Jimma University

Jimma Institute of Technology

School of Graduate Studies

School of Civil and Environmental Engineering

Chair of Hydraulic and Hydrology Engineering

Evaluating Hydraulic Performance of Urban Water Supply Distribution System: A Case Study of Woreta Town South Gondar Zone, Amhara Region, Ethiopia

A Thesis Submitted to the School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Hydraulic Engineering

By

Fentaw Mekuria

February 2016

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By

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February 2016

Jimma, Ethiopia

DECLARATION

This thesis is my original work and has not presented for a degree in any other university.

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Candidate	Signature	Date

This thesis has been submitting for examination with my approval as a university supervisor.

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Advisor	Signature	Date

This thesis has been submitting for examination with my approval as a university supervisor.

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Co-Adviser	*Signature*	*Date*

ABSTRACT

Water supply distribution systems facing challenges to deliver quality and adequate quantity of water with the require velocity and pressure. These challenges come from rapid population growth, migration to urban cities, scarce water resource, poor design, poor operation and poor maintenance of the system. These also expose infrastructural decay, inefficient distribution system and water demands on pipeline increasing every day. Woreta, which found northern Ethiopia in south Gondar zone Amhara Region, is rapidly grown commercial town. To assure this growth a potable, reliable and adequate quantity water supply is needed. Then evaluating the hydraulic performance of water supply distribution system of the town was paramount important to upgrade the distribution system or add new resource to meet current and future demand. The main objective of this research was to evaluate hydraulic performance of water distribution system based on widely accepted key standard. Evaluating hydraulic performance of water distribution system is useful to identify the gap between supply and demand, as well as the deficiency of hydraulic parameters in water supply distribution system. To achieve the main objective of the research three specific objectives were set. These were evaluating the water supply versus demand, assess water in the distribution system and simulate hydraulic parameter with EPANET. To strength, this research previous works were reviewed, which were related to the studied area. Primary and secondary dates were collecting to achieve the objective of this research. After the necessary data collected data analysis, model built and simulation were done. Hydraulic network Simulation with EPANET was carrying out because of to track water supply, water demand, flow velocity and pressure at each node. Both single and extended period simulation carried out for one ward. According to the research result, the following were found. Woreta population in 2011, 2012, 2013, 2014 and 2015 was 32757, 34165, 35634, 37167and 38765 respectively. Woreta town was exposed for sever water shortage in 2011-2015. Because of supply (l/p/d) of each year were below unit demand (20l/s). Water loss (percentage) in 2011-2015 was 25.1, 25.28, 26.18, 28.51, and 29.07 respectively. Additionally, water supply service coverage in Woreta town was 78.9%. 21.1% of Woreta town population was lived without water supply service connection. During steady state simulation water supply was fail at each node to meet nodal demand. Moreover, at extend period pressures at 12 out of 27 junctions were higher than recommended pressure (60m) and the flow velocity 30 out of 36 links were below minimum allowance velocity (0.6m/s). Based on the research outputs, the following activities were recommended. These were population should be clearly know, new water source should be add to meet current and future demand, water loss should be combat, water quality analysis should be done and upgraded existing water supply system into three clearly defined pressure zone, such as for elevated area, commercial(tall building) area and lower area.

Key words: Water Distribution System, Water Demand, Water Loss, Water Supply, EPANET, Hydraulic Performance and Hydraulic Parameter

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ACRONYMS

ADB	Asian Development Bank
ADSWE	Amhara Design Supervise and Water Work Enterprise
ASoCE	American Society of Civil Engineers
AWRDB	Amhara Water Resource and Development Biro
AWWA	American Water Works Association
CSA	Central Statistics Agency
DUoT	Delft University of Technology
ECSC	Ethiopian Civil Service College
GTP	Growth and Transformation Plan
GWP	Global Water Partnership
IJoESE	International Journal of Earth Sciences and Engineering
IJoSK	International Journal of Scientific Knowledge
NAoS	National Academy of Sciences
MoWIE	Ministry of Water Irrigation and Electric
UoASNS	University of Applied Science Northwestern Switzerland
WB	World Bank World Development Report
WDS	Water Distribution System
WFEDO	Woreta Finance Economics Development Office
WHO	World Health Organization
WWSS	Woreta Water Supply Service

Abbreviation

App.Fig	Appendix Figure
App.Table	Appendix Table
D	Diameter

DCI	Ductile Iron
DCI	Ductile Cast Iron
DN	Nominal Diameter
E.C	Ethiopian Calendar
Fig	Figure
GI	Galvanized Iron
gpm	gallon per minute
HL	Head Loss
JU	Junction
Km	Kilometer
L	Length
Lps	Liter per Second
Lpcd	Litter per Capital per Day
M	Meter
m^3	Cubic Meter
m/s	Meter per Second
mm	Millimeter
PI	Pipe
PU	Pump
Q	Discharge
RE	Reservoir
St	Steel
TA	Tank
UFW	Unaccounted For Water
V	Velocity

1. INTRODUCTION

1.1. Background

A safe, reliable, affordable, and easily accessible water supply is essential for good health and development(WHO, 2009; Hunter et al., 2010; WHO, 2011; WHO & UNICEF, 2012). To transport potable water a good water distribution system is needed. Then water supply systems are the most important public utility(Swammee et al., 2008; Elsheikh et al., 2013). The practice of transporting water for human consumption has been went several millennia. The most extensive water distribution systems in ancient times were the roman aqueducts which built in 312 B.C, and pressure pipe built 3,500 years ago, those conveyed water long distances with gravity through a collection of open and closed conduits(Walski et al., 2003 and Josi et al. 2014).

In Ethiopia, the first pipeline was lay from Entot Mountain to the old palace or MenelikII palace to distribute the developed spring water by storing in the reservoir and distributed it by gravitational force to the palace and higher royal official's residence. After Geffersa and Legadadi dams built to distribute potable water to Addis Ababa city, the system of pumping water is by relay method from water plant to different parts. After 1971, pipe water system used throughout the country towns(Behailu, 2012).

Woreta town was got its first piped water service in 1980. However, this project now a day no gives service for Woreta town but it give service for Woreta agricultural collage. The water supply distribution system, which give service for Woreta town at the movement was constructed in 2001(ADSWE, 2010).

A water distribution system is a pipe network, which delivers water from single or multiple supply sources to consumers. A water distribution system consists of complex interconnected elements such as pipes, nodes, pumps, control valves, storage tanks, and reservoirs (Rossman, 2000). Additionally, water distribution system has three main components, which are water sources and intake works, treatment works and storage, transmission and distribution (Swammee et al., 2008). Depending upon the methods of distribution, the distribution system is classified gravity system, pumping system and dual system(Leirens et al., 2010; Ramesh et al., 2012).

The main objective of water supply system is deliver potable water to satisfy combination of domestic, commercial, industrial, and firefighting demands at required time with sufficient hydraulic performance(Zyoud, 2003; M. & Babelb, 2014). However many of the developing countries, drinking water supplies are inadequate to meet consumers' demands because of water schemes designed to continuous supply but they operated as intermittent systems(Gottipati & Nanduri, 2014). This was due to the rapid increase in population, urbanization, high pressure on the existing infrastructure, erratic power supply that hampers continued operation of the water supply system, which usually results in infrastructural decay; there by disrupted the efficient water distribution system(Bello & Tuna, 2014). Moreover, water supply networks regularly experience pressure drops and interruptions of water supply when there is an unexpected increase in water demand and transport potable water over vast geographical areas to millions of consumers(Leirens et al., 2010; Ehlers et al., 2006). Therefore, Computation of flows and pressures is crucial to provide water to the consumers and has paramount importance in designing a new water distribution network or expanding the existing one (Saminu & Sagir, 2013).

Water modeling is becoming an increasingly important part of hydraulic engineering. One application of hydraulic modeling is pipe network analysis. Using programmed algorithms repeatedly to solve continuity and energy equations, computer software can greatly reduce the amount of time required to analyze a closed conduit system. Such hydraulic models can become a valuable tool for cities to maintain their water systems and plan for future growth(Datwyler, 2014). Water distribution modeling is the latest technology began two millennia ago when the Minoans constructed the first piped water conveyance system(Atiquzzaman, 2004; Elsheikh et al., 2013).

Today water distribution modeling is a critical part of designing and operating water distribution systems that are capable of serving communities reliably, efficiently, and safely, both now and in the future (Walski et al., 2003). The advent of the computers significantly enhanced our ability to analysis flow. Computer models for analyzing pipe flows and pressures in water distribution networks used throughout the world is essential tools for the efficient operation and improvement of very complex systems. Most analysis and design problems do not have a single correct answer. The design may begin the solution process by developing a mathematical model of the physical system(Ali, 2000).

Modeling the water flows, pressure heads and quality in urban water distribution system was a challenging exercises hydraulic complexity and stochastic inputs to the system. Increasing hydraulic complexities associated with water distribution systems necessitated precise estimation of flows and pressures in various parts of the system. Because of solution of single pipe flow problem was no longer adequate. Therefore, analyzing the all most entire water distribution network was gave birth to water distribution network (Oyelowo, 2013; Henshaw & Nwaogazie, 2015).

In this study the Hydraulic, model EPANET version 2.0 was used for evaluating the hydraulic performance of urban water supply system to the studied area. Because EPANET was open-structured, economical free accessible, simple operation, worldwide computer modeling program and due to its hydraulic and quality capability that performs steady and extended period simulation of hydraulic and water quality behavior within pressurized pipe networks. A network consists of pipes, nodes (pipe junctions), pumps, valves and storage tanks or reservoirs. EPANET tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprised of multiple time steps(Rossman, 2000; Ramesh et al., 2012).

The hydraulic network modeling process involved data collection, system operation and monitoring, network schematization, assign parameters model building, model testing, the analysis of the problem and recommended solution. Generally adopting computer models to design water distribution networks such as EPANET, one will have enough results including number of graphs, tables, and caparison figures as well for most favorable decision-making (Walski et al., 2003; Ramesh et al., 2012; Umar et al., 2012).

In the present study, both single period and extended period simulation were carryout, for hydraulic parameters, which were simulated pressure, flow rate and velocity. Steady-state simulations represent a snapshot in time and used to determine the operating behavior of a system under static conditions. This type of analysis can be useful in determining the short-term effect of fire flows or average demand conditions on the system(Datwyler, 2014). Extended period simulations (EPS) used to evaluate system performance over time. This type of analysis allows the user to model tanks filling and draining, regulating valves opening and closing, and pressures and flow rates changing throughout the system in response to varying demand conditions and automatic control strategies formulated by the modeler(Walski et al., 2003; Ormsbee & Lingireddy, 1997; Datwyler, 2014).

1.2. Statement of the Problem

Water is essential to sustain life so that a satisfactory supply must be available to all. Therefore improving access safe drinking water can result in tangible benefits to health (Hunter et al., 2010; WHO, 2011). However, it is widely recognized that many countries in the world are entering an era of severe water shortage and about a billion of people in developing countries have not safe, reliable, affordable, easily accessible and sustainable water supply (WHO, 2009; Hunter et al., 2010; WHO, 2011). In developing countries, urban water distribution systems designed for continuous water supply at adequate pressure and flow however often operated intermittently. Because of due to the rapid increase in population, urbanization make high pressure on existing infrastructure, which usually results in infrastructural decay, there by disrupted the efficient water distribution system. Moreover, urban water supply networks are large-scale systems that transport potable water over vast geographical areas to millions of consumers. As a result, water supply networks regularly experience pressure drops and interruptions of water supply. When there is an unexpected increase in water demand. Then evaluating hydraulic performance for safe and efficient operation of these networks is crucial(Leirens et al., 2010; Gottipati & Nanduri, 2014).

Woreta area is the only place in Ethiopia where rice is grown widely and calls agro-economic zone. and it is intermediate town between Bahir-Dar and South Gondar and North Gondar(Gebremedhin et al., 2009). Due to the reason Woreta was attract and became home of different banks, education sector, government and non-governmental organizations. Moreover, in Woreta many people immigrated for searching job and many regional conference set. This increase population growth which make stress and exposed for sever water shortage in the town. In other hand, Woreta was considering one of the rapidly grown commercial towns and has good prospect for development. Hence, to assure its progress a potable, reliable and adequate water supply system needed(ADSWE, 2010).Therefore, evaluating hydraulic performance using hydraulic network modeling software was key thing to know capacity of utility, deficiency of hydraulic parameter and to expansion or renew Woreta existing the water distribution system.

1.3. Objectives of the Study

1.3.1. Main Objective

The main objective of this study was evaluating hydraulic performance of urban water supply distribution system based on existing safe and secure water demand with widely acceptable key standard.

1.3.2. Specific Objective

- ❖ To evaluate water supply versus water demand
- ❖ To assess water loss in the distribution system
- ❖ To identify the deficiency of hydraulic parameters with EPANET

1.4. Research Question

- ❖ Is the existing water supply satisfied current demand of Woreta town?
- ❖ Is there water loss in water distribution system before reach customer?
- ❖ Is Woreta water supply system has good hydraulic parameters?

1.5. Scope of the Study

The scope of this research include, evaluating water supply versus demand, water loss and hydraulic network modeling existing water supply system. Analysis of water balance include evaluating water supply, water demand and water loss in existing water supply based on production and consumption data and projection of total population data in studied area. Hydraulic network modeling includes nodal-demand allocation, population projection for each node, evaluating flow and pressure in distribution system, evaluating nodal demand versus nodal supply after collection of necessity raw data. The output of water balance analysis and hydraulic network modeling evaluated based on widely accepted by MOWIE design standard. In this study population forecast with acceptable formula that CSA use, water that delivers to community has assumed quality water and all population use only from existing Woreta water supply system, the demand of livestock assumes from river or other source and un-meter water loss and illegal water connection are considering as loss.

1.6. Significance of the Study

The significance of this research was to evaluating hydraulic performance of Woreta town water supply system. This used to know the gap between supply and demand, to assess water loss and to identify the deficiency of hydraulic parameters in distribution system. Based on research output, basic solution recommended to improve hydraulic performance of the distribution system and filled gap between supply and demand in the studied area. According to the research output, there were shortage of water, high water loss, low velocity and high pressure. The significant of this research was showing this problem and recommended solution to solve the problem. Therefore became improve the performance of water distribution system and fill the gap of water supply to meet current and future demand.

1.7. Limitation of the Study

During the time of this study, there were challenges to calibrate hydraulic networks and allocated nodal demand. All Pressure gages in distribution system, were not work to measure pressure and there were no enough bulk meter to measure water loss at field surveying. This was difficult, to calibration of hydraulic network model to compare simulated and field survey results. The other problem was nodal demand allocation. Because there were no clearly known the population from each node. This was also challenge to allocate nodal demand.

2. LITERATURE REVIEW

2.1. General Concept of Water Supply

Safe and adequate supply of drinking water is a large crucial component of human life system. However, billions of people in the world have not access water today. Two third of this number of population is from the developing countries(WHO, 2009; Hunter et al. 2010; WHO 2011). The Provision of safe and adequate water supply to population have large effects. Such as on health, productivity, quality of life, reduction of poverty and ensure sustainable socioeconomic development (MOWIE, 2015). The pace of urban development is increasing of urban water demand due to urban population growth and increasing of urban living facilities requiring high water consumption placing a challenge on the demand side of urban water supplies. In addition to construction of new and expansion existing urban water supply schemes, both these challenges should have to be properly addressed in Operation and Maintenance too (Laura, 2006; MoWR, 2005).

2.2. Element of Water Distribution System

Urban water supply networks are large-scale systems that transport potable water over vast geographical areas to millions of consumers. These systems consist of elements, such as pipes, pumps, valves, storage tanks, reservoirs, meters, fittings, and other hydraulic appurtenances needed to carry water from source of potable water to the various point of use(Laura, 2006; Leirens et al., 2010; Elsheikh et al., 2013).

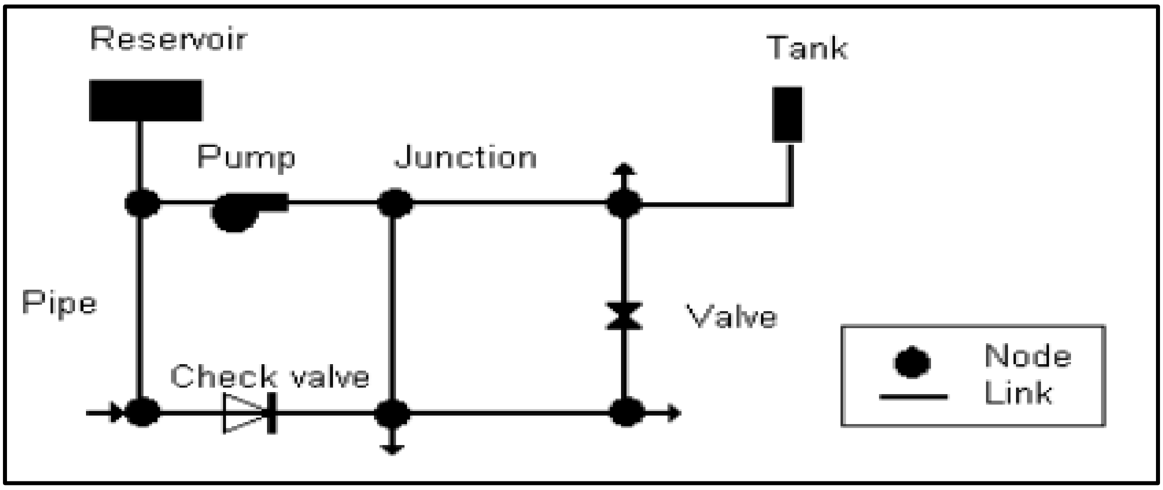

Fig.2.1: Physical Components of Water Distribution System

2.2.1. Pipes

Pipes are main components of water distribution system and found in different lengths, materials and diameters laid down in the network. These are mainly grouped into three transmission lines, distribution lines and service pipes. The transmission line is the pipe between the source and the storage elements; it carries water from source or pump station to the storage tank. while the capacity is enough for both serving the consumers and carrying excess water to the storage tank. Also it delivers water from storage tank when the source or pump is not able to meet the demand. The distribution lines deliver water to the pressure zone and distribute the water to the service nodes. On the other hand, service pipes are the pipes that mainly send water to the consumers(Ramana et al., 2015).

2.2.2. Pump

Pump is hydraulic machine that adds energy to the water flow by converting the mechanical energy into potential energy to overcome the friction loses and hydraulic grade differentiations with in the system. The pump characteristics are presented by various performance curves such as, power head and efficiency requirements that are developed for the friction rate. These curves are used in the design stage to find out the most suitable pump for the system. A booster pump stations designed where distribution system areas remote from pumping stations, high rise building areas where normal pressure is inadequate, localized area of higher elevation or extension to existing distrbution system where the cost of additional elevated storage is prohibitive (Misirdali, 2003).

2.2.3. Storage Facility

A storage facility provides a reservior in which the inflow and outflow of water can better match the hourly cunsumer demand and can be a supply source during emergency situtions such as interruptions in the normal supply service or high demand for fire fighting. The maximum and minimum elevation of water in the tank determine the pressure in the distribution system and should be designed accordingly. The required volume determines the surface area of the tank which is based on daily use and fire flow demand. Reserviors should be located within or adjacent to load centers (areas of high demand) of the distribution grid to meet water demands those areas without causing high velocities and head losses in the distribution mains(MoWR, 2006; HQDoA, 1986).

Where ground elevation are relatively uniform, an elevated tank will be considered to maintain pressure in insted of ground storage facilities where practical. The height of the tank will be determined from the topography of the area served, height of the buildings and the pressure losses in the distribution system. In addition,altitude valves, check valves and shut off valves are necessary to control the level of water in the tank and to provision or isolate portions of the distribution system during emergencies. Ground level storage can consist of steel standpipes and steel or concrete ground storage reserviors. These are to be designed where there is sufficient difference in ground elevation to maintain adequate pressure in the distribution system. Concrete reserviors can be designed for any size system. If the differences in natural ground elevations is insufficient to maintain pressures, booster pumps may be required in conjunction with ground storage to increase system pressure(HQDoA, 1986).

2.2.4. Water Meter

Water metre these are the devices which are installed on the pipes to measure the quantity of water flowing at a particular point along the pipe. The readings obtained from the meters help in working out the quantity of water supplied and thus the consumers can be charged accordingly. The water meters are usually installed to supply water to industries, hotels, big institutions etc. metering prevents the wastage of purified water(Venkateswara, 2005).

2.2.5. Valves

Valves are an element that opens, throttles, or closes to satisfy a condition of flow and the behavior of it is determining by the upstream and downstream conditions. Types of valve are listing below.

Pressure Reducer Valve: Pressure reducer Valve throttles to prevent the downstream hydraulic grade from exceeding a set value. If the downstream grade rises above the set value, the pressure reduce valve will close. If the head upstream is lower than the valve setting, the valve will open fully.

Pressure Sustaining Valve: Pressure sustaining valves throttle to prevent the upstream hydraulic grade from dropping below a set value. If the upstream grade is lower than the set grade, the valve will close completely.

Pressure Breaker Valve*:* Pressure breaker valves use to force a specified pressure (head) drop across the valve. These valves do not automatically check flow and will actually boost the pressure in the direction of reverse flow to achieve a downstream grade that is lower than the upstream grade by a set amount.

Flow Control Valve*:* Flow control valves is use to limit the maximum flow rate through the valve from upstream to downstream. Flow control valves do not limit the minimum flow rate or negative flow rate (flow from the To Pipe to the From Pipe).

Throttle Control Valve: Throttle control valve uses as controlled minor losses. A throttle control valve is a valve that has a minor loss associated with it where the minor loss can change in magnitude according to the controls that are implementing for the valve.

General Purpose Valve: General-purpose valve are used to model situations and devices where the flow-to-head loss relationship is specified by you rather than using the standard hydraulic formulas. General-purpose valve can used to represent reduced pressure backflow prevention valves, well drawdown behavior, and turbines (Rossman, 2000; Water CAD User's Guide, 2003).

2.3. Components of Urban Water Supply Systems

The Main components of drinking water distribution systems are water sources and intake works, treatment works and storage, transmission mains and distribution network. The common water sources are rivers, lakes, springs, and manmade reservoir and groundwater sources. The intake structures and pumping stations constructed to extract water from these sources(Swammee et al., 2008).

The raw water is transporting to the treatment plants for processing through transmission mains and it stored in clear water reservoirs after treatment. The clear water reservoir provides a buffer for water demand variation and design for average daily demand. When water is carrying over long distances through transmission main, if pressure head maintained by pump called pumping main. On the other hand, if the flow in a transmission main is maintains by gravitational potential available because of elevation difference, is calling gravity main. There are no intermediate withdrawals in a water transmission main. Similar to transmission mains, the flow in water distribution networks is maintains either by pumping or by gravitational potential(Laura, 2006). Transmission mains and distribution network are calling water supply delivery system. The distribution network delivers water to consumers through service connections (Leirens et al., 2010).

2.4. Methods of Water Supply System

Water can delivers to customer continuous supply system or intermittent supply system.

2.4.1. Continuous System

Continuous water supply system is the best system and water is supply for all 24 hours and 7 days in a week. This system is possible when there is adequate quantity of water for supply. In this system, supply water is always available for firefighting. In addition, due to continuous circulation, water always remains fresh. In this system less diameter of pipes are required and rusting of pipes will be less. Losses will be more if there are leakages in the system(Sharma, 2008; Venkateswara, 2005)

2.4.2. Intermittent System

When adequate quantity of water is not available, the supply of water is dividing into zones and each zone is supply with water for fixed hours in a day or on alternate days. As the water is supply after intervals, it is call intermittent system. The system has disadvantages such as Pipelines are likely to rust faster due to alternate wetting and drying, increases the maintanance cost, polluted water through leaks during non-flow periods and more wastage to collect fresh water at each supply time. In this water supply system the high-elevated area, get adequate pressure by dividing the city in zones. The repair work can easily do in the non-supply hours (Anden & Kelkar, 2007; Behailu, 2012).

2.5. Methods of Water Distribution

For efficient distribution it is required that, the water should reach to every consumer with required rate of flow. Therefore, some pressure in pipeline is necessary, which should force the water to reach at every place. The methods of distribution system classified as gravity system, pumping system and combined system(Behailu, 2012).

2.5.1.Gravity System

When some ground sufficiently high above the city area is available, this can best utilized for distribution system in maintaining pressure in water mains. This method is also much suitable when the source of supply such as lake, river or impounding reservoir is at sufficiently higher than city. The water flows in the mains due to gravitational forces. As no pumping is required, therefore it is the most reliable system for the distribution of water

2.5.2. Pumping System

Constant pressure head had maintained in the system due to direct pumping into mains. Rate of flow cannot vary easily according to demand unless numbers of pumps are operating in addition to stand by ones. Supply affected during power failure and breakdown of pumps. Hence, diesel pumps standby also in addition to electrical pump. During fires, the water pumped in required quantity by the stand by units.

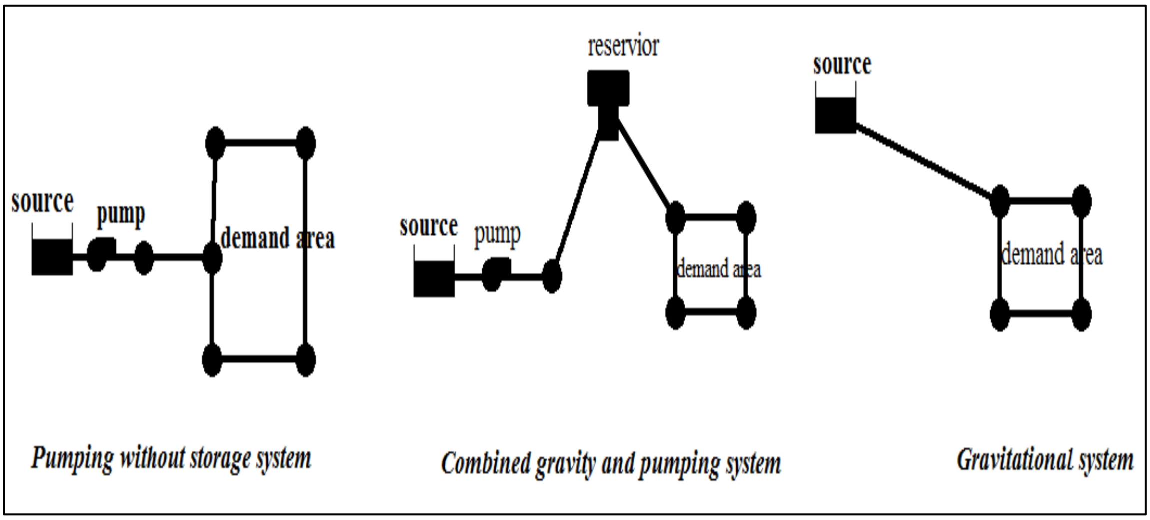

Fig.2.2: Water Distribution Methods

2.5.3. Combined System

The pump connected to the mains as well as elevated reservoir the system also known as dual system. In the beginning when demand is small the water is stored in the elevated reservoir, but when demand increases the rate of pumping, the flow in the distribution system comes from the both the pumping station as well as elevated reservoir. This system is more reliable and economical, because it requires uniform rate of pumping but meets low as well as maximum demand. The water stored in the elevated reservoir meets the requirements of demand during breakdown of pumps and for firefighting (Venkateswara, 2005).

2.6. Water Losses in Distribution System

The volume of water lost between the point of supply and the customer meter due to various reasons. It can be express as the difference between system inputs volume, and authorized consumption, and consists of apparent and real losses. Apparent losses can be subdividing into unauthorized consumption, meter inaccuracies and data handling errors. Real losses are make-up of leakage from transmission and distribution pipes, leakage from service connections and losses from storage tanks (Jalal et al., 2008; Sharma, 2008; Swammee et al., 2008). Water losses occur in every water distribution network in the world. For economic and technical reasons, it has to be accepting that real water losses cannot eliminate. Nevertheless, there has been a large increase in the knowledge and development of state-of-the-art equipment, allowing us to manage water losses within economic limits(Stevens et al., 2004; Cunlifffe, 2014).

2.7. Layouts of Pipe Networks

The configuration of the distribution system is determined primarily by size and location of water demand, street patterns, location of treatment and storage facilities, degree and type of development of the area, and topography(Misdial, 2003). Generally, two patterns of distribution main systems commonly used are:

2.7.1. Branching Or Dean End System

Branched configurations are also providing depending upon the general layout plan of the city roads and streets. The rural water networks have branched configurations.

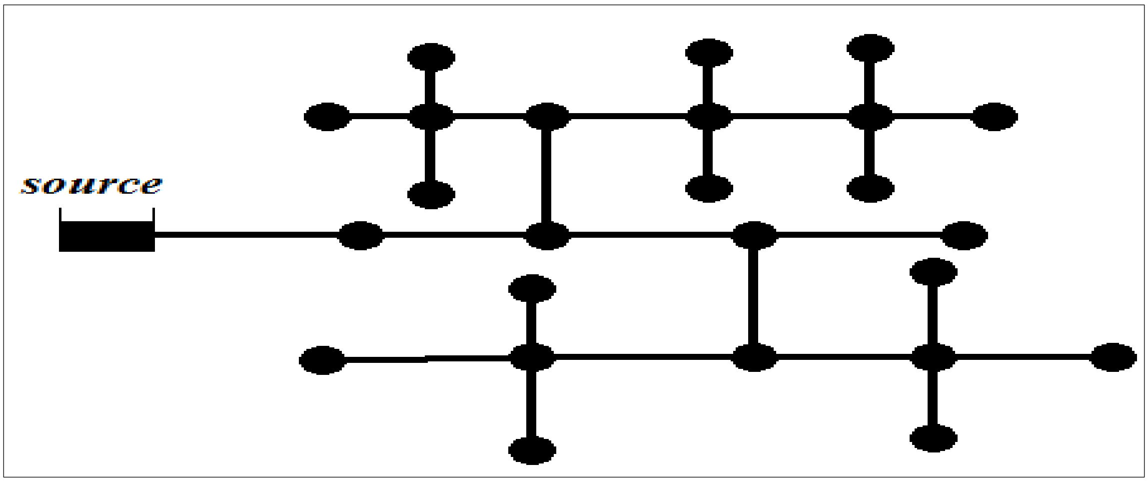

Fig.2.3: Branching System

2.7.2. Looped System

Urban water networks have mostly looped configurations. The looped system has the hydraulic advantage of delivering water to any location from more than one direction, thereby avoiding dead ends (HQDoA, 1986).

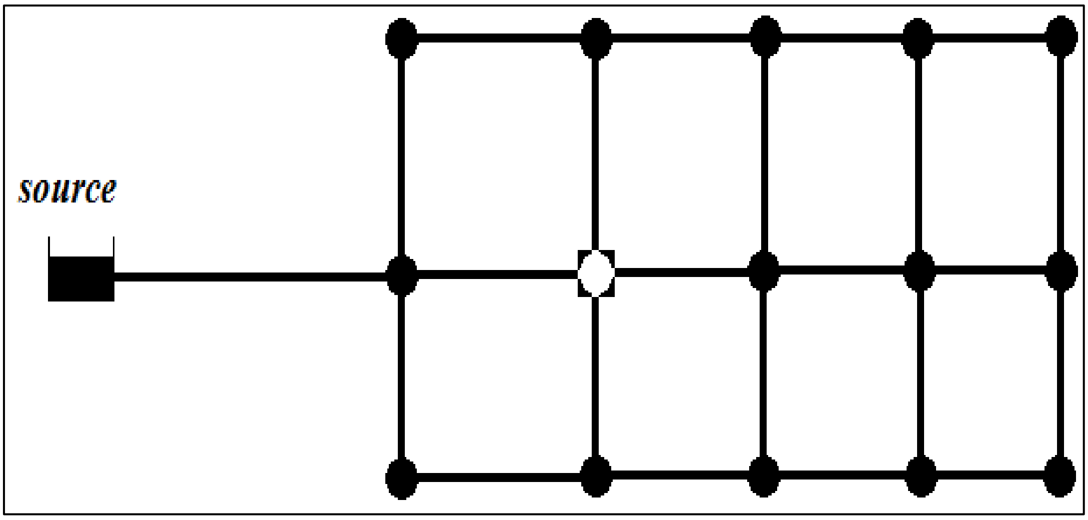

Fig.2.4: Looped System

The use of looped feeder system is preferable because of the looped feeder supplies water to the area of greatest demand from at least two directions. Looped configurations are preferred over branched configurations. Then looped feeder system should be using for water distribution systems whenever practicable. Water distribute into different area is dividing into zones, and each zone serving with a separate distribution reservoir and a separate distribution main(Gottipati & Nanduri, 2014; WHO, 2014).

2.8. Design of Hydraulic Network

Water distributions systems are design adequately satisfy the water requirements for a combination of are residential, commercial, industrial, public water uses, fire demand and unaccounted for system losses. The essential parameters for network sizing are the projection of residential, commercial and industrial water demand, per capita water consumption, peak flow factors, minimum and maximum pipe sizes, and pipe material and reliability considerations(Swammee et al., 2008; Venkateswara, 2005).

2.8.1.Estimation of Water Demand

The estimation of water demand for the sizing of any water supply system or its component is the most important part of the design methodology(Venkateswara, 2005; Sarbu, 2011; Arunkumar, 2015).

Population Forecasting: The average percentage of the last few decades/years is determined, and the forecasting has done on the basis that percentage increase per decade/year will be same(Alemayehu, 2010). Thus, the population at the end of n years given:

$Pn = Pox(1 + r)^n$……………………………………………….………………… (2.1)

Where:

R=annual growth rate of the population

Pn=population at time n in the future

Po = present population

n = periods of projection

Residential Water Demand: Residential water demand includes the water required in residential buildings for drinking, cooking, bathing, lawn sprinkling, gardening, sanitary purposes, etc. In most countries, the residential demand constitutes 50 to 60% of the total demand(Venkateswara, 2005; Alemayehu, 2010).

Institution and Commercial Demand: Universities, Institution, commercial buildings and commercial centers including office buildings, warehouses, stores, hotels, shopping centers, health centers, schools, temple, cinema houses, railway and bus stations etc. comes under this category. Commercial use of water amounts to about 10 to 30% of total consumption(Belay, 2012).

Industrial Water Demand: The quantity of water demand for industrial purpose is around 20 to 25% of the total demand of the city(Alemayehu, 2010).

Public Use Demand: It is for parks, public buildings, and streets contribute to the total amount of water consumed per capita. Fire demands are usually included in this class of water use. The total quantity of water used for firefighting may not be large, but because of the high rate at which it is required, it may control the design of the facilities. About 5 to 10% of all water used is for public uses(Alemayehu, 2010; Belay, 2012).

Firefighting Demand: The quantity of water required for firefighting is generally calculating by using different empirical formulae(Alemayehu, 2010). Ethiopia National Board of Firefighting calculates fire demand:

QF = $231.6\sqrt{P}(1 - 0.01\sqrt{P})$ ………………....……………………………………… (2.2)

Where, QF = fire demand (m3/hr.);

P = Population in 1000’s

Per Capital Demand: the total quantity of water required by various purposes by a town per year and 'p' is population of town, and then per capita demand will be (Venkateswara, 2005):

$$P_{cd}\,\frac{Q}{P*365} \quad (2.3)$$

Where:

Pcd=per capital

Q= Discharge

P= population

Factors affecting Per Capita Demand: The water demand varies from seasonally, daily, even hourly. The main factors affecting for capita demand are Climatic conditions, Size of community, Living standard of the people, Industrial and commercial activities, Pressure in the distribution system, System of sanitation, Cost of water, System of supply & Size of the city. The rate of water consumption increase in the pressure of the building and even with the required pressure at the farthest point, the consumption of water will automatically increase. This increase in the quantity is firstly due to use of water freely by the people as compared when they get it scarcely and more water loss due to leakage, wastage etc. (Swammee et al., 2008).

Peak Factor: The water demand is not constant throughout the day and varies greatly over the day. Generally, the demand is lowest during the night and highest during morning or evening hours of the day. Peak daily demand over a 12-month period required for the design of a distribution system upstream of the balancing storage calculate as during a 12-month period over average daily demand of the same period. Peak hour demand or maximum hour demand over a 24-hour period required for the design of a distribution system. Thus, the peak hour factor can define as the ratio of peak hour demand on peak day over average hour demand over the same 24 hours. Peak factor for a water distribution design can also estimate from the ratio of peak hourly demand on a maximum demand day during the year over the average hourly demand over the same period. Then demand factor calculated with below formula:

$$P_f = \frac{P_{hd}}{A_{hd}} \quad (2.4)$$

Where:

Pf = peak factor

Phd= Peak hour demand

Ahd = Average hour demand

$$P_{df} = \frac{P_{dd}}{A_{dd}} \text{...... ....} (2.5)$$

Where;

Pdf= Peak day factor

Pdd= Peak day demand

Add= Average daily demand

$$P_{hd} = A_{hd} X P_{hf} \text{...} (2.6)$$

Where:

Phd= peak hour demand

Ahd= average hour demand

Phf= peak hour factor

Table2.1: Demand Factor

Population	Maximum day demand factor	Peak hour factor
< 20,000	1.30	2.00
20,000 to 50,000	1.25	1.90
50,000 and above	1.20	1.70

Annual Average Demand: The annual average demand is average daily demand over a period of one year. Used for economical calculations and firefighting.

Q day-avg = Pd x p……………………………………………………………………. (2.7)

Where:

Pd=per capital demand

P= population

Maximum Day Demand: maximum day demand is the amount of water required during the day of maximum consumption in a year. It is important for design of water treatment plants and water storages.

Q day-max = Q day-avg X Mdf……………………………………….………………… (2.8)

Where:

Qday-max=maximum day demand

Qday-avg=average day demand

Mdf=maximum Day Factor

Peak Hour Demand: peak hour demand is the amount of water required during the maximum hour in a given day. It is Important for design of distribution systems.

Q peak-hour = Q day-avg x Phf……………………………………………. (2.9)

Fire Flow Rate: firefighting calculated from the following formula

$QF = 231.6\sqrt{P}(1 - 0.01\sqrt{P})$………………………………………………. (2.10)

QF=fire flow rate

P=population

Coincident Draft: Coincident is the sum of maximum daily demand and the fire demand.

$Qcd = 231.6\sqrt{P}(1 - 0.01\sqrt{P})$ + Q day-avg X Mdf…………………….……………. (2.11)

2.8.2. Water Losses

water losses due to defective (pipe joints, cracked and broken pipes, faulty valves and fittings), Losses due to, consumers miss use public tap, Losses due to unauthorized and illegal connections. It accounts about 10 to 15% of total consumption (Venkateswara, 2005; Belay, 2012). A figure of 15% generally regarded as good, and uneconomical to try to reduce.

Table2.2: Water Losses Percentage each Design Period

Start year	5 years	10 years	15 years	20 years
40%	35%	30%	27.5%	25%

Source=(MoWR 2006).

2.8.3. Base (Nodal) Demand

Although water utilities make a large number of flow measurements, such as those at customer meters for billing and at treatment plants and wells for production monitoring, dates are not usually compile on the node-by-node basis needed for modeling. The most common method of allocating baseline demands is a simple unit loading method. This method involves counting the number of customers [hectares of a given land use, number of fixture units, or number of equivalent dwelling units] that contribute to the demand at a certain node, and then multiplying that number by the unit demand [for instance, number of gallons (liters) per capita per day] for the applicable load classification. Two basic approaches exist for filling in the data gaps between water production and computed customer usage(Datwyler, 2014; Belay, 2012).

2.8.4. Water Supply Service Coverage

Population with access to water services, either as a domestic water connection or through public water points. Calculated as the population served (connections and public water points) divided by the total population living in the service area.

$$Coverage\ \% = [\frac{(B+(C*D))*E}{A} * 100] \quad \ldots \ldots (2.12)$$

Where:

A=Total population of the town,

B=Number of domestic customers,

C=Number of public water points,

D= Number of households using public water point

E= Average family size (CSA)

2.8.5. Hydraulic Parameters

Transmission Main: Rising and gravity transmission mains from source to distribution should be design for the maximum day demand, based on the design hours of water source operation. Storage facilities at the termination of the transmission main(s) should cater for the peak hourly flow in the distribution system. Where transmission or gravity mains involve working or static pressures that are higher than advisable in relation to pump capacities or pipe pressure ratings, and then break pressure tanks and/or booster stations should considered. No house connections should be made to transmission mains (MoWR, 2006).

Distribution Systems: The distribution network should design for the peak hourly demand. The minimum pipe size considered for primary and secondary networks should be DN 50.8 mm. Tertiary pipes may be below DN 50.8 mm but not below DN 25.4 mm. Large-scale networks may conceivably have a larger minimum diameter for primary and secondary pipes. Distribution systems planned with either one large diameter pipe suitable for the final planning horizon, or multiple smaller diameter pipes installed at various intermediate-planning horizons. An economic analysis should carry out to determine the cheapest solution (MoWR, 2006; Swammee et al., 2008).

Pressure Head: The minimum design nodal pressures prescribed to discharge design flows onto the properties. Generally, it based on population served, types of dwellings in the area, and firefighting requirements. AS it is not economic to maintain high pressure in the whole system just to cater to the need of few high-rise buildings in the area, the provision of booster pumps are specify. Moreover, water leakage losses increase with the increase in system pressure in a water distribution system. The static state pressures in pipelines must be less than the pipe nominal pressure rating. In the case of long mains where water hammer risk is expected, due attention must be given to the pipe material and a proper water hammer analysis carried out. The operating pressures in the distribution network shall be as follows:

Table2.3: Pressure Limit

Pressure head	Normal condition	Exceptional condition
Minimum	15m	10 m
Maximum	60m	70 m

Minimum water head envisaged where distribution pipes are close to reservoirs in terms of perhaps both location and elevation, and in small sections of the distribution system that would require a pressure reduce valve otherwise mean raising pressures generally to achieve a 15 m minimum pressure (AWRDB, 2012; MoWR, 2006).

Flow Rate: The main hydraulic parameters in water distribution networks are the pressure and the flow rate, other relevant design factors are the pipe diameters, velocities, and the hydraulic gradients. The distribution flow rate, design based on the maximum of day demand plus fire demand and maximum hour rate. Because of Velocity is directly proportional to the flow rate. For a known pipe diameter and a known velocity, the flow rate through a section can estimated. Low velocities affect the proper supply and will be undesirable for hygienic reasons that sediment formation may cause due to the longtime of retention(Zyoud,2003; Venkateswara, 2005; MoWR, 2006). The effect of the velocity on the diameters of pipe system can observed from the following equation:

$$V = \sqrt{\frac{4Q}{\pi D^2}} \quad \text{(2.13)}$$

Where:

D=diameter of the pipe (m)

Q: discharge (m3 /sec)

V: velocity (m/sec)

From the above equation, it is clear that the velocity increasing should decrease the diameter value. Water velocities maintained at less than 2 m/sec, except in short sections. Velocities in small diameter (<100m) pipes may need even lower limiting velocities. A minimum velocity of 0.6 m/sec can take, but for looped systems, there will be pipelines with sections of zero velocity. Experience shows that a pipe designed to flow at a velocity between 0.6 and 1.5 m/sec, depending on diameter, is usually at optimum condition. Short sections, particularly at special cases, e.g. at inlet and outlet of pumps, may be designed for higher velocities(MoWR, 2006; AWRDB, 2012; Datwyler, 2014).

Head Loss: Head loss relates to velocity and pipe roughness. The maximum head loss with therefore be governed by the maximum velocity criterion. Any internationally recognized formula may use in the hydraulic computations, with coefficients taken as follows: For Hazen-Williams (C-value):

Table2.4: Hazen William C-Factor

Type of pipe	Upvc	Steel	DCI/GI
New	130	110	12O
Existing	100-110*	90-110*	100-110*

Table2.5: Colebrook-White and Darcy-Weisbach C-Value

Type of pipe	Upvc	Steel	DCI/GI
New	0.25	0.85	0.55
Existing	1.35-0.85*	2.60-0.85*	1.35-0.85*

* Depending on age and condition

The above C- and mm- values are applicable to transmission mains and similar lengths of pipelines with few appurtenances. For distribution systems, it generally recognized that a C-value of 100, or 1mm for Colebrook-White/Darcy-Weisbach, universally used (MoWR, 2006).

2.8.6. Reservoirs

Operational reservoir(s) should be providing to command a distribution system, located at elevation(s) providing the required pressure for water flow within the system. They should have sufficient storage to cover the difference between hourly peak demand and actual supply from the source, firefighting demands if to allow for, and for a limited emergency volume in case of power breakdown, repairs or O&M activities. In order to provide for security of supplies above the need for balancing purposes it is recommending that the minimum total reservoir storage capacity be in the range of 30% to 50% of the average daily demand(MoWR, 2006).

2.8.7. Power Supply and Pumps

The design working capacity of pumps (duty point) determined taking into account the system requirement and the number of units working simultaneously. A mechanical flow meter (water meter) should be installed on the outlet of a pumping station (after the manifold)(AWRDB, 2012). Maximum flow velocities for pumping systems will be as follows:

- Discharge flange: unlimited,
- at inlet branch: 2m/sec,
- at outlet branch: 3.5 m/sec,
- at inlet manifold:1.2 m/sec,
- at outlet manifold: 3 m/sec,
- In riser pipe from submersible pump to borehole head: 2 m/sec

Borehole pump installation should have an arrangement for measuring the water level in the tube well (dip tube). There must be a low water level protection device for the pump-motor set. Where multiple boreholes are needed for any particular scheme, consideration will be given to providing 50% stand-by borehole capacity, fully equipped, depending on the vulnerability of the scheme(MoWR, 2006).

2.8.8. Operation and Maintenance

Urban water supply networks are large-scale systems that transport potable water over vast geographical areas to millions of consumers. As a result, safe and efficient operation and maintenance of these networks is crucial. As the utility existed to serve the demand of the urban community, it is very challenging to satisfy the rapidly growing development activities of the community, which has direct relation with water supply. Intended to encourage the utility to handle between its operation and maintenance activities rehabilitation need on the water supply service, which has to be planned and achieved to diminish shortfalls in the system (Leirens et al., 2010; MoWR, 2012). Because of factors that could cause a water distribution system to lose its hydraulic flow pattern such as changes in flow and pressure caused by poor operational controls of valves and pumps and impacts of repairs and maintenance(WHO, 2014).

2.9. Hydraulic Analysis of Water Distribution System

The flow hydraulics in water supply distribution system govern the basic principles of flow such as continuity equation, equations of motion, and Bernoulli's equation for close conduit. Another important area of pipe flows is to understand and calculate resistance losses and form losses due to pipefittings (bends, elbows, valves, enlargers and reducers), which are the essential parts of a pipe network. Suitable equations for form losses calculations are required for total head-loss computation as fittings can contribute significant head loss to the system (Swammee et al., 2008; Elsheikh et al., 2013).The continuity equation for steady flow in a circular pipe of diameter D is

$$Q = \frac{\pi}{4} D^2 xV \qquad (2.14)$$

Where:

V = average velocity of flow, and Q =volumetric rate of flow

2.9.1. Conventional Method

The most common conventional method (not using computers) that is using in designing hydraulic networks is the Hardy Cross algorithm method. It involves iterative trial and error. Now a day, manual computation for hydraulic network analysis is only acceptable when applied to systems with only a single pipeline or branched network with no loop. For networks with loops, it is highly recommended to use the more accurate, fast and convenient network modeling computer software (Ormsbee & Lingireddy, 1997; Walski et al., 2003; Atiquzzaman, 2004).

2.9.2.Hydraulic Network Modeling Software

Computer models have become an essential tool for the management of water distribution systems around the world. The models for analyzing and designing water distribution systems have been available since the mid-1960s even if many advances work had done with regard to the sophistication and application of this technology. The primary reason for the growth and use of computer model has been the availability and widespread use of the microcomputer (Ormsbee & Lingireddy, 1997). Many methods were using in the past to compute flows in network of pipes such methods range from graphical analogies and finally to the use of mathematical models to find the hydraulic flow and head relationships as well as the resulting water quality concentration(Saminu & Sagir, 2013).

Hydraulic modeling of water distribution systems can allow to determine system pressure and flow rates under a variety of different conditions without having to go out and physically monitor your system (Dawe, 2000a). There are numerous purposes for using a computer model to simulate the flow conditions within a system. a model can be employed to ensure adequate quantity and quality portable water to community, evaluate planning and design alternatives, assess system performance, verified operating strategies for better management of the water infrastructure system, perform vulnerability studies to assess risks that may be presented and affect the water supply. For this purposes, a model is constructed in which data describing network elements of pipes, junctions, valves, pumps, tanks, and reservoirs are assembled in systematic manner to predict pipe flow and junction hydraulic grade lines(HGL) or pressures within a water distribution system(Dawe, 2000; Water CAD User’s Guide, 2003; Atiquzzaman, 2004).

2.9.3. Selection of Hydraulic Modeling software

Computer models are significant investments for water companies.to ensure a good invest meant return and correct use of the models, the models must be capable of correctly simulating flow conditions encountered at the site. Pipe network analysis mathematical models become increasingly accepting, within the water industry as a mechanism for simulating the behavior of water distribution systems. The selection of a particular model and the setup of a model schematization determine depends on the research question at hand, the behavior of the system, the available time and budget and future use of the model. The research question and the behavior of the water system determine the level of the model schematization. The time scale of the dominating processes and the spatial distribution of the problem are key elements in the selection of a model(Rossman, 2000; Dawe, 2000b; Ramesh et al., 2012).

2.9.4. Hydraulic Modeling of Water Distribution Network

EPANET is a computer program that performs steady and extended period simulation of hydraulic and water quality behavior within pressurized pipe networks. EPANET tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprised of multiple time steps (Rossman, 2000). Main principle of network analysis with EPANET is basing on the continuity and conservation of energy theory. For incompressible fluids, continuity equation implies, the algebraic sum of the flow rates in the pipes meeting at a node together with any external flows is zero also called conservation of mass(Dawe, 2000b; Rossman, 2000; Newbold, 2009).

Mass Conservation: Hydraulic performance of pipe network system is based on mass continuity and energy conservation. That, a fluid mass entering any pipe system will be equal to the mass leaving at any pipe system.

$Inflow\ at\ node\ =\ out\ flow\ at\ nodes$……………………………..…………….. (2.15)

A1xV1=A2xV2+A3xV3

Qa=A1xV1 =inflow into node

Qb = outflow =A2xV2

QC = external flow into the system or withdrawal=A3XV3

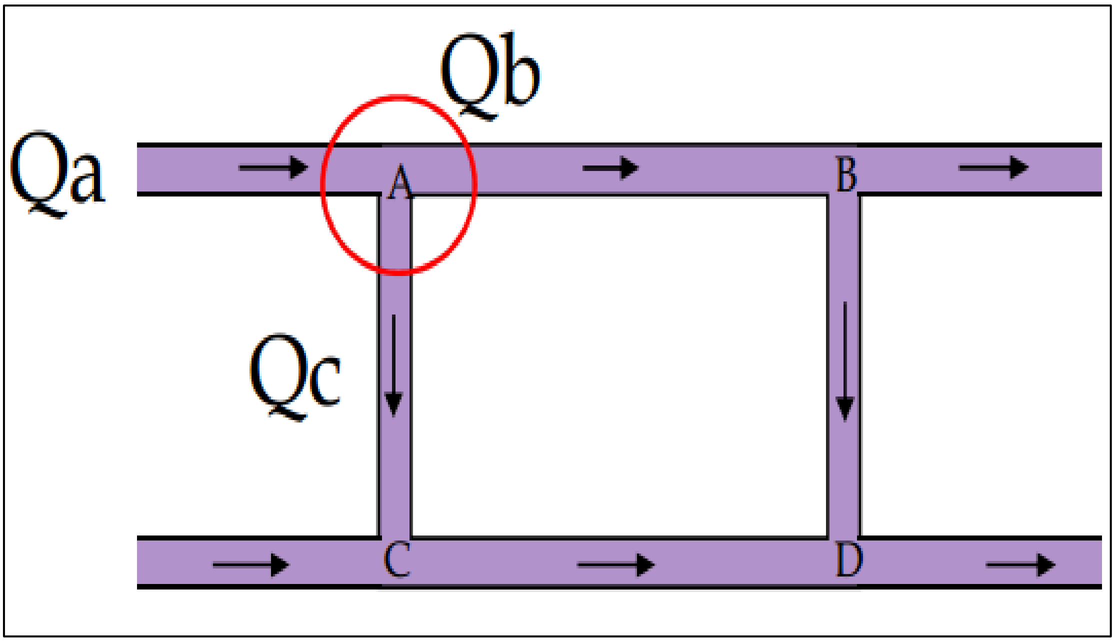

Fig.2.5: Conservation of Mass

Conservation of Energy: The conservation energy implies that, for all paths around closed loops and between fixed grade nodes, the accumulated energy loss including minor losses minus any energy gain or heads generated must be zero and called steady state.

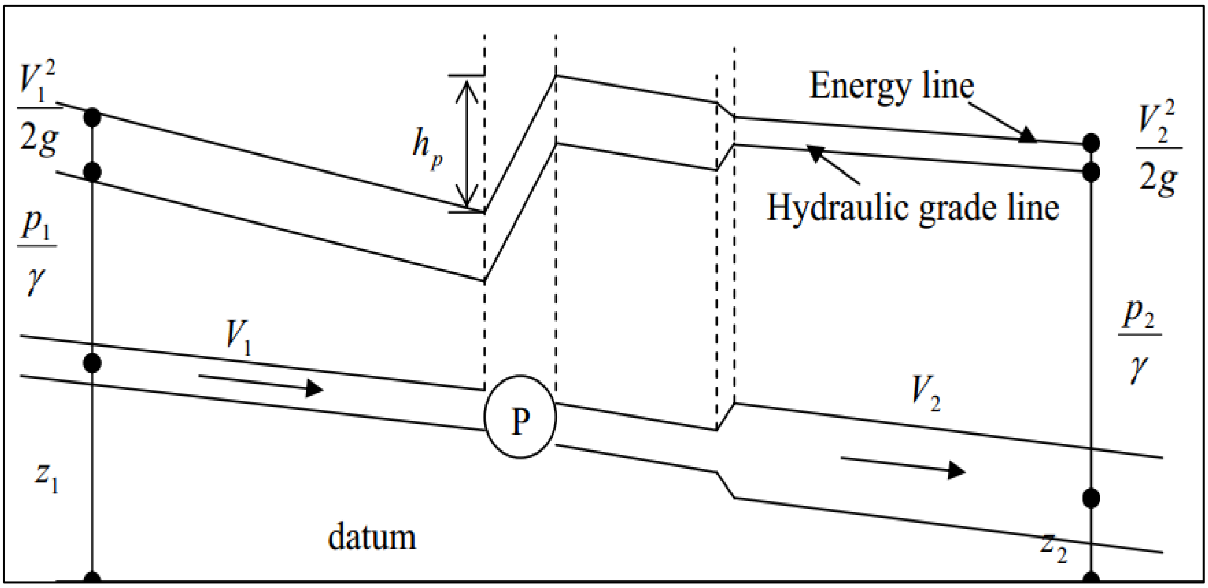

Fig.2.6: Conservation of Energy

$$\frac{p1}{\gamma}+\frac{v1^2}{2g}+Z_1+h_p=\frac{P2}{\gamma}+\frac{v2^2}{2g}+Z_2+h_l \qquad (2.16)$$

Where: $\frac{p1}{\gamma}$ And $\frac{P2}{\delta}$=pressure head, $\frac{v1^2}{2g}$ and $\frac{v2^2}{2g}$=velocity head, Z_1 and Z_2=elevation at two section, h_p= energy gains due to pumps, h_l =major loss and minor loss

Head Loss: Head loss calculated with famous Hazen-Williams equation

$$h_f=10.69[\frac{Q}{C_{CHW}}]^{1.852}x\,\frac{L}{D^{4.87}} \qquad (2.17)$$

Where:

Hf = head loss (m),

L = pipe length (m),

D = pipe diameter (m),

Q = flow rate in the pipe (m3/s), and

CHW = Hazen-William Coefficient(Dawe, 2000b).

EPANET due to its full featured and accurate hydraulic modeling, open-structured, public domain economical free (free cost), accessible modeling due to simple operation and worldwide it is the best software to hydraulic network modeling than other commercial(Rossman, 2000).

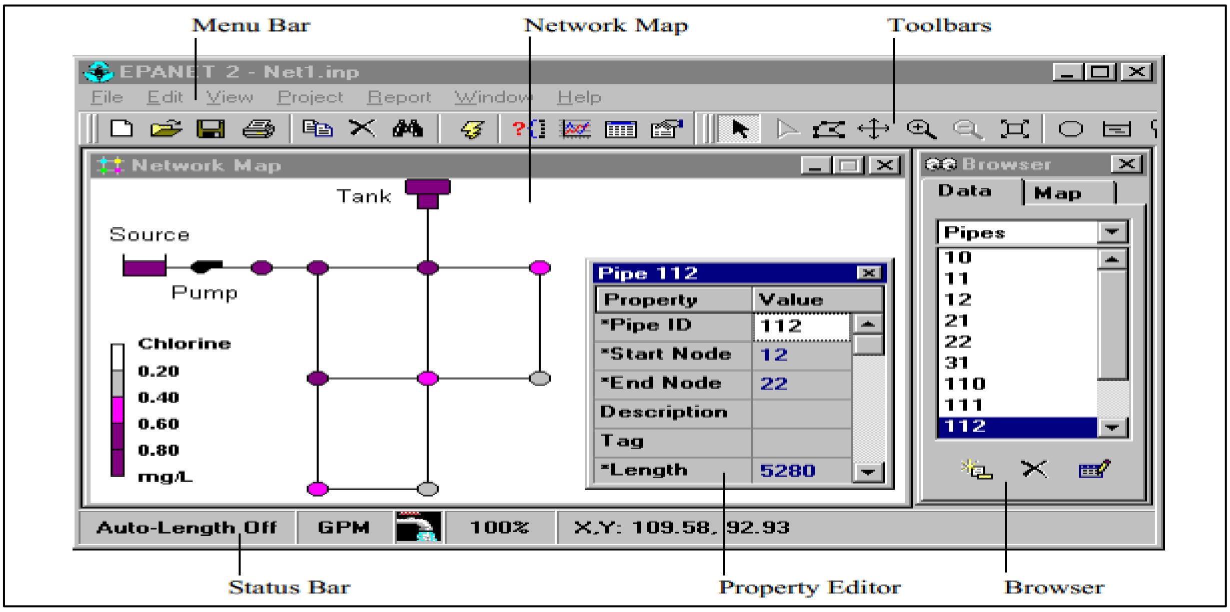

Fig.2.7: EPANET Workspace

When hydraulic network modeling of steady and extend period simulation with EPANET done, nodal demand allocation, analysis of water source, modeling of pumps, analysis of tanks, skeletonization of water network, assigning network parameters and evaluate output result are key activities (Ormsbee & Lingireddy, 1997; Sarbu & Valea, 2011; Datwyler, 2014).

3. MATERIAL AND METHODOLOGY

3.1. Description of Study Area

Study area was in Woreta town, which is located at geographical location of 10^0 51'55'' N and $39^0$10'35''E, and about 636km, North of Addis Ababa along an asphalt road in South Gondar Zone of Amhara Region, East of Tana and South of Addis-Zemen. It is situate in Wyenadega climatic zone at elevation of 1828 meters above sea level. The general topography of the town is hilly east part of asphalt road and flat west part. According CSA population was 28,876(CSA, 2008). Presently Population live in Woreta town is 38,765.

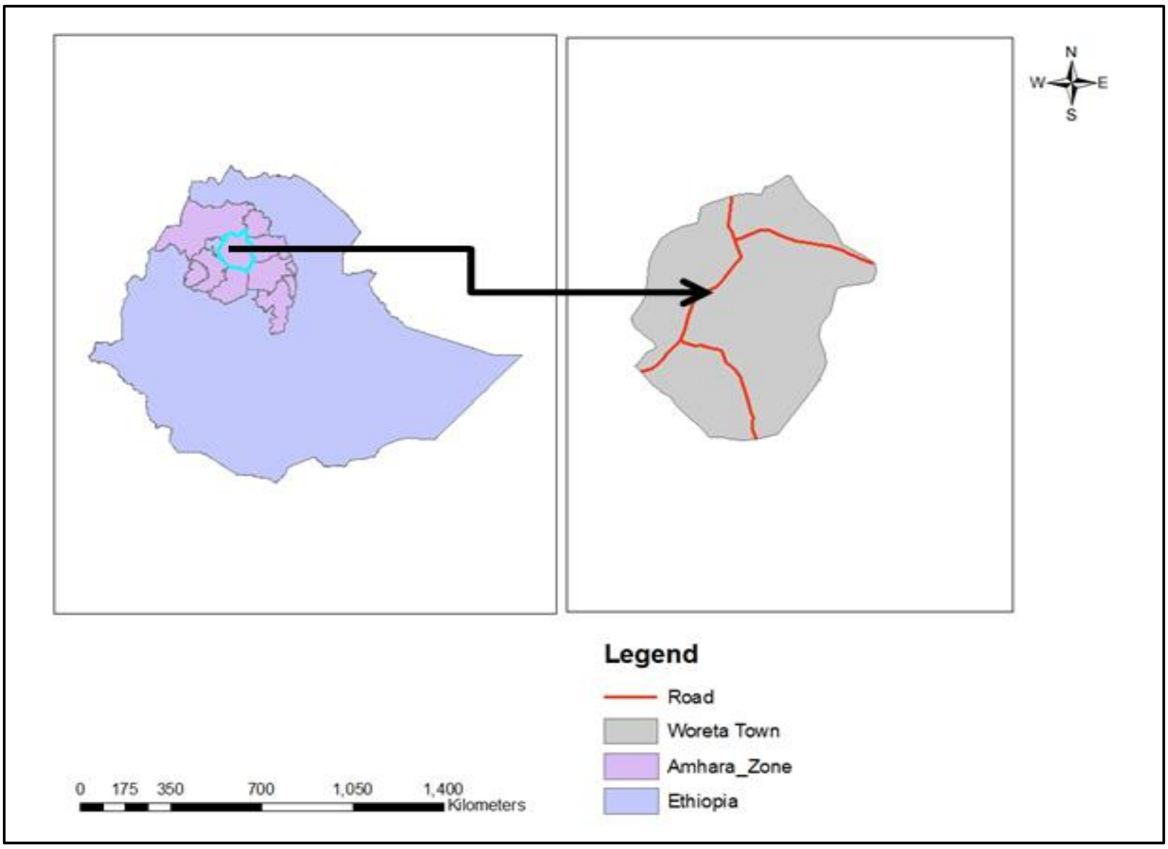

Fig.3.1: Map of Woreta Town

3.2. Material

The materials used in this research to achieve the research goal. Because of materials are key elements to facilitate the research work. Materials, which were used in this research topographical map, computer, digital camera, EPANET software, GPS, utility map, Mendeley software, edrawmax software, excel and word.

3.3. Methodology

In this research work of evaluation of hydraulic performance of urban water supply distribution system in Woreta town, that was both secondary and primary data used. Based on the research objectives and questions how the research carried out discussed here. After successful completion of data collected from the study area data analyzed to evaluating water supply versus water demand, to assess water loss and to identify deficiency of hydraulic parameters as shown below using flow chart fig.3.2.

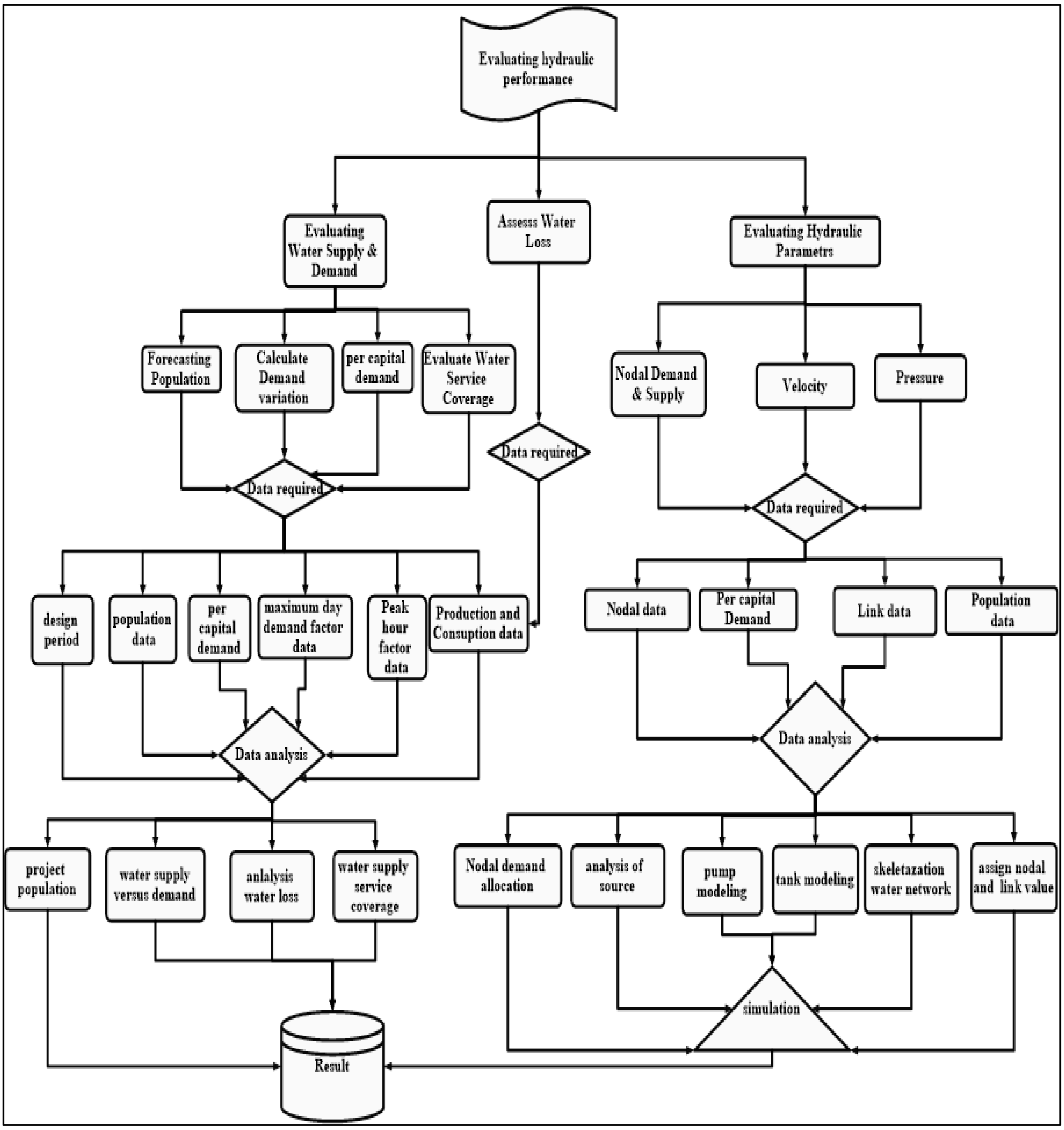

Fig.3.2: Flow Chart of Activities to Evaluating Hydraulic Performance

3.3.1. Input Data Collection

In order to carry out water supply versus demand analysis and simulation of hydraulic parameter in Woreta town, input data were required. Basic data needed to estimation of demand were design period, design population, design per capita water consumption, production and variation water demand (average day demand, maximum day demand and peak hour demand).The input data required for Hydraulic network modeling were link and node data. Data associated with each link include pipe identification number, pipe length, pipe diameter, and pipe roughness. Data associated with each junction node include junction identification number, junction elevation, and junction demand. In addition to the network pipe and node data, physical data must be obtaining that describe all tanks, reservoirs, pumps, and valves. Physical data power or data for use in describing the pump flow- head characteristics curve.

3.3.2. Data Source and Method of Collection

Both primary and secondary data collected method were using. Primary data were collecting from field survey and a random sample of 70 households that 20 of 70 from elevated area, 20 of 70 from lower elevated area 30 of 70 from commercial area who are water user from Woreta water supply system while secondary data was collected from documents. Table3.1 shows data source.

Table3.1: Data Source and Data Type

No.	Data item	Types of data		Data source
		Primary	Secondary	
1	Elevation(nodal point)	√		Field survey
2	Flow yield		√	WWSS
3	Water data (production, consumption)		√	WWSS
4	Utility map		√	WWSS
5	Distribution system (well, Pipe, reservoir, pump) data		√	WWSS
6	Per capital demand		√	MOWIE
7	Population		√	CSA
8	Design period		√	WWSS

3.4. Methods of Data Analysis

3.4.1. Analysis of Water Supply and Water Demand

In order to estimate total water demand that a quantities of water produced to meet all water needs (residential, Institution and commercial, industrial, public use, firefighting and losses) and total number of population needed to know barrier between production capacity of the scheme and consumption of water in the town. Official records for production and water consumption (water billing) data were used in this research to undertake water balance analysis and subsequently to quantify losses. Additional data collected includes reservoir data, Borehole data and Pump data. The water production and consumption found from Woreta water supply service office that recorded 2011-2015. The distribution system were designed to adequately handle the peak hourly demand or maximum day demand and fire flows, whichever is greater, during peak hourly flows; storage reservoirs supply the demand in excess of the maximum day demand. Then evaluating demand variation based on population size was key element to determine the whole capacity of distribution system. To calculate variations of water demand in water distribution system peak hour factor, maximum daily demand factor and per capital demand are essential. This demand factors presented in table3.2.

Table3.2: Demand Factor

Population	Maximum day demand factor	Peak hour factor
< 20,000	1.30	2.00
20,000 to 49,999	1.25	1.90
50,000 and above	1.20	1.70

Source:(Alemayehu, 2010).

3.4.1.2. Population Data:

Historical population data collected by projected Ethiopian urban rank CSA 2008. Population data from 2011 to 2015 were forecasting by geometric, increase method. In 2008 population size of Woreta town and annual growth rate was 28870 and 4.3% respectively(CSA, 2008). The following formula was used:

$Pn = Pox(1 + r)^n$………………………..…………………………………..……… (3.1)

Where:

R=annual growth rate of the population

Pn=population at time n in the future

Po = present population

n = periods of projection

3.4.1.2 Average Annual Demand

The total volume of water delivered to the system in a full year expressed in liters. When demand fluctuates up and down over several years, the average daily demand over a period of one year is used as an average day demand.

Q day-avg = pcd x p………………..………………………………………………… (3.2)

Where:

Qday-avg=average demand

Pcd=per capital demand

P=population

3.4.1.3. Maximum Day Demand

The amount of water required during the day of maximum consumption in a year. The water supply, treatment plant and transmission lines should be design to handle the maximum day demand. In addition, the storage reservoirs must be supply excess water higher than the maximum day demand during peak flow.

Q day-max = Q day-avg X Mdf……………………..…………………………….. (3.3)

Where:

Qday-max= maximum day demand

Mdf= Maximum Day Factor

3.4.1.4. Peak Hour Demand

The maximum amount of water required during single hour in a given day is expresses in liters per day. It is an important parameter for design of distribution systems. Additionally, distribution system is design based on maximum daily demand plus firefighting demand.

Qpeak-hour = Q day-avg X PHF…... (3.4)

Where:
Qpeak-hour= peak hour demand

PHF=Peak Hour Factor

3.4.1.5. Water Supply Service Coverage

Estimation of water supply coverage used to determine how much of population get access adequate water services, either as a domestic water connection (individual or shared) or through public water points. Calculated as the population served (connections and public water points) divided by the total population living in the service area(MoWR, 2005).

$$Coverage\,\% = \left[\frac{(B+(C*D))*E}{A} X100\right] \ldots \quad (3.5)$$

Where:

A=Total population of the town,

B=Number of domestic customers,

C=Number of public water points

D= Number of households using public water

E= Average family size (CSA)

3.4.2. Hydraulic Parametres Analysis

The model would be constructing in EPANET 2 software due to its Hydraulic and Water Quality modeling capabilities. EPANET is a computer program that performs extended period simulation of hydraulic and water quality behavior within pressurized pipe networks and tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprised of multiple time steps. The modeling process involved the following steps: Input data collection, source analysis, pump modeling, tank analysis, and network schematization, assigning network parameter, model building and model evaluated and problem analysis(Gupta et al., 2013; Datwyler, 2014).

3.4.2.1. Source Analysis

When analyzing any water system, it is critical to understand the sources supplying water to the system. Without adequate source, even the best-designed water systems will fail to deliver the required flow to water users(Datwyler, 2014). Typical water supply sources include reservoirs, storage tanks, and external water supply at junction nodes such as groundwater wells. Reservoirs and storage tanks furnish the water supply to the water distribution network. Reservoirs treated as inexhaustible sources of water, and as such, their water level never varies. However, as a storage tank empties, its water level lowers and it has to be refill by pumping from either a reservoir or a groundwater well. In EPANET, groundwater well pump should model the same as a pumped reservoir. As pumping of the groundwater occurs, drawdown of the water table elevation at the groundwater well can occur. At higher pumping rates, the groundwater well may not be able to recharge fast enough to maintain the pumping rate specified by the defined groundwater well pump curve.so during simulation assume that the source is not fluctuated(Rossman, 2000).

3.4.2.2. Modeling of Pumps

To model parallel pumps, it is necessary to insert the pumps on the same from and to nodes. To model pumps in series (where the outlet of the first pump directly discharges into the inlet of the second pump), it is necessary to insert the pumps one after the other on the same pipe. If desired, the two or more pumps can be modeling as an equivalent composite single pump that has a characteristic curve equal to the sum of the individual pump curves. For pumps that are in parallel, the discharge values for the individual pump curves are added together to end up with the equivalent single pump curve(Rossman, 2000; United state of Department of Energy, 2005).

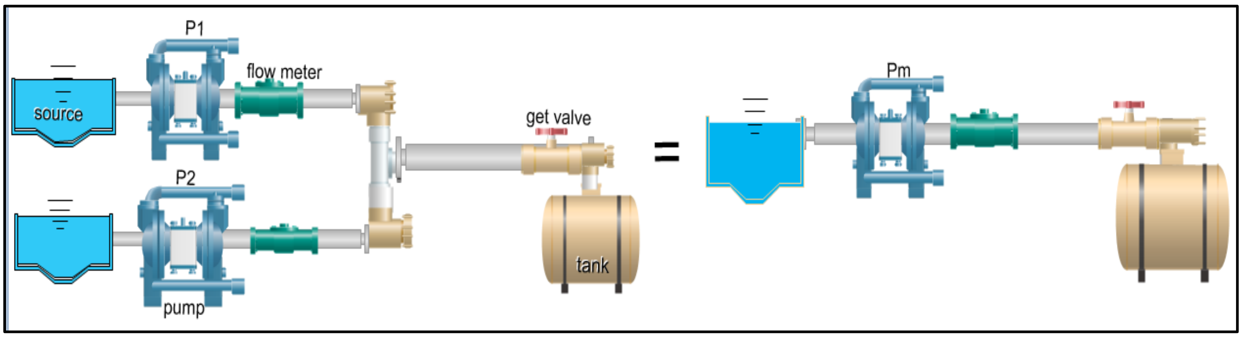

Fig.3.3: Modeling of Parallel Pumps

$Q_{P1} + Q_{P2=Q_{pm}}$………………………...………………………………………………… (3.6)

$C_{p1\cong C_{P2}} = C_{CM}$………………………...………………………………………………… (3.7)

Where:

Qp1= discharge of p1,

Qp2= discharge of p2,

Qpm= discharge of pm,

p=pump, pm= model pump,

Cp1=curve of pump1,

Cp2= curve of pump2 and

Cm= modeling pump curve

If the pumps are connected together in series, then the head values are for the individual pump curves are added together to end up with the equivalent single pump curve. Therefore, by the above point of view pump change to single equivalent pump(Rossman, 2000).

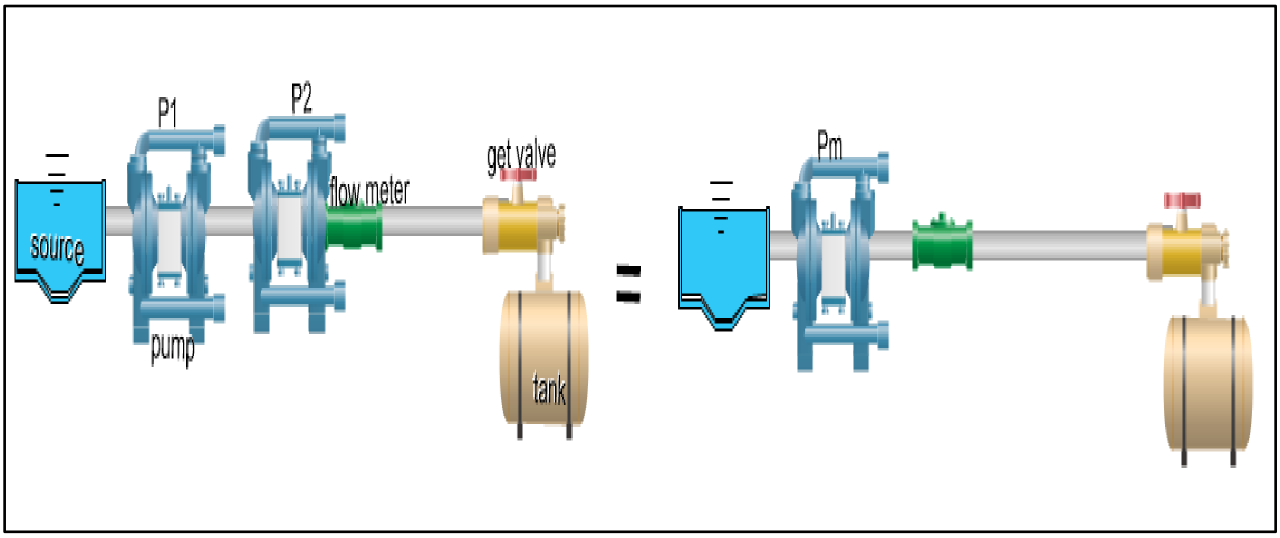

Fig.3.4: Series Pump Modeling

$C_{p1} + C_{p2} = C_{pm}$………………………………………...…………………………… (3.8)

$Q_{p1} \cong Q_{P2=Q_{PM}}$………………………………...……………………………… (3.9)

Where:

Cp1= Curve of pump1,

Cp2=Curve of pump2 and

Cm=curve of modeling pump

Qp1= discharge of pump1

Qp2= discharge of pump2

Qpm= discharge of modeling pump

3.4.2.3. Analysis of Tanks

When performing an extended period simulation, if two or more storage tanks are hydraulically adjacent to each other it is possible that oscillations can occur between the tanks as the water bounces back and forth between them. This fluctuation is causing by small differences in flow rates as the tanks refill individually, causing the water level in the tanks to differ over time thereby causing the oscillation between the tanks. To prevent this effect from occurring, it is recommending that hydraulically adjacent tanks be modeling as a single composite tank with an equivalent total surface area and storage volume equal to the sum of the individual tanks(Rossman, 2000; Bouman, 2014; Datwyler, 2014).

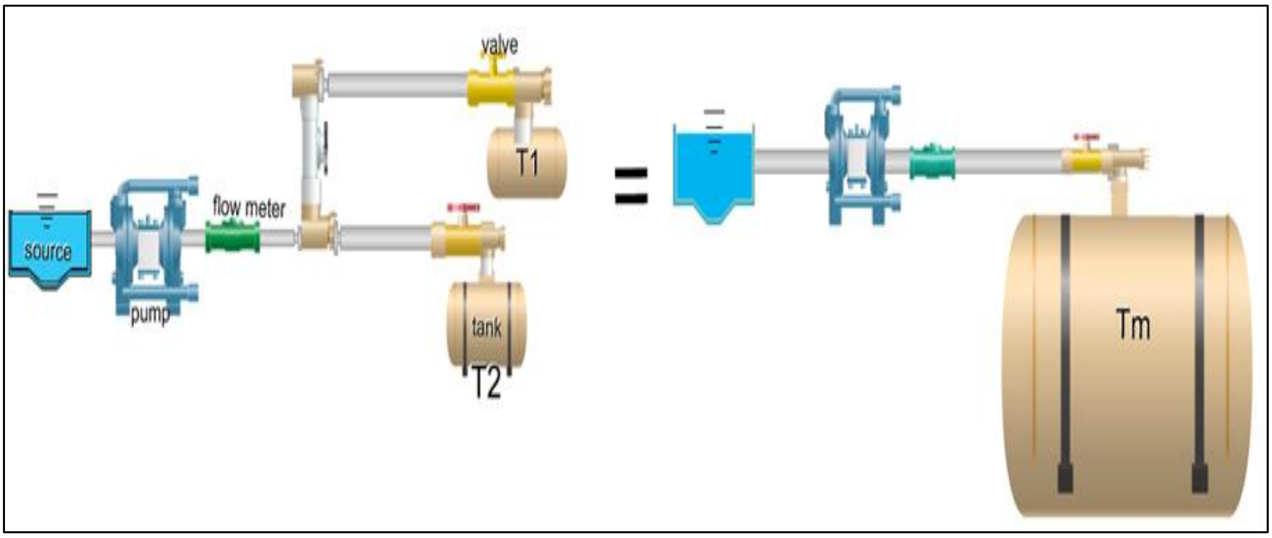

Fig.3.5: Tank Modeling

$$V_{T1} + V_{T2} = V_{TM} \quad (3.10)$$

VT1=Volume of tank1

VT2=Volume of tank2

VTM=Volume of modeling tank

3.4.2.4. Skeletonization of Water Network

The next step in using EPANET was to skeletonize the network and assign node numbers to the nodal points. Figure shows below the skeletonization of the network on the Woreta town map. The skeletonization of water distribution layout based on master plan of the town, and then placed as a map on EPANET platform as shown on.

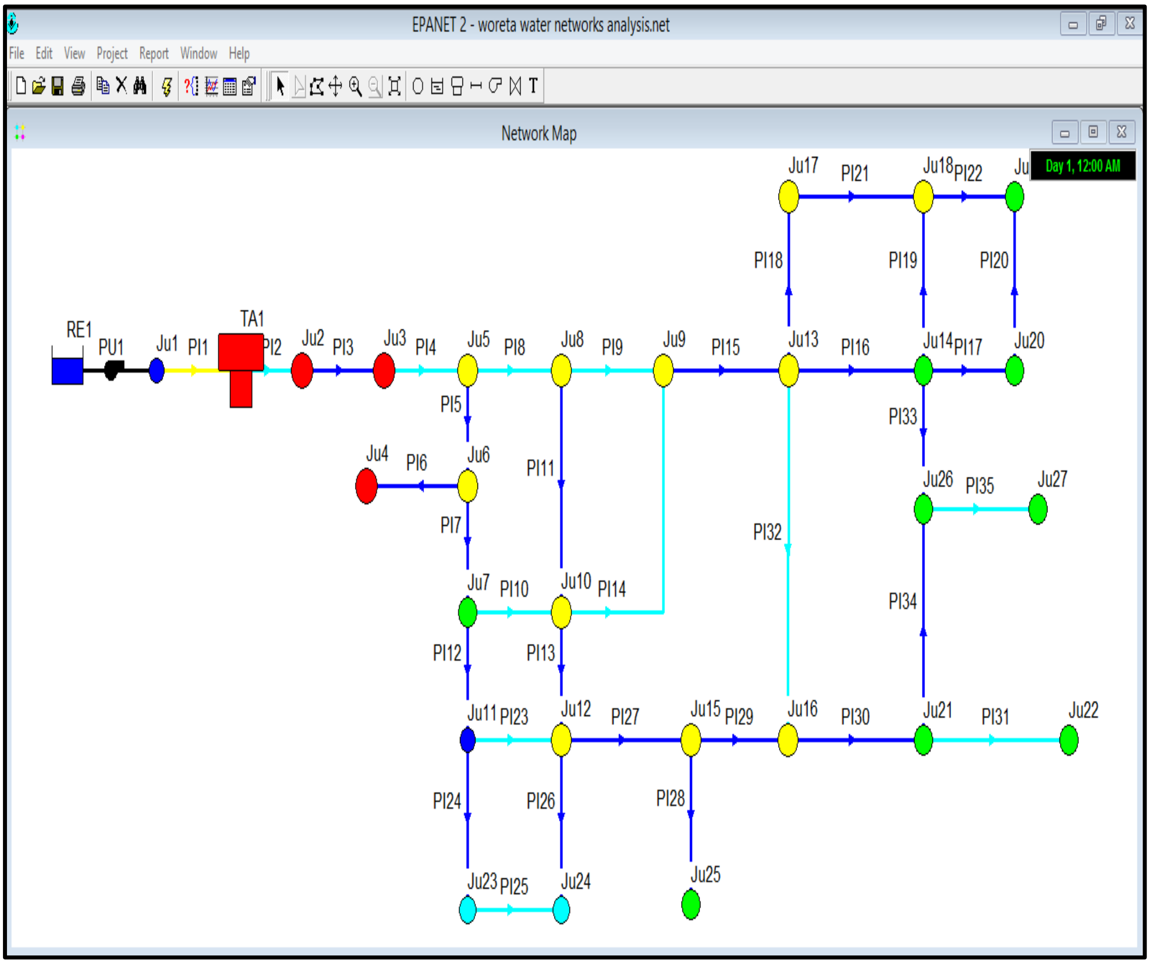

Fig.3.6: Skeletonizations of Water Networks

3.4.2.5. Assign Network Parameters

After the skeletonization of the network on EPANET platform, the next step was to assign network parameters. The networks parameters include pipe lengths, pipe diameters, roughness coefficients (Hazen-William), Nodes numbers, and Nodal elevations. The node and pipe data sets contain geographic co-ordinates, ground levels, basic demand information, internal diameter and friction coefficients, pump curves, pump discharge, service reservoir geometry.

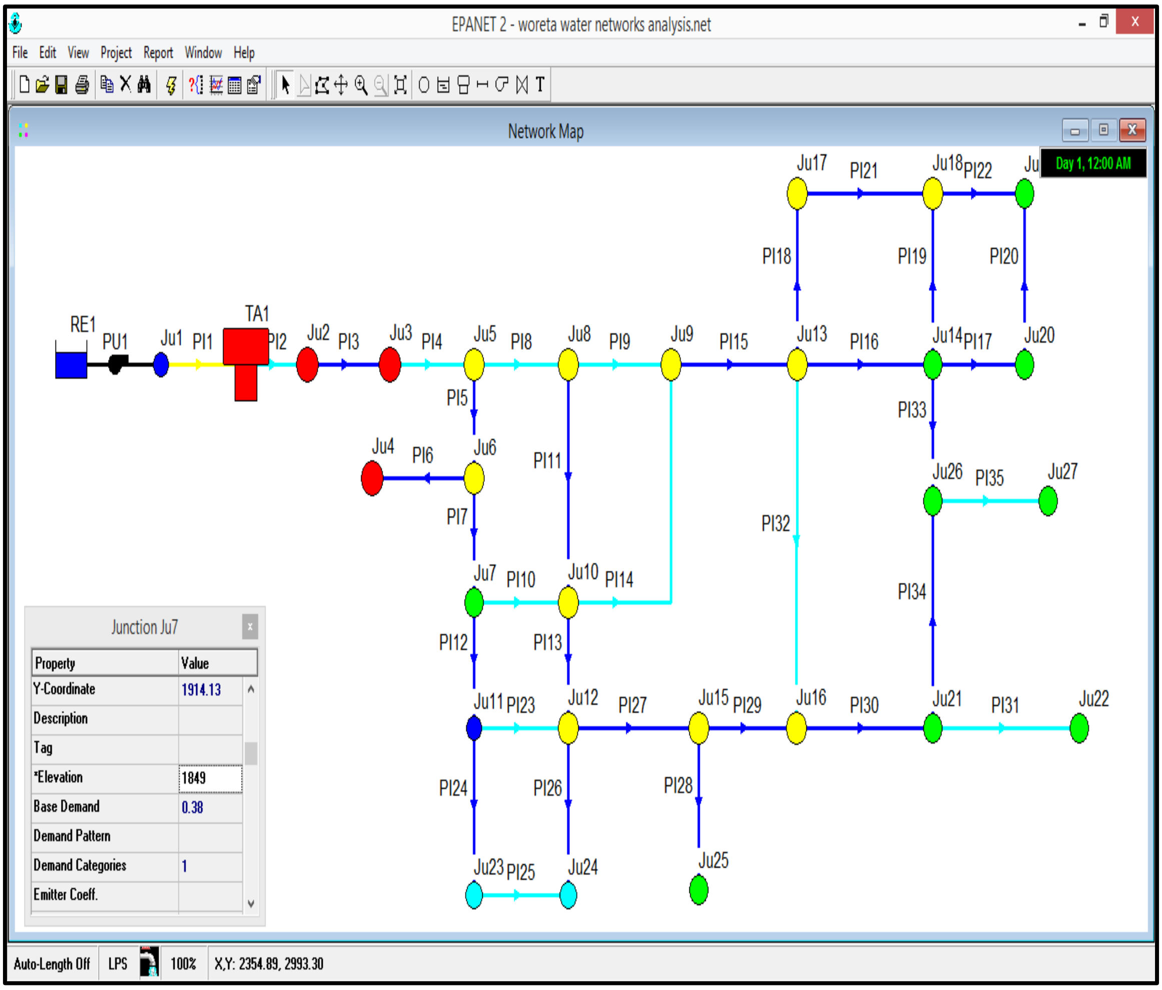

Fig.3.7: Assign Network Parameters

3.4.2.6. Nodal Demand Allocation

A special aspect of the model building process is the determination of nodal demands. The survey of numerous users spread all over the network carried out. and using an average household occupancy of Woreta 4.3 and a daily per capita water consumption of 20 l/c/d of Woreta town, their demand was concentrated into a limited number of pipe junctions in order to make the network presentation suitable for a computer model. The starting point is the calculation of the average demand. This yields the demand of a certain area, which has to be converting into demand at a point (pipe junctions). Node usually had one of the two main functions; it receives a supply for the system or it delivers the demand required by consumers. Generally, acceptable in modeling to lump half of the demands along a line to the upstream node and the other half of the demands to the downstream node(Belay, 2012; Joshi et al., 2014). Demand allocation to consumption points were estimated using formulae.

$$Nd = \sum pi.Dj \quad (3.11)$$

Where:

Nd=Nodal demand

Pi=population in each service area

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

i = subscript referring to the i-th service area

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

3.4.2.7. Evaluate Model Result

Data should be entering into the computer in a format compatible with the selected computer model. After data have been assembled and encoded, the associated model parameters should then be estimated actual model application. Model result evaluation based on Ethiopia ministry of water irrigation and energy urban water supply design criteria at normal condition. To compare the standard and output maximum and minimum, allowable velocity and pressure stated below. Experience shows that pipe designed to flow at a velocity between 0.6 and 1.5 m/sec (MoWR, 2006).Maximum velocity and minimum velocity limit given below.

- ❖ Maximum velocities of transmission mains < 2.5m/s
- ❖ Maximum velocities of distribution mains < 2 m/s
- ❖ Minimum 0.6 m/s

Ministry of water resource irrigation & energy put standard for urban water supply design maximum and minimum allowance pressure head. It tabulated below.

Table3.3: Maximum and Minimum Pressure Limit

Pressure	Normal condition	Exceptional condition
Minimum	15m	10 m
Maximum	60m	70 m

Source: (MoWR, 2006)

4. RESULT AND DISCUSSION

The Woreta water supply design period is 20 years(ADSWE, 2010). It constructed in 2001 and served the last 15 years. The distribution network of Woreta town is looped type distribution pipes system, which covers all part of the town and it get power source from the national grid line. Woreta town water supply distribution system has two unclearly defined pressure zones (higher elevated area and lower elevated area). The elevated areas get water from 100m3 elevated reservoirs the other part of the town get from the 300m3 service reservoir by gravity. Woreta water supply method is combination of pumping and gravitational system. Water deliver from two boreholes, which currently supply a yield of about 6.0 l/s and 6.5l/s; to service reservoir and elevated tank by pumping system for 18 hour/day. When water reach from elevated tank and service reservoir, it distributed by gravitational system to the whole customer. Now a days Woreta water supply system feeds about 38,765 populations. In the distribution system, there are 13 public fountains, out of which 12 are functional. Those public fountains lactated at lower places are functional regularly and those public fountains located at relatively elevated places are functional only when there is enough pressure or when there is less demand in the distribution system.

4.1. Population Forecasting

Accurate estimation of water use by particular society is rarely achievable, since water use is practically reliable to change. However, fair estimation is reachable under the circumstances in which current and future population dynamics are well known. Even good estimation is also viable if official records of local economic activities along with the climatic conditions are accessible. Direct population count and projection based on pre-counted population are two possible approaches to collect population data. However, since direct population count at any time requires a great deal of resource and time, it is not usually preferred(CSA, 2008; Alemayehu, 2010; Belay, 2012). The Ethiopian statistic authority uses the formula $\mathrm{pn} = \mathrm{po}\,(1 + \mathrm{r})^{n}$ for most water supply schemes in the country to project population at the end of required decade/year (ADSWE, 2010). Due to the above fact and limited population data geometric increase is using for population projection of Woreta town. In order to forecast 2011-215 of the Woreta town based on Ethiopian urban rank. The growth rate and population in 2008 was 4.3 %, and 28,870 used for the current projection(CSA,2008). Woreta town water service office current population recorded data show total population in 2015 was 38,254 and Woreta town finance and economics office registered total population in 2015 was 40,386. However, this research done based on 2008 Ethiopian urban rank.

$Pn = Pox(1 + r)^{N}$ ………………………………………..……………………….. (4.1)

Where:

Pox= current population

N= number of years

R=annual growth rate of the population

Pn=population at time n in the future

Table4.1: Projection of Woreta Population 2011-2015

Year	Population
2011	32757
2012	34165
2013	35634
2014	37167
2015	38765

4.2. Water Supply and Demand in Woreta Town

The main objective of water distribution system is to deliver adequate quantity and quality within required velocity and pressure (Alemayehu, 2010). A safe, reliable, affordable, and easily accessible water supply is essential for good health, but for several decades almost 1 billion people in developing countries have lacked access to such a supply(Hunter et al., 2010; Alemayehu, 2010; WHO & UNICEF, 2012). Water supply schemes designed to supply 24-hour a day and 7 days in a week but they operated intermitted. Intermittent water supply is piped water supply, which delivered less than 24 hours a day. It results in waste of water, requiring larger pipes in the network to deliver the same amount of water in a shorter time. It also allows contaminated water to enter the piped network when the pipes are empty. When associated with public tap supplies, this unreliability promotes stress and fighting among the urban poor who struggle to get their share of water each day(Mcintosh 2014; WHO 2014). There was the same case in Woreta town water supply system, which was design for period 20 years' to give service for continuous 24 hour a day and7 days a week. However, it was impossible to deliver water for 24-hour day and 7 day a week to meet the existing demand. Due too many factors, Water demand in Woreta town was high and shown severe shortage of water in the town. During the study time, Woreta was used intermitted water supply system. Because of, in Woreta town the quantity of water that wells produced not enough to meet the needs of consumers and system flushing, and other needs. The problem arises due to limited source capacity, high population growth in town & poor operation and maintenance, inequity of water in distribution due to the topography. This also high pressure on the existing infrastructure, which usually results in infrastructural decay, there by disrupted the efficient of water distribution system. Moreover, another problem of water supply in Woreta town is associated with erratic power supply that humped continued operation of the water supply system. In Woreta following rapidly development of the town construction field such as buildings and expansion of road, increase rapidly. This also highly challenge and make stressed on water supply system. In other hand Woreta rapidly grown commercial town and has good prospect for development. To support the progress a potable, reliable and adequate water supply system must be established(ADSWE, 2010). Generally, the capacity of water distribution system that supplied and needed to supply to satisfy all demand within its design period tabulated below. To calculate demand variant; per capital demand, maximum day demand factor and peak hour factor were needed. Since Woreta population, number ranged 20,000-50,000 then calculated by taking maximum day demand factor and peak hour factor was 1.25 and 1.9 respectively(Alemayehu, 2010), and per capital demand, 20 l/c/day(MoWE, 2015).

Table4.2: Water Demand in Woreta Town 2011-2015

Year	Population	Average day demand(m^3/day)	Maximum day demand (m^3/day)	Peak hour demand (m^3/da y	Per capital demand (l/c/day)
2011	32757	655.14	818.925	1245	20
2012	34165	683.3	854.13	1298	20
2013	35634	712.68	890.85	1354	20
2014	37167	743.34	929.18	1412	20
2015	38765	775.3	969.125	1,474	20

Table4.1

Table4.3: Water Supply in Woreta 2011-2015

Year	Population	Production (m^3/year)	Average supply (m^3/day)	Water Supply (l/p/d)
2011	32757	224493.60	614.63	18.76
2012	34165	230198.81	630.25	19.31
2013	35634	220596.39	603.96	18.35
2014	37167	253370.27	693.69	18.66
2015	38765	247230.42	676.88	17.46

As shown the above table4.2 and table4.3 the distribution system could not capable to deliver expected enough water for satisfied different demand of woreta town. The design period of Woreta water supply system will end the next five years at that time Woreta total population will be 49,904. The new standard for urban water supply access with GTP-2 category minimum service level a town whose population ranged 20,000 to 49999 will 50 l/c/d(MoWIE, 2015). Therefore Woreta water distribution system at the end of next five year forced to deliver 3,119 m^3/day and 4,740.88 m^3/day to satisfy maximum day demand and peak hour demand respectively. It is impossible to achieve this demand because it was fail to satisfy current demand, even within 20 l/c/d per capital demand. During the research time, Woreta town was used intermittent water supply system. Because of the scheme cannot provide 24 hours water supply to the public. For more understanding water production and demand in the town was presented in fig4.1.

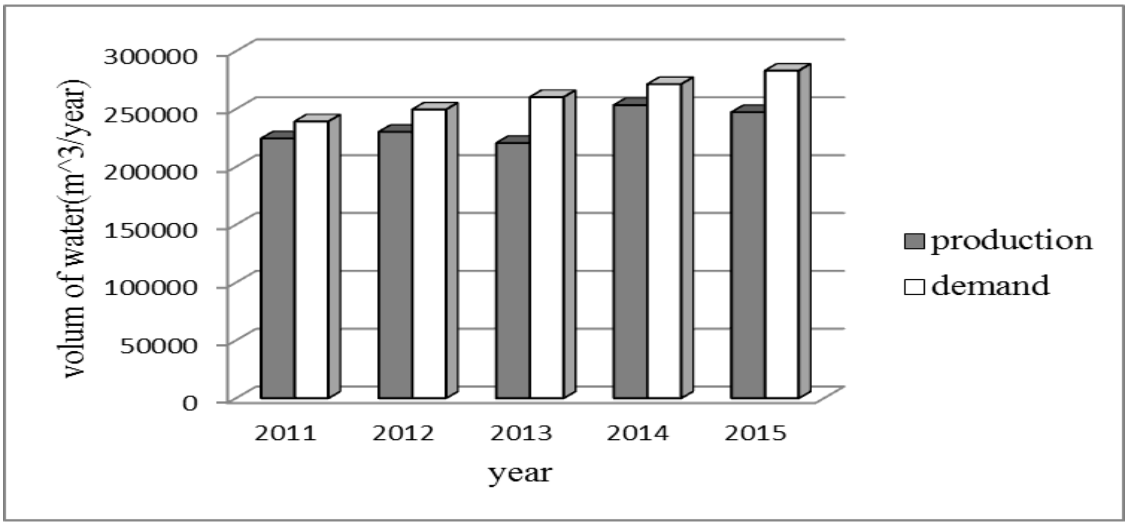

Fig.4.1: Water Supply versus Demand

4.3. Water Loss in the Distribution System

Water loss is volume of water lost between the point of supply and the customer meter due to various reasons. It expressed as the difference between systems input volume and authorized consumption, and consists of apparent and real losses. Apparent losses can be subdivided into unauthorized consumption, meter inaccuracies and data handling errors (Bello & Tuna, 2014b; Nigam et al., 2015). In this research, real water loss discussed. Real losses are made up of leakage from transmission and distribution pipes, leakage from service connections and losses from storage tanks(Sharma, 2008)**.** Water loss in Woreta town was severe problem according to consumption and production data. Additionally, during field surveying there was indication of case of high water lose in the town due to expansion of different infrastructure such as road expansion, suddenly burst of pipe. Water loss was analyzed from water production and consumption data from the year 2011-2015, which is presented in fig.4.2 below.

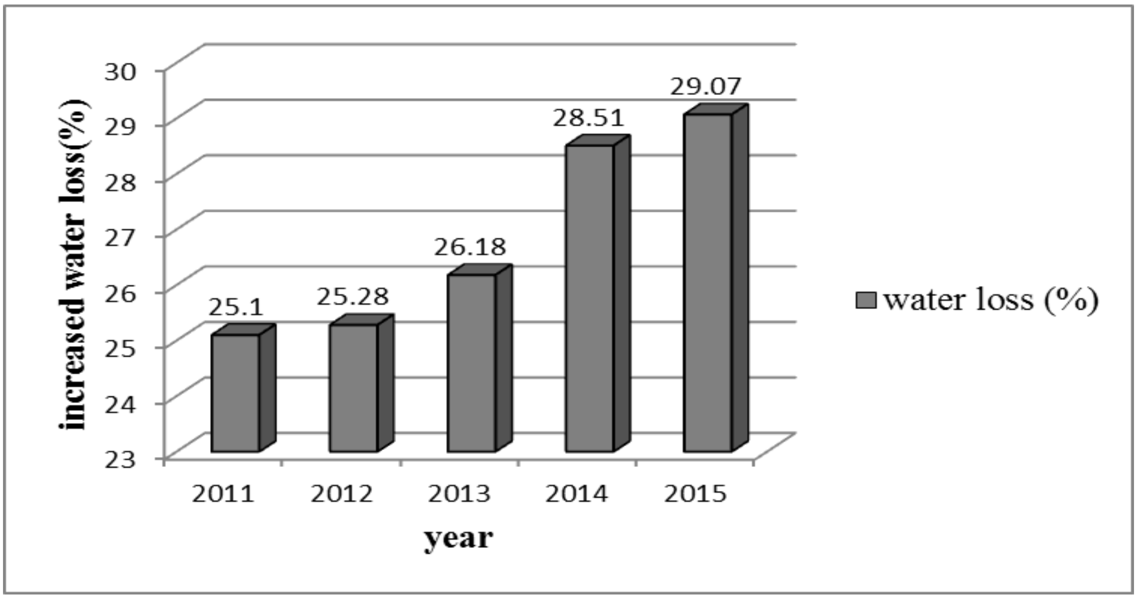

Fig.4.2: Water Loses in Woreta Town Water Distribution System

The other problem, in Woreta water distribution network was insufficient maintenance, poor rehabilitation measures and intermittent water supply that increase water loss form 2011-2015. The long served pipe and intermittent water supply in Woreta town were problems by pipe remains empty for long time and exposed for corrosion. This leads to lose its resistance for water pressure during water supply and made pipes break and burst within water distribution networks(Walski et al., 2003).

4.4. Water Supply Service Coverage in Woreta Town

Service coverage means a piped connection to each household, ideally with water available 24 hours a day and7 days a week to meet existing demand. The best measure of a good water supply service in a city is the number of people with 24-hour access to piped water at home. That is why service coverage must be the most important performance parameter of any water utility(MoWE, 2012; Mcintosh, 2014). Because of pressurized piped water not supplied throughout the day is not potable. Contaminate may enter when the pipes are empty(WHO & UNICEF, 2012; WHO, 2014). In Woreta, existing water supply system does not fully service coverage to satisfy demand of the population. The problem arises due to the following reasons, limited source capacity, rapid population growth, expansion of new area and illegal settlement around the town. According to Woreta town water supply service office, the domestic water (yard, house and neighborhood) users were 3,256 households and total public tap users were 3,865 households in 2015. Then in Woreta, water supply service coverage was 78.9 % of from population 38765.then 21.1% people had no access connection of water supply. Fig.4.3 shows that woreta town water supply service coverage.

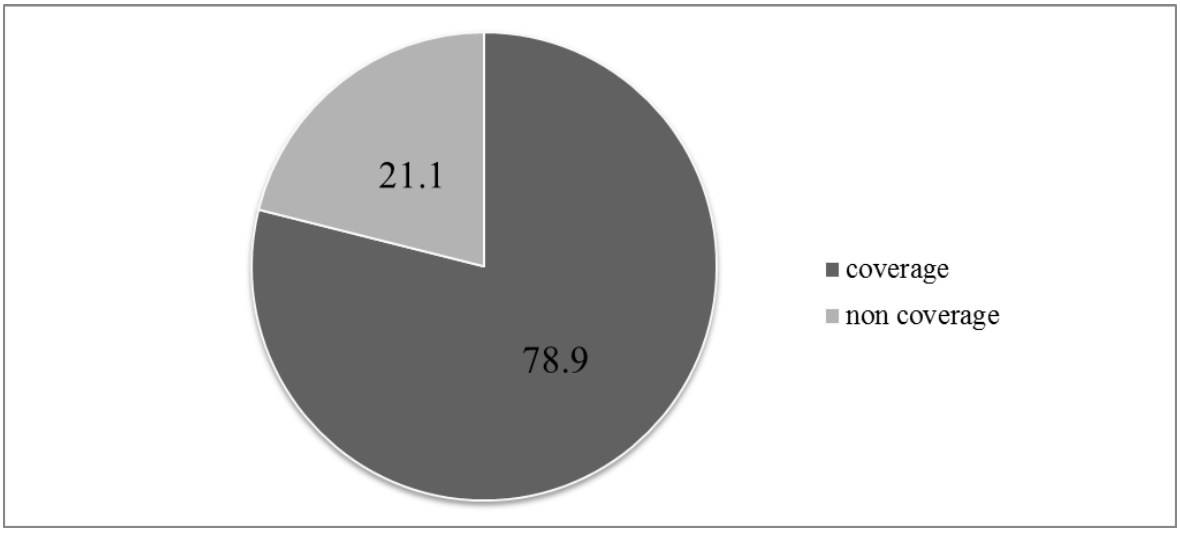

Fig.4.3: Water Supply Service Coverage

4.5. Hydraulic Analysis of Water Supply Network

In this study, the Hydraulic model EPANET version 2.0 was using for evaluating the hydraulic performance of water supply system to the study area. EPANET is a computer program that performs steady and extended period simulation of hydraulic and water quality behavior within pressurized pipe networks(Rossman, 2000; Dawe, 2000b; Water CAD User's Guide, 2003). Extended period simulation indicates whether the system has the ability to provide acceptable levels of service over a period of minutes, hours, or days (Water CAD User's Guide, 2003). EPANET tracks quantities of flow and head loss in each pipes, and residual pressure at each node, the height of water in each tank, and the concentration of a chemical species(Rossman, 2000). When the water distribution system being modeling does not have a combination of pressure boosting station and pressure-reducing valves, an instantaneous model developed. This consists of a "snapshot" of the demands on a model in a static scenario. While extended period model was creating and use to evaluate system performance over time. EPANET uses programmed algorithms to repeatedly solve the continuity and energy equations to determine the flow and residual pressures at specific nodes in the pipe network(Datwyler, 2014). The Hazen-Williams equation used to calculate the friction losses in the pipes. After application of the software simulation steady state and 72 hour extended period, results obtained viewed on the network map. The analysis and modeling of distribution network in Woreta water supply was consists of 36 pipes of different material, 29 nodes, 1 tank, 1service reservoir and 2 source reservoir from which water was pumped to the elevated tank as shown in fig.4.4.

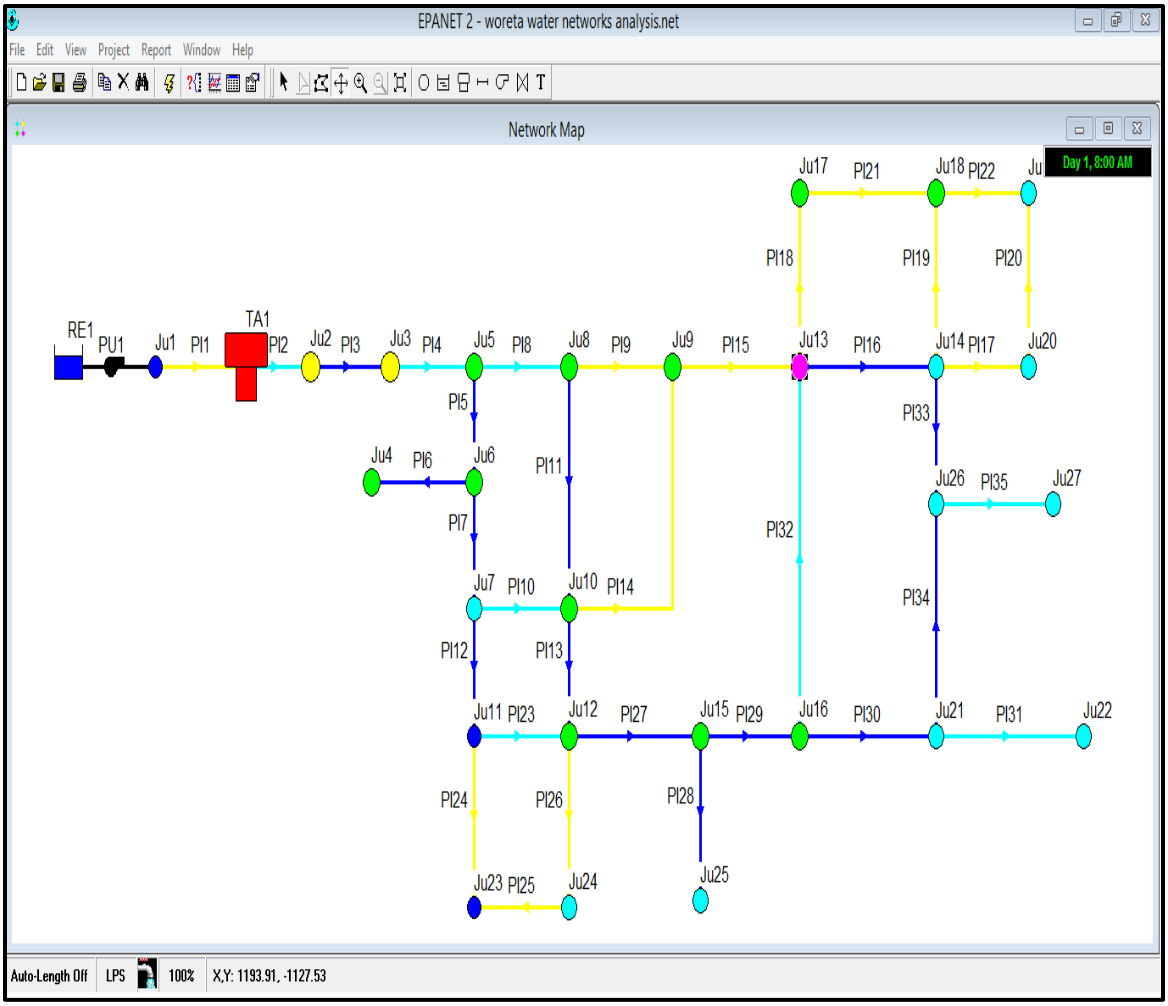

Fig.4.4: Hydraulic Modeling of Water Distribution Network

4.5.1. Nodal Demand and Nodal Supply

Water demand in a distribution system fluctuates over time during simulation as shown from fig.4.5 where as water product was constant. Not only water dmand and also flow in fig.4.6, pressure, velocity and hydraulic head vary time to time due to demand pattern. At night from 12am-6 am demand was low and at 6 am-12 noon demand was high because people sleep at night demand be come low and peope at the moornig wake up to perform their activity water is needed then water demand be come high. When demand increase flow rate and velocity increase where as peressure head decreased at each demand pattern.

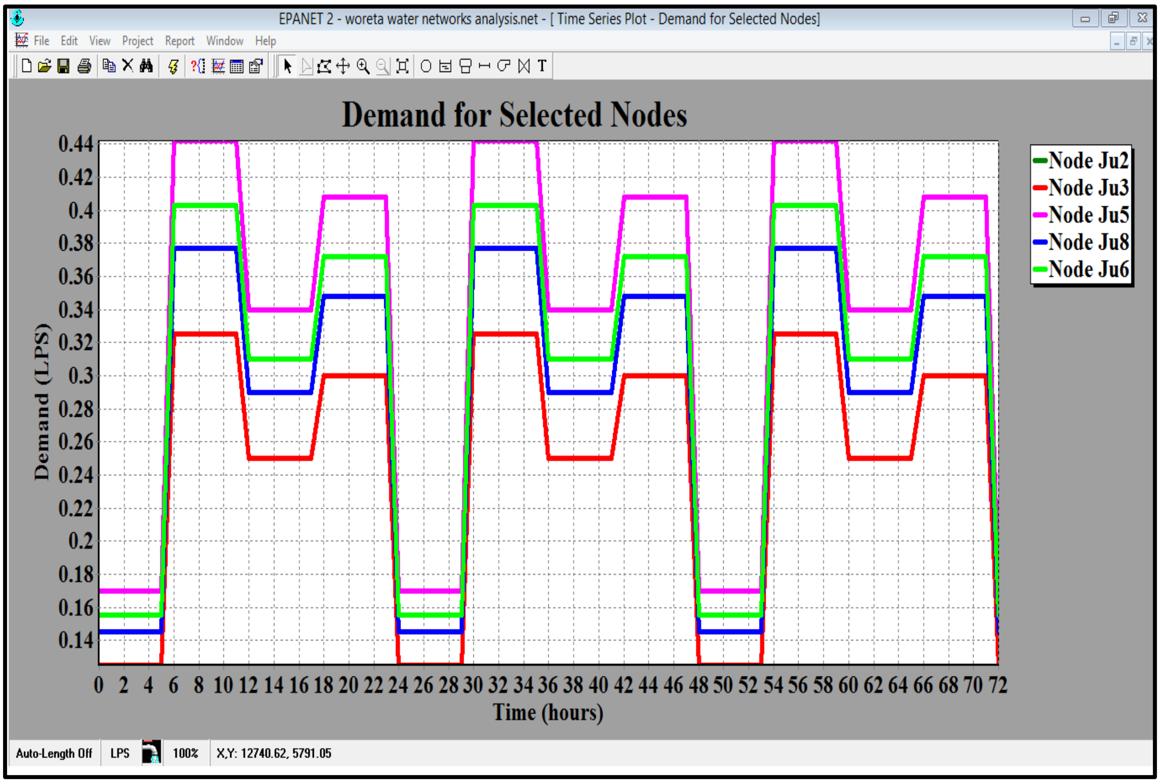

Fig.4.5: Fluctuate of Water Demand with Time

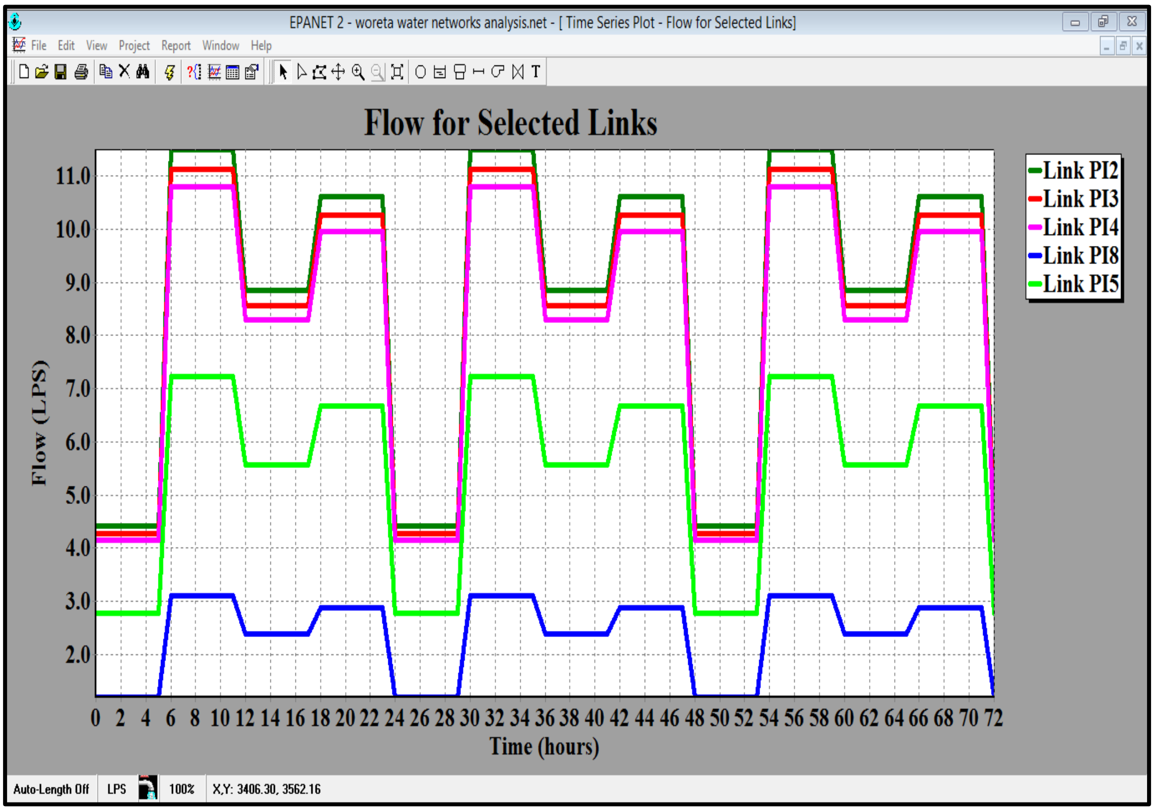

Fig.4.6: Flow Fluctuate With Time

This variation in demand over time can be modeled using demand patterns. Demand patterns are multipliers(flow/average flow) which vary with time and applied to a given base demand, most typically the average daily demand(Dawe, 2000b). A steady state model tells whether the system has the capability or not to meet a certain average demand (Boulos et al., 2014; Sarbu, 2011; Water CAD User's Guide, 2003). The average current supply interns of nodal draw-off in liters per second(lps) and the analysis of actual current nodal demands for each of the nodes in the distribution network node points(nodal draw offs) in Fig.4.6 and Fig.4.7 shows very clearly that the current supplies at the nodal points failed.

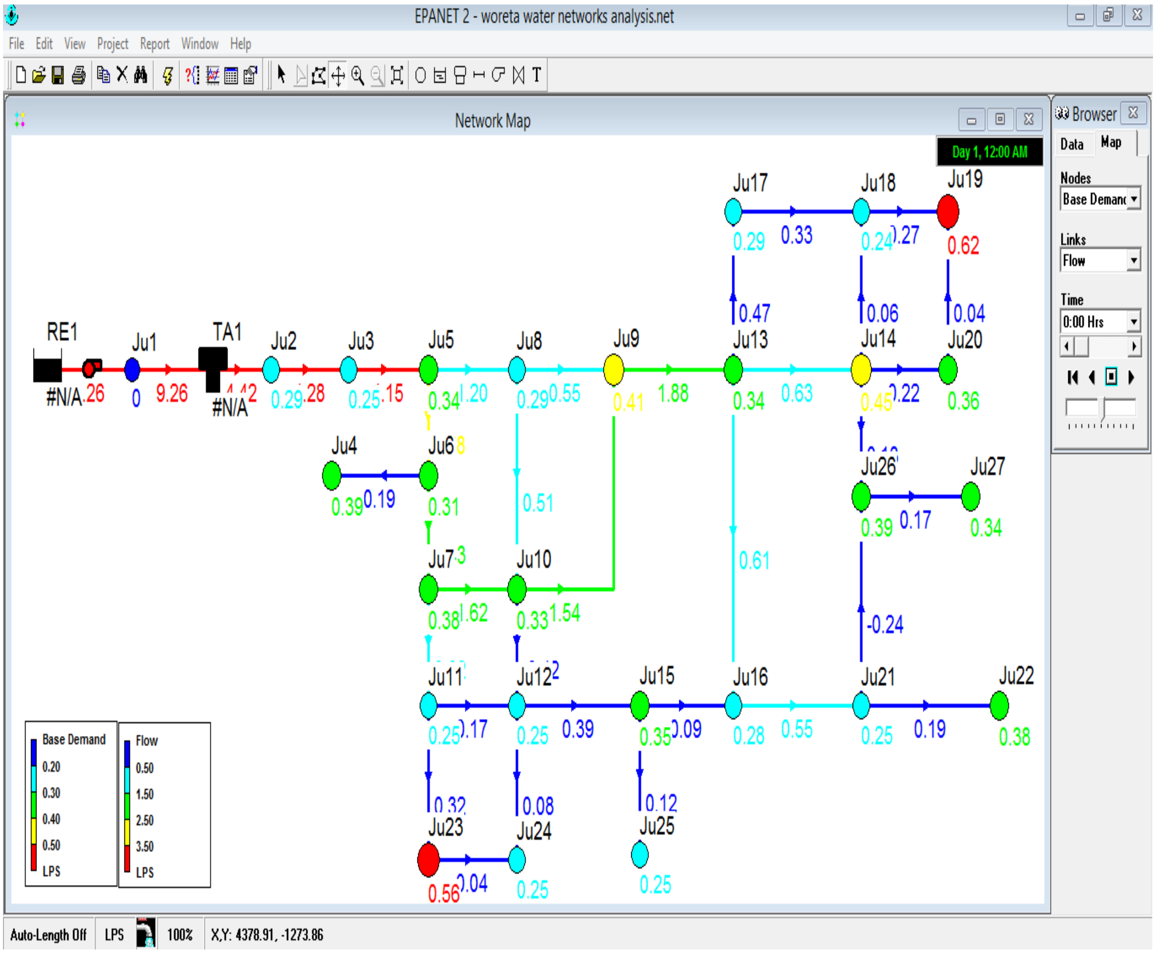

Fig.4.7: Output of water Supply and Demand at Single Period

Woreta water distribution system feed 38,765 populations and standard per capital demand was 20 liters/day. Then the distribution system forced to delivered 17 l/sec peak hour demand to this population. Because water distribution is designs to meet peak hour demand. Therefore, to meet current and future demand the sources of water must be improve and additional new source should be introduce to the system.

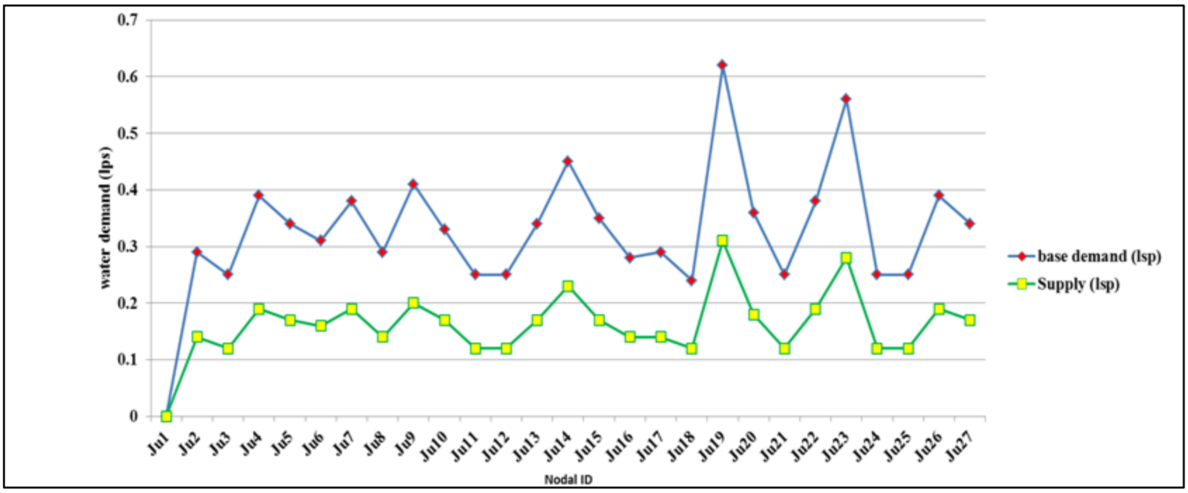

Fig.4.8: Nodal Demand versus Nodal Supply

4.5.2. Water Flow Velocity in Distribution System

Water velocity should maintain at less than 2m/sec, in distribution system except in short section and not more than 2.5 m/s in transmission system. A minimum velocity of 0.6 m/sec had taken, but for looped systems, there would be pipelines with section of zero velocity(MoWR, 2006; AWRDB, 2012). During implementing, the simulation showed a further drop of most links velocity below 0.6 m/sec. The velocity varies from time to time through demand pattern but most of linked below minimum.

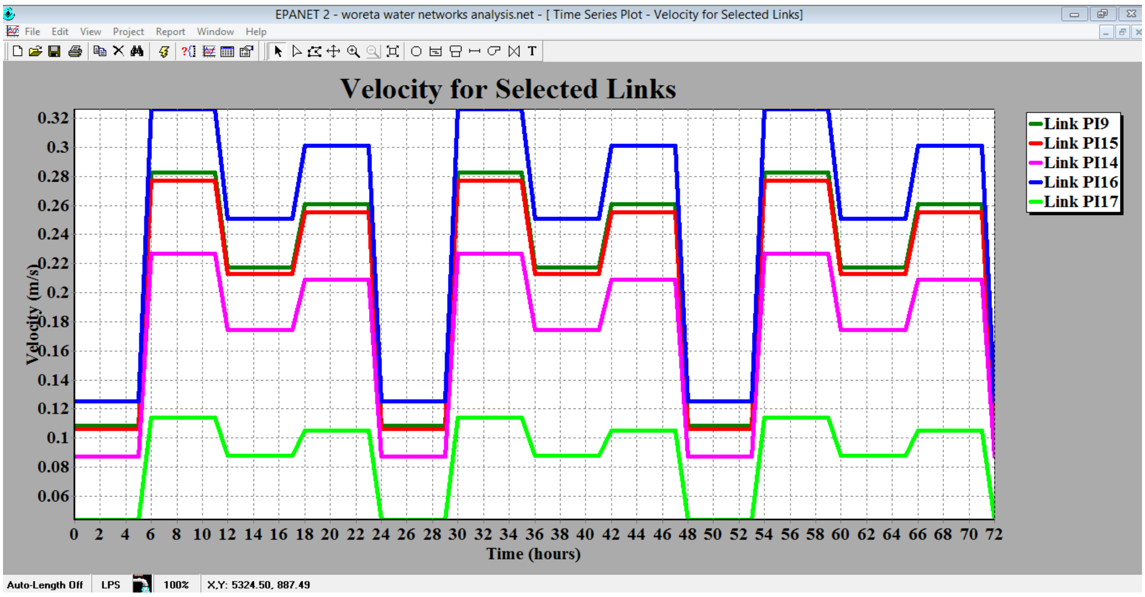

Fig.4.9: Velocity Variation for Selected Links

Low velocities are undesirable because they lead to low pipe flows, since discharge is a function of velocity. Also low velocities are undesirable for reasons of hygiene and sedimentation problem. In opposite way, high velocity, not more than 2.0 m/s and 2.5 m/s in distribution system and transmission system respectively to prevent erosion and high head losses(Dawe, 2000b; Gupta, 2006; MoWR, 2006).

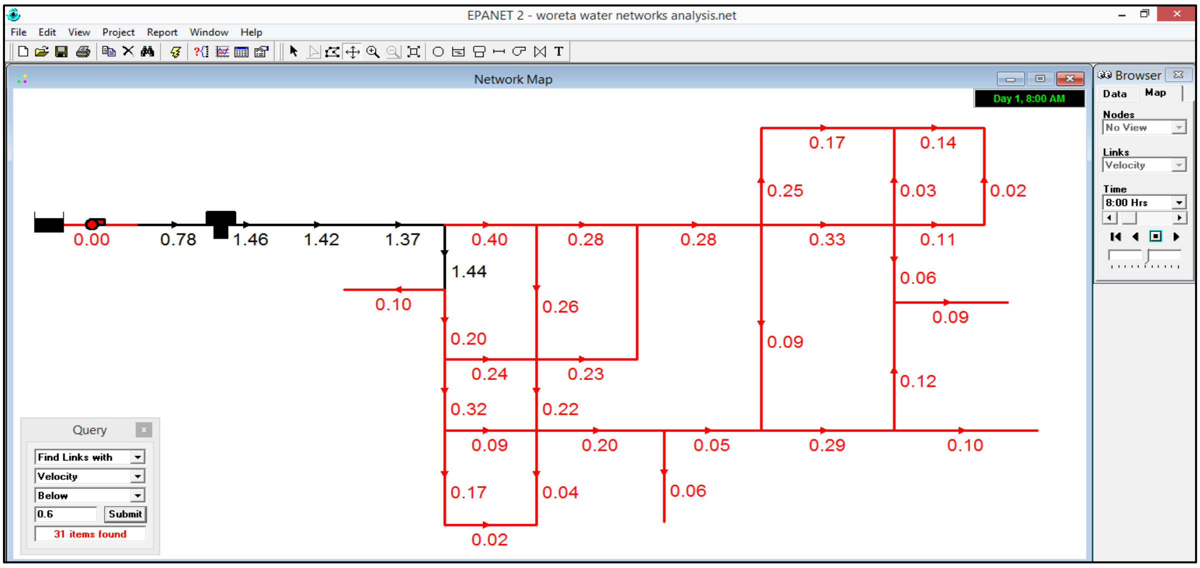

Fig.4.10: Query of Velocity below 0.6 m/s at Peak Hour Demand

Most links those 30 out of 36 velocities in Woreta town water supply distribution system below 0.6 m/s during peak hour demand as shown above the fig4.10 and below fig.4.11. This also exposed the water network for high sedimentation and poor water quality problem.

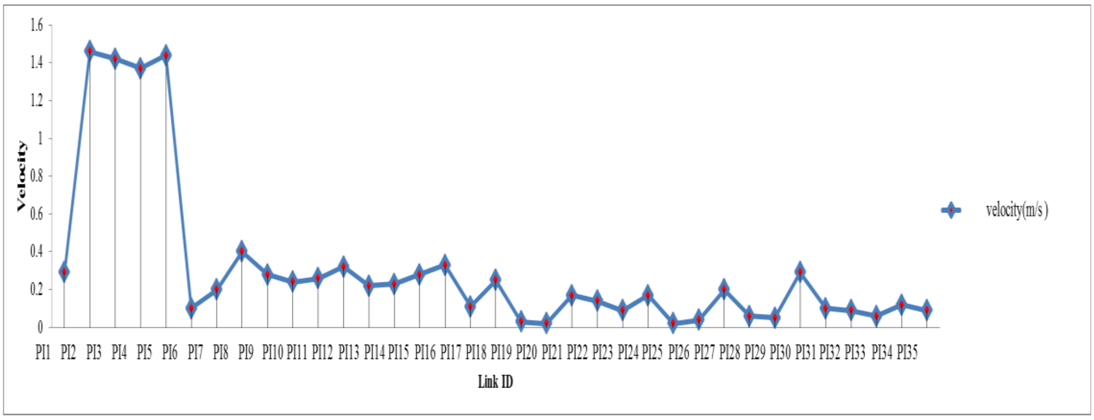

Fig.4.11: Nodal Velocity at Peak Hour Demand

Pipes, especially older, unlined, metal pipes, may experience an internal buildup of deposits due to mineral deposits and chemical reactions within water supply pipe when allowance velocity below minimum. This buildup can result in loss of carrying capacity, reduced pressures, and poorer water quality (Walski et al., 2003). Then corrosion pipe also cannot resist high pressure then become pipe burst and water leakage.

4.5.3. Pressure Head in Water Distribution System

Pressure modeled at each node on the distribution mains as shown below fig.4.12 EPANET workspace.

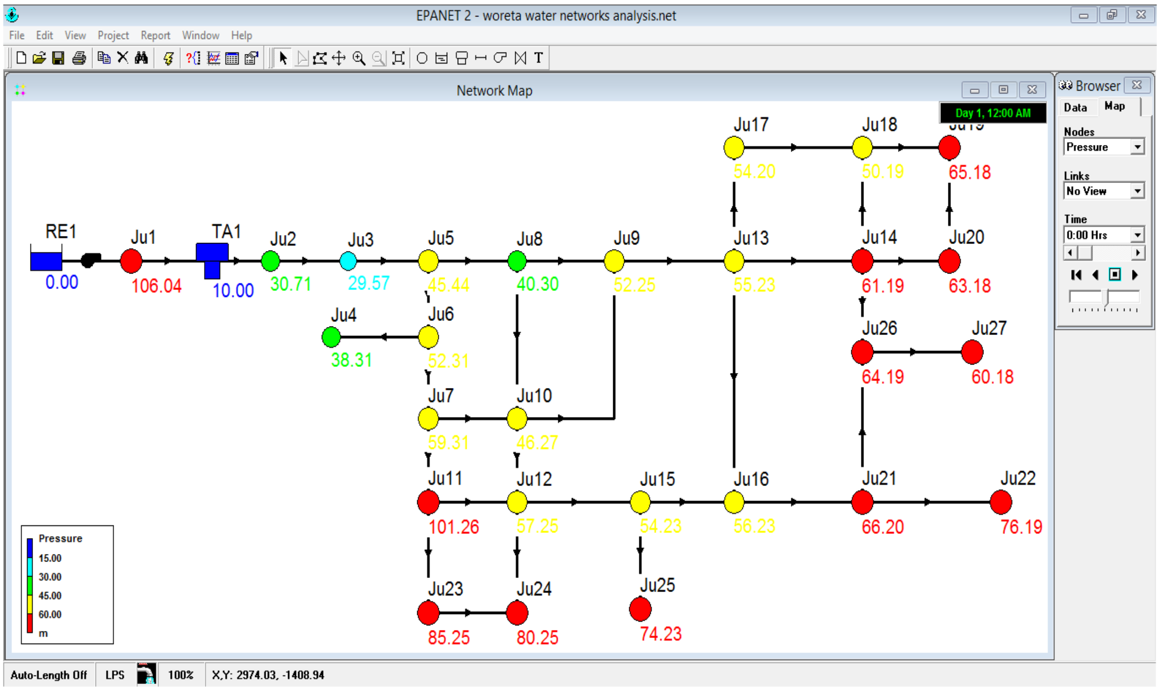

Fig.4.12: Simulation of Pressure Head

As shown from the output pressure differs from time to time when demand pattern varies. The result shows that pressure decrease when demand increase at peak time and increase when demand decrease as shown in fig.4.13.

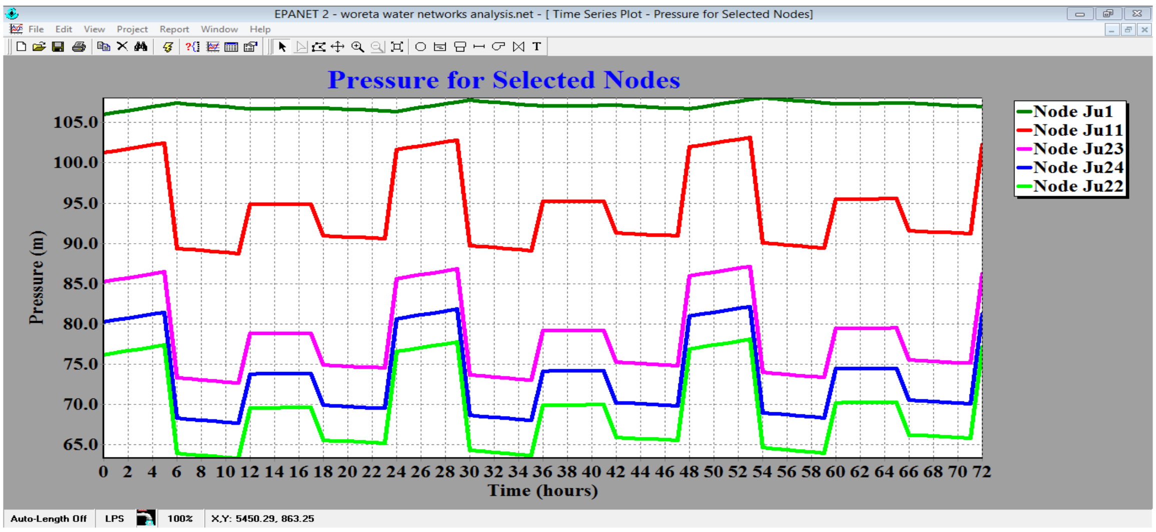

Fig.4.13: Fluctuation Pressure Head with Time

A query was done using EPANET software for all nodes, which observed from the fig.4.14 EPANET workspace 14 out of the 29 nodes have pressure heads above the required minimum pressure head of 15.00m and below maximum pressure require head 60m. However, 12 of nodes had maximum pressure head above maximum limited of pressure head. Off course at node JI-1, pressure head 106.04m above maximum limit and at reservoir and tank below minimum limit. At exceptional condition minimum and maximum perssure head would be 10 and 70 m respectively (MOWR, 2006). Transmission pipelines are component of the water supply system importance of the transmission pipe is like a neck of the system. Due to high-pressure (106.04m) break or flow block on the transmission pipe will completely cease the water supply system.

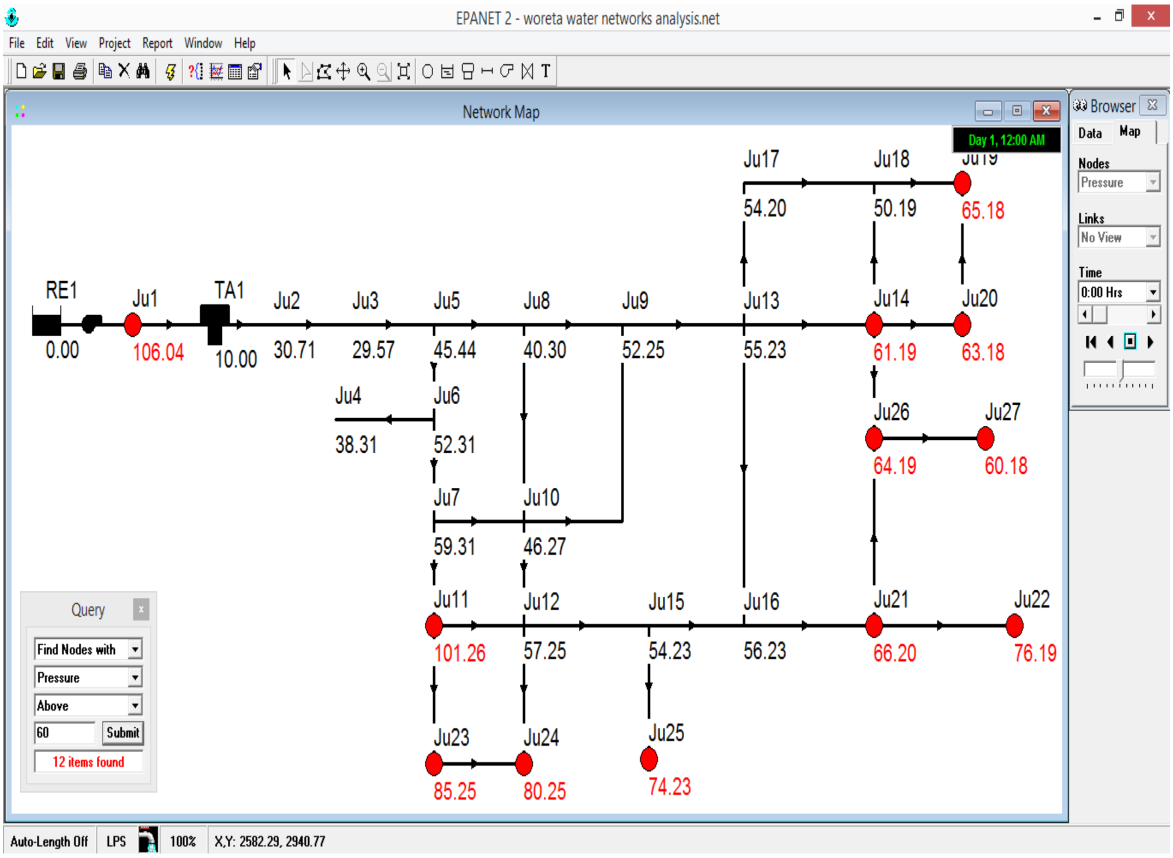

Fig.4.14: Query of Pressure above 60 m

The operating pressures at normal condition in the distribution network minimum and maximum pressure head should be 15m and 60m respectively. And also pressures at the various nodes as a rule, a minimum of head(15m) was considered adequate during peak hour demand to satisfy for most of the day(MoWR, 2006). From fig.4.14 above shown that the output had good pressure head between 15 and 60 m were JI2, JI3, JI4, JI5, JI6, JI7, JI8, JI9, JI10, JI12, JI13, JI15, JI16, JI17 and JI18.

Simulation of hydraulic network shows that the lower elevation area was exposed higher-pressure head as shown below from the contour map fig.4.15, 4.16 and table4.4.

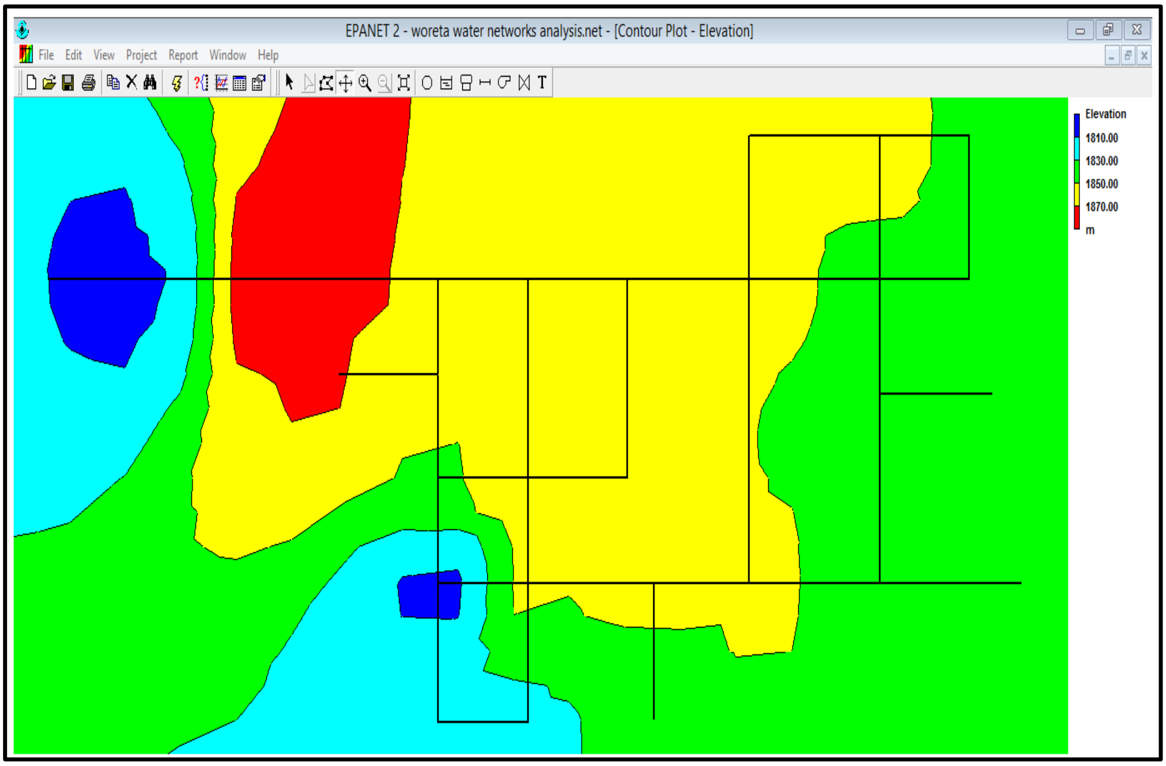

Fig.4.15: Conotur Map of Elevation

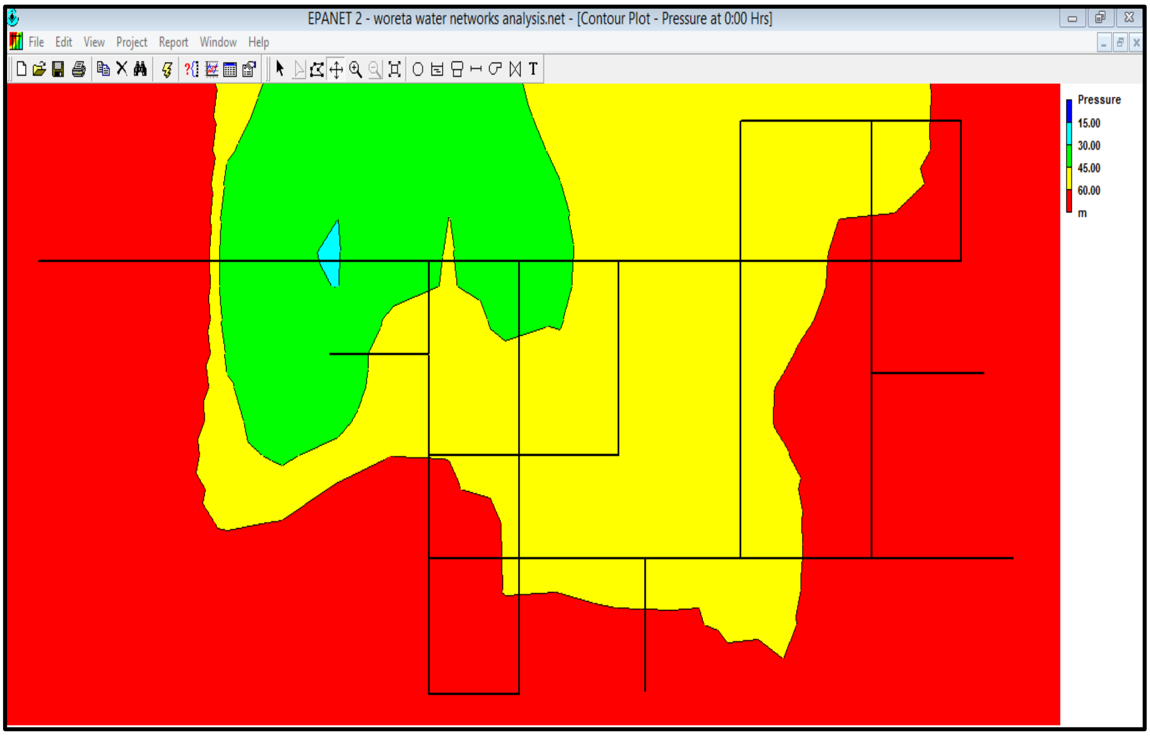

Fig.4.16: Contour Map of Pressure Head

Table4.4: Pressure Head and Elevation

Node ID	Elevation (m)	Pressure (m)
Ju1	1807	106.04
Ju2	1879	30.71
Ju3	1880	29.57
Ju4	1870	38.31
Ju5	1863	45.44
Ju6	1856	52.31
Ju7	1849	59.31
Ju8	1868	40.3
Ju9	1856	52.25
Ju10	1862	46.27
Ju11	1807	101.26
Ju12	1851	57.25
Ju13	1853	55.23
Ju14	1847	61.19
Ju15	1854	54.23
Ju16	1852	56.23
Ju17	1854	54.2
Ju18	1858	50.19
Ju19	1843	65.18
Ju20	1845	63.18
Ju21	1842	66.2
Ju22	1832	76.19
Ju23	1823	85.25
Ju24	1828	80.25
Ju25	1834	74.23
Ju26	1844	64.19
Ju27	1848	60.18
RE1	1807	0
TA1	1901	10

This also exposed pressure deficiency at elevated area and high buildings when there was high demand from lower area and in opposite at low demand pipe breakage, burst, and high water leakage occur. The pressure in the network is generally not good and the network was not efficient to deliver quality water. Moreover, according to interview data in Woreta town, even though it is not logical to expect a consumer to draw water directly from the transmission main all the time, due to the higher-pressure deficiency from buildings and elevated area some people had connection from transmission line. This also problem by itself and exposed the dropped of pipes pressure head when distribution lines receive water. Additionally during surveying, distribution system had unclear defined two pressure zone because between high pressure and low-pressure zones many interconnections. According to this research output for Woreta town, three clearly defined pressure zones (for elevated area, lower area and commercial or institutional area) should be design. Because of during intermit supply pressure become above simulated pressure head. this also affects the hydraulic performance of the network(Zyoud, 2003; Ramesh et al., 2012; Datwyler, 2014). Moreover, as shown above (fig.4.7) nodal supply failed to feed nodal demand and flow conditions change slowly due to low flow velocity, the resultant pressure changes are small and might not threaten the pipeline. When add new source and updating existing water supply system flow conditions change rapidly and large pressures generated. Then develops sufficient magnitude of pressure head to burst pipes and damage equipment(Karney, 1990).

5. CONCLUSION AND RECOMMENDATION

5.1. Conclusion

The main goal of this research was to evaluating hydraulic performance of urban water supply in Woreta town. Evaluating water supply, water demand, water loss and simulation of hydraulic parameters to identify deficiencies were main part of this study. At the end of the analysis, the following results were founds:

- Estimation and know number of population is key tool to find the exact resource to meet the demand. In Woreta, population number did not know clearly. In 2015 Woreta town, population registered by WWSS, WFEDO, based on CSA 2007 census and based on CSA 2008 urban rank were 38,254, 40,386, 35,269 and 38,765 respectively. Since population essential to estimate resource needed by the community and to know the gap between water supply and water demand, the exact number of population of Woreta town should be clearly know.
- In Woreta town, there was extreme shortage of water and its water supply distribution system was incapable to deliver enough water to meet the past and current demand of the town. From the year 2011-2015 water supply (litter per person per day) of the town was less than unit demand/per capital demand/ (20l/s).This was due to the rapid increasing population growth, urbanization, infrastructure breakage; this made high pressure on the existing infrastructure and disrupted the efficient water distribution system. Moreover, another problem in Woreta town was erratic of power supply that hampered continuous operation of the water supply system. Additionally, water supply service coverage in Woreta town was 78.9%. Due to limited source capacity, rapid population growth, expansion of new area and illegal settlement around the town 21.1% of Woreta town population was lived without water supply service connection.
- Water loss was high in Woreta town and increasing in 2011-2015. That was due to expansion road expansion, insufficient maintenance, poor rehabilitation measures and intermittent water supply.
- In this research hydraulic network, modeling was constructing with EPANET2 software due to its hydraulic and water quality modeling capabilities. Moreover economical free and has ability to perform steady and extend period simulation. During perform steady state simulation nodal supply was fail to satisfied average

nodal demand at single period. This shows that water supply unable to meet nodal demand. Then additional sources should be necessary to satisfy current average nodal demand and future demand.

- From extend period simulation the flow velocity, 30 out of 36 links were below minimum allowance velocity (0.6m/s). Low velocity exposed water age, sedimentation and hygiene problem. In other direction, Woreta is rapidly grown town and to assure this growth rapid expansion of water distribution system to deliver water for new area is needed. The expansion of water supply coverage for new area will exposes for water crisis and water age due to deficiencies of velocity.
- Pressures at 12 out of 27 junctions were higher than recommended pressure (60m) during low demand period. The higher the pressure exposed to breakage, leakage and burst of pipe during intermittent supply and excess water deliver at low demand. This increase water loss and lowered performance of water distribution system. Then to optimize the pressure it should be design three clearly defined pressure zones for elevated, lower and commercial area.

5.2 Recommendation

In this research, recommendations were stating based on the output of the study, after analysis of raw data. These recommendations used for, as indication of farther observation and carefully study should be do for the future in this area. Then after carefully examined the output of study the following recommendations were stated:

- Clearly unknown number of population in Woreta town was a big challenge to determine the gap between demand and supply. Then population of Woreta town should be clearly register and known to search water source to meet the demand. Additionally known population of the town will use to know population from each pressure zone. This also uses to know population at each node.
- From Production and population data analysis, additionally from hydraulic network modeling simulation shown that supply was fail to meet current demand. Therefore, add new source and upgrade capacity of existing water distribution system needed to deliver adequate water.
- According to production and consumption data analysis, water loss was increase with time for the last 5 years. Then old component of distribution system should be replace with new component and farther study should be done on the studied area.

- ❖ Hydraulic network simulation shown flow fails to achieve minimum allowances velocity (0.6 m/s) at peak hour demand. This exposed water age, sedimentation and hygiene problem. Due to this reason farther water quality analysis must be do.
- ❖ Pressure during hydraulic simulation in the distribution system found that high at low elevated area (above 60 m) when there was low demand. These made elevated and commercial (which has high buildings) area exposed shortage of water during limited source because of water draw dawn to low elevated area. Additionally during intermittent supply or when new source added on the system pressure increase. This also increases breakage, burst, and leakage of pipe. In order to tackle this problem three clearly defined pressure zone for elevated area, commercial area and lower elevated area should be design.

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Appendix

APP.Table1: Water Balance in Woreta Distribution System

year	water production (m^3/year)	Water consumption (m^3/year)	water loss (m^3/year)	Water loss (%)
2011	224493.6	168145.71	56347.89	25.1
2012	230198.81	172004.55	58194.26	25.28
2013	220596.39	162844.25	57752.13	26.18
2014	253370.27	181134.41	72235.86	28.51
2015	247230.42	175360.54	71869.88	29.07

APP.Table2: Demand vesus Supply

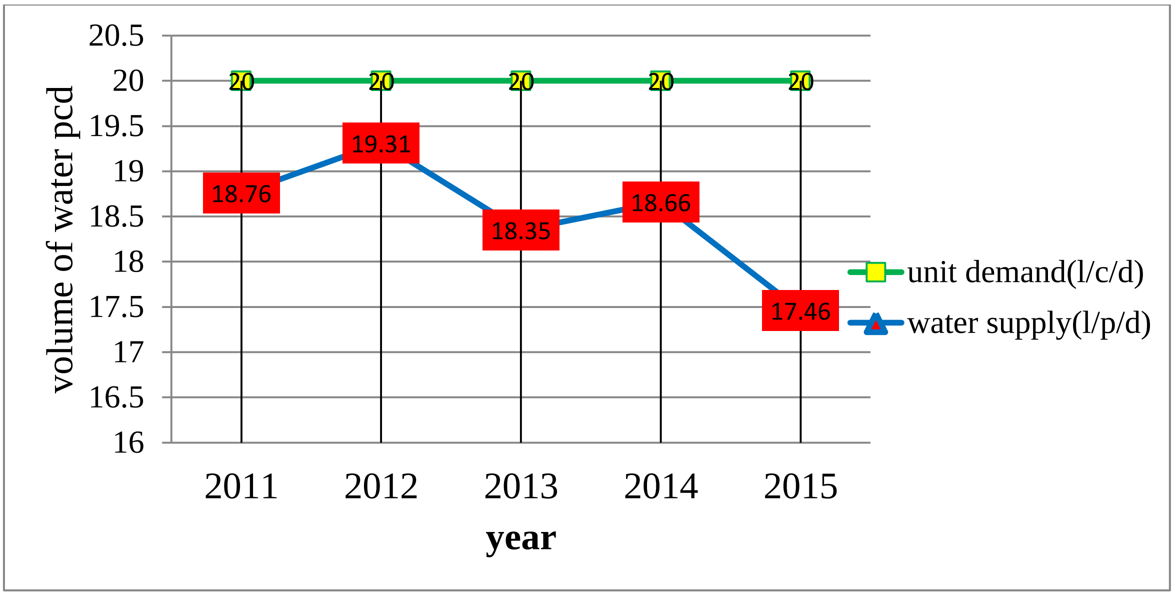
Node ID	base demand (lps)	Supply (lps)
Ju1	0	0
Ju2	0.29	0.14
Ju3	0.25	0.12
Ju4	0.39	0.19
Ju5	0.34	0.17
Ju6	0.31	0.16
Ju7	0.38	0.19
Ju8	0.29	0.14
Ju9	0.41	0.2
Ju10	0.33	0.17
Ju11	0.25	0.12
Ju12	0.25	0.12
Ju13	0.34	0.17
Ju14	0.45	0.23
Ju15	0.35	0.17
Ju16	0.28	0.14
Ju17	0.29	0.14
Ju18	0.24	0.12
Ju19	0.62	0.31
Ju20	0.36	0.18
Ju21	0.25	0.12
Ju22	0.38	0.19
Ju23	0.56	0.28
Ju24	0.25	0.12
Ju25	0.25	0.12
Ju26	0.39	0.19
Ju27	0.34	0.17
RE1	-	-9.26

APP.Table3: Network Links at 8:00 Hrs.

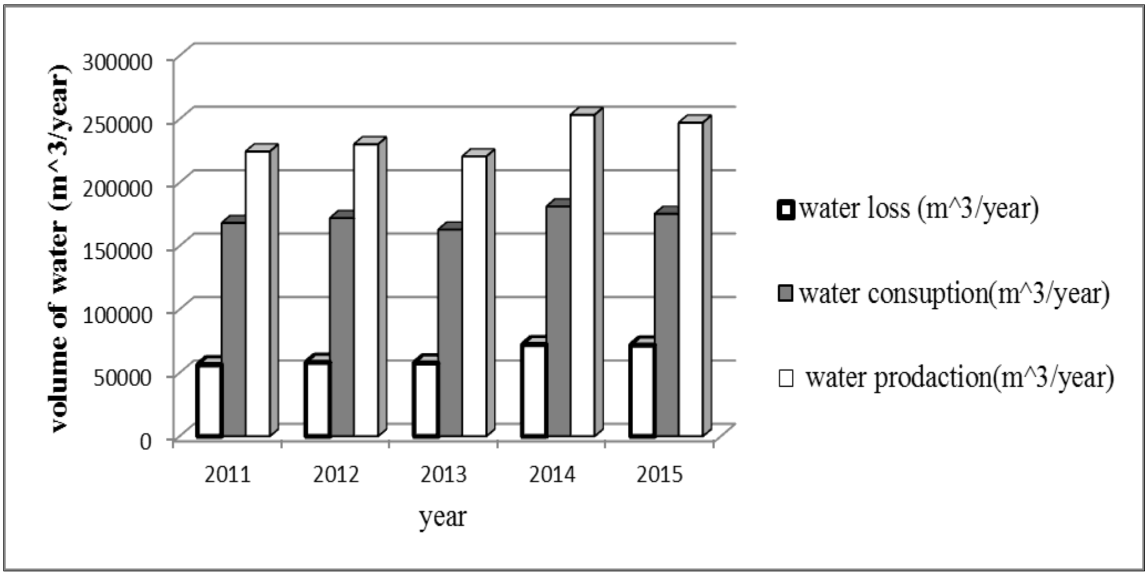
Link ID	Length m	Diameter mm	Roughness	Flow LPS	Velocity m/s	Unit Head loss m/km	Friction Factor
PI1	2221.8	200	100	9.1	0.29	0.89	0.042
PI2	188.9	100	100	11.49	1.46	40.08	0.037
PI3	21.95	100	100	11.11	1.42	37.68	0.037
PI4	186	100	100	10.79	1.37	35.66	0.037
PI5	14.85	80	100	7.23	1.44	50.42	0.038
PI6	18.25	80	100	0.51	0.1	0.37	0.057
PI7	25.1	200	100	6.32	0.2	0.46	0.044
PI8	233.5	100	100	3.11	0.4	3.57	0.045
PI9	118.25	80	100	1.42	0.28	2.47	0.049
PI10	252	150	100	4.22	0.24	0.87	0.045
PI11	67.7	80	100	1.32	0.26	2.15	0.049
PI12	91.1	80	100	1.61	0.32	3.13	0.048
PI13	90.6	80	100	-1.1	0.22	1.54	0.051
PI14	186	150	100	4.01	0.23	0.79	0.045
PI15	65	150	100	4.89	0.28	1.14	0.044
PI16	78.4	80	100	1.64	0.33	3.23	0.048
PI17	86	80	100	0.57	0.11	0.46	0.056
PI18	91	80	100	1.23	0.25	1.91	0.05
PI19	89.6	80	100	0.16	0.03	0.04	0.067
PI20	71	80	100	0.1	0.02	0.02	0.068
PI21	86	80	100	0.86	0.17	0.97	0.052
PI22	55.6	80	100	0.7	0.14	0.67	0.054
PI23	253	80	100	0.45	0.09	0.29	0.058
PI24	78.4	80	100	0.84	0.17	0.94	0.053
PI25	253	80	100	0.11	0.02	0.02	0.072
PI26	78.4	80	100	0.21	0.04	0.07	0.065
PI27	75	80	100	1.01	0.2	1.31	0.051
PI28	78	80	100	0.33	0.06	0.16	0.06
PI29	66	80	100	0.23	0.05	0.08	0.063
PI30	78.5	80	100	1.44	0.29	2.55	0.049
PI31	175	80	100	0.49	0.1	0.35	0.057
PI32	158.4	150	100	1.58	0.09	0.14	0.052
PI33	78	80	100	0.33	0.06	0.16	0.061
PI34	80.4	80	100	-0.62	0.12	0.54	0.055
PI35	178	80	100	0.44	0.09	0.28	0.058
PU1	-	-	-	9.1	0	-107.18	0

APP.Table4 Network Table - Nodes at 0:00 hrs.

	Co-ordinate		Elevation	Base Demand	Demand	Head	Pressure
Node ID	x	y	m	LPS	LPS	m	m
Ju1	358795	1319033	1807	0	0	1913.04	106.04
Ju2	358956	1318212	1879	0.29	0.14	1909.71	30.71
Ju3	358937	1318206	1880	0.25	0.12	1909.57	29.57
Ju4	358815	1318334	1870	0.39	0.19	1908.31	38.31
Ju5	358789	1318301	1863	0.34	0.17	1908.44	45.44
Ju6	358795	1318336	1856	0.31	0.16	1908.31	52.31
Ju7	358806	1318368	1849	0.38	0.19	1908.31	59.31
Ju8	358587	1318441	1868	0.29	0.14	1908.3	40.3
Ju9	348481	1318487	1856	0.41	0.2	1908.25	52.25
Ju10	358627	1318479	1862	0.33	0.17	1908.27	46.27
Ju11	358579	1318971	1807	0.25	0.12	1908.26	101.26
Ju12	358384	1318920	1851	0.25	0.12	1908.25	57.25
Ju13	357960	1318872	1853	0.34	0.17	1908.23	55.23
Ju14	357869	1318629	1847	0.45	0.23	1908.19	61.19
Ju15	358185	1318872	1854	0.35	0.17	1908.23	54.23
Ju16	358043	1318755	1852	0.28	0.14	1908.23	56.23
Ju17	357792	1318432	1854	0.29	0.14	1908.2	54.2
Ju18	357792	1318388	1858	0.24	0.12	1908.19	50.19
Ju19	357616	1318508	1843	0.62	0.31	1908.18	65.18
Ju20	357567	1318807	1845	0.36	0.18	1908.18	63.18
Ju21	357977	1318816	1842	0.25	0.12	1908.2	66.2
Ju22	357856	1318883	1832	0.38	0.19	1908.19	76.19
Ju23	358549	1318853	1823	0.56	0.28	1908.25	85.25
Ju24	358347	1319111	1828	0.25	0.12	1908.25	80.25
Ju25	3518853	1318955	1834	0.25	0.12	1908.23	74.23
Ju26	357930	1318955	1844	0.39	0.19	1908.19	64.19
Ju27	357652	1318913	1848	0.34	0.17	1908.18	60.18
RE1	358795	1319033	1807		-9.26	1807	0
TA1	359122	1318312	1901		4.84	1911	10
pump			1812	-	-	105	-



App.Fig.1: Woreta Town Per Capital Demand versus Unit Demand 2011-2015



App.fig.2:Water Balance in Woreta Distribution System 2011-2015

Breakage of pipe during road expansion

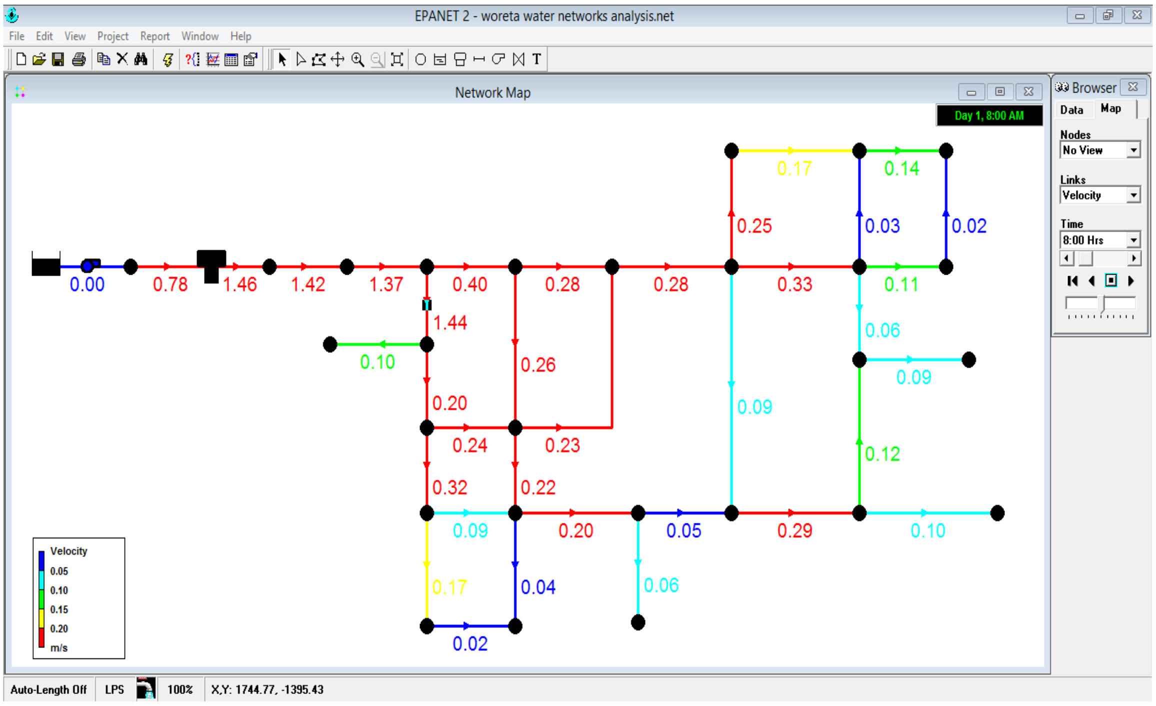
Breakage of pipe near to source

App.Fig.3: Water Loss During field Survey

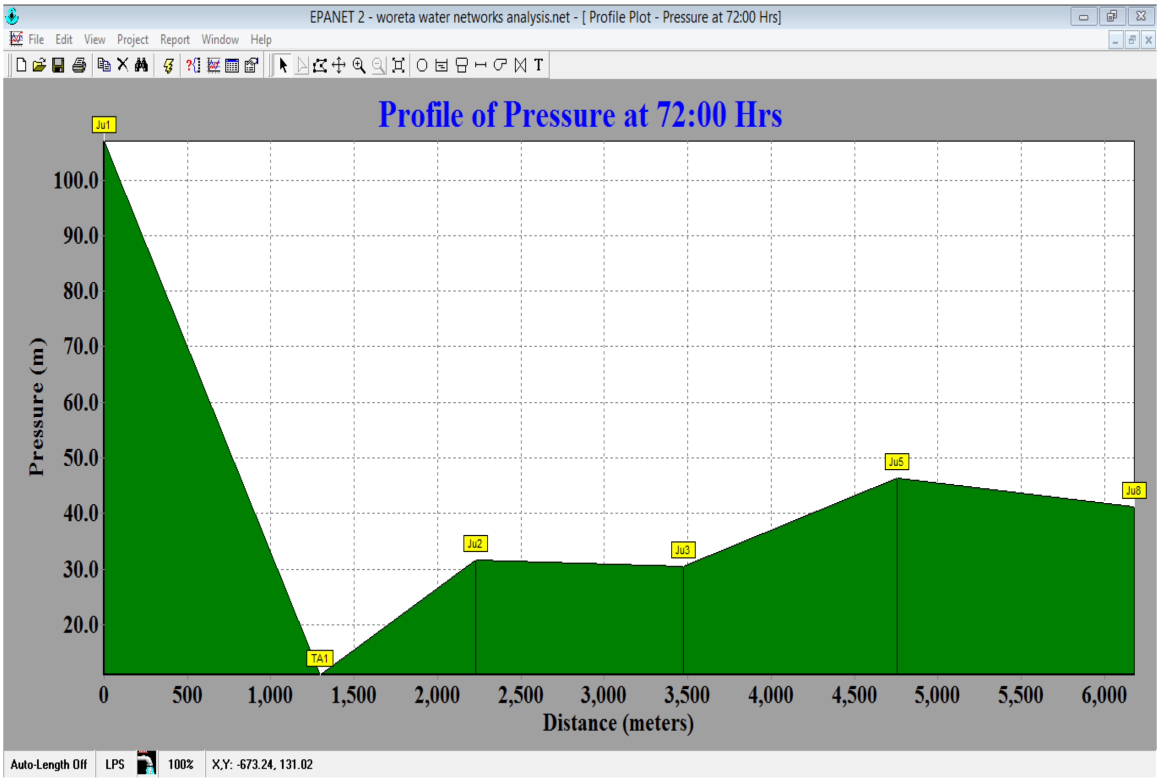
Rice Land around Woreta Immigrants to Woreta Town People for Search Jol

App.fig.4: Immigrant People to Woreta Town for Search Job

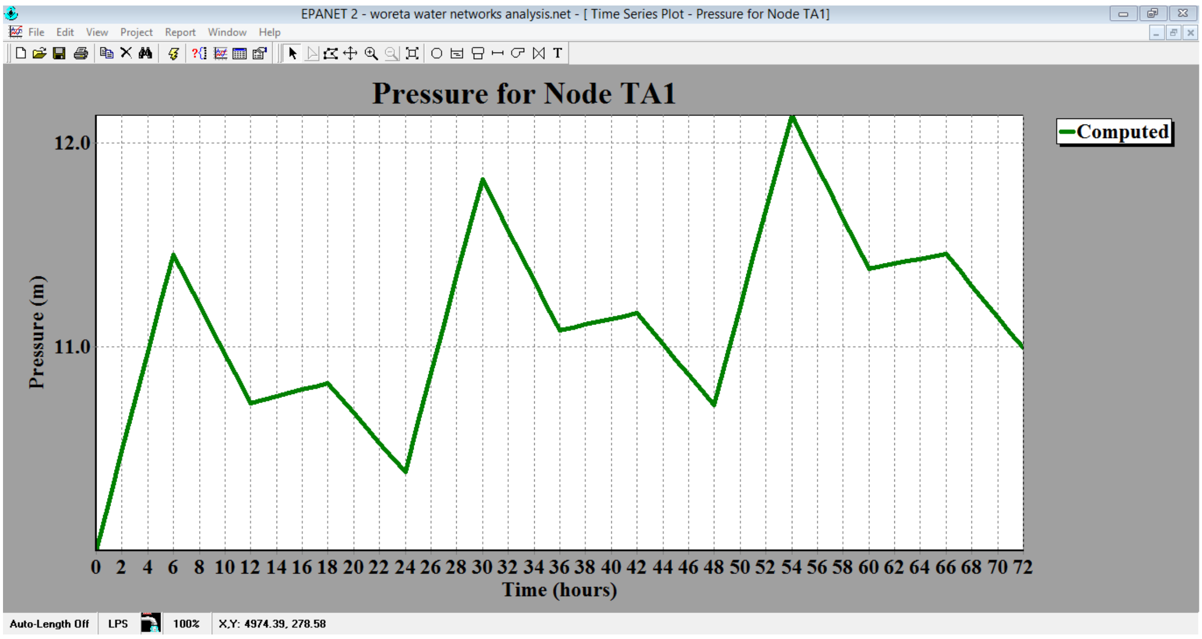
App.Fig.5: Commercial Area in Woreta Town



App.Fig.6: Modeling of Velocity on EPANET Workspace



App.Fig.7: Pressure Profile at Each selected Node



App.Fig.8: Tank Pressure Head in Time Interval

QUESTIONERS:

1. What are Woreta water supply source…………………………….………………….?

2. Is Woreta water supply continuous supply? If it is no, what is the cause of it?

…………………………………………………………………………………..….?

3. Do you think the existing water supply meet the current demand of Woreta………..…?

If no, is there scarce of resource……………………………………………………………?

Where communities fetch water to meet their demand--------------------?

4. What you think? Why the existing water supply not satisfied the community demand---?

5. How shortages of water affect community life----------------------------------?

6. Is there high water loss in Woreta--?

7. How many times you see over flow from reservoir-----------------------------------

8. Do you see Pipe breakage----------------------------------? How it occurs ---?

9. Do you see back flow--?

10. Please mention all problems of Woreta town, water supply system

---?

---?