Evaluation of Hydraulic performance of Gravity Water Supply System (The Case of Likimse-Abela Water Supply System)

BY

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Abstract

Water supply distribution network components needs to design carefully because of cost involved and its life time required to serve. However, in many of the developing countries, like ours the hydraulic and physical performance of water distribution network is inadequate to meet consumers' satisfaction on water supply and losses in system. Likimse-Abela, gravitational water supply system in Wolayta Zone, Humbo Woreda has been experiencing disruption and uneven distribution of water supplies for days to a week. This study was conducted in Likimse-Abela, to evaluate hydraulic performance of existing water supply distribution system of the rural PAs. Both primary and secondary data sources were used in this study. Primary data were collected though Key informant interview and focus group discussion with Likimse-Abela Committee members, Woreda Water and Irrigation office experts, field observations, photographs of relevant sites and infrastructures were taken. For secondary data collection, documents review was used to collect valuable information. To analyze the data which is collected from different sources, both qualitative and quantitative methods was used. The computer software application that is excel was used to analyze the data obtained from office. The field survey data for distribution system was evaluated by using the engineering software called EPANET-2. As per the analyzed results; the current average per capita demand of the area was estimated 11.62lit/cap/day however, enough of water flowing from the source to the system and low system efficiency were observed to satisfy the user community. According to simulated results; the maximum of 191.8m and minimum of -73.99m water pressure were examined in the transmission and distribution main, respectively. Further, the analyzed water losses result in Likimse-Abela water supply system indicates that about 12.19% and 58.22% of production is Non-Revenue Water both on the pressure main and distribution network. In general, weak connection, failure in gate valves and taps, small capacities of night storage reservoir and large volume of water loss were leads the point to work unevenly. Hence, few years after most of residences were not satisfied to the service of the system. Finally, it is recommended that the reduction of NRW, evenly distribution and safeguarding resources through appropriate water demand management strategies should be given priority. As a result, welfare of human being will be maintained holistically.

Key words: *Hydraulic performance, level of service, EPANET, Likimse-Abela, Humbo and Wolayta.*

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Abbreviations and Acronyms

BoFED	Bureau of Finance and Economy Development
GO	Governmental Organization
GPS	Geographical Positioning System
GTP	Growth Transformation Plan
IRC	International water and Sanitation Center
NGO	Nongovernmental Organization
PAs	Peasant Association
RWSS	Rural Water Supply System
SNNPR	Southern Nation, Nationality and People Region
TC	Technology Choice
UN	United Nation
WASH	Water supply, Sanitation and Hygiene
Water CAD	Computer Aid Design
WLTF	Water Loss Task Force
WVE	World Vision Ethiopia
PIs	Performance indicators
WSS	Water Supply system
m.a.s.l	Meter above sea level
EPA	Environmental Protecting Agency of United state
KPIs	Key Performance Indicators
ISO	Insurance Service Office

Chapter One

1.1. Introduction

1.1.1. Background

Starting from the ancient time, a safe supply of water is the necessity of mankind and the whole living things, therefore water supply systems development become the most important public utility. To develop upgrading or maintaining this utility an enormous amount of money is spent every year around the world. Comparing in the system itself the allocated budget for the water supply system development the major share of capital investment goes to the water conveyance system which is conventionally called water distribution network. Nearly 80% to 85% of the cost of water supply project is used for construction of the distribution system; therefore, using rational methods of designing water distribution system will result in considerable savings and keep the system efficient.

Water supply infrastructure varies from complex to simple corresponding to the capital investment spent on it, the capacity of sources and the size of the distribution system. Similarly, its type from community managed system to a complex computerized, remote-controlled, multisource system; however, the objective of all the water systems are the same to provide safe water to the cheapest cost in a sustainable way to the end users. (K. Swamee and Sharma, 2008)

Since the ancient time, as that of the system intended to be studied in this paper, gravity water supply systems have been utilized to conduit water from source to where it is needed for purposes. However, in those times the conducting devices were earthen or lined channels, wooden or bamboo pipes, etc. lay on the ground or suspended above the ground but following the natural water path, contour. In these days, the mechanism left regularly in agriculture and domestic purpose. Sealed pipes of iron and plastic invented to transport water in different topography from the source to the point of requirement. In this case, the flow can be made is dependent on the amount of pressure formed in it may resisted by the pipe and losses created due to friction against the wall of the pipe.

In many rural areas, gravity water supply system is best option which can be chosen by the community since the investment and O&M cost is reasonable compared to pumped system. In most of the cases, gravitational water supply system looks simple and usually preferable, however do not function properly and fail to distribute the water evenly among the water points. This is because of factors influencing the effective service and infrastructures which results hydraulic failure, the focus of this paper.

The primary objective of a water supply utility is to have this asset operate at its maximum possible efficiency with minimum cost throughout its design life time. To achieve this objective, the first major to be taken is to evaluate and assess the existing efficiency of all the components of WSS using suitable performance indicators (PIs). Since the whole component of the distribution system is responsible for the impact of the water utility and customer satisfaction priority has provided to it to be studied not only in terms of technical concept but in its holistically situation.

Despite the key role distribution system plays in providing service to the public, few or no standards exist to evaluate its performance. Owning to the above case, this study examines existing distribution system performance standards.

Hydraulic performance which relate to the efficient delivery of water is measured in terms of pressure, flow rate, velocity, etc. As explained on the above, performance measures include customer complaints, which is a measure of how quickly a utility allays customers' concerns; and customer satisfaction, which gages customers' perceptions of the overall service they receive using analysis of pipe system models (EPANET-2).

Therefore, performance assessment is the key to understand the sustainability of the system. Hence performance assessment can be defined as "any approach that allows for the evaluation of the efficiency or the effectiveness of a process or activity throughout its life time. (Aleger & Coelho, 2012)

1.1.2. Problem Statement

The basic theory of this research is that the service level of rural water supply systems is strongly related to appropriate design and its assumption, workmanship of implementation, utilization of quality of materials and management practice.

This study is intended to conduct the assessment on the extent of hydraulic performance of rural water distribution system, the case of Likimse-Abela water distribution system that has influenced by low capacity of utilities, weak connection, damage on the valves, uneven distribution, low respond to operation and maintenance and poor management system.

According to the design document of Likimse-Abela water supply system, the system has served for 18 years but, currently like many other rural water supply systems the problem of uneven and intermittent water distribution, water loss is growing concern. Out of the major indicators of such water distribution are the capacities and configuration of system components, material quality and workmanship.

The other observed problem in Likimse-Abela water supply system is frequent disconnection of joints in the water distribution network during which the woreda water office does not have immediate response for maintenance. Further, surveys indicated that the water released from the reservoir, approximately 58.22% is loss in the distribution because of malfunctioning taps and gate valves that cause the system open and weak or unauthorized connection to the farm yard. Thus, it made crisis on some of water points.

To achieve the appropriate service level of the case system this paper try to assess the hydraulic parameters, the variations, and the relations between them and other factors, which control the performance of the water supply networks; also, it must investigate the effects of local conditions and improve them for increasing the efficiencies of the water distribution systems.

1.2. Objective

1.2.1. General Objective

The overall objective of the thesis was to evaluate the hydraulics performance of rural gravitational water supply system the case of Likimse-Abela Water Supply System.

1.2.2. Specific Objectives

1. To evaluate the existing water supply situation of the town
2. To evaluate water losses of existing water distribution system
3. To simulate hydraulic parameters of existing distribution system
4. To evaluate the present water demand and forecast future demand

1.3. Research Question

1. What is current situation of the system?
2. What are causes for Leak of pipes and losses of water?
3. How is the performance of water supply network of Likimse-Abela?
4. What are the remedial measures which reverse the system's challenge to extend the service up to its life time?

1.4. Significance of the Study

This research assesses the situation of hydraulic performance of rural gravitational water distribution system, the case of Likimse-Abela found in Humbo woreda, Wolayta zone. Based on findings factors that are negatively contributing for satisfaction level of the service will be listed out and alternative system operation, maintenance and management will be recommended. In addition, hopefully, the insights that has drawn from this study will initiate further research on similar sites and will contribute to solving the existing problems of rural water distribution system.

1.5. Thesis Out line

This research includes the following five chapters: -

- The first chapter is the introductory section which depicts the overview of the research background, statement of the problem, objective, research questions, significance of the study
- The second chapter comprises a review of related literatures on the hydraulics of the water supply which are useful to understand the indicators, the principle and situation of performance of services.
- Chapter three deals with explanation of the study area, existing water supply situation of woreda and site and model selection criteria and water demand of the area including detail descriptions of the methodology work and materials used in the research.
- The fourth chapter present results and discuss findings.
- The last but not the least chapter is the conclusion and recommendations part.

Chapter Two

2.1. Literature review

2.1.1. Introduction

Challenges in providing satisfactory water to the rapidly growing population especially for rural area in developing countries are increasing from time to time. Thereby, most of the existing water supply systems are unable to meet the various demands of water. Beside to this; infrastructural aging problem, poor management of the existing system components/assets and utilities capacity shortages were increases the level of water losses in the distribution system. In most of the cases water supply systems are often unable to meet existing demands and are not able to perform to meet the satisfaction of the consumers. Which means consumers take disproportionate amounts of water compared to the standard set by different organization. In addition to the overall shortage of water or inefficiency of the system, many areas are faced a problem in distributing the available water impartially among the residents. Besides, the poor management of existing infrastructure increases the level of water losses in water supply. As this research deals with performance evaluation of distribution system, issues related to water loss, pressure fluctuation, uneven flow rate, leakage and their cause are identifying and reviewed in this chapter.

2.1.2. Network Hydraulics

'In networks of interconnected hydraulic elements, every element is influenced by each of its neighbors; the entire system is interrelated in such a way that the condition of one element must be consistent with the condition of all other elements' (Walski, et al., 2003). According to (K. Swamee and Sharma, 2008) interconnections of hydraulic elements are defined in concepts of conservation of mass and energy.

2.1.2.1. Conservation of Mass

'The principle of conservation of mass dictates that the fluid mass entering any pipe will be equal to the mass leaving the pipe (since fluid is typically neither created nor destroyed in hydraulic systems). In network modeling, all outflows are lumped at the nodes or junctions' (Walski et al., 2003).

$$\sum_{Pipes} Qi - U = 0 \quad \text{....2.1}$$

Where, Qi = inflow to node in i-th pipe (L3/T)

U = water used at node (L3/T)

The term for accumulation of water at nodes is required to describe stored and withdrawn water from tanks, while extended period simulation is regarded (Walski, et al., 2003).

$$\sum_{Pipes} Qi - U - \frac{ds}{dt} = 0 \quad \text{....2.2}$$

Where, dS/dt = changes in storage (L3/T)

2.1.2.2. Conservation of Energy

'The principle of conservation of energy dictates that the difference in energy between two points must be the same regardless of the path that is taken' (Bernoulli, 1738 cited in (Walski, et al., 2003). The equation for conservation of energy is written in terms of head as follows:

$$Z_1 + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} + \sum h_p = Z_2 + \frac{P_1}{\gamma} + \frac{V_2^2}{2g} + \sum h_l + \sum h_m \quad \text{....2.3}$$

Where, Z = elevation (L)

P = Pressure (M/L/T2)

γ = fluid specific weight (M/L2/T2)

V = velocity (L/T)

g = gravitational acceleration constant (L/T2)

hp = head added at pump (L)

hL = head loss in pipes (L)

hm = head loss due to minor losses (L)

Therefore, in connected network the difference in energy at any two point is equal to the energy increases from pumps and energy losses in pipes (frictional head loss) as well as energy losses in bending and fittings (minor head loss) that occur in the path between them.

2.1.3. Principles of Pipe Flow Hydraulics

When water flows through pipe follows basic principles of hydrodynamics. Hydrodynamics helps to understand how the fluid behaves while moving in the pipe or channel. In relation to this it can answer what dimension and type of pipe can be suggested to conduct fluid from one place to the other. Similarly, the types energy used to push water should be known in relation to the capacity of holding and pushing pressure.

Thanks to scientists, during the past 250 years they have found new ways of answering the question about size, shape, and strength rather using trial and error. Different experiments show the situation of pipes, pumps and other utilities in the laboratories and come up with mathematical models before the system get install. Hence much of our current knowledge of water flow in pipes and open channels has come from this kind of experimentation; empirical formulae were derived from the data collected to link water flow with different size of pipes and channels. Today we use formulae for most design problems, and come up with models which can perform complicated trials and mathematical calculation within few minutes.

While studying the water flows in channel or pipe new factors appear, which were not in hydrostatic, so to come to simplified situation, engineering approach tries to omit some of factors that have brought insignificant impact on the outcome with rational reasons. For instance, viscosity usually ignored while determining pipe size and makes the calculation process easy and it makes no difference to the final choice of size.

The development of hydraulic theory has produced *three* important basic tools (equations) which are fundamental to solving most hydrodynamic problems: (Kay, 2007)

- ♣ Discharge and continuity
- ♣ Energy
- ♣ Momentum.

These are important theories of hydraulics for the design, evaluation and assessment of hydraulics of water supply and other systems.

2.1.3.1. Discharge and continuity

Discharge refers to the volume of water flowing along a pipe or channel each second. There are two ways of determining discharge. The first involves measuring the volume of water flowing in a system over a given period.

Discharge can also be determined by multiplying the velocity of the water by the area of the flow. To understand this, imagine water flowing along a pipeline. In one second the volume of water flowing past X-X will be the shaded volume.

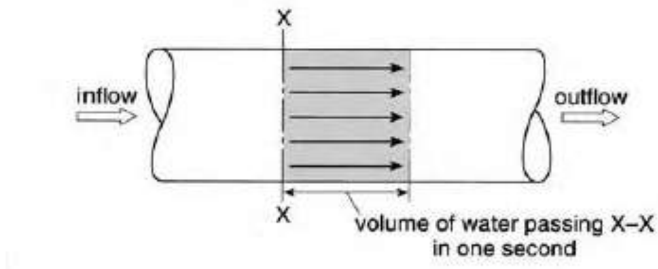

Figure 1: Discharge across a cross section of the pipe

This volume can be calculated by multiplying the area of the pipe by the length of the shaded portion. But the shaded length is numerically equal to the velocity v and so the volume flowing each second (i.e. the discharge) is equal to the pipe area multiplied by the velocity.

$Q = V * A$…………2.4

The continuity equation builds on the discharge equation and simply means that the amount of water flowing into a system must be equal to the amount of water flowing out of it.

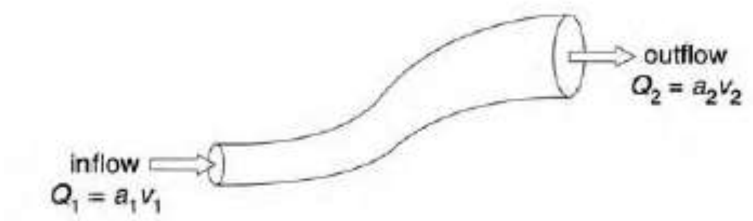

Figure 2: Continuity equation across the pipe

Inflow = Outflow………………2.5

$$Q_i = Q_o$$

$$A_1 \frac{dx_1}{dt} = A_1 \frac{dx_2}{dt}$$

But from the discharge equation:

$$\mathrm{Q = VA}$$

And so:

$$V_1 a_1 = V_2 a_2$$

So, the continuity equation not only links discharges it also links areas and velocities as well. This is a very simple but powerful equation and is fundamental to solving many hydraulic problems. However, continuity holds true only the flow is steady state flow. In case of reservoir or tanker if the release may not be equal to inflow the situation of continuity slightly different from the above with the rate of change of storage.

$$Q_i = Q_o + \Delta Q$$

It is sometimes called water balance equation which is easily observed in the reservoir to store night flow for future use.

2.1.3.2. Energy

The other basic tool uses energy to make the link between pressure and velocity in pipes and channels. Energy is the ability to do useful work and in case of water in the system can possess energy in three ways:

- Pressure energy
- Kinetic energy
- Potential energy

When water is under pressure it can do work by rotating turbien which in turn rotate generator for producing electric. This pressure energy is calculated as:-

$$\text{Pressure energy} = \frac{P}{\rho\, g}$$

Where, P is pressure

g is gravitational acceleration

ρ is density of water

Pressure energy is like pressure head and measure in meter (m).

When water possess energy because of its movement it is call kinetic energy it is sometime called velocity energy and expressed by: -

$$\text{Kinetic energy} = \frac{V^2}{2g}$$

Where, V is velocity

g is gravitational acceleration

It is also sometimes referred velocity head and measure in meter (m)

The third energy form is potential that is created because of the location of the water. It is determined by the height of water above some fixed datum.

$$\text{Potential energy} = \mathrm{Z}$$

Where Z is the height of water above a datum

This energy is use full to generate hydro power and in case of gravity-flow water system the driving force to reach the water to the required place is energy.

A gravity-flow water system is “powered” by gravitational energy. The amount of such energy in the system is determined by the relative elevations of all points in the system. Once it has been constructed, all points in the system are immovably fixed (i.e. buried into the ground) and their relative elevations cannot change. Thus, for any system, there is a fixed, specific quantity of gravitational energy available to move water.

So, the basic conservation of energy equation evolved from the total energy concept considering the loss in energy through the pipe line in the hydraulic systems used to solve the hydraulic problems.

The Swiss mathematician Daniel Bernoulli (1700-1782) made this most important discovery. Indeed, it was Bernoulli who is said to have put forward the name of hydrodynamics to describe water flow. It led to one of the best-known equations in hydraulics-total energy equation. It is also referred to as the Bernoulli equation. (Kay, 2007)

$$\text{Total energy} = \frac{\mathrm{P}}{\rho\,\mathrm{g}} + \frac{\mathrm{V}^2}{2\mathrm{g}} + \mathrm{Z}$$

Simply knowing the total energy and put those together in the system Bernoulli understood it will be the same throughout a system. Which is makes it much more useful.

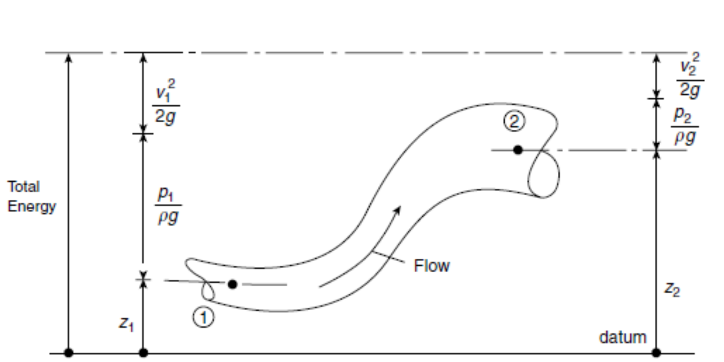

Figure 3: Energy equation at two points within the pipe

When water flows through a pipe the total energy at section 1 and section 2 is the same. So, the total energy equation/Bernoulli equation re write as follow: -

$$\frac{P_1}{2g} + \frac{V_1^2}{\rho\ g} + Z_1 = \frac{P_2}{2g} + \frac{V_2^2}{\rho\ g} + Z_2$$

In this case, it doesn't mean that pressure, velocity and elevation at 1 and at 2 correspondingly equal however, their sums are equal. This also again true if and only if there is no energy loss occurred. It is obvious that the system needs energy to be driven and in the meantime, there will be loss of energy like observed on the above on conservation of energy that should be considered in the system design, management, evaluation and assessment. So, one can calculate change in pressure and velocity of fluid by knowing some of the above principles and natural topographic elevation.

Continuity and energy in this regard can explain many interesting phenomena; however, for this research we will see the application of the equation for measuring discharge, pressure changes and velocities.

2.1.3.3. Momentum

The momentum equation is about movement and the forces which cause it. The link between force, mass and velocity and is used to determine the forces created by water as it moves through pipe and hydraulics structures

$$\text{Forece(N)} = Q(V_2 - V_1)$$

Where V_1 and V_2 represents velocities in a system, and so

This is now in a form that is useful for calculating forces in hydraulics. Based on these basic principles of hydrodynamics and real fluid (water), but ignoring viscosity since it is insignificant and taking a friction force that exists between the fluid and the flow boundary Bernoulli equation rewrite as follow but modified. bear in mind that the friction between the particles of flowing water is negligible.

$$\frac{P_1}{2g} + \frac{V_1^2}{\rho g} + Z_1 = \frac{P_2}{2g} + \frac{V_2^2}{\rho g} + Z_2 + h_f$$

h_f is important in determining the size of pipe to carry a given flow considering the factors influencing its value.

2.1.3.4. Frictional force/Loss

The magnitude of energy lost due to friction against some obstacle is determined by several factors and creates the above situation in the pipe. The major factors would be the roughness of the obstacle, and the velocity of the flow. Minor factors would include water temperature, suspended particles, dissolved gases, etc.

Holding this h_f (head loss because surface resistance or frictional loss) can be calculated using Darcy–Weisbach, Hazen–Williams or Chezy-Mannig, however for this research and to adhere to conventional way of design, evaluation and assessment the following suggestion has looked through.

Liou (1998) pointed out the limitations of the Hazen–Williams equation, and in conclusion he strongly discouraged the use of the Hazen–Williams equation. He also recommended the use of the Darcy–Weisbach equation with the Colebrook–White equation.

Swamee (2000) also indicated that the Hazen–Williams equation was not only inaccurate but also was conceptually incorrect.

Brown (2002) examined the historical development of the Darcy–Weisbach equation for pipe flow resistance and stated that the most notable advance in the application of this equation was the publication of an explicit equation for friction factor by Swamee and Jain (1976). He concluded that due to the general accuracy and complete range of application, the Darcy–Weisbach equation should be considered the standard and the others should be left for the historians. Considering the above investigations, only the Darcy–Weisbach equation for pipe flow has been used in the entire portion of the research.

Ramalingam et al. (2002) published a brief history of water distribution network analysis over 100 years and included the chronology of pipe network analysis methods. Several methods have been used to compute the flow in pipe networks ranging from graphical methods to the use of physical analogies and finally the use of mathematical/numerical methods.

The preferable equation, Darcy-Weisbach (Colebrook-White) equation is common in the use of analysis of pressure pipe systems. For any flow rate and incompressible fluid.

$$V = \sqrt{\frac{8g\,RS}{f}} \qquad 2.6$$

Where V: flow velocity (m/s)

g: gravitational acceleration (m/s^2)

R: hydraulic radius (m)

f: Darcy-Weisbach friction factor

S: Friction slope

The friction factor, f of Darcy-Weisbach can be found using the Colebrook equation that is why all the time Darcy-Weisbach equation write under bracket with Colebrook-White.

$$\frac{1}{\sqrt{f}} = -2\log\left[\frac{K}{14.8R} + \frac{2.51}{R_e\sqrt{f}}\right] \qquad 2.7$$

Where K: roughness height (m)

R_e: Reynolds number, it is an index used to classify flow as either laminar or turbulent flow and calculated as follows: -

$$R_e = \frac{4VR}{\nu} \qquad 2.8$$

Where R_e: Reynolds number

V: Mean Velocity (m/s)

R: Hydraulics radius (m)

ν: Kinematics Viscosity (m^2/s)

Without the above principle, suggestion and consideration of determinants the design, evaluation and assessment of water supply system may not provide the required efficient service, in other way the customer satisfaction will not be achieved.

Minor losses

The other determinants in designing, evaluation and assessment of performance are minor losses these are result of localized areas of increased turbulence and are frictional head losses, which cause energy losses within a pipe. A drop in the energy and hydraulic grades caused by valves, meters, and fittings, the value of these minor losses is often negligible relative to friction and for long pipes, and they are often ignored during analysis. However, the importance of such losses will depend on the layout of the pipe network and the degree of accuracy required.

The resulting head loss is computed from these losses are calculated using the following equation

$$H_m = \frac{KV^2}{2g}……………….2.9$$

Where K: minor loss coefficient for specific fitting

H_m: Minor loss (m)

V: Velocity (m/s)

g: gravitational force (m^2/s)

These minor and/ or major losses may occur by the following dynamics which contributes a lot to create problem on the water supply system.

2.1.4. Types of water distribution system

According to (Walski, et al., 2003), the water distribution networks have classified as explained below;

2.1.4.1. Branched system

This system named because of its similarity with tree branch that the water has only one possible path to reach to the leaves from the source. Similarly, the water from the source follows only one path to a customer. Thereby, these are applicable for small-capacity water suppliers, and are common in most developing countries. The advantage of these system is the most economical because of its low cost, but it has some disadvantages as presented below;

- Low reliability, affects all users especially located downstream of any breakdown in the system. So that, their water services were interrupted until the repairs are finished.
- Fluctuating in water demand, producing rather large pressure variations in the system.
- When there is a need for developing the network, new branches follow that development and new dead ends will be constructed.
- It is also danger of contamination during the network without water.

2.1.4.2. Looped system

As the name suggests, in looped systems it uses different paths that water can follow to get from the source to a customer. The systems are generally more desirable than branched systems because it coupled with sufficient valves and accessories, and can provide reliability in the water distribution. In this system because of more than one path for water, the system capacity is greater and it improves the hydraulics of the distribution system. For example, it considers a main break occurring near the reservoir in each system depicted. In the looped system, that break can be isolated and repaired with little impact on customers outside of that immediate area. While, the effect of water service interruption is more significant to branch system.

2.1.5. Water Distribution Force

Since the ancient time, numerous trials and innovations have been reviled for the development of water extraction and distribution mechanism. Starting from dwelling by the river side, digging wells nearby the compound up to household level pipe connection. Water supply and distribution system has passed a complex journey to satisfy various objectives to meet public requirement. Because of significant development of urbanized areas and construction of thousands of small and large-scale water supply and distribution systems in recent decades, many people have access to clean water and adequate sanitation. However, the quality of service which is provided by water utilities is often questionable and the cost of new systems is still often prohibitive. So, this ever-increasing need push scientists and engineers to study and use mathematical models for design, evaluation and assessment of distribution system from the source to the required point based on the principles explained above. The water distribution system also increasingly modified and consist various components such as pipes, pumps, reservoir and controlling elements. They collectively form a monolithic system, as body of human being with different organ works together and supply the required quantity of water.

The driving power that required in the distribution systems are several different way, as local conditions or other considerations may dictate as follows: - (CHATTRTJEE, 1998)

2.1.5.1. Gravitational distribution

This is possible when the sources of supply are at some elevation above the required point, so that sufficient pressure can be maintained in the mains. This is most reliable method if the conduit leading form sources to points is adequate in size and well safeguarded against accidental breaks. It is economical comparing to systems driven by pump which uses external energy to drive and by itself it is also system that needs follow up of its performance.

2.1.5.2. Pumping

Of course, it is common method generally used in practice. In this method, the excess of water pumped during periods of low consumption is stored in elevated tanks or directly without storage. The water is forced into the main and then to the consumers. For a power failure would mean complete interruption in water supply. If pumping is done electrically, the peak power consumption of water plant is likely to occur during high current consumption and this increases power cost.

While distributing water adequate pressure, flow rate from source to all customers become issue of demand. It is generally desired that the water should be supplied in a continuous manner; however, this is an ideal condition. As a result, measuring and evaluating a systems' performance is become focus to assure the continuity of the system in its life time, however evaluation itself a complex problem.

Hence, the distribution system should be able effectively satisfy major consumptions by including the environmental and health impact.

To provide reliable, accessible, cost effective structure for systems' operation, it is necessary to identify the most critical priorities and preferences of all major customers, and then try to establish effective management tools to achieve all predefined goals. For all these reasons, providing a framework for customer-oriented operation is essential.

Now in a day, in our country context water supply networks are crucial systems which have physical complexity in their construction, installation, operation, with enormous economic concerns and environmental implications. They also contribute significantly to public health. Despite this, design and operational challenges have often been underestimated by professionals and engineers and their direct or indirect impacts.

2.1.6. Pressure related phenomenon in the distribution line

2.1.6.1. Water Hammer

The phenomenon that may occur in the distribution system is water hammer. When water flowing in pipe line is abruptly stopped or suddenly changes its velocity and the momentum is destroyed. This exerts a thrust on the valve and additional pressure on the pipe shell behind. The pressure so developed is known as water-hammer pressures and may be so high as to cause bursting of the pipe shell.

The maximum pressure developed in pipe lines due to water-Hammer is given by the formula

$P_2=(14.762.V)/\sqrt{(1+(K.d)/t)}$……………….20

Where V: Velocity of Water just before the closing of the valve (m/s)

d: diameter of pipe (m)

t: Thickness of pipe shell (m)

K: constant= (Modulus of elasticity of pipe material)/(Bulk modulus of elasticity of water)

The value of K for steel comes out to be 0.01, forecast iron 0.02 and for cement concrete is 0.1

The water hammer pressure can be calculated by using the above equation. But for the design purpose its value is generally taken as 8.4kg/cm2 for small size pipes of 75mm to 250mm.

2.1.6.2. Cavitation

The water supply system may suffer from cavitation and it can cause lots of problems, particularly in pumps and control valves. It occurs when a fluid is moving very fast; consequently, the pressure can drop to very low values approaching zero (vacuum pressure).

2.1.6.3. Boundary layer

Friction between water flow and its boundaries and the internal friction (viscosity) within the water gives rise to an effect known as the boundary layer. Water flowing in a pipe moves faster in the middle of the pipe than near the pipe wall. This is because friction between the water and the pipe wall slows down the flow. Very near to the pipe wall water sticks to it and the velocity is zero, although it is not possible to see this with the naked eye. Gradually the velocity increases further away from the wall until it reaches its maximum velocity in the center of the pipe. To understand how this happens. Imagine the flow is like a set of thin plates that can slide over each other. The plate nearest the wall is not moving and so it tries to slow down the plate next to it – the friction between the plates comes from the viscosity of the water.

2.1.7. Components of water distribution network

2.1.7.1. Transmission and distribution mains

In the water distribution system, piping is often categorized as transmission mains and distribution mains. Transmission mains were consisting of components that are convey large amounts of water over great distances, typically between major facilities within the distribution system. In most water supply system, transmission main is mainly used to transport water from treatment plant to service reservoirs/ storage tanks. Whereby, individual customers are usually not served from these mains. Distribution mains are an intermediate pipeline used to delivering water from transmission main to customers. The mains are smaller in diameter than transmission mains, and typically follow the general topology and alignment of the town streets. Different fittings such as elbows, tees, reducers, crosses and numerous other accessories are used in the main to connect pipes. While, other maintenance and operational appurtenances, such as fire hydrants and valves are also connected directly to the distribution mains. Further, services also called service line were laid and transmit water from the distribution mains to end customers. (Tomas, et al., 2003) cited by (Benyam, 2016)

The main part of the distribution component that has influencing the hydraulic performance of the system is pipe. At the same time in the hydraulic of the water supply system, as explained above it is pipes contributing much percent of losses that alter the performance of the system, because of this pipe, type, dimension, age and roughness are factors that always go together. If one of the above characteristics of pipes is not fulfilled, the satisfaction to the level of service also will not be attained.

Hence, to understand the failure contribution to achieve satisfactory level to the service the above situation was observed as follows. The pipe type could be GI, HDPE, PVC, etc. In this case, the pipe as that of other materials have their own design life to serve before it get deteriorate. For instance, GI pipe is considered deteriorated earlier compared to the PVC and HDPE because of the early reduction of coefficient of roughness. Regarding the pipe related to its age which required for the intended service and design period of the whole system (0' Conner, 1995) cited by (Mansoor, 2007) showed that "*a pipe is newly installed, the failure intensity can be high and as a result high failure rates. This can be described as a settling in period, possibly* *due to construction practices. After early faults have settled down, the failure intensity will be smaller and remain relatively constant for long periods of its useful life. Then as the pipe ages, the intensity will begin to increase and the pipes start deteriorating. This is the period of most interest, as eventually the intensity will exceed certain level, and it will become cost efficient to replace the pipe."*

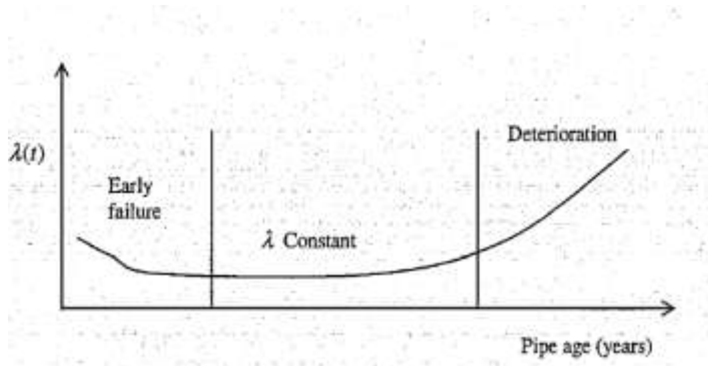

Figure 5: Bathtub Curve (0' Conner, 1995)

The second factor related to the pipe, which negatively impact to the distribution of pipe is roughness of material utilized for the system. This is because the deterioration rate varies from one material to another. This is reflected in the decaying rate of Hazen William friction coefficients (C value) with pipe age. Therefore, the roughness coefficient of the materials was factors observed but not expected with major change referring to Yan, 2006 cited by (Mansoor, 2007).

The diameter of the pipe obviously known to influence the distribution corresponding to the required water to be distribute for target area. In this case*," Kettler and Goulter (1985) cited by* (Mansoor, 2007) *provided a regression equation for the number of breaks versus diameter and time for cast-iron and asbestos-concrete mains in Winnipeg, Canada. Their expressions indicate a strong inverse relationship with pipe failure and diameter*."

In addition, the above factors which affects the level of service the type of the distribution network that has developed using pipes is one of the determining factors for the reduction in hydraulic performance. So, in branched system in which a main breakage cause all downstream consumers to be out of service comparing to looped in high demands time, the velocities are faster, as a result the head losses are higher. The other issue that usually missed in pipes are the positioning of pipes related to nature of flow through determines the direction where more flow to happen. If the flow is laminar flow in a pipe and the T-branching not installed on level position the flow tends to the lower direction by the influence of force of gravity. In this case, if situation forces, there must incorporated controlling devises either to create turbulent situation little further down the junction or balance flows using gate valves.

Therefore, in general all the above factors that have explained related to the pipes and the external factors influence the efficiency of pipe negatively.

2.1.7.2. Connecting fittings

Types of joints used in water distribution system have a significant impact on failure. Joints can be characterized by their strength to withstand stresses, ability of flexibility to withstand movements and water tightness. As mentioned by May, 2000 cited (Mansoor, 2007) the joint strength vary with pipe material and size. Connection particularly on secondary pipes these may be associated with the poor workmanship or the type of materials installed as mentioned above by May, 2000. It was also well known that these portions are hydraulically stressed comparing to the pipe line without connection stretching. Therefore, much of the losses next to valves and taps on the system is expected due to the failure in connection.

2.1.7.3. Valves

In water distribution system regulation and control of either discharge or pressure is frequently achieved through use of valves. Different types of valves are available, among them the most widespread are: Pressure reducing valves, pressure sustaining valves, flow reducing valves, check valves and general purpose valves. The most common type of valve in water distribution systems is the isolation valve, which can be manually closed to block the flow of water. There are several types of isolation valves that may be used, including gate valves (the most popular type), butterfly valves, globe valves, and plug valves.

2.1.7.4. Reservoir and storage tanks

In the water distribution system, reservoir and storage tanks are mainly provided to meet the fluctuations of water demand and to stabilize pressure within the distribution system. Similarly, these components were reserve water for emergency requirements. Accordingly, the common reservoirs established in the water supply system are circular and/or rectangular type which build either from concrete or steel materials. The recommended location of such facilities is mainly in elevated area beyond the center of service area. (Benyam, 2016)

2.1.8. Water Supply System Performance Classification

The topic, performance of a system has several meanings in all aspect of engineering. This may be used for evaluation of systems, for assessment in the planning, design and operation, etc. The goals of term performance of a system are almost universally recognized, and it is commonly taken to consensus that the system should satisfy the demand of almost all customers. Indirectly, it is to mean that the system provides sufficient flows with adequate pressures and acceptable quality. The utilities recommended also are normally selected based on satisfying the requested need holding that it is sustainable and within reasonable economic limit. Therefore, any inconveniences in the quality and quantity of supply through the system is considered as poor performance.

However, once one goes beyond general statements and attempts to flesh these issues out in more quantitative detail, a great deal of variation is observed in specific way systems are assessed and evaluated. To begin to frame these issues, it is helpful to go back to a very basic classification. Basically, it is helpful to classify performance based on physical and chemical characteristics of the supplied water into two primary aspects of quantity and quality. Meanwhile quantity of supplied water can be measured based on two major physical characteristics of supplied water, as quantity of pressure and quantity of outflows in the service life. The quality of water in other hand depends on chemical characteristics of the water and its constituents in service life.

Different consultant and researcher in different time have undertaken evaluation and or assessment of system classified as functional, not functional, problematic, but working and sustainable. To qualify the system as sustainable and meet best performance some organizations used the following criteria.

- Regular supply in all standpipes and not more than 30-60 minutes fetching travel time
- Regular operations and maintenance
- Water point radius to the community with in the standard level of GTP-2, 1km for rural and 250m for urban area.
- The Per capita standard demand available, 25lit/cap/day for rural and 100lit/cap/day to 40lit/cap/day for urban corresponding to categories of the town.
- Collection fee covering the running project; inclusion of the poorest in the community
- Consistent support from the Woreda water office

Some other organization also suggested KPIs which fall under the following five categories: service delivery; financial credibility; technical effectiveness; human resources; background information. (Still, 2017). However, David Still concluded that the three most important indicators of a water supply scheme's health are:

- Water quality [where the basic questions are: does it look good? does it taste good? does it smell good? and is it disinfected? Is the source protection in order?]
- Reliability [measured as working tap days as a percentage of the maximum possible]; and
- Source sustainability [where this is an indicator showing either the level in the dam, the flow in the spring or the level in the borehole, relative to some minimum allowable level].

Which are also elaborated by (Husnain, et al., July, 2013), Performance is the degree to which infrastructure provides the services to meet the community expectations and it is a measure of effectiveness, reliability, and cost (NRC 1995). The performance of a WSS depends on efficient and reliable working of all functional components including water resources, physical assets, operational activities, personnel, and environmental and financial activities. The performance of a WSS is evaluated to indirectly estimate the conditions and rehabilitation needs to ensure continuous and reliable working of these components of a WSS during their entire service life before the occurrence of a failure. Once a failure has occurred, the cost of corrective action is much more than planned preventive action would have been. The difference between the planning and management cost and the cost of corrective actions justifies the need for PA. There are several methods of performance evaluation of water utilities given in the literature, including PIs, total factor productivity indices, and production frontiers (Coelli, et al., 2003) cited by (Husnain, et al., July, 2013). However, for methods other than PIs, more sophisticated data are required, which is difficult to acquire or is sometimes even missing in the case of most SM-WSS.

A water supply system may face several problems associated with its continuous aging process, pressure fluctuation, water loss, water quality deterioration and so on, however to operate and maintain a WSS at its maximum possible efficiency at a reasonable cost is one of the prime objectives of water utility. The entire situation in hand the goal can only be achieved by adopting rational and optimum maintenance, rehabilitation, and renewal strategies. Hence, the primary step towards a sustainable WSS is to evaluate the performance of a given WSS, which further provides the basis for detailed investigations (detailed condition assessment and (or) M/ R/R strategies). The performance of a WSS can be assessed by selecting suitable indicators as indicated below.

Table 1: Performance indicators for water supply services *(Husnain, et al., July 2013)*

Item No	Indicators categories	Description of subgroup of Performance Indicators
1	Water Resources	• Water resources availability • Usage efficiency and reuse/Multipurpose use
2	Personnel	• Personnel data • Personnel per function • Technical services personnel per activity • Personnel qualification and training • Personnel health and safety
3	Physical	• Treatment • Storage, transmission and distribution • Metering coverage • Automation and control
4	Operational	• Inspection and maintenance of physical assets • Electrical and signal transmission equipment installation • Mains, valves, and service connection rehabilitation • Water losses • Failures • Water metering efficiency
5	Quality of service	• Coverage • Public taps and standpipes • Pressure and continuity of supply • Quality of supplied water • Service connection and meter installation and repairs • Customer complaints
6	Financial and economic	• Revenues • Costs • Composition of running costs per main function of the water undertaking • Composition of running costs per technical function activity • Composition of capital costs • Investments • Average water charges • Efficiency indicators • Leverage indicators • Liquidity • Profitability • Water losses

2.1.9. Water Demand

To evaluate a WDS, it would be ideal to identify all demands in the area and then to determine their expectation with consideration to associated consequences. Major customers may include those facilities that constitute significant portion of supply demand in the area (e.g., residential, industrial, and firefighting, hospitals or health). In rural area, domestic and cattle watering are main customer facilities.

To investigate the quantity of water needed for each individual customer, the period they need water for, and the appropriate level of water quality that is suitable for their need. The overall standard would consider that different customers have different needs in terms of quantity and quality of water as well as the time that they need water for. Yet this is clearly not trivial to achieve since one system generally serve all users.

The estimation of the quantity of water should reflect customer preferences and expectations efficiently. If the customer needs are met, the level of satisfaction is higher and the water utility is managed well. Of course, it is ideal to have an accurate water utilities and estimation of demand during service time. There is an allowed percent uncertainty both for the utilities operation and demand estimation since the demand and preferences usually depends on various dynamic factors such as land use, population growth and migration, demographic structure, various development factors including urbanization and, in some places, rising (falling) standards of living. Water demands are extremely variable over the period of system operation. For example, the residential water demands have considerable fluctuations over days, weeks, months, seasons and years. Therefore, not only the total quantity of water is interested in, but it its various time variations are still important.

Land use has an important influence on water demand. If a customer's activity is known, their associated water demands can be estimated. This estimation might sometimes be carried out by metering actual water usage for all major types of land use with different densities. Otherwise, a rough (and more uncertain) estimate of water demand for presumed activities might be required, as it is discussed in more detail in the next chapter.

The level of water consumption is also related to a customer's life style which indirectly depends on the level of income. Even in this research it has tried to understand and compare water use for different group families who are better off use 3-4 times fold water since their capacity to means of fetching helps them to use more. Those who are poor may use 1-2 plastic jerrycan of water, 20 litter capacities. In addition, different study also presented similar justification for the water demand. Recently, water use for different groups and for different periods was compared. Households with average summer use of less than 25% of all households are classified as *Low* volume users, while those in the highest 25% comprise *High* volume users; the rest of the households are designated as *Med* (medium). Later, continued the previous relationship for price elasticity. Researcher found that given a 10% increase in price of residential water demand can be expected to 6% decrease in consumption. But evaluation of pricing system for water efficiently is not possible unless all customers are metered. Kenney et al. (2008) cited by (Mahdi, 2008)

Pressure management can be an efficient tool for demand management. The water pressure in the pipe, reservoir, and finally at the tap should be neither too low nor too high. Low pressure may cause unwanted flow reductions and conversely, high pressure may increase leakage and causes faucets to leak, valves seats to wear out quickly. As a result, the Uniform Plumbing Code requires that pressure not exceed 56m head at service connections, unless the service is provided with a pressure-reducing device.

For the hydraulic calculations used for the sizing of a gravity fed system, we always measure the pressure in m WG. We must differentiate the static pressure from the dynamic pressure:

- ♣ The static pressure is the force exerted by water on the pipes walls when all taps are turned off (water does not circulate in the pipes),
- ♣ The dynamic pressure is the force exerted by water on the pipe walls when 1 or several taps are open (water circulates in the pipeline).

The static pressure is the maximum pressure which can exist in the pipes. It allows determining the pressure to which the pipe must resist, as well as the need to install pressure breaking devices to protect the pipe. The pipes used for gravity fed systems are resistant to a certain pressure; call Nominal Pressure (NP): if the pressure in the pipe is higher than this NP, there is a risk of rupture. The range of pipes nominal pressure generally used for the gravity fed system are given in table below:

Table 2: Pipes Pressure Level NP (Source: (ACF, 2008))

Pipe type	Nominal Pressure	Maximum Pressure (P_{static})
Plastic pipe (PVC or PE)	NP 6	60 meters
	NP 10	100 meters
	NP 16	160 meters
Galvanized Iron (GI)	NP 16	160 meters
	NP 25	250 meters

The dynamic pressure is the force which water exerts in pipes when water flows, i.e. when the taps are open, and pipes are full of water. The dynamic pressure is lower than the static pressure because when water circulates in pipes, it loses energy. Indeed, pressure losses due to the frictions of water against the pipe's walls can be observed when water circulates in the pipe. These losses of pressure are called "head losses" (ACF, 2008). In both cases demand may fluctuate and the performance of the system altered either by providing less water to the customers or breaking the pipe and creating loss which totally imbalances demand and supply.

Generally, water demand expectation can be considered from point of views of the following issues: -

1. Preparation of a water demand forecast based on demographic trends, historical water use, economic indicators, and climate conditions;
2. Supply-side planning by considering safe yields of existing supplies, and if inadequate for future needs, location of alternative supplies to meet all or part of future needs;
3. Demand-side planning which identifies additional water conservation measures and wastewater recycling to reduce demand, and quantifies their costs and savings;
4. Carry out a supply reliability evaluation which examines the probability of a supply shortage in comparison with the short-term feasible demand reductions;

5. Come up with resource strategies that combine new supply development with demand reduction alternatives into a manageable number of combinations. The strategies should ideally take account of the water quality, economic considerations, environmental impacts, and the utility policies and goals, including financial objectives; and monitoring evaluation to keep the process updated.

Since the expectation of water demand vary from place to place, time to time it is important to produce a certain criterion to achieve the minimum requirement expectation of the community.

2.1.10. Pressure

Optimum pressure is one of the major criterion to be achieved in water distribution system; so as per the objective of the research the system must be decides either hydraulically efficient or not. If it holds true, the water available to each demand category even during high withdrawal period will be confirmed. At the same time leakage maintained minimum and almost no breakage across the system. These two cases maintained by setting minimum pressure at junctions and differently achieved maximum allowable pressure to keep the system free of breakage in pipes.

According to (Sharma, 2008) the minimum design nodal pressures are prescribed to discharge design flows onto the properties. It is based on population served, types of dwellings in the area, and firefighting requirements. The general consideration is that the water should reach up to the user point sufficiently. With these considerations, various codes recommend minimum ranging from 8 m to 20 m for residential areas.

Similarly, (M. Johnson, et al., 2009) recommended; cited by (Dereje, n.d.)

1. Minimum pressures at peak hour demand: sufficient to serve the highest supply point in the network. Typically, a mains pressure of not less than 15 to 20 m would be required to serve up to three stories building. However, in this case it is enough to have 7-15m of water column otherwise in case of some of water point’s high queue and hassle may be occurred.

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

2.1.11. Water loss in distribution network

Water losses occur in all water distribution networks, since the connection will not be perfect regarding to the workmanship, material quality, etc. Due to this reason, an amount that reaches to 10% of the total production is allowed. Thereby, the volume of this losses reflects the level of service of the system. (Dighade, et al., 2014) cited by (Benyam, 2016). In general, 'water losses consist of real and apparent losses. And to most water utilities, the level of Non-Revenue Water (NRW) is a key performance indicator of hydraulic efficiency. Utility managers should use the water balance to calculate each component and determine where water losses are occurring. By quantifying NRW from the water balance concept, volumes of lost water from the system can be calculated. Therefore, the water balance can guide water loss estimation in the distribution system while also indicating the level of accuracy of the Non- Revenue Water calculation. Non-revenue water (NRW) is the total amount of water losses in the system from the water source to the outlet meter of the customers and it consists of real loss and apparent losses. Thus, it is described as the difference of total amount of water production and authorized consumption. (Farley, et al., 2008)

Losses are categorized depending on the places where it lost

- Physical/Real losses
- Apparent losses
- Unbilled authorized consumption
- Billed authorized consumption (Farley, et al., 2008)

2.1.12. Criteria for Performance Evaluation

The performance of the system is measured based on its ability of the system to deliver good quality and quantity of water at all the times under suitable set of operating conditions. This performance depends on several criteria. Planning of these systems are very important and the factors that need to be considered are as follows: (BHADBHADE, 2004)

- Design life, reasonable design period of water supply system considering to the satisfaction of the level of service should be maintained even at the end of the life time of the system. The determination of the life time of the components should consider the expect population forecast, development plan of the area, the deterioration time of materials used, etc.

- Appropriate advantages of topographic features to reduce energy costs
- Projected population growth
- Projected industrial and commercial growth
- Water consumption data: average daily consumption, per capita consumption and
- peak flow factors
- Minimum and maximum acceptable pressures.
- Storage facilities (K. Swamee and Sharma, 2008)

A water supply system to be satisfactory in its level of service the criterion used above seems a bit complex but essential and not to missed for the design and evaluation. The Ministry of Water, Irrigation and Electricity of Ethiopia in this regard has issued national guidelines for public drinking water system. As per this guideline, a community is being supplied with reliable source should be minimum in cost and consider the criterion like the above. Also, “if a village, town or city is supplied water through a source, then the capacity of the source must be equal to or greater than the design maximum day demand and the design average day demand. In case of failure of portion of system there should have optional means of providing water during the peak periods in the day maintaining optimum pressures.

In addition, storage tanks should be able to provide enough storage facility to meet the regular average daily demands satisfying peak hourly periods but most importantly fire flow demands at a key location peak hours, however in this research no fire flow demand considered.

Generally, the peak hourly flow factors are depending on the economical capacity of the area that is 1.5-2 times the average daily flows. Also, the maximum design variation in the storage levels should not vary more than 10m to maintain the required pressures. In case the distribution system does not provide fire protection, then it should have storage capacity of 24 hours and must be able to maintain a pressure of at least 18m throughout the distribution system.

As per the Insurance Services Office (ISO), towns having fire class greater than 8 should be able to provide a flow of 20 lit/sec at peak daily demand at a pressure of at least 14m (ISO Mitigation Online, 2009). “Dead ends should be minimized by looping them to the main network system” (ODEQ, 2009). A hydrant or a flushing device should be preferably installed at dead ends to not have issues of water contamination due to stagnation (ODEQ, 2009). Main water lines should be at least 6 inches in diameter, and the least diameter of the pipe in the system should be 50mm. The design velocities in the pipes can range from 1 to 2 m/s (Salvato, 1992) cited by (BHADBHADE, 2004)

Table 3: Water supply standard as per GTP-2 Standard

Description of Indicators	GTP-2
Average water use for any rural household	25lit/cap/day
The maximum distance from household to water point	1000mm
Queuing time at a water tap	30-60minutes
Time taking to fill 20 lit jerrycan container	3minutes
Maximum number of people at the tap	250 people
The minimum flow on the water tap	7.5lit/min

2.1.13. Network Synthesis

With the advent of fast digital computers, conventional methods of water distribution network design have been replaced with different model which are used to simulate the system. The conventional design practice in vogue is to analyze the water distribution system assuming the pipe diameters and the input heads and obtain the nodal pressure heads and the pipe link discharges and average velocities.

The nodal pressure heads are checked against the maximum and minimum allowable pressure heads. The average pipe link velocities are checked against maximum allowable average velocity. The pipe diameters and the input heads are revised several times to ensure that the nodal pressure heads and the average pipe velocities do not cross the allowable limits. Such a design is a feasible design satisfying the functional and safety requirements. Providing a solution merely satisfying the functional and safety requirements is not enough. The cost must be reduced to a minimum consistent with functional and safety requirements and reliability considerations.

The main objective of the synthesis of a pipe network is to estimate design variables like pipe diameters and pumping heads by minimizing total system cost subject to several constraints.

These constraints can be divided into safety and system constraints. The safety constraints include criteria about minimum pipe size, minimum and maximum terminal pressure heads, and maximum allowable velocity.

Many researchers applied different way of approaches to conduct design, assessment and evaluation to water supply system using different software. One of them was (Muranho, et al., 2013) who explore technical Performance Indexes (TPIs) to assess the operational performance of Water Distribution Networks (WDNs) using EPANET model for Water Distribution Network (WDN) synthetic models, do pipe sizing, compute technical performance indicators, and allowed demand-driven and pressure-driven simulations. Finally, they have presented new performance evaluation tools and concluded that the traditional TPIs based on state variables have been implemented in EPANET software.

2.1.14. Need for hydraulic modeling

In most of the cases, rural water supply systems have poor data and recording system comparing to towns of which their system is designed using consultant firms. Similarly, from the experience observed design of rural water supply systems is conducted manually with almost on recording and controlling systems. In most of the time data and design documents are found on the hands of individuals who are personally interested. And, most rural communities do not have information regarding their network system. In such cases, when one wants to assess the performance of the system and the structure in the distribution system, it is advantageous to use user friendly computer models to connect it with data base. Computer models make use of hydraulic simulation software which imitating the real-time system behavior and predicting the performance of the future using ‘what if’ scenarios. (Haested, 2003). In the rural area like the one suggested to the research hydraulic simulation models are useful to provide decision support in operation, maintenance and management of the systems.

Chapter Three

3.1. Methods and Materials

3.1.1. Profile of the study area

Humbo woreda is one of the 12 woredas in Wolayta zone and it is located south of the town Soddo 20 km of asphalt road, 380km from the capital Addis Ababa. Some part of the woreda belongs to the rift valley and the right escarpment of the valley passed through it. From the woreda town, Tebela to kebeles there is no serious accessibility problem but most of the roads are dry weather roads and few of them are paved road, constructed by the project URAP. The woreda comprised 36 PAs.

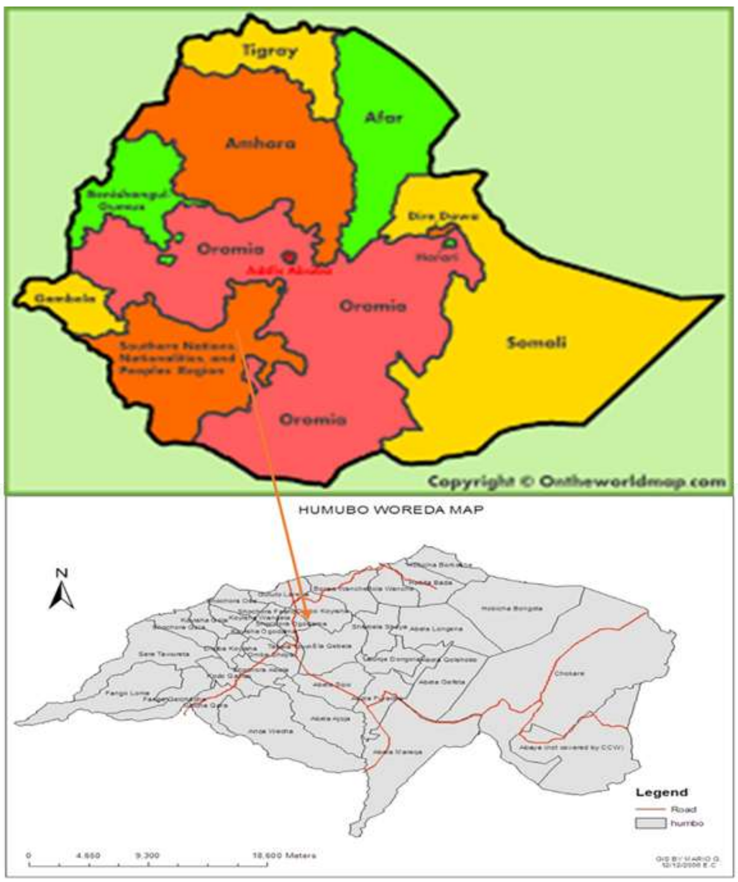

Figure 6: Location Map of the woreda

The woreda is bordered on the southeast by lake Abaya which separates it from Oromiya region, on the south by Gamo Gofa zone, on the west by Offa on the north-west direction by Soddo Zuria, on the north-east by Damot Woyde, and on the east by Belate river which separates it from Sidama zone. It falls within coordinates of 6^{o}44.898'-7^{o}11.32'N and 37^{o}20"-37^{o}58'00.87"E; and the total area of the woreda is 859.21 km^2, which is largest woreda in the zone with 35 rural Kebeles and one rural town. Topography of the area comprises different topographic terrains that vary from hilly terrains and rugged to pen planar surfaces ranging from below 1250meters above sea level in the South-East part to meters above sea level 1700 in the North-West part.

3.1.2. Existing Water Supply Situation of the woreda

From the introduction and geographical description, the woreda is in the Rift Valley, just besides to one of the greatest of lakes of Rift valley, Lake Abaya. Comparing to other woredas in Wolayta zone it covers 859.21 km^2. The woreda is divided into tropical and subtropical climatic zone. This climatic condition is one of the determinants to be considered in the water sector with high water demanding. In addition, because of the global weather condition change the area is prone to recurrent drought and the resulting water sacristy.

The available types of water sources in the woreda are deep and shallow wells drilled by machine, small and medium yield of springs, traditional hand dug wells and some roof water catchment structures. These are resources managed by woreda water, mines and irrigation office. According to inventory conducted in 2015 in the zone the water coverage is reported 68%.

Table 4: Types of water supply and their functional status *(Wolayta Zone Water, 2015)*

Status of source	Source type						Total Sources	Total Wps of Likimse	Total Wps of other sources
	BH	SW	HD W	LS	MS	SS			
	12	**8**	**8**	**4**	**4**	**8**	**44**	**41**	**129**
Functional	**5**	**4**	**5**	**3**	**3**	**7**	**27**	**26**	**94**
Non-Functional	**7**	**4**	**3**	**1**	**1**	**1**	**17**	**15**	**35**

Compared to other woredas in the zone it has less number of water supply systems. The situation of potable water supply and efficiency of distribution structure in the PAs are remarkably low. However, the population size of the woreda is higher and falls on top third position.

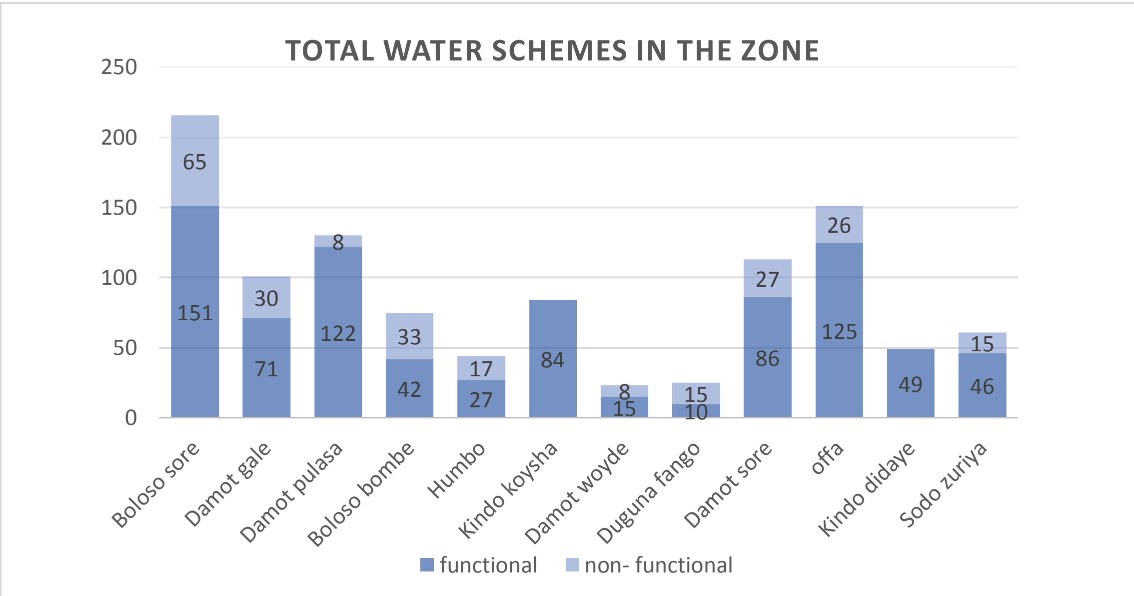

Figure 7: Functional and Nonfunctional schemes in the Zone (Wolayta Zone Water, 2015)

3.1.3. Existing situation of Likimse-Abela

Likimse-Abela is a gravitational water supply system established by World Vision in 1998 in Humbo woreda. The source spring is found in Soddo Zuria Woreda, Offa Sere kebeles. The specific location of the spring is at the toe of dissected moderate hill 'Damota, at a geographical position 6^0 48.37' E and 37^0 46.29'N having an elevation of 1744 m.a.s.l. The recharge area of the spring is large enough and found on higher altitude which gets abundant rainfall. The nearby surrounding area have good vegetation cover which facilitate favorable conditions for heavy infiltration and reduce evaporation which in turn are helpful for the occurrence of abundant ground water.

The source extended to Humbo woreda of 13 kebeles. Using 76.9km of both pressure main and distribution pipe line, two 150m^3 and one 50m^3 capacity masonry reservoirs, 41 water points, 10 cattle troughs but not working and 27 washing basins which are supplied manually using buckets or plastic jerrycan. The end tip of the pipe extends and supply water to 'Abela-Mareka' kebele at

1295 m.a.s.l of elevation. The system totally has 121m and 384m gravitational head difference between source and reservoir and reservoir to end tip of the distribution respectively. The source position has good head to drive water to end users, if handle properly with energy controlling device. (World Vision, 2001). In addition, the system was designed to be closed system but through time gate valve and taps were damaged and made the distribution system is open which flows free in all its 14 hours working time, even in times when there are no users.

In the woreda as observed on Table 4 above there are different type of water sources are available. By the drought response project (humanitarian assistance) in 1985, World Vision drilled boreholes in 11 PAs. These boreholes had been provided service to the community including to some of Pas under this study, but now sealed and replaced with water points from Likimse spring. This had happened just after 10 years of service time of boreholes because of going back of the community to their original village from settlement and some of them are abandoned dry up. The new turning point and the beginning of a big leap forward in the history, these 13PAs started getting service after the commissioned of the system in 1998 and replaced all the boreholes, found in the area by the water point from the spring.

The capacity of the source spring yield is 36lit/sec at a maximum dry season and 12lit/sec of it is diverted to Likimse-Abela water supply system and the remaining was left flowing to tribute the river Hamessa, but currently, it is observed that the overflow portion of the yield of the spring is taken to be included in the town water supply system. The amount planned to be taken is 20 lit/sec. (Mott MacDonald in association with AG, n.d.)

To visualize the capacity of the spring the daily water production is 3110.4m^3 which is huge amount to serve 24,883 families, taking ideally no loss and the distribution system is 100% efficient. In the system, no observed treatment unit installed, but the reservoir attendant told that there is frequent washing and flushing of reservoir.

The existing water tariff of the area is a flat rate system decided by the community, which is 60ETB per House hold Per Year. This is low water fee and is recognized as uncontrolled water supply system.

Status of the source with distribution is of course functioning but embraces different problems such as, breakage of fittings and taps, leakage losses, uneven distribution, demand and supply imbalance and system over stress of expansion.

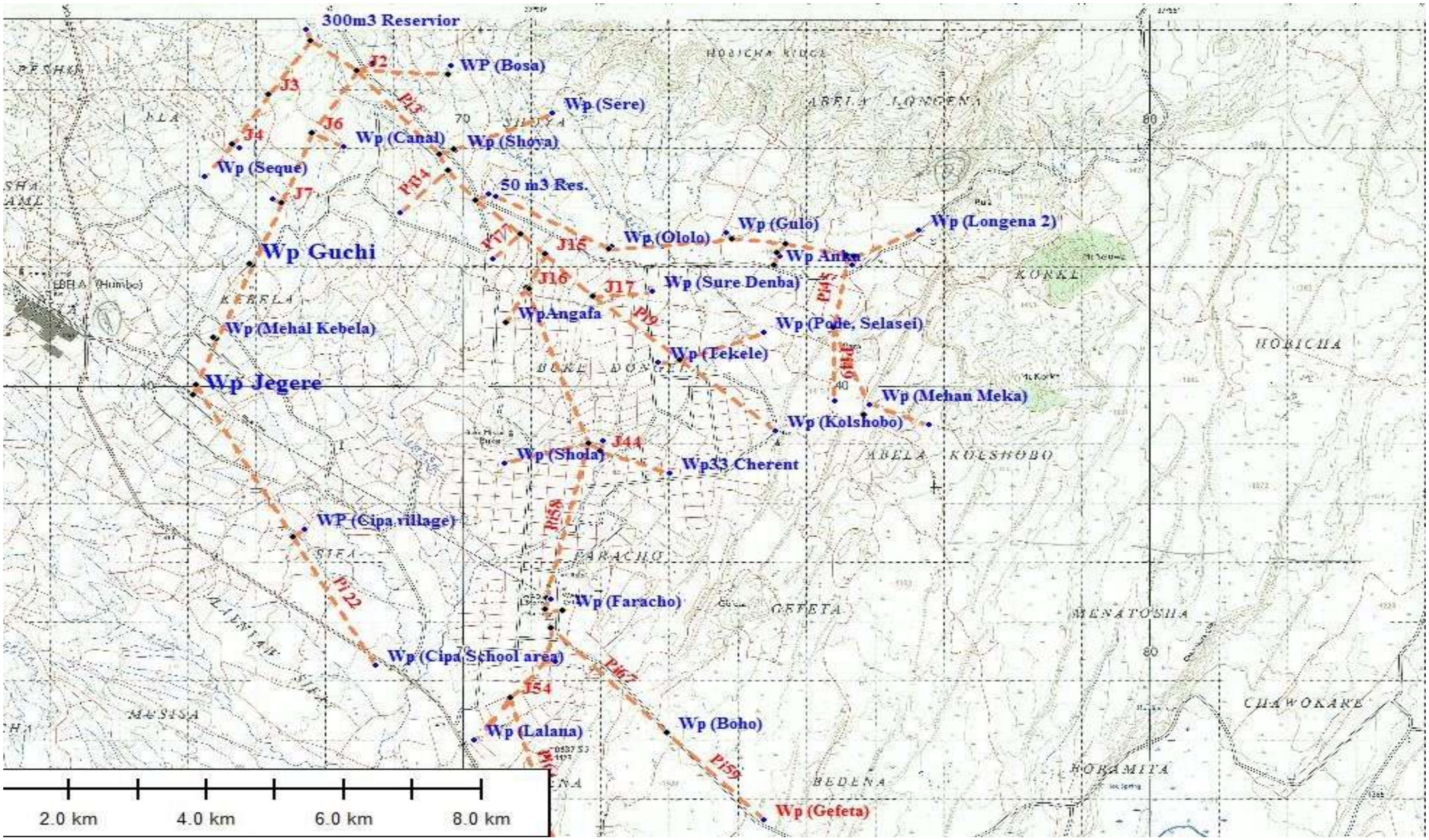

Figure 8: Distribution pipeline over laid on contour map (Source: Own field survey)

3.1.4. Water supply coverage

3.1.4.1. Mode of connection

Likimse-Abela Water supply system is managed by the Water Board established by the committees of each of kebeles and based at Humbo town under the Woreda Water, irrigation and Energy Office. The system serves 13 PAs holding using distribution mode of connection shared, public fountain considered equal amount of water to points to be distributed corresponding to population exist in the area.

3.1.4.2. Population distribution

Table 5: Existing Points flow rate, coordinate and No. of household *(Source: field survey)*

It. No	Description	North	East	Elevation	User Population (HH)	Avg. Discharge (lit/sec) on points
1	Wp5 (Jegere sefer)	6.6924	37.7884	1500	105	0.05
2	Wp6 (Mehal Kebela)	6.7011	37.7911	1542	50	0.102
3	Wp7(Guchia)	6.7125	37.7958	1574	100	0.08
4	Wp17 (Shola)	6.6820	37.8295	1397	170	0.161
5	Wp2(Mehal Ela)	6.7223	37.7988	1597	126	0.08
6	Wp10 (Tawla sefer)	6.6613	37.8357	1402	195	0.083
7	Wp (Abela Faracho)	6.6593	37.8371	1401	160	0.05
8	WP (Angafa)	6.7035	37.8297	1479	75	0.5
9	Wp4 (Seque)	6.7257	37.7899	1622	75	0.09
10	Wp19 (lalana)	6.6397	37.8255	1376	144	0.061
11	Wp20 (Olafino)	6.7202	37.8157	1461	139	0.5
12	Wp18 (Kulia)	6.6516	37.8363	1385	120	0.333
13	Wp16 (Zewde)	6.6854	37.8425	1408	120	0.156
14	Wp21 (Edget)	6.7132	37.8279	1502	110	0.5
15	Wp1 (Zekaras Zasa)	6.7302	37.7945	1615	120	0.05

It. No	Description	North	East	Elevation	User Population (HH)	Avg. Discharge (lit/sec) on points
16	Wp (canal)	6.7303	37.8082	1601	250	2
17	Wp24 (Shoya)	6.7228	37.8282	1504	100	0.1
18	Wp10 (Cherenet)	6.6806	37.8513	1412	180	0.192
19	Wp15 (Tekele)	6.6975	37.8496	1419	189	0.555
20	Wp23 (Shoya)	6.7299	37.8228	1520	80	0.3
21	Wp26 (Ololo)	6.7153	37.8436	1437	120	2.333
22	Wp15 (Sura Denba)	6.7083	37.8489	1443	158	0.444
23	Wp32 (Zaro)	6.7431	37.8119	1529	297	0
24	Wp33 (Bosa)	6.7427	37.8227	1562	299	0.6
25	Wp25 (Sere)	6.7355	37.8356	1513	140	0.0832
26	Wp27(Gulo)	6.7174	37.8586	1438	187	0.111
27	Wp28 (Abela Zegre)	6.7137	37.8657	1437	162	1.8181
28	Wp31Pode, Selase	6.7021	37.8635	1425	200	0.833
29	Wp29 Longena	6.7124	37.8752	1443	80	0.5
30	Wp30 Longena	6.7178	37.8839	1447	80	0
31	Wp12 Kolshobo	6.6871	37.8651	1432	140	0.46
32	Wp13 (Girara)	6.6916	37.8729	1419	100	0.1562
33	Wp14 (Mehanmeka)	6.691	37.8775	1407	210	0.167
34	Wp 35 (Abela Mareka)	6.6111	37.8398	1295	80	0
35	Wp (Abela Cipa)	6.651	37.8125	1380	143	0.15
36	Wp (Abela Gefeta)	6.6393	37.8516	1308	170	0
37	Wp Cipa (school sefer)	6.6719	37.8031	1400	160	0
38	Wp (Boho)	6.4577	37.8511	1317	210	0.55
39	Wp (Korke)	6.6896	37.8768	1398	143	0

It. No	Description	North	East	Elevation	User Population (HH)	Avg. Discharge (lit/sec) on points
40	Wp (Anka)	6.7121	37.8648	1431	234	0.66
41	WP (Gututo Larena)	6.76206	37.7829	1727	128	0.25
				Total	**6049**	**15.0585**

3.1.5. Criterion for the case site selection

The main reasons of selection of this site for this assessment is because it serves 13 PAs, which is holds 1/3 of the total population or covers almost ½ of the area of the woreda. The second reason is the system management of the water supply system is not easy comparing to other water supply systems since water trans-flow from woreda to woreda and covers much of kebeles. The existing population found from the Federal Democratic Republic of Ethiopia Central Statistical Agency, 2014-2017 and field survey is 30, 245. The management system is different for the rural and town water supply system. The last prioritized motivation is to assist stakeholder in planning and managing their infrastructure and service in a sustainable situation. Hence, due to the above reasons the system was found rational to be selected for this study.

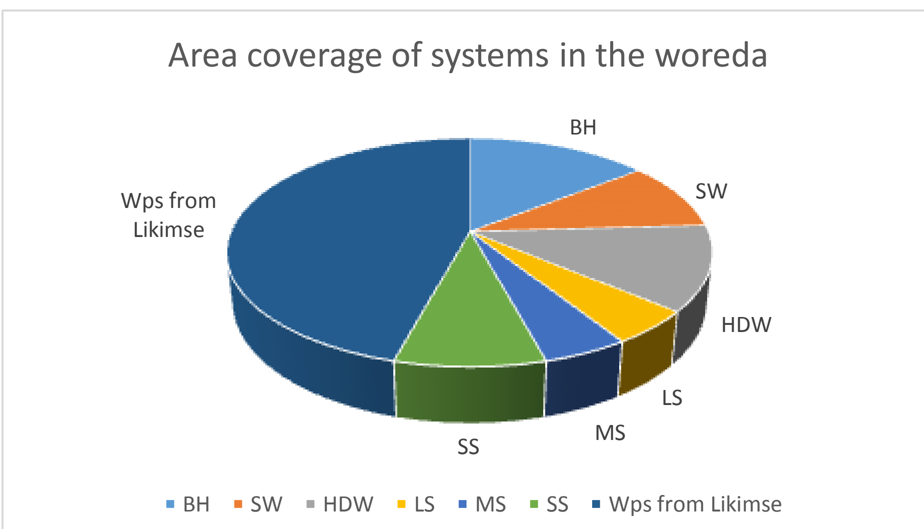

Figure 9: Area coverage of the water supply systems in the woreda

(Source: Inventory document)

3.1.5. Selection of hydraulic simulation software

In this study, EPANET-2 model was used because of its capability to conduct hydraulic simulation. It is a model that performs extended period simulation of hydraulic and water quality behavior within pressurized pipe networks. Because, this days the mathematical models have become increasingly accepted in the water industry as a mechanism for simulating the behavior of water distribution systems to facilitate decision-making on management, operation and maintenance. EPANET is an open-structured, public domain hydraulic and water quality model developed by the EPA, and is used worldwide (Muranho, et al., 2011). Beside this, the model is user friendly and easily accessible. While other hydraulic and water quality modeling software's such as: Water Cad, KY pipe, Info Water, H20Map Water and Hydraulic CAD are not accessible for the site at rural level. However, EPANET has gained wide acceptance in the world; for the merely fact that it provides quantitative basis for undertaking new modeling activities and assessment of existing water distribution system.

3.2. Materials

For this research, Topographic map of Soddo and Tebela, 1: 50,000 from the Ethiopian Mapping Agency was used to detect the exact location of water supply system utilities and develop system map using data collected by GPS and tape meter. picture of the existing water distribution system and service situation. V-notch, stop watch, plastic jerrycan of 3, 5, and 20 litter capacity were used to measure the required date for the model and generally for the assessment. In addition, water meter, pressure gagging, recording papers were parts of the materials used to collect data.

3.2.2. Data Sources

The source of data was involved both primary and secondary data. For the study, the primary data were obtained from pressure reading, elevation surveying, field observation, wealth ranking and focus group discussion with community and water committee to obtain additional relevant information on the subject matter. While, secondary data were collected from different literature reviews, design report, annual and inventory reports of the water supply system.

3.3. Methods

3.3.1. Data collection methods

The performance to which the infrastructure provides the service to meet the satisfaction of community on the water supply system was assessed using field observation, measurement, Focus Group Discussion (FGD), Wealth ranking, KII and data review from different offices. The above methods used to assess the situation of the system, the degree of satisfaction and involvement of the community and institutions support and responsibility in any phase of the system.

To understand the realities of the situation of water supply system field observation together with measurement were conducted. Meanwhile informal discussion with different part of the community was made to get direct information on the system situation. During this time, it was observed the functionality of system, distribution situation, the part of system leaking, part of system with break, junction places where expansion did to some villages, the amount of water taken by the household, the time they spent on fetching, the queue and the distance traveling. The time taken to observe the whole system is ten days, from March 1-10, 2017. The above field observation was conducted together with the woreda water office's expertise and local language translator.

3.3.1.1. Water source

Likimse-Abela water supply is a system provided water from Offa Sere spring source found in Soddo woreda at about 8km for the reservoir used as night storage. The spring has a total capacity of 36lit/sec yield leaving some water to percolate and joined to the river. But, for the case system the source supply only 12lit/sec of water for the community found in 13PAs in Humbo Woreda on table 9. However, the case source has no any recording device installed that tells the amount of flow produced to the system. So, the above-mentioned yield to the system is measured using calibrated v-notch on suitable places on the channel to the river Hamessa. The total water produced from the source is 3110.4m^3 per day. This is cross checked on field measurement for this research purpose and on the design document prepared both Likimse-Abela water system and for Soddo town water supply expansion project from the same source (Mott MacDonald in association with AG, n.d.), *15 towns Water Supply Project*.

The above daily average production of the source is enough to serve 124,416 people using the GTP 2 standard, 25lit/day/capita.

Figure 10: Source Likimse-Abela Spring

3.3.1.2. Distribution Pipes

Water distribution systems are often susceptible to failure events, mainly due to component malfunctions. As explained on the literature part above, pipes are materials that contribute much portion of the loss unless controlled and considered as central part of the system, which estimated to reach to 60% of the total cost of the system. In the system pipes could be exist in different type and size which conducting fluid from one point to another in the network or system. In the model pipes are considered as link to connect nodes.

Likimse-Abela water supply system contains pipes of various diameters but one type, GI.

The distribution pipes of this system were observed categorized in to primary and secondary lines. Primary lines are main lines enclose multiple connections along its length. Secondary lines are lines of smaller diameter and supply water directly to the end users. For this study, primary lines of diameter vary from 150mm up to 80mm were included while secondary lines of lower diameters were concentrated to nearby nodes. A total of 76,905m of pipe extended and stretched in 13PAs.

Table 6: Pipe length and dimension of the water supply system

Item No	Pipe Diameter (mm)	Length(m) On the ground
1	150	2913
2	100	12575
3	80	8268
4	63	7578
5	50	7578
6	40	23611
7	25	18591
	Total	**80394**

In terms of material used in the system the whole pipes are GI pipe confirming to be ISO-65-1973(E) specification a length of each 6meter and hydraulic test pressure 50 bar (10^6 N/m^2

3.3.1.3. Service Reservoirs

In Likimse-Abela water supply system currently has two 150m^3 capacity of trapezoidal concrete reservoirs and one 50m^3 of circular masonry reservoir for night storage and service reservoir. The second reservoir was constructed in 2010/11 by World Vision for reserving water for irrigation purpose to create income generation mechanisms to groups. However, because of hassle created by the downstream users currently it has become part of the system.

The existing reservoir designed using 1/3 of maximum daily demand. (Source: M0WR 2006). This was also determined using population data taken by the designer, the population that has forecasted for 15 years, 40,510 people. Due to this the capacity of the reservoir was estimated 300 m^3. Water is driven directly to these reservoirs using gravitational force thus the community in all 13 PAs located at the downstream of the reservoirs is severed using similar driving force except, one water point constructed between the source and reservoir (Gututo Larena Water point).

Figure 11: Reservoirs in the distribution system (Source: Own filed visit Photo)

3.3.1.4. Power supply units

The power use to drive the whole system is gravity force. The water supply system is operated for 14 hours during the day time but with no stand by power or loop provided for the contingency. If failure happen on the upper stream the system ceases functioning to the rest of portion on the downstream.

3.3.2. Elevation data

Setting elevation is one of the significant requirements to simulate the hydraulic characteristics of water in distribution system. For this study, the elevation data of utilities were obtained by field record using Global Positioning System (GPS) to cross check the existing utilities position on the design. In appendix-G, all the assigned node elevation data were listed with coordination system.

3.3.3. Base Water Demand data

To estimate the daily water consumption of the user community the population data of the 13 kebeles was collected from each of the kebele administration offices. In addition, since the establishment of the system no observed influencing factors occurred in the area that affect the population distribution, size and growth rate. Most of the factors influencing the population forecast are war, natural disasters, developmental transformation, etc. The population data collected was also cross checked with data found from Federal Democratic Republic of Ethiopia Central Statistical Agency, 2014-2017 to increase the precision of data. But the population data on the design document was observed have difference compared to the current population found in both situation.

Topographic map found from Ethiopian Mapping Authority, with the scale of 1: 50, 000 and twenty-meter contour interval is used to draw the existing utilities using Global Mapper shown on Figure 8. Therefore, by the help of the map drawn and population model is developed by the EPANET-2 software.

The level of water consumed for domestic purpose has been aggregated to analysis the distribution of the water coverage among different localities. Evaluating the domestic water supply coverage using volume of consumption may allow realizing the distribution comparison among the villages. For this reason, wealth ranking on three sample kebeles were conducted using focus group discussion. To achieve one of the criterion of hydraulic performance of providing minimum required water for the area it must be analyzed and compare with the GTP-2 and other standards, 25 lit/cap/day. The samples were selected to represent the geographical position of water points in the system. The kebeles were Ampo, Buke Dongola and Faracho. The focus group discussion held on March 12, 2017 by discussing with community representative, members of water committee, and kebeles staffs. This is because all over the world it has vividly indicated that volume of household's water consumption is influenced by the families' economic status. In this research, the economic status mainly manifested by annual income and possession of assets like land, domestic animals, and water containers holdings at house level. So, the respondents were asked to estimate their holdings and income raised both from agricultural and non-agricultural sources.

Table 7: Wealth ranking criteria of Ampo community

Better Off	Medium	Poor
3 and above oxen	Only 2 oxen	1 and below ox
2 and above cows for milk	1 Cow	0-1 Cow
2 and above donkey	1 Donkey	No donkey at all
10 and above sheep	5-7 sheep	0-4 sheep
3 and above ‘Timad’ of land	2-3 ‘Timad’ of land	1-1/2 ‘Timad’ of land
50 and above foot of eucalyptus	15 foot of eucalyptus trees	12 and below foot eucalyptus

Table 8: Wealth ranking criteria of Buke Dongola community

Better Off	Medium	Poor
2 and above oxen	1Donkey	No oxen and donkey
1 and above cows for milk	No Cow	No Cow
2 and above donkey	1 Donkey	No donkey at all
7 and above sheep	5-7 sheep	1 and below sheep
4-3 Timad of land	2-3 Timad of land	1-1/2 Timad of land
1000 birr and above of cash crop	500-1000 birr of cash crop	0-400 birr of cash crop

Table 9: Wealth ranking criteria of Abela Faracho (Source: computation based on survey)

Better Off	Medium	Poor
2 and above oxen	Only 1 ox	No ox
2 and above cows for milk	1 Cow	0-1 Cow
2 and above donkey	1 Donkey	No donkey at all
10 and above sheep or goats	4-8 sheep or goats	2 sheep or goats
8 and above ‘Timad’ of land	4 ‘Timad’ of land	2 ‘Timad’ of land
2 corrugated houses and above	1 corrugated house	Hut

To know the number and status of user households and conduct wealth ranking of water treasure book for fee collection was used. In sampling the participant for the wealth ranking geographical position of villages of the following number of households were used to respective kebeles 810, 700 and 670 from the total of 6049 houses. This was done by sample size determination technique. Among different methods, the one which has been developed by Carvalho (1984), as cited by Zelalem (2005), cited by Wonduante was used (see Table 11 below). Hence regarding Carvalho (1984) the sample size of this research is 2180 HHs and population size is 6049 which is feasible and rational to accept it for the research.

Table 10: Sample size determination (Source: Carvalho, 1984)

Population size	Sample size		
	Low	Medium	High
51-90	5	13	20
91-150	8	20	32
151-280	13	32	50
281-500	20	50	80
501-1200	32	80	125
1201-3200	50	125	200
3201-10000	80	200	315
10001-35000	125	315	500
35001-150000	200	500	800

Ranking of the community was conducted by calling to the name of head of the family to discuss and categorize. On the focus group discussion, the community identified as Better-off families use donkey, cart, and labor fee mechanism to fetch water. Even in a places or time where or when there is no water they travel long distance to fulfill their consumption. Comparing to the poor itis much more different and poor use labor intensive mechanism to fetch water especially using women and children. The number of available container they have for watering purpose is also vary accordingly. To visualize the situation the materials used to fetch water in the area are 20lit plastic jerrycan, 10lit plastic jerrycan, 5lit plastic jerrycan and 3lit jerrycan.

As a result, the following ranking was identified, 11% of the user households as 'Better-Off', 34% as 'medium' households and 55% as 'poor' households, based on criterion set by Ampo the community (see Table 8). For Buke Dongola PA it was identified 27% of households as 'Better-Off', 33% as 'medium' and 40% as 'Poor' based on criterion set by the community (see Table 9). Through the wealth ranking exercise to 670 households used in the design of the system for Abela Faracho kebeles 14% of the households were ranked by the community as 'Better-Off' and 26% as 'Medium' and 60% as 'Poor' again using the ranking criterion used by the community on Table 10. For the ranking purpose the average number of persons per household was taken 5.

The other important situation observed on the field was some of the water points did not give service and some of them provide below and above the standard flow rate on the point. Hence the per capita demand of families is totally altered and people in the area are forced to travel to the nearby water points or scoop holes and other unprotected sources. The variance of water in the distribution ranges from excess to zero flow rates.

In addition to the above observation using focus group discussion household demand per day was estimated corresponding to the rank done above. The result shown below on the table

Table 11: Consumption per day to wealth categories (Source: computation based on survey)

Name of water points	Better -Off	Medium	Poor	Reliability
Sure Denba	5 by 20lit Jerrycan per day	3 by 20 lit Jerrycan per day	1 by 20 lit Jerrycan and 2-3 by 5 lit Jerrycan	12 O'clock -2 O'clock
Buke Dungola/ Tekele	8-9 by 20 lit Jerrycan per day	6-7 by 20 lit Jerrycan per day	1-2 by 20 lit Jerrycan per day	1 O'clock -1 O'clock
Buke Dungola/ Zewde	8 by 20 lit Jerrycan per day	5 by 20 lit Jerrycan per day	1-2 by 20 lit Jerrycan per day	1 O'clock -1 O'clock
Jegere sefer	2 by 20 lit Jerrycan per day	2 by 20 lit Jerrycan per day	2 by 20 lit Jerrycan per day	2 O'clock- 12 O'clock
Ella	6 by 20 lit Jerrycan per day	4 by 20lit Jerrycan per day	2 by 20 lit Jerrycan per day	1 O'clock -1 O'clock
Mahena Meka	1 by 20 lit Jerrycan per day	1 by 20 lit Jerrycan per day	1 by 20 lit Jerrycan and 2-3 by 5 lit Jerrycan	2 O'clock-10 O'clock
Mehal Ella	5 by 20lit Jerrycan per day	3 by 20 lit Jerrycan per day	2 by 20 lit Jerrycan per day	3 O'clock-7 O'clock
Sheqe	6 by 20 lit Jerrycan per day	4 by 20lit Jerrycan per day	2 by 20 lit Jerrycan per day	1 O'clock - 1 O'clock
Abela Faracho	42 by 20 lit Jerrycan for all the community	42 by 20 lit Jerrycan for all the community	40 by 20 lit Jerrycan for all the community	3 O'clock-8 O'clock
Abela Zegre	6 by 20 lit Jerrycan per day	4By 20 lit Jerrycan per day	2 by 20 lit Jerrycan per day	12O'clock -2 O'clock
Gulo	3 by 20 lit Jerrycan per day	2-1 by 20 lit Jerrycan per day	1 by 20 lit Jerrycan per day	1 O'clock - 1 O'clock
Kulia	5-6 by 20 lit Jerrycan per day	2-3 by 20 lit Jerrycan per day	1-2 by 20 lit Jerrycan per day	3 O'clock-2 O'clock

3.3.4. Demand multiplier factors

Though the peak demand determination is a probabilistic method for modeling, peak hour demand scenario was adopted. As per the standard set of the per capita demand for each supply node was applied by taken demand multiplier factors of 24-hour flow duration. However, in most of the cases engineers used their experience to determine the peak which may lead to failure on the system design by either over or under estimating. Therefore, for this study by considering the peak flow time, minimum flow condition and the actual condition of population served from the system; the demand multiplier factors were adopted data obtained from the regional water, energy and mineral bureau. Therefore, the proposed peak factor and patterns for demand multiplier factors were 1.2 for max demand and 1.8 for peak hourly demand. (Source: FDRE, MoWR)

3.3.5. Roughness coefficients for pipeline

In water supply system efficiency pipeline is the component contributes much directly related to frictional effect either with the capacity it can carry, smoothness of internal surface, age and material type of the pipe. In this case, Hazen-Williams developed equation considering these factors. Higher C-factors represent smoother pipes (with higher carrying capacities) and lower C-factors describe rougher pipes. The value of roughness coefficient, C-factor is depending on pipe materials and its age; this effect can be shown in table below Yan, 2006 cited by (Mansoor, 2007).

Table 12: Typical roughness coefficient of pipe for different type's material (Yan, 2006)

Pipe Material	Age in years								
	New	10	20	30	40	50	60	70	80
DI	140	130	130	120	120	120	110	100	-
PVC	150	140	140	140	140	140	130	-	-
HDPE	140	130	130	130	130	130	120	-	-
AC	150	130	130	120	120	120	100	-	-
PE	130	120	120	120	120	120	110		
PC/RCC	130	120	110	95	70	70	70	-	-
Steel/GI	150	130	130	100	100	100	60	60	60
CI	150	110	100	90	80	70	70	60	-

EPANET usually provides default roughness coefficient 150 however, for this case simulation the roughness from the chart below was adopted.

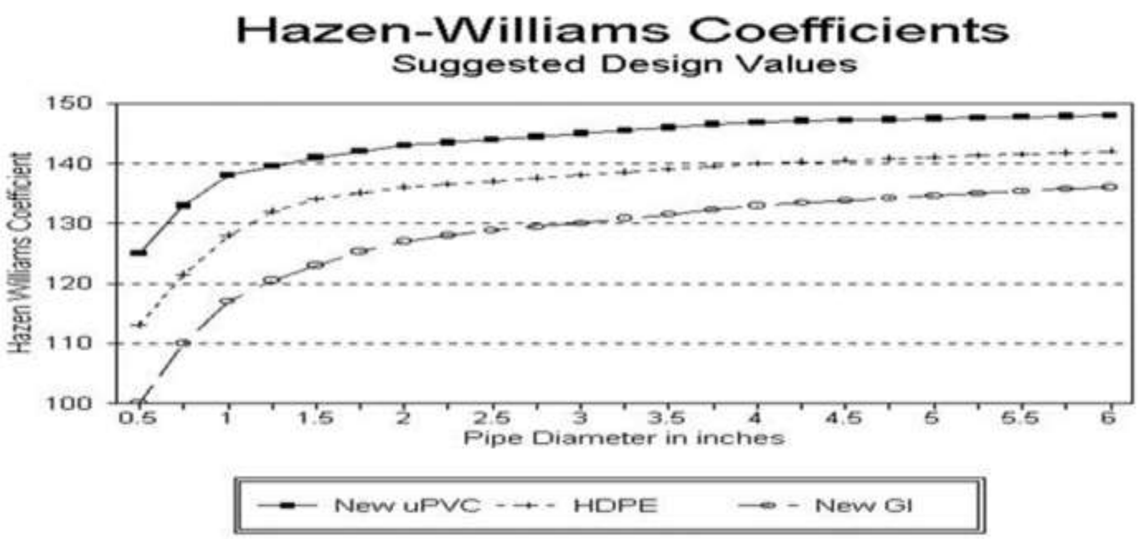

Figure 12: Hazen-Williams Roughness coefficient

3.3.6. Model Presentation

To build and simulate the hydraulic model, the EPANET-2 stand-alone, distribution modeling software was used. The water distribution network map has done as built by collecting geographical position data on the field combining with the design found from the hand of individuals. In this case, topographical map of scale 1: 50,000 with the help of Global mapper is used. In addition, for modeling system map was also done using the above information found from the field.

The network simulation was taken extended periods by consideration of hourly demand variation pattern over 14-hour flow duration analysis work only. Since at the start the system is intermittent system, not giving service for the night time. For this study, the network operational set-up was done by system international; SI unit and the project liquid were taken water at 20°c. The other model input was taken and carried out as mentioned below; In most of the cases models in the water supply system includes Reservoir usually we call it sources, Pipes, Tanks in some cases explained reservoirs, pump and valves are components that as much as possible the resulting sketch fairly represent the actual water network. In this case reservoirs, tanks and nodes considered as junction and pipe and valve are links.

Holding this the following assumption was taken into consideration to run simulation for assessment of hydraulic performance of the system: -

- The water distribution system was designed to cope mainly with the domestic demand and 20 lit/capita/day considering the future population growth.
- The average growth rate of the area is 2.7% (Federal Democratic Republic of Ethiopia Central Statistical Agency, 2014-2017)
- Average family size is 5 person/house. (Federal Democratic Republic of Ethiopia Central Statistical Agency, 2014-2017)
- Yield of spring is 36lit/sec, however only 12lit/sec is provided to Likimse-Abela water supply system, the rest of water is left to Soddo town water supply and environmental balancing flow to downstream.
- The demand of consumption has been approximated depend on the inhabitant around fair distance to water points.
- Since there is no available demand pattern in the area it is considered that similar flow of water during service time (14 hours).

- Similarly, because of unavailability of data peaking factor are 1.2 and 1.8 both for maximum and peak hourly demand respectively based on design guideline of Ministry of Water, Irrigation and Electricity of Ethiopia.

3.3.7. Model Calibration and Validation

To understand the credibility of a model it is simply to compare with model result precisely reflects measured field values. Thus, to have a confidence on model result it needs to calibrate and validate a model. An effort to perform hydraulic model calibration and validation for this case study is presented as follows. Similarly, our model EPANET also need to calibrate and validated to have confidence result. To conduct calibration data was entered a file and registered with the project.

3.3.7.1. Hydraulic Calibration

In this case pressure were measured along the main line. Off course the main challenge was to dig the junction and get appropriate place to measure, hence the pressure at nearest water points taken and drawn back to get measured result at the junction. According to Pearson the measured and simulated data was correlation below: -

As explained above the system is open system working in all 14hours service time so, it is difficult to get the maximum and minimum demand of the system. Due to this reason, the pressure is measured only one time at nodes selected by undermining the time since the discharge at points were taken similar throughout the service hours of the day.

To understand the measured and simulated pressure data Pearson formula was used as shown below: -

$$R^2 = \frac{\sum(X-\overline{X})(Y-\overline{Y})}{\sqrt{\sum(X-\overline{X})^2\,(Y-\overline{Y})^2}}$$

Where,

- R is Person correlation
- X: Measured pressure on the field
- Y: Computed pressure by the model
- $\bar{X}$: Mean value of X

  $\bar{Y}$: Mean value of Y

3.3.8. Method of Data Analysis

The data collected by applying the above methods interpreted using descriptive statistics, percentage, average, exact value and ordinal to analyze the findings by the help of Microsoft excel. Qualitative data collected from the user community on the water points, technical staff members and water committees using informal interviews and discussions was organized and changed to quantifiable result. The result analyzed displayed using tables and graphs in the way that are used for modeling the system and become criterion for the performance assessment of the system.

Based on the analyzed data to visualize the situation of the water supply system in different operation condition and give recommendation model was used. Now a day in different thematic area models are introduced to facilitate easy manipulation of sophisticated mathematical equations and formulas. With the progressive update of computers models increasingly accepted in the water industry too. In this case, EPANET-2 model is preferred as analyzing tool to understanding the water supply system situation in different operating condition.

3.3.9. Water Loss Assessment

As per the information provided on the problem statement part water loss is the biggest challenge observed in the system. In this regard losses were categorized in different ways like physical and administrative losses.

During site visit and data collection time in some places losses were observed. The major loss on the system was loss on tap of water points since the system open throughout it service hours in a day with malfunction taps on every of water points, which allowed much water to loss. The other observed losses were leakage through disconnection of joint, over flow on the night storage reservoir and leakage on the gate valves around the reservoir area and in the distribution network. Some of observed cases were deliberately and some were unintentionally. To understand the above losses, the system had been divided in two parts from the source to reservoir and from the reservoir to each distribution nodes.

Table 13: The IWA 'best practice' standard water balance (Source: IWA water loss task force)

System Input Volume	Authorised Consumption	Billed Authorised Consumption	Billed Metered Consumption	Revenue Water
			Billed Unmetered Consumption	
		Unbilled Authorised Consumption	Unbilled Metered Consumption	Non-Revenue Water
			Unbilled Unmetered Consumption	
	Water Losses	Commercial Losses	Unauthorised Consumption	
			Customer meter Inaccuracies and Data Handling Errors	
		Physical Losses	Leakage on Transmission and Distribution Mains	
			Leakage and Overflows from the Utilities Storage Tanks	
			Leakage on Service Connections up to the Customer Meter	

3.3.9.1. Loss from the source to reservoir

This section is pressure main section which has 8,282m length of pipe and connects wet-well and night flow storage reservoir. The flow in this section is gravitational flow that doesn't use pump to push because of the 134m height elevation difference. In addition, the type of pipe used is GI pipe of diameter 100mm and 50mm. During construction time, some portion of the recommended 150mm of diameter in the 4174m has changed to two parallel pipes of 100mm and 50mm. The reason for such modification was not explained in any of the portion of construction report to the authorized office and it is totally unknown. But the hypothesis to the change made by the designer is; the parallel line the made is thought equivalent to the recommended size. However, in hydraulic output it is totally different. In the key informant interview it was examined and found that some portion of the pipes buried deep up to 3-4m to comp onset the head loss exerted due to change otherwise the observed head loss comparing the two situations was estimated 13m and 60m.

The water point which is connected to the 2 inch of pipe along the pressure main works 24 hours in a day without controlling device like to that of other water points in the system. The researcher suggested similar 14 hours of service time per day of the system, so the remaining 10 hours of the flow will be lost without reaching to the other portion of the system. Hence, with the discharge measured on the spot of the tap, 0.25li/sec and the water that will be lost was calculated 9m^3/day. In addition, during fetching time on the tap for an individual 3minute standard was set however, an average time of getting and leaving the tap on reality took 6-7minute, it is doubled to the standard for change of the user. Taking a minimum estimated 2 minute of loss it was calculated as 40% of loss on the tap that is about 5.04m^3/day. The other loss observed was in the chamber before getting to the reservoir was measured using velocity-area method. As a result, at average of the reservoir 1.3lit/sec will be lost and joined river Hamessa. This was also calculated 112.32m^3/day of water. Totally, in this section 126.36m^3 of water every day will be lost.

3.3.9.2. Loss between the reservoir and end tips

The first loss observed in this section is over flow of the reservoir during the night time. This is because of the less capacity of the reservoir that had recommended on the design document. This is mentioned above on the description of the system, 300m^3. The water that come from the source was observed 10.54lit/sec deducting the loss in between. So, simply to see how much time required to fill the reservoir it is calculated 8 hours and the rest of 2 hours can be considered loss on the reservoir. This is calculated 75.888m^3/day.

On the field observation, it was also measured that the flow rate on the total 40 water points after reservoir is 14.8lit/sec, the current requirement is 12.33lit/sec, but the pick hourly demand used in the design document from the reservoir is 18lit/sec see Annex C. So, it is easy to observe that there is loss in the middle. At the same time, it is easily understandable that the amount released from the reservoir and the output on the taps is different with the amount of 3.19lit/sec. The variant presented is loss in the pipe connection and failure on the valves in the chamber of the reservoir. These losses were easily observed and estimated using the stop watch Vs bucket and velocity methods. As a result, 1.5lit/sec and 1.69lit/sec losses estimated both in pipe connection and valve chambers of the reservoir sum up respectively.

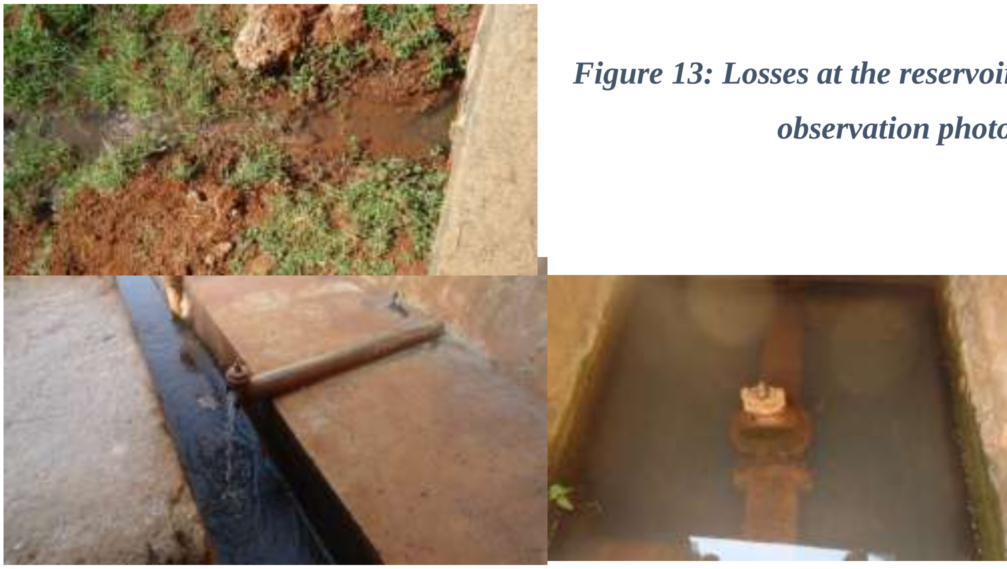

Figure 13: Losses at the reservoir chambers (Field observation photos)

According to data collected during field observation time taking for a person to fetch was recorded 6-7minute of time on some water points and 10-12minute on some water points which have flows. This was found because of hassle occurred due to low flow rate and high queue. That had created the inconvenience of in and out to the high flow rater on the water points and narrow mouth of fetching materials made the time to be extended. But for the calculation of loss on points an average of 5 minute has taken to the water point. This is because either with hassle occurred to get water or high pressure of flow rate. From this justification 2-minute loss is estimated on each of water fetching process. Hence the system was efficiently gives service for about 8.4 hours only. The rest of 5.6 hours of time spent without giving service, so, the annually water loss was estimated 95,570.9m^3 on 17 water points which have more flow rate.

In addition, unauthorized connecting of water points to the farm yard and illegal disconnecting joints for other purposes are considered as either the loss on the connection or joint failure loss. This is because to simplify the research and at the same time considering the case acceptable rather wasting the resource.

During field visit, it was also observed that the utilities do not have any recorded data related with average leak flow, number of reported bursts and average leak duration; due to these physical losses in the main was assessed base on the available data, and it was adopted by considering the minimum achievable annual physical losses (unavoidable annual real loss) in the system.

Chapter four

4.1. Result and Discussion

4.1.1. Estimated water demand

Estimating the expected water demand of the community in all of 13 kebeles were used for assessing and sizing system components such as reservoirs, and transmission and distribution pipe line.

4.1.2. Population forecasting

In water supply system designing an accurate population survey of the village is necessary, since the main objective of the system is to satisfy the user community with potable water throughout the life time of the system. This indirectly tells the size of system utilities required to distribute water to the satisfactory level and utilize to forecast future demand. In this regard, the current population data of user community of the case site collected from the Federal Democratic Republic of Ethiopia Central Statistical Agency, 2014-2017 and cross checked directly by counting from the treasure book of fee collection on each of water points is 30, 245. This is the existing user population number. However, the design engineer was forecasted much more which totally different from the reality on the ground.

As a result, the designer misleads to forecast the system life only for 15 years which is not economical comparing to the capital investment incurred.

The formula used to project the population for the given design period is geometrical progression forecasting method.

$$P_n = P_o(1 + r)^n$$

Where:

P_n = Estimated population figure

P_o = Base population figure

r = Growth rate and,

n = Number of year

The other data that utilized to calculate the design population was 3% growth rate, but again here the growth rate of the area is 2.7%. (Source: Federal Democratic Republic of Ethiopia Central Statistical Agency, 2014-2017 and Wolayta Zone water resource assessment report).

Therefore, regard to the above information; forecasted population is 39,478 for the design period up to 2027.

4.1.3. Per capital water consumption

To understand the per capita consumption of water in the case system various demand portion of water used for domestic purpose were taken aggregated. This aggregated amount of water depends on the level of development of the area, the weather condition, religion, living standard of the family, etc. However, in this case system it is only the living standard of the families were considered to determine the per capita water consumption of the community, taking the other situations area similar. To identify the economic status of the families as explained above on the methodology wealth ranking method was used on three sample kebeles. The kebeles were Ampo, Buke Dongola and Faracho. This is because all over the world it has vividly indicated that volume of household's water consumption is influenced by the families' economic status. In this situation, for the wealth ranking 5 persons of family size was considered. As a result, the average per capita demand of the area estimated 11.62lit/cap/day which is below the requirement set by of the national growth and transformation plan-2 (GTP-2) per capita demand, 25lit/cap/day with in 1km of distance.

Table 14: Daily Per Capita Demand of sample kebeles (Source: computation based on survey)

Ranking	Ampo		Buke Dongola		Abela Faracho	
	Popu.	Lit/capita/day	Popu.	Lit/capita/day	Popu.	Lit/capita/day
Better-Off	89	18.89	189	26.67	94	7.78
Medium	275	12.22	231	18.33	161	5
Poor	446	6.67	280	5	402	4
Avg. lit/c/d		12.6		16.66		5.6

Similarly, according to existing water supply design report; the average per capital water demand of those PAs at the end of the decided design period, 2013 adopted to be 20l/c/d. However, the result found on the ground on table 16 compared below design demand. Therefore, it was necessary to adjust the recent per capital water consumption as per the national growth and transformation plan-2 (GTP-2) requirement considering that the uneven distribution of system aggravated the situation more.

4.1.4. Average water demand

There are several mathematical methods of estimating the water demands of a given town; including extrapolating historical trends and correlating demand with the socio-economic variables of the town. But, the most common means of forecasting future water demand is estimating current per-capital water consumption, and multiply this by the projected population figure. Therefore, at the end of the design period, 2023 the average water demand for Likimse-Abela was calculated as;

$$Q = 37{,}430 * 25\ l/c/d = 935.744 m^3/d$$

4.1.5. Water distribution network analysis

In the modern water supply system, clear water shall be delivered to the service reservoirs directly through the transmission main and which is completely isolated from the distribution system. Similarly, the existing Likimse-Abela water supply which was constructed before 17 years ago take water directly from the service reservoir. So, the existing network configuration impact and the capacity of distribution system components were described as below.

4.1.6. Existing reservoirs capacity

The capacities of reservoirs in the water supply system were determined using different methods. The most appropriate and economical approach of determining storage volume of reservoir is the 24 hours' supply demand simulation mass curves. To develop such type of curves, it requires reliable recorded historical data of hourly water demand figures of the area. But, in the absence of such type of data, to determine the size of reservoirs, it was adopted the commonly practiced in many water supply systems and based on the rural water supply design criteria of the ministry of water resources; it was used for sizing the reservoir volume as one third of the maximum daily demand. Therefore, as per the design criteria of the FDRE; MoWIE, the maximum day factor usually varies between 1.0 and 1.3. Hence, a maximum day factor of 1.2 was adopted for assessing the maximum day water demand and reservoirs capacity for Likimse-Abela water supply system and applied. In the design document, similar factor was used.

Maximum day demand = 1.2 * average day demand

$= 1.2 * 935.744\ m_3/d = 1122.89 m^3/d$

Accordingly, the required reservoirs volume capacity for the user community was estimated as;(Source: M0WR 2006).

Reservoir capacity = maximum day demand *1/3

= 1122.89*1/3 = 374.30m3

Consider the consumption for cattle and other misalliance loss 15% additional water the clear water reservoir capacity becomes $450m^3$. But, in the existing water supply system of Likimse-Abela water system has less reservoir capacity, $300m^3$. So, because of this capacity it become factor for the overflow loss on the reservoir. Again, in here by using informal interview of the reservoir attendants the reservoir service pattern was tried to figure out on a one day schedule. This is based on the experience of reservoir attendants. The reservoir open at 6 O'clock in the morning and close at 8 O'clock in the evening. However, in the day time the reservoir will be finished at about 9 hours which means around 9:00-9:30 O'clock the reservoir continues working without reserving freely flow. During the night time the reservoir fill around 3:00 am at night and then overflow for the rest of time, about 3 hours up to it starts service. In this regard, it was clearly understood that the existing capacity of reservoir there is loss both before and after the reservoir. Otherwise, the reservoirs were found in good condition and able to serve for extra years.

4.1.7. Transmission main line

The pressure main of the system is cover a total of 8.28 km and with the length of this interval, the elevation difference between clear water tank and service reservoir is 121m. The composition of pipe on the design document was 4174m of 150mm and 4108m of 100mm GI pipe but, on the ground the installed pipe in terms of 150mm diameter has changed to two parallel pipes of 100mm and 50mm. The reason for such modification was not explained in any of the portion of construction report to the authorized office and it is totally unknown. But the hypothesis to the change made by the implementer is; the parallel line the made was thought equivalent to the recommended size. However, in hydraulic output it is totally different, 13m head loss replaced by 60m, but not created any further change on the transmission of water. In the key informant interview it was examined and found that some portion of the pipes buried deep up to 3-4m to pass the pick areas that had been created blocking of flow. (see Annex B)

Figure 14: Pressure Pipeline Changed with two parallel pipes

4.1.8. Distribution Pipes

Regarding the topographic nature of the area Likimse-Abela water distribution system has an advantage of distributing water using gravitational force. The head deference of the reservoir and the end tip of water point 385m. The maximum and minimum water pressure in the distribution system was 191.18 and -77.91m head between the reservoir and water points. According to the design criteria of the FDRE; MoWIE, the maximum and minimum water pressure in the distribution system is 80m and 15m respectively. Beside these comparison, the system currently operates out of the recommended limitation. This is because of the freely flows of water with no controlling valve functioning. Otherwise, as explained above the design period not yet reached to be complained old system, no deterioration observed but mainly due to system components malfunctioning. The distribution pipes of this system were observed categorized in to primary and secondary lines. Primary lines are main lines enclose multiple connections along its length. Secondary lines are lines of smaller diameter and supply water directly to the end users. For this case study, Primary lines of diameter varies from 150mm up to 80mm were included while secondary lines of lower diameters were concentrated to nearby nodes. A total of 76,906m of pipe extended and stretched in 13PAs.

The system has no any recording device installed that tells the amount of flow used by the user community or loss. So, this portion holds much part of contribution of losses occurred in the system.

The other important issue observed most of the pipes designed and recommended have changed with no reason known, due to this there exist the discrepancy of pipes on the ground and design. Therefore, to modify the service level of the system some arrangement on pipe and utilities simulation was conducted by adjusting the diameter and length of the pipe. As a result, no node will be observed without water. The table below shows the different three situations. Much of the length of pipes reduced are 150mm and 25mm diameters.

Table 15: Pipe length Vs diameter comparison

Pipe Diameter (mm)	Pipe length(m) On the design	Length(m) On the ground	Modified length(m) Using simulation
150	5818	2913	5014
100	12040	8428+4147	10474
80	6485	4121+4147	4121
63	13395	7578	7318
50	14638	7578	7596
40	29504	23611	30784
25	105	18591	11599
Total	**81,985**	**76,906**	**76,906**

4.1.9. Pressure variation in the distribution system

Variation of water pressure in the distribution system is mainly because of hourly fluctuation of water demand. The pressures in Likimse- Abela water supply system was also a function of this factor. Variation of elevation difference in most part of the area has also an impact for the rising and reduction of water pressure in the network. Therefore, with in the service time of 14 hours some of the water points listed on Table 5 was disconnected from the system, especially the water points that had been expanded according to the data observed. The time that these water points get only for few hours both in practical observation and on the simulation. But the system components that had designed at the initial get water the whole time, and the maximum pressure observed in the pipe is 191.18m at Junction J31. The middle areas like kebeles of Boke Dengola extending to Kolshobo and Ela up to Mehale Kebela have pressure amount of 60- 191m which as a result damaged the whole taps of the system. Therefore, to bring the system evenly distribute the water and increase the level of satisfaction to the community two optional remedial action have provided and the pressure at specific junction reduced to 56.33m. The time that the pressure was high at 8:00 hours and 12:00 hours after the start of giving service, in the morning.

4.1.10. Negative pressure

Situations that give rise to negative pressures should always be avoided. Hence, pressure in the distribution system is one of the factors for intermittent water supply. For this study, all negative pressure shown on the reality indicated; the system was disconnected during that demand time and water was not reaching to customers. Whereby, these was mainly because of; unintentional expansion of the system, utilization of different capacity of pipe during construction by omitting the recommended size and type (inadequate pipe capacity) and positioning of water points on higher ground elevation. Due to the above reason 11 water points face negative pressure when we observe the simulation result on the ground. See table below: -

Table 16: Negative pressure observed on the existing system

Item No	Water points	Head (m)	Pressure (m)
1	Junc WpGefetaKebele	1238.12	-71.88
2	Junc Wp35Mareka	1262.07	-45.93
3	Junc WpSeque	1581.45	-40.55
4	Junc WpZekaryas	1585.71	-29.29
5	Junc J4	1590.23	-27.77
6	Junc Wp30lalana	1351.53	-24.47
7	Junc Wp(Korke)	1381.37	-16.63
8	Junc Wp23Longena	1445.01	-1.99
9	Junc Wp8CipaSchool	1369.96	-0.04
10	Resvr SpringLikimse	1800	0
11	Junc WpKulia	1385.08	0.08

4.1.11. Model Calibration and Validation

To understand the credibility of a model it is simply to compare with model result precisely reflects measured field values. Thus, to have a confidence on model result it needs to calibrate and validate a model. An effort to perform hydraulic model calibration and validation for this case study is presented as follows. Similarly, our model EPANET-2 also need to calibrate and validated to have confidence result. To conduct calibration data was entered a file and registered with the project.

4.1.11.1. Hydraulic Calibration

In this case pressure was measured along the main line. Off course the main challenge was to dig the junction and get appropriate place to measure, hence the pressure at nearest water points taken and drawn back to get measured result at the junction. According to Pearson the measured and simulated data was correlation below: -

Table 17: Time series of pressure calibration

Time	Junction	C. Pressure	M. Pressure	X-Xavg	Y-Yavg	Squr(x-xavg)	squr (Y-Yavg)	(x-xavg)*(Y-Yavg)
4:00	J2	6.92	7.5	-46.15	-50.84	2129.82	2584.60	2346.22
5:00	J5	30.82	32	-21.65	-26.94	468.72	725.71	583.23
6:00	J12	100.5	96	42.35	42.74	1793.52	1826.79	1810.08
7:00	J14	87.75	74	20.35	29.99	414.12	899.46	610.32
7:30	J15	18.1	16	-37.65	-39.66	1417.52	1572.84	1493.16
8:00	J25	167.7	157	103.35	109.94	10681.22	12087.02	11362.40
9:00	J26	50.45	50	-3.65	-7.31	13.32	53.42	26.68
10:00	J27	54.65	48	-5.65	-3.11	31.92	9.67	17.57
10:30	J28	37.23	36	-17.65	-20.53	311.52	421.44	362.34
11:00	Vp Kolshob	23.47	20	-33.65	-34.29	1132.32	1175.74	1153.82
Sum		**577.59**	**536.5**			**18394.03**	**21356.69**	**19765.82**
Average		**57.759**	**53.65**					

$$R^2 = \frac{\sum(X-\overline{X})(Y-\overline{Y})}{\sqrt{\sum(X-\overline{X})^2\,(Y-\overline{Y})^2}}$$

Where,

- R is Person correlation
- X: Measured pressure on the field
- Y: Computed pressure by the model
- $\bar{X}$: Mean value of X
- $\bar{Y}$: Mean value of Y

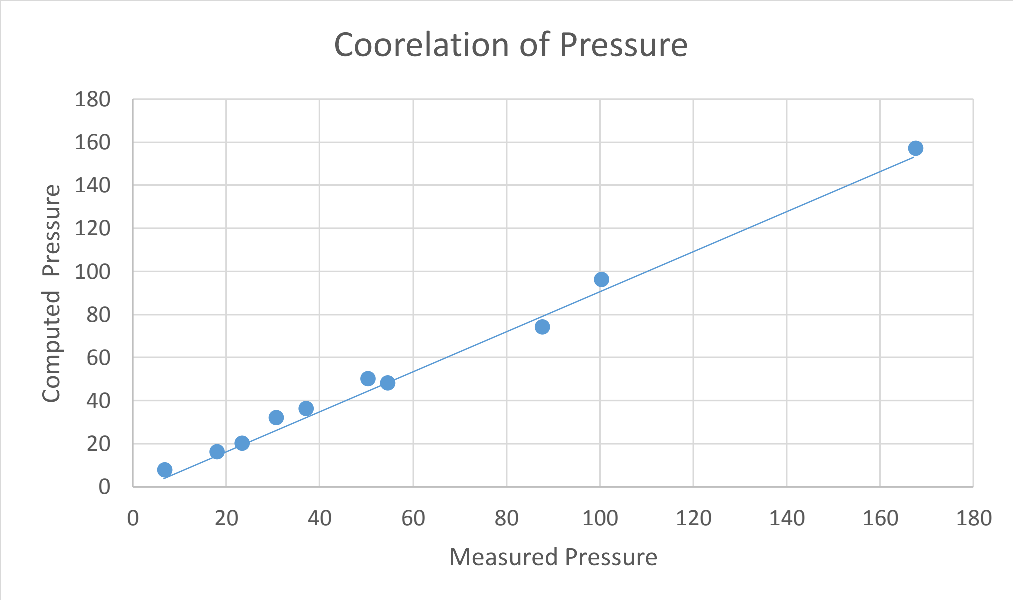

Figure 15: Measured Vs. Computed pressure values along the pipe line

Hence

$$R^2 = \frac{19765.82}{\sqrt{18394.03 * 21356.69}} = 0.997$$

4.1.12. Simulation Results

4.1.12.1. General information of simulation

The aim to conduct the hydraulic performance assessment model is to evaluate and quantify the consequences due to failure events in water distribution systems. The performance assessment by the help of model developed in this thesis mainly consists the hydraulic part only. Regarding component failure and performance measure mathematical models were used to clearly understand the status of the level of service.

In this case, the EPANET-2 software was preferred to perform hydraulic simulation since it is freely available and user friendly, particularly for water supply like the one under the study. The EPANET-2 performs extended period and single snapshot simulation of hydraulic and water quality with in pipe networks. The software tracks the flow in the distribution system, pressure on the node and head of water in the reservoir to understand the reality situation on the ground. For that matter, running a single snapshot simulation was helpful while performing preliminary model calibration. However, it should not work for the assessment of distribution system. Hence, the extended period simulation was exclusively used for entire model calibration and assessment effort. In most of the cases it uses as a research tool for improving our understanding of the movement and fate of drinking water constituents in the system. The core intention of the research is to understand the hydraulic situation of the component; however, the model can also help assess alternative management strategies for improving water quality in the system. Generally, EPANET contains a state-of-the-art hydraulic analysis engine that includes the following capabilities:

- No limit on the size of the network that can be analyzed
- Computes friction head loss, including minor head losses for bends, fittings, etc.
- Models constant or variable speed pumps
- Computes pumping energy and cost models' various types of valves including shutoff, check, pressure regulating, and flow control valves
- Allows storage tanks to have any shape (i.e., diameter can vary with height)
- Considers multiple demand categories at nodes, each with its own pattern of time variation
- Models pressure-dependent flow issuing from emitters (sprinkler heads) can base system operation on both simple tank level or timer controls
- And on complex rule-based controls (Rossman, 2000)

For the extended period of simulation, the demand pattern has taken as 1 for 14 hours and 0 for 10 hours because the system is totally open point discharge throughout the service time and closed for the given 10 hours of time.

4.1.12.2. System Maps development

In building system model, it is typically use full to draw system map for the water distribution system because it illustrates a wide variety of valuable characteristics. System maps may include information such as:

- Pipe alignment, connectivity, material, diameter, and so on
- The locations of other system components, such as tanks and valves

- Pressure zone boundaries
- Elevations
- Miscellaneous notes or references for tank characteristics
- Background information, such as the locations of roadways, streams, planning zones, and so on
- Other utilities

For this study system map was done by the data collected from the site. It is as built system map the map on the design has big variation on arrangement and positioning compared to the existing.

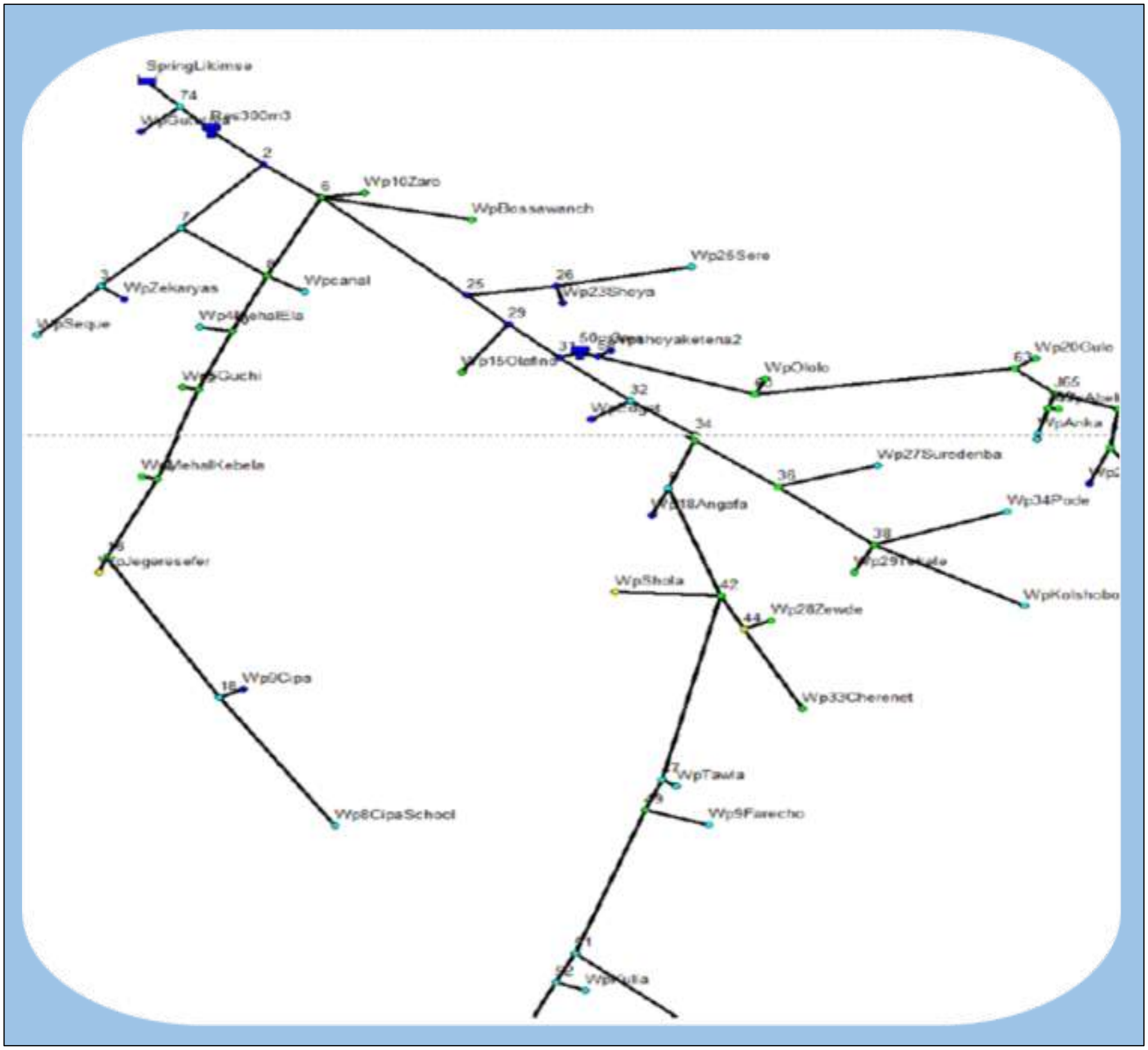

Figure 16: Layout of Likimse-Abela Water Supply System

(Source: As built drawing based on field survey)

Table 18: Shows pressure result at 4:00 hrs

Range of Pressure	Number of nodes in the modified	Number of nodes on the ground	% of modified	% on the ground
< 15	27	28	33.3	34.57
15-20	10	8	12.34	9.8
20-30	15	13	18.52	16.05
30-40	14	15	17.28	18.52
40-50	9	4	11.1	4.9
50-60	2	4	2.4	4.9
60-70	1	3	1.2	3.7
>70	3	8	3.7	9.8

From summary of simulation result pressure in the distribution line at a given time in the service hour is in permissible range. In this regard, the available pressure in the system may not cause bursting of pipe this may be supported by the open water points all over the system. Instead leakage may occur due to, workmanship and illegal disconnecting in the back yard. However, on the field it was observed that the distribution of pressure was not even, some places face excessive pressure, one of the main causes of this uneven values of pressure in the system is the wide range of levels between the sources of water (reservoirs) and the consumption nodes, also the absence of pressure reducing valves to counteract the high pressure from the reservoirs contributes in increasing this problem of high pressures in the system.

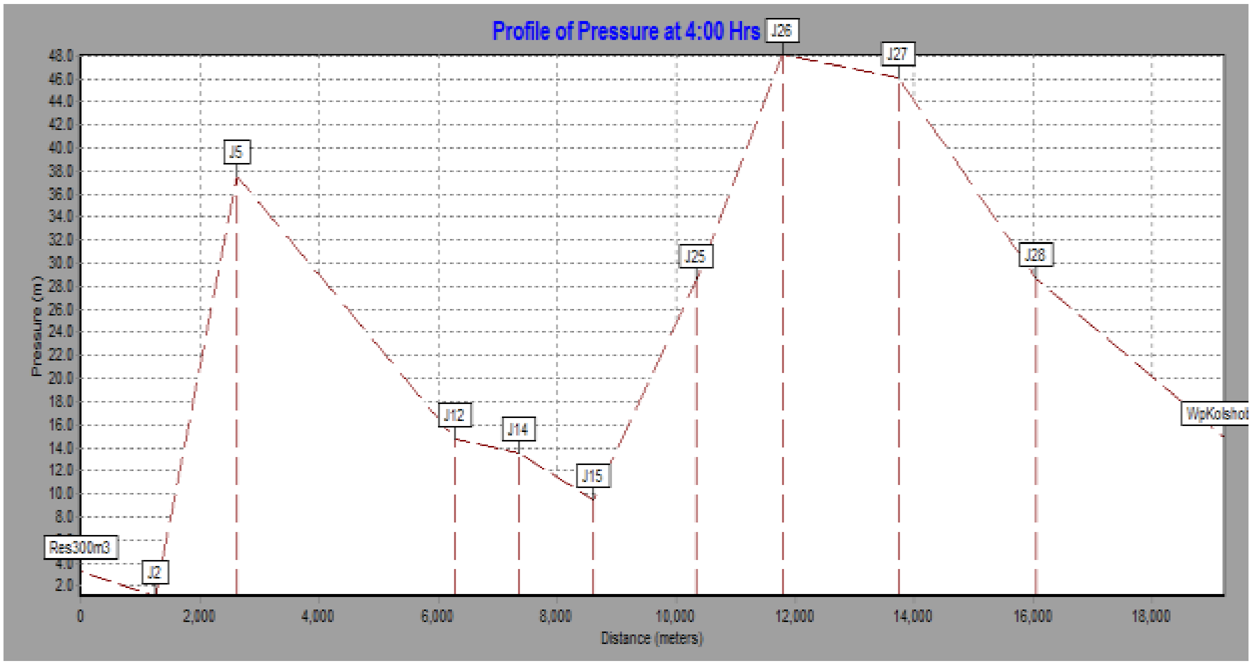

Figure 17: Pressure distribution at 4:00hrs.

In some consumption nodes zero values of flow rate exhibited as shown on Table 5 which is an indication of presence of negative or zero pressures at the consumption nodes this as explained in the research because of factors such as reduce in roughness coefficient, joint disconnection and improper positioning the junction on the ground.

When totally observed the velocity in the links is within the permissible velocity of 0.3-3m/s, however some places exhibited less slant. For instant at 3:00 hour in the pipe (Pi25) the velocity is 0.25m/s and in (pi32) is 0.28m/s but acceptable for the system supplying intermittently.

Table 19: Summary of system elements (Source: Summary analyzed by model)

System component	Numbers
Number of Junctions	80
Number of Reservoirs	2
Number of Tanks	2
Number of Pipes	82
Number of Pumps	0
Number of Valves	0
Flow Units	LPS

4.1.14. Water Loss Analysis

One of the major challenges of water utilities is high volume of water loss in distribution networks. If a large quantity of supplied water is lost; it is difficult to meet the level of satisfaction of the user community. Whereby, water loss for Likimse-Abela water supply system was assessed and discussed as below;

Because of the type of the system the water is distributed equally to the community but, with similar challenge mentioned above it performed inefficiently against to its objective.

Major sites for the loss are: -

- Taps on every water points are open.
- Disconnection of joints
- Malfunctioning or damage of valves.
- Vandalism and illegal disconnection
- Overflow on the reservoir

Figure 18: Photo shows losses. (Source: Own field visit photo)

4.1.14.1. Loss from the source to reservoir

In most of the case in this section loss is minor compared to the distribution main in practice which exhibited to contribute estimated 60% of the loss in the system. In addition, the type of pipe used is GI pipe of diameter 100mm and 50mm so, the connection points are too much since its standard length of connection is 6m.

So, only considering the visible portion water that will be lost in this section $9m^3$/day loss was calculated within 10hours during the night. In addition, during fetching time it is estimated 40% of water reached to water points lost and estimated $5.04m^3$/day. The other loss observed was in the chamber before getting to the reservoir estimated to $112.32m^3$/day of water. Totally, in this section $126.36m^3$ of water everyday will be lost. It is 12.19% of the total produced water.

4.1.14.2. Loss between the reservoir and end tips

The first loss observed in this section is over flow of the reservoir during the night time. This is because of the less capacity of the reservoir that had recommended on the design document. This is mentioned above on the description of the system 300m^3. In this portion, there is 2 hours' loss calculated 75.888m^3/day.

On the field observation, it was also measured that the flow rate on the total 40 water points after reservoir is 14.8lit/sec, the current requirement is 12.33lit/sec, but the pick hourly demand used in the design document from the reservoir is 18lit/sec see Annex C. So, it is easy to observe that there is loss in the middle but comparing to the current requirement it is still enough. The other losses that were easily observed in the pipe connection and valve chambers estimated 1.5lit/sec and 1.69lit/sec sum up respectively.

As per the standard time set on the national guideline time taking to fill 20 lit jerrycan of container on each of community tap was considered 3minutes however, on field observation the time was estimated 6-7minute of time on some water points and 10-12minute on some water points which have flows. This is because of hassle occurred due to low flow rate and high queue, the inconvenience of in and out to the high flow rater water points and narrow mouth of water fetching materials made the observed time to be extended. But for the calculation of loss on points an average of 5 minute has taken to the water point. From this justification 2-minute loss is estimated on each of water fetching process. Hence it is to mean that the system was efficiently gives service for about 8.4 hours only. The rest of 5.6 hours of time spent without giving service, so annually 95,570.9m^3 on water will be lost from 17 water points which have more flow rate, which is 34.5% of the total water reached on the tap.

The other situation which was considered for this research is that the whole communities are using the water system in the 14 hours of service time. But, on the field visit it was observed that there are days that the water points will be idle, flowing free. The days are market day, Sunday, or days given to social activities.

In addition, as mentioned above, unauthorized connecting of water points to backyard farm and illegal disconnecting joints for similar purpose are considered as either the loss on the connection or joint failure loss. This is because to simplify the research and at the same time considering the case acceptable rather wasting the resource.

Water loss expressed as per number of connection

$$Water\ loss\ =\ Annual\ total\ loss\ *1000/(Number\ of\ connection*365)$$

Water loss expressed as per length of pipes

$$Water\ loss\ =\ Annual\ loss/(Length\ in\ Km\ *365)$$

In any of discipline achievement of 100% may be ideal hence, there should be acceptable margin from an economic point of view of a country and considering also the development level of once country. Due to this reason 10% of loss can be consider but, in our case it is observed much more than the acceptable limit. Therefore, the loss summarized below is within the matter of concern that must be considered reduce.

Table 20: Summary of losses in the distribution system

Area of NRW	Amount (m3/d)	Annual loss (m3)	Annual Sum (m3)	Production (m3)	% of losses
In the valve chamber from the source to reservoir (24 hours)	112.32	40996.8	Annual losses between the source and reservoir		
On the water point b/n source and reservoir night time (10 hours in the night)	9	3285			
On the water point b/n source and reservoir day time (14 hours in the day)	5.04	1839.6	**46121.4**	378432	12.19%
Over flow on the reservoir (2hours in the night time)	75.888	27699.12	Annual losses between the reservoir and end user		
In the valve chambers after reservoir	73.08	26674.2			
On water points from the reservoir to end tips (14 hours in the day)	261.8	110817.65			
Loss on the connection in the distribution (14hours in the day)	75.6	27594	**192784.97**	331128	58.22%
Total loss in the system	**612.728**	**238906.37**			

An Acceptable Water Loss

- It is a compromise between the cost of reducing water loss and maintenance of distribution system and the cost (of water) saved.
- AWWA Leak Detection and Accountability Committee, (1996), recommended 10% as a benchmark for NRW.
- NRW levels and action needed :< 10% Acceptable, monitoring and control, 10-25% Intermediate, could be reduced,> 25% Matter of concern, reduction needed.

4.1.14.3. Reasons of water losses

- The system is totally open system it flows throughout service time whether the user come or not
- Leakage due to weak connection of fittings
- Leakage occurred due to high pressure in some points
- Over flow due to less capacity of night storage reservoir
- Unknown reason of change of design and poor workmanship during construction
- Unauthorized connection to water points and illegal disconnection of joints to the back-
- yard gardening

Figure 19: Points of different flow rate at the same service day (Source: Own filed visit Photo)

4.1.15. Operation and Maintenance Situation

Even though there is trained water committee and operators, the O&M of the system is much more difficult to manage by these people. It is because the system serves trans woreda and kebele. Due to this, in some kebele the may be happen break or disconnection that would be responded on time unless the break creates on their water point. No skilled operators who control the system dwelling permanent base in the area. Most of the operation system was controlled by the utility guards. The overall maintenances of system components were not checked by schedule and it was maintained during failure or damage is occurring. Valves and all accessories installed in the main were not properly used and maintained regularly. no functional controlling valves within the network. Accordingly, when failure is happened; the utility will be maintained by closing the reservoir. Whereby, the other users forced to stay until the water come again. The other collected information was due to limitation of budget; when failure is occurred at customer's service line, the customers forced to supply all the necessary accessories. Accordingly, until the required accessories are purchased and replaced a considerable quantity of water is lost.

4.1.16. Challenge Encountered

It was reported on the above that availability of the design document and other relevant information on the system was the main challenge. However, probing to different source rough design document has found on an individual hand. The second challenge that has created on the proceeding of the research was the discrepancies of information of the paper and of the reality. For instant, the lay out on the document and the existing system is not totally resembled. In addition, pipe length and diameters, population served and design populations also are different. The other issue come across on the document was design population determination. It was simply adopted as geometric progression without considering or observing the plan of the government in the area, hence with the huge investment short design period was considered. The third challenge observed on the field was the system status. It is totally an open and uncontrolled system without taps and valves. Due to this case, it is difficult to understand the peak hourly demand and some of water points malfunctioned. Hence, the researcher obliged to do as built drawing to clearly identify the problem occurred. Validation and calibration was done manually.

Chapter Five

5.1. Conclusion and Recommendation

5.1.1. Conclusion

In this water supply system, inadequate operation of water distributing components, weak management system and absence of inspection largely contribute to inefficient hydraulic performance which as a result create poor level of service to the satisfaction of user community

In this study, some the fundamental performance challenges identified and taken as cases to be considered on the system are: -

- Insufficient availability of water on some of water points, uneven distribution
- Evaluation indicated that acceptable minimum and maximum pressures have not been met. During the service hour of the system, parts of the distribution receive water with low pressure and under some circumstances on some of points water is not observed because of the pressure and velocity in the distribution system is contradicting to the permissible minimum requirement.
- 58.22% of the water produced and distributed is lost without giving service to the community. Because of this most of the customers are not satisfied in the level of service of the system.
- Malfunctioning of taps and valves, frequent disconnection of joint is observed.
- Poor workmanship and lack of supervision on the construction brought the pipe type and dimension installed different with recommended type and size on some portion of the system.
- Poor management, absence of inspection and O&M

Therefore, about the satisfaction level of service, the system was observed in poor performance which deliver fluctuating amount of water to various demand categories within the user community.

5.1.2. Recommendations

Based on the findings, the following recommendation are made

- Above all resource conservation must be given priority to guarantee future consumption, hence the system must be maintained to be closed system, which means all taps and gate valves should be replaced.
- Pressure sustaining valves or controlling valves must be installed between J5 and J12 on the distribution system in the model to control the occurrences of variation of pressure. These valves start closing and opening either automatically or manually to avoid fluctuation of pressure by maintaining minimum required pressure.
- In addition to the above technical recommendation it is also good the think on establish asset management plan specially to control the water loss.
- The system age is 18 years, the pipes are not deteriorated except the disconnection of joints, breakage of valves and taps so with some improvement on reservoir capacity to 450m^3, change on the pipe diameter and installation of pressure reducing valve the future demand can be covered and the design period could also be extended to 25 year.
- The system is large system which can be run by water service so, it requires establishment of utility or satellite water office in central part of the user community.
- Expansion should be based on the careful study with the help of models build for the existing system on the ground.
- To alleviate question of information on the system there must be as built drawing of lay out which helps to be guide the maintenance crew rather going to field to check the pipe size and arrangement.
- In the system, it was observed that a masonry reservoir constructed for income generating by using irrigation. This reservoir has created challenge on water points because it breaks the pressure for the water points located downstream. So, it must be looped to the original pipe and provide water for the reservoir like other water points. This by far modify the flow rate of water points in the downstream.

- Water point installed on the transmission main, which is similarly feed with water at the same time with the reservoir. So, to reduce loss both in head and flow it is better to put tanker considering the population in that area so, it is possible to loop and close the pipe and reduced loss by directing water to the main reservoir.
- Establish chlorine injection system on the reservoir.
- Finally, concerned government organization or other stakeholder should frequently follow up the system and should be pay enough attention and put forward dimensions on water loss management strategies through the processes with involving engineering approaches.

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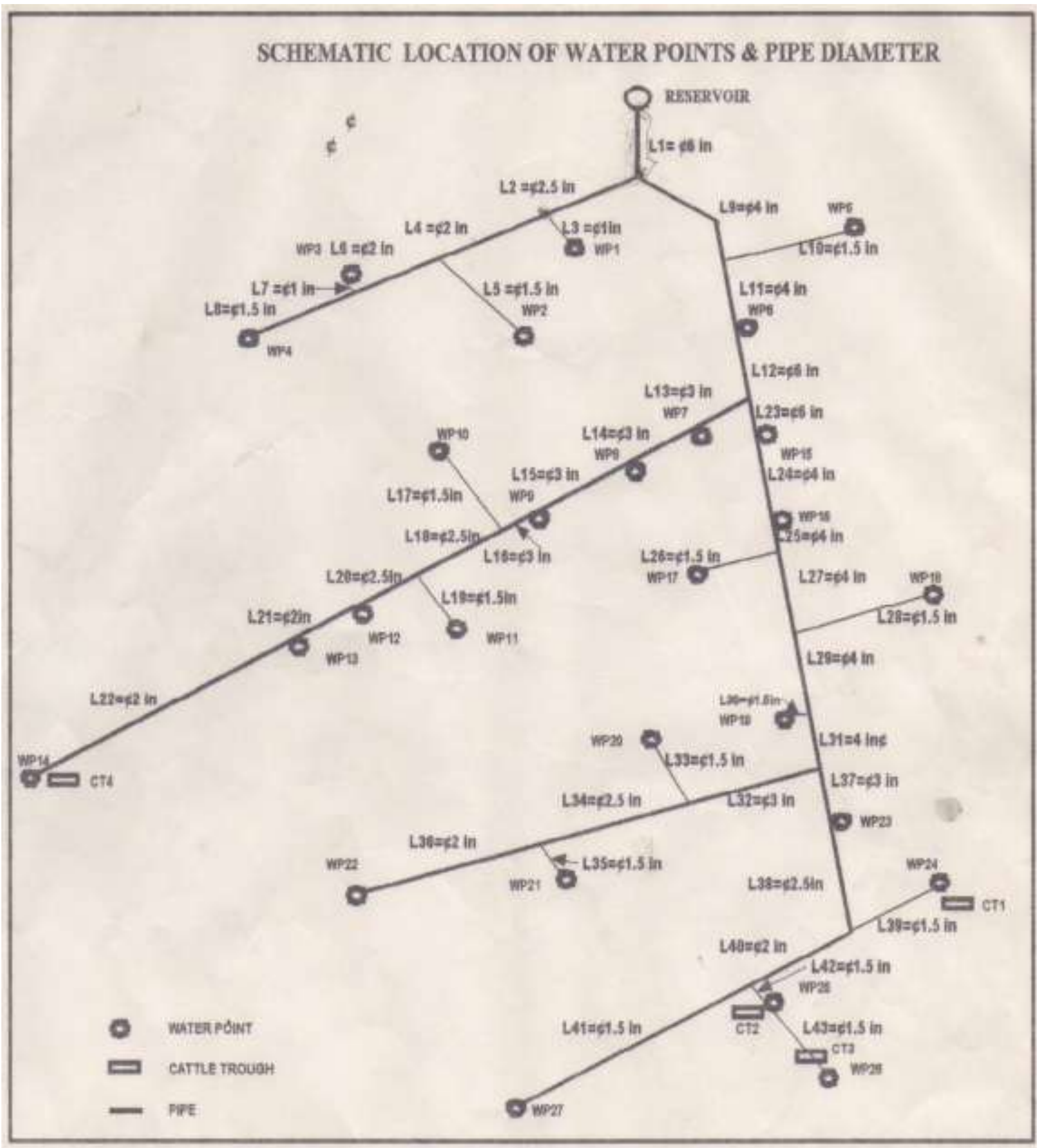
Appendixes



Appendix A: Lay out on the design document

Table - Hydraulic calculation of Gravity main

Elevation of source of spring = 1800m

Q	D	V	HGL	L	ELV	ST.H	H.L	ΣH.L	F.H	REMARK
lit/sec	mm	m/sec	m/m	m	m	m	m	m	m	
12	150	0.7	0.0033	128	1797.97	2.025	0.42	0.42	1.6	Wet well
12	150	0.7	0.0033	4046	1769.81	30.19	13.47	13.89	16.42	
12	100	1.5	0.0238	4108	1677.78	122.22	98	111.9	10.32	Reservoir

Appendix B: Hydraulic Calculation of Gravity main of Likimse-Abela water supply system

LAWSS Hydraulic Calculations of Distribution Systems

Reservoir Elevation= 1677.78 m

P Line	N Node	D mm	Q lit/sec	V m/sec	HGL m/m	L m	ELV m	ST.H m	H.L m	(H.L m	F.H m	REMARK
L1	1	150	18	1.01	0.0069	156	1669.41	8.37	1.08	1.08	7.3	V.C
L2	2	63	3	0.96	0.0149	1147	1615.79	61.99	17.1	18.18	43.81	
L3	3	25	0.75	1.53	0.0954	63	1612.6	65.18	6.01	24.19	40.98	WP1
L4	4	50	2.25	1.14	0.0265	675	1598.09	79.69	17.88	42.07	37.62	
L5	5	37	1	0.93	0.0238	825	1583.18	94.6	19.66	61.73	32.87	WP2
L6	6	50	1.25	0.64	0.0084	1055	1578.4	99.38	8.81	70.54	28.84	
L7	7	25	0.5	1.01	0.041	16	1578.4	99.38	0.66	71.2	28.2	WP3
L8	8	37	0.75	0.7	0.0135	1151	1561.23	116.55	15.54	82.74	29.81	WP4
L9	9	100	15	1.91	0.0372	1390	1593.98	83.8	51.69	52.77	31.03	
L10	10	37	1	0.93	0.0238	1827	1527.88	149.9	43.53	93.3	53.6	WP5
L11	11	100	14	1.78	0.0323	2163	1501.24	176.54	69.86	163.16	13.38	WP6
L12	12	150	13.6	0.77	0.004	641	1491.35	186.43	2.58	165.74	20.68	V.C
L13	13	80	5.2	1.03	0.0135	1207	1468.53	209.25	16.31	182.05	27.2	WP7
L14	14	80	4.8	0.95	0.0115	1097	1457.21	220.57	12.6	194.65	25.92	WP8
L15	15	80	4.4	0.87	0.0096	2135	1393.96	283.82	20.59	215.24	68.58	WP9
L16	16	80	4	0.8	0.008	402	1386.11	291.67	3.24	218.48	65.77	
L17	17	37	1	0.93	0.0238	715	1385.5	292.28	17.04	235.52	56.76	WP10
L18	18	63	3	0.96	0.0149	2242	1389.1	288.68	33.43	268.95	19.73	

LAWSS Hydraulic Calculations of Distribution Systems

Reservoir Elevation= 1677.78 m

P Line	N Node	D mm	Q lit/sec	V m/sec	HGL m/m	L m	ELV m	ST.H m	H.L m	(H.L m	F.H m	REMARK
L19	19	37	0.8	0.74	0.015	286	1390.82	286.96	4.3	273.25	13.71	WP11
L20	20	63	2.2	0.705	0.008	1164	1375.3	302.48	9.36	282.61	19.87	WP12
L21	21	50	1.8	0.91	0.0168	570	1364.2	313.58	9.62	292.23	21.35	WP13
L22	22	50	1.4	1.3	0.0104	4541	1299.51	378.28	47.06	339.29	38.99	WP14
L23	23	150	8.5	0.5	0.0016	847	1471.3	206.48	1.33	167.07	39.41	WP15
L24	24	100	8	1	0.0104	1244	1435.07	242.71	12.93	180	62.71	WP16
L25	25	100	7.5	0.95	0.0093	636	1428.12	249.66	5.91	185.91	63.75	V.C
L26	26	37	1	0.93	0.0238	569	1418.88	258.9	13.56	199.47	59.43	WP17
L27	27	100	6.5	0.82	0.0069	1007	1425.79	252	7.03	206.5	45.5	
L28	28	37	0.75	0.69	0.0134	557	1430.11	247.67	7.46	213.96	33.71	WP18
L29	29	100	5.75	0.73	0.0054	905	1424.92	252.86	4.94	218.9	33.96	
L30	30	37	0.75	0.69	0.0132	28	1424.86	252.92	0.37	219.27	33.64	WP19
L31	31	100	5	1.03	0.0041	587	1427.85	249.93	2.42	221.69	28.24	
L32	32	80	2.5	0.5	0.003	1231	1425.94	251.84	3.76	225.45	26.39	
L33	33	37	0.75	0.69	0.0131	578	1424.87	252.93	7.58	233.03	19.9	WP20
L34	34	63	1.75	0.56	0.0051	1842	1421.15	256.63	9.39	242.42	14.21	
L35	35	37	0.75	0.69	0.0131	216	1425.91	251.87	2.83	245.25	6.62	
L36	36	50	1	0.51	0.0053	1667	1402.84	274.98	8.81	254.06	20.92	WP22

LAWSS Hydraulic Calculations of Distribution Systems

Reservoir Elevation= 1677.78 m

P Line	N Node	D mm	Q lit/sec	V m/sec	HGL m/m	L m	ELV m	ST.H m	H.L m	(H.L m	F.H m	REMARK
L37	37	80	2.5	0.5	0.0031	413	1431.68	246.1	1.3	222.99	23.11	WP23
L38	38	63	2	0.64	0.0066	746	1414.14	263.64	4.96	227.95	35.69	
L39	39	37	0.65	0.6	0.0099	961	1430.97	246.81	9.53	237.48	9.33	WP24
L40	40	50	1.35	0.68	0.0095	2175	1396.74	281.04	20.96	258.44	22.6	
L41	41	37	0.5	0.5	0.0039	2721	1382.44	295.44	10.81	269.25	26.19	WP25
L42	42	37	0.81	0.75	0.0155	126.31	1405.58	272.2	1.96	271.21	24.22	WP26
L43	43	37	0.5	0.5	0.0103	1570.69	1365.65	312.13	16.21	287.42	24.7	WP27

Appendix C: Hydraulic calculation of Distribution line of Likimse-Abela Water Supply System

ANNEX-2(a)- HYDRAULIC CALCULATION OF EXTENDED LINE TO ELLA

P Line	N Node	Q lit/sec	D mm	V m/sec	L m	Elev. M	H.G. L. M/m	ST. H m	H. L. M	C. H. L m	F. H. M	Remark
L_2	2	3	63	0.96	1147	1615.79	0.0149	61.99	17.1	18.18	43.81	
EL_1	3'	1.1	50	0.56	1229	1601.05	0.0287	76. 73	7.87	26.05	50.68	
L_4	4	1.9	50	1	675	1598.09	0.0276	79.69	8.62	36.8	42.89	
L_3	Wp_1	0.55	25	1.12	10	1601.05	0.0512	76.73	0.512	26.56	50.17	Only place chage
EL_2	EWp_1	0.55	37	0.51	814	1591.17	0.0512	86.61	5.84	31.89	54.72	
L_5	WP_2	0.63	37	0.6	825	1583.18	0.0099	94.6	8.19	44.99	49.61	
L_6	6	1.27	50	0.65	1055	1578.4	0.0086	99.38	9.1	45.9	53.48	
L_7	WP_3	0.635	25	1.29	16	1578.4	0.0688	99.38	1.1	47	52.38	
L_8	Wp_4	0.635	37	0.6	1151	1561.23	0.0099	116.5 5	11.43	57.33	59.22	

Elevation of Reservoir 1677.78m

Cumulative head loss at point 1 is 1.08m.

ANNEX-2(b)- HYDRAULIC CALCULATION TO GEFETA (PREVIOUS SOLAR SYSTEM)

P Line	N Node	D mm	Q lit/sec	V m/sec	L m	Elev. M	H.G. L. M/m	ST. H m	H. L. M	C. H. L m	F. H. M	Remark
L_{20}	Wp_{12}	63	1.635	0.52	585	1375.3	0.0044	302.48	2.55	274.05	28.43	l is half of previous L [illegible]
EL_3	EWp_2	63	1.635	0.52	2350	1305.72	0.0044	372.06	10.29	218.79	90.27	
L_{21}	WP_{13}	50	1.09	0.56	570	1364.2	0.0069	313.58	3.648	277.7	35.88	
L_{22}	WP_{14}	37	0.546	0.51	4541	1299.51	0.0072	378.27	32.57	310.27	68	*
EL_4	2'	50	1.09	0.56	300	1310.02	0.0064	367.76	1.92	283.71	84.05	
EL_5	EWp_3	37	0.546	0.51	521	1306.73	0.0072	371.05	3.74	287.45	83.6	
EL_6	EWp_4	37	0.546	0.51	2600	1268.7	0.0072	409	18.65	306.1	102.9	

_ Elevation of reservoir = 1677.78m

_ Elevation of junction to Gefeta = 1375m (A)

_ Cumulative head loss at point of junction = 271.50m

* _ The path line L_{22} can be changed as shown in schematic drawing

_ Advantage:- the line could supply to the previous elevated tanker of Village near by the main road to Arba Minch, at the junction to Abella Faricho.

:- Easy for installation as well as maintenance.

:- Could serves for more family head not considered in design population.

_ Additional need for this purpose is only about 1500m, 50mm pipe (G.I).

_ L_{22} should be changed to 37mm.

ANNEX-2(c)- HYDRAULIC CALCULATION OF EXTENDED LINE TO GEFETA

P Line	N Node	D mm	Q lit/sec	V m/sec	L m	Elev. M	H.G. L. M/m	ST. H m	H. L. M	C. H. L m	F. H. M	Remark
L_{39}	Wp_{24}	37	0.546	0.5	961	1430. 97	0.0069	246.81	6.63	234.58	12.23	l is half of previous L_{39}
L_{40}	40	63	2.184	0.7	2175	1396.74	0.0251	281.04	17.26	245.21	35.83	
L_{42}	42	50	1.638	0.83	126.31	1382.44	0.0141	295.34	1.78	246.99	48.35	**
42	WP_{25}	37	0.546	0.5	10	1382.44	0.0069	295.34	0.069	247.06	48.28	**
L_{43}	1'	37	1.096	1.02	1570.69	1365.65	0.0287	312.13	45.07	292.06	20.07	
EL_7	Wp_{26}	37	0.546	0.5	10	1365.65	0.0069	312.13	0.069	292.13	20.001	
EL_8	EWp_5	37	0.546	0.5	1650	1337.3	0.0069	340.48	11.38	303.44	37.04	
EL_{41}	Ep_{27}	37	0.546	0.5	2721	1405.58	0.0069	272.2	18.76	263.97	80.23	

_ Elevation of Reservoir = 1677.88m

_ Cumulative head loss at point 38 = 227.95

** _ L_{40} is changed fro G. I pipe 50mm to G. I pipe 63m.

** _ L_{42} is changed fro 37mm to 50mm.

*_ WP_{11} (L_{19})

_ The position the supply line to the above mentioned water point was changed due to the reason that to supply to WVA camp simultaneously with it.

Appendix D: Hydraulic calculation of Expansion of Likimse-Abela Water Supply System

It. No	Name of Kebeles	Existing types of water sources						
		BH	SW	HDW	LS	MS	SS	Wps from Likimse
1	Abala Mareka	2						2
2	Abaya Bisare			5				
3	Abaya Chawkare	1			1			
4	Abela Ajaja	1						2
5	Abela Faracho	1						4
6	Abela Gefata	2						2
7	Abela Longena							2
8	Abela Qolshebo							4
9	Abela Shoya							5
10	Abela Sipa	1						2
11	Abela Zegire							3
12	Ambe Shoya					1	1	2
13	Ampo Koysha							3
14	Ancka Wocha	1						
15	Boke Dongola							5
16	Bola Wanche							
17	Bosa Wanche		3				1	1
18	Demba koysha				1			
19	Ela kebela					1		5
20	Fango Gelchecha							
21	Fango Lome							
22	Gelecha Kara	1						
23	Gututo larena	1	1					1
24	Hobicha Bada							
25	Hobicha Bongota							
26	Hobicha Borkoshe							

It. No	Name of Kebeles	Existing types of water sources						
		BH	SW	HDW	LS	MS	SS	Wps from Likimse
27	Hobicha Digiso							
28	Kodo Kanko						1	
29	Koysha Gola		1					
30	Koysha Ogodama				1	1	2	
31	Koysha Wangala	1	1	2				
32	Offa							
33	Sere Tawreta				1			
34	Shochora Abala							
35	Shochora Fisho						1	
36	Shochora Ogodama					1		
37	Shocora Gola	1					1	
38	Shochora Ose	1	2	3				
39	Tebela 02	1						
	Total	**12**	**8**	**10**	**4**	**4**	**7**	**41**

Appendix E: Type and Number of Water supply system in Humbo Woreda

Parameters	Description
Pipe line	• 80km of pipe of 150mm-38mm • Pipe types are GI • No drawing at all • GI pipe installed in 1998
Water distribution points	• 41 Water points with two faucets each
Night storage Reservoir	• 150m^3 of two concrete reservoirs • 50m^3 of masonry reservoir
Cattle troughs	• 10 cattle troughs but not working
Treatment component	• No any treatment component included • No problems reported
Source	• Spring capping with discharge of 36lit per second
Pump station	• The system has no pump system, it is gravitational
Hydrant	• No installed hydrant (hydrant not considered)
User committee	• There are committees on each of points • There is one water committee board at the woreda level
Regular maintenance	• There is maintenance support on request

Annex F: Information on the system (Source: Key informant interview during secondary data collection)

It. No	Description	North	East	Elevation
1	Wp5 (Jegere sefer)	6.6924	37.7884	1500
2	Wp6 (Mehal Kebela)	6.7011	37.7911	1542
3	Wp7(Guchia)	6.7125	37.7958	1574
4	Wp17 (Shola)	6.6820	37.8295	1397
5	Wp2(Mehal Ela)	6.7223	37.7988	1597
6	Wp10 (Tawla sefer)	6.6613	37.8357	1402
7	Wp (Abela Faracho)	6.6593	37.8371	1401
8	WP (Angafa)	6.7035	37.8297	1479
9	Wp4 (Seque)	6.7257	37.7899	1622
10	Wp19 (lalana)	6.6397	37.8255	1376
11	Wp20 (Olafino)	6.7202	37.8157	1461
12	Wp18 (Kulia)	6.6516	37.8363	1385
13	Wp16 (Zewde)	6.6854	37.8425	1408
14	Wp21 (Edget)	6.7132	37.8279	1502
15	Wp1 (Zekaras Zasa)	6.7302	37.7945	1615
16	Wp (canal)	6.7303	37.8082	1601
17	Wp24 (Shoya)	6.7228	37.8282	1504
18	Wp10 (Cherenet)	6.6806	37.8513	1412
19	Wp15 (Tekele)	6.6975	37.8496	1419
20	Wp23 (Shoya)	6.7299	37.8228	1520
21	Wp26 (Ololo)	6.7153	37.8436	1437
22	Wp15 (Sura Denba)	6.7083	37.8489	1443
23	Wp32 (Zaro)	6.7431	37.8119	1529
24	Wp33 (Bosa)	6.7427	37.8227	1562
25	Wp25 (Sere)	6.7355	37.8356	1513
26	Wp27(Gulo)	6.7174	37.8586	1438

It. No	Description	North	East	Elevation
27	Wp28 (Abela Zegre)	6.7137	37.8657	1437
28	Wp31Pode, Selase	6.7021	37.8635	1425
29	Wp29 Longena	6.7124	37.8752	1443
30	Wp30 Longena	6.7178	37.8839	1447
31	Wp12 Kolshobo	6.6871	37.8651	1432
32	Wp13 (Girara)	6.6916	37.8729	1419
33	Wp14 (Mehanmeka)	6.691	37.8775	1407
34	Wp 35 (Abela Mareka)	6.6111	37.8398	1295
35	Wp (Abela Cipa)	6.651	37.8125	1380
36	Wp (Abela Gefeta)	6.6393	37.8516	1308
37	Wp Cipa (school sefer)	6.6719	37.8031	1400
38	Wp (Boho)	6.4577	37.8511	1317
39	Wp (Korke)	6.6896	37.8768	1398
40	Wp (Anka)	6.7121	37.8648	1431
41	WP (Gututo Larena)	6.76206	37.7829	1727

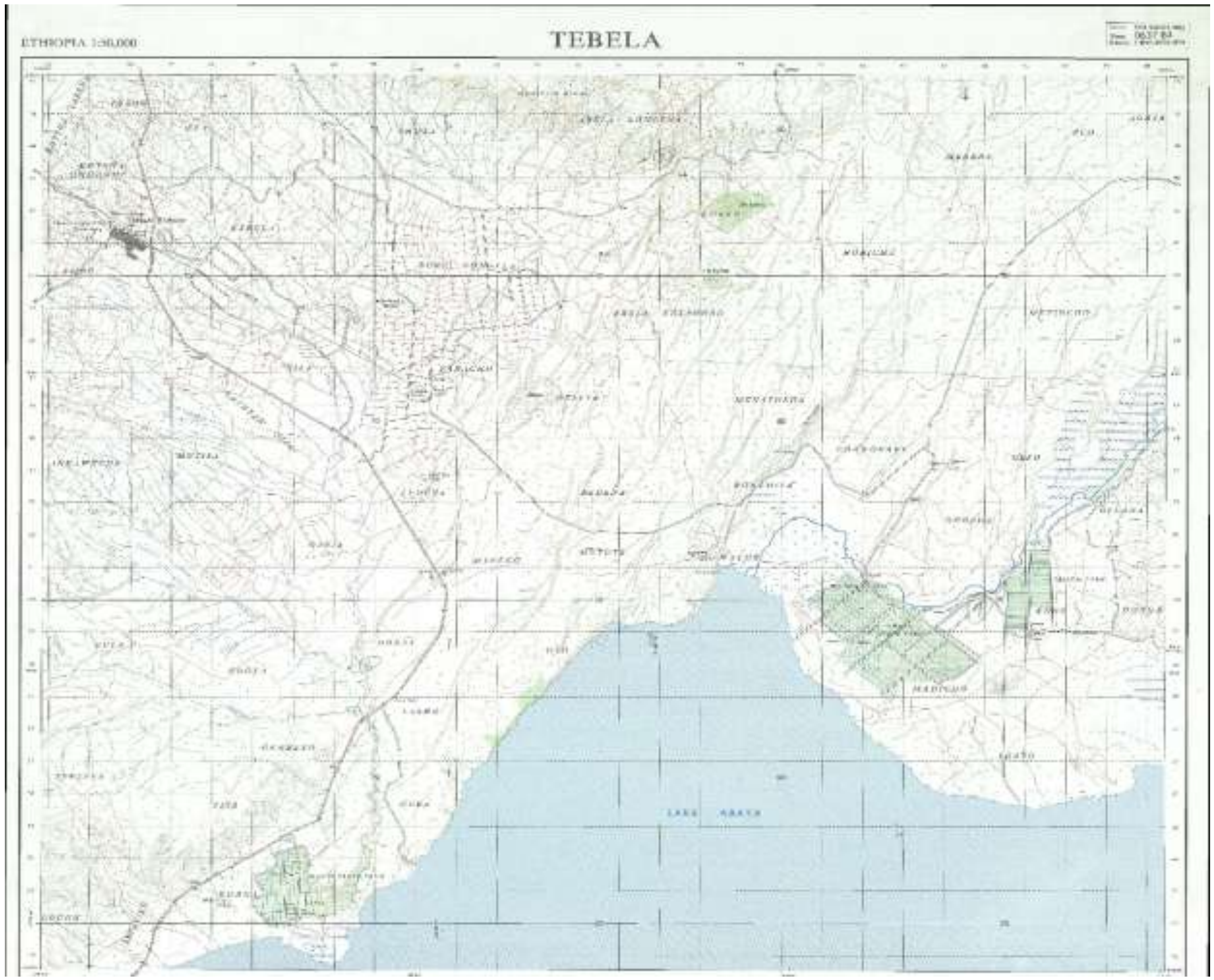
Annex G: Elevation data collected on the field observation

Sr. No	Name of Water points	No. Houses	No. people	Avg. Demand (lit/day)	Avg. Demand (m3/d)	Max. Demand (m3/d)	Peak hrly Dem.(m3/d)	Peak hrly demand (lit/sec)	Period of service
1	wp5 (Jegere sefer)	105	525	10500	10.5	12.6	18.9	0.219	14/24
2	wp6 (Mehal Kebela)	50	250	5000	5	6	9	0.104	14/24
3	wp7(Guchia)	100	500	10000	10	12	18	0.208	14/24
4	wp17 (Shola)	170	850	17000	17	20.4	30.6	0.354	14/24
5	wp2(Mehal Ela)	126	630	12600	12.6	15.12	22.68	0.263	14/24
6	wp10 (Tawla sefer)	195	975	19500	19.5	23.4	35.1	0.406	14/24
7	Wp (Abela Faracho)	160	800	16000	16	19.2	28.8	0.333	14/24
8	Wp (Angafa)	75	375	7500	7.5	9	13.5	0.156	14/24
9	wp4 (Seque)	75	375	7500	7.5	9	13.5	0.156	14/24
10	wp19 (lalana)	144	720	14400	14.4	17.28	25.92	0.300	14/24
11	wp20 (Olafino)	139	695	13900	13.9	16.68	25.02	0.290	14/24
12	wp18 (Kulia)	120	600	12000	12	14.4	21.6	0.250	14/24
13	wp16 (Zewde)	120	600	12000	12	14.4	21.6	0.250	14/24
14	wp21 (Edget)	110	550	11000	11	13.2	19.8	0.229	14/24
15	wp1 (Zekaras	120	600	12000	12	14.4	21.6	0.250	14/24

Sr. No	Name of Water points	No. Houses	No. people	Avg. Demand (lit/day)	Avg. Demand (m3/d)	Max. Demand (m3/d)	Peak hrly Dem.(m3/d)	Peak hrly demand (lit/sec)	Period of service
	Zasa)								
16	wp (canal)	250	1250	25000	25	30	45	0.521	14/24
17	wp24 (Shoya)	100	500	10000	10	12	18	0.208	14/24
18	wp10 (Cherenet)	180	900	18000	18	21.6	32.4	0.375	14/24
19	wp15 (Tekele)	189	945	18900	18.9	22.68	34.02	0.394	14/24
20	wp23 (Shoya)	80	400	8000	8	9.6	14.4	0.167	14/24
21	wp26 (Ololo)	120	600	12000	12	14.4	21.6	0.250	14/24
22	wp15 (Sura Denba)	158	790	15800	15.8	18.96	28.44	0.329	14/24
23	wp32 (Zaro)	297	1485	29700	29.7	35.64	53.46	0.619	14/24
24	wp33 (Bosa)	299	1495	29900	29.9	35.88	53.82	0.623	14/24
25	wp25 (Sere)	140	700	14000	14	16.8	25.2	0.292	14/24
26	wp27(Gulo)	187	935	18700	18.7	22.44	33.66	0.390	14/24
27	wp28 (Abela Zegre)	162	810	16200	16.2	19.44	29.16	0.338	14/24
28	wp31Pode, Selase	200	1000	20000	20	24	36	0.417	14/24
29	wp29 Longena	80	400	8000	8	9.6	14.4	0.167	14/24
30	wp30 Longena	80	400	8000	8	9.6	14.4	0.167	14/24
31	wp12 Kolshobo	140	700	14000	14	16.8	25.2	0.292	14/24

Sr. No	Name of Water points	No. Houses	No. people	Avg. Demand (lit/day)	Avg. Demand (m3/d)	Max. Demand (m3/d)	Peak hrly Dem.(m3/d)	Peak hrly demand (lit/sec)	Period of service
32	wp13 (Girara)	100	500	10000	10	12	18	0.208	14/24
33	wp14 (Mehanmeka)	210	1050	21000	21	25.2	37.8	0.438	14/24
34	wp 35 (Abela Mareka)	80	400	8000	8	9.6	14.4	0.167	14/24
35	Wp (Abela Cipa)	143	715	14300	14.3	17.16	25.74	0.298	14/24
36	Wp (Abela Gefeta)	170	850	17000	17	20.4	30.6	0.354	14/24
37	Wp (Cipa school A.)	160	800	16000	16	19.2	28.8	0.333	14/24
38	Wp (Boho)	210	1050	21000	21	25.2	37.8	0.438	14/24
39	Wp (Korke)	143	715	14300	14.3	17.16	25.74	0.298	14/24
40	Wp (Anka)	234	1170	23400	23.4	28.08	42.12	0.488	14/24
	Total	5921	29605	592100	592.1	710.52	1065.78	12.335	
41	Wp Gututo Larena	128	640	12800	12.8	15.36	23.04	0.267	24/24

Annex H: Estimated peak hourly and maximum demand



Annex I: Top map of the area (Source: Ethiopia Mapping Agency)

Node ID	Existing pressure m	Pressure by PRV m	Modified pressure m
Junc J2	8.99	8.12	2.99
Junc J4	-25.7	35.62	30.49
Junc WpSeque	-38.48	22.85	17.71
Junc WpZekaryas	-27.22	34.11	28.97
Junc J5	52.39	41.15	40.72
Junc Wp10Zaro	30.53	19.29	18.86
Junc J6	45.94	34.7	34.27
Junc Wpcanal	44.25	33.01	32.58
Junc J7	41.94	30.7	30.27
Junc Wp4MehalEla	28.42	17.18	16.75
Junc J8	37.44	26.2	25.77
Junc Wp5Guchi	37.34	26.1	25.67
Junc J9	28.76	17.52	17.09
Junc WpMehalKebela	26.55	15.31	14.88
Junc J10	24.34	13.1	12.67
Junc WpJegeresefer	22.37	11.13	10.7
Junc J11	92.3	81.06	80.62
Junc Wp9Cipa	83.96	72.72	72.29

Junc Wp8CipaSchool	15.82	4.58	4.15
Junc J12	143.49	72.72	36.5
Junc J13	138.27	67.5	31.28
Junc Wp23Shoya	132.95	62.18	25.96
Junc Wp25Sere	138.54	67.77	31.55
Junc J14	152.67	65.78	37.32
Junc Wp15Olafino	161.91	75.02	46.56
Junc J15	148.66	13.81	8.57
Junc J25	167.7	32.86	27.62
Junc WpEdget	172.85	38.01	32.77
Junc J26	187.25	52.41	47.17
Junc Wp18Angafa	148.69	13.84	8.61
Junc J27	185.28	50.44	45.2
Junc Wp27Suredenba	168.78	33.93	28.7
Junc J28	167.81	32.96	27.73
Junc Wp34Pode	158.53	23.69	18.45
Junc Wp29Tekele	151.69	16.84	11.61
Junc WpKolshobo	154.03	19.18	13.94
Junc J30	185.06	50.21	44.97
Junc WpShola	146.78	11.94	6.7
Junc J31	191.18	56.33	53.82

Junc Wp28Zewde	178.68	43.83	41.33
Junc Wp33Cherenet	173.08	38.23	35.73
Junc J33	146.38	11.54	13.18
Junc WpTawla	145.97	11.13	12.77
Junc J34	149.09	14.25	22.27
Junc Wp9Farecho	132.21	-2.64	5.39
Junc J35	142.65	7.81	15.83
Junc J37	144.5	9.65	17.68
Junc WpKulia	130.94	-3.91	4.12
Junc J38	151.47	16.62	24.65
Junc Wp30lalana	106.4	-2.32	5.7
Junc J16	3.67	3.67	3.63
Junc Wpshoyaketena2	8.18	8.18	8.15
Junc J17	58.37	58.37	58.34
Junc WpOlolo	69.31	69.3	69.27
Junc J18	35.21	35.21	35.17
Junc Wp20Gulo	27.61	27.61	27.58
Junc Wp23Longena	-1.73	-1.73	9.83
Junc J19	33.16	33.16	33.12
Junc WpAbelaZegre	19.26	19.26	19.22
Junc J21	33.34	33.34	33.31

Junc J22	28.18	28.18	28.14
Junc Wp22longena	13.17	13.17	13.13
Junc J23	21.76	21.76	21.72
Junc Wp24Girara	9.64	9.64	9.61
Junc Wp25MehanaMeka	8.38	8.38	8.35
Junc Wp35Mareka	84.93	-49.92	9.1
Junc 74	33.95	34.03	34.02
Junc WpAnka	3.82	3.81	3.78
Junc J20	19.6	19.59	19.56
Junc WpBossawanch	55.66	44.42	43.99
Junc WpBoho	167.78	32.94	40.96
Junc J36	165.19	30.34	38.37
Junc WpGefetaKebele	58.98	-75.86	36.59
Junc WpGututola	11.37	11.45	11.43
Junc J29	168.24	33.4	28.16
Junc J3	26.5	25.64	20.5
Junc J24	9.51	9.51	9.48
Junc Wp(Korke)	-16.36	-16.37	14.94
Junc Jn1	179.85	45.01	47.75
Junc Jn2	91.94	80.7	80.27

Annex J: Likimse-Abela Water Supply System Network Table - Nodes at 8:00 Hrs