JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

CHAIR OF HYDROLOGY AND HYDRAULIC ENGINEERING

EVALUATING HYDRAULIC PERFORMANCE OF URBAN WATER SUPPLY DISTRIBUTION SYSTEM: A CASE STUDY OF DEBRE MARKOS, EAST GOJJAM ZONE, AMHARA REGIONAL STATE, 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

DANIEL YALEMZEWD

DECEMBER, 2016
JIMMA, ETHIOPIA

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DECEMBER, 2016

JIMMA, ETHIOPIA

DECLARATION

I, undersigned, declare that this thesis, entitled as **Evaluating Hydraulic Performance of Urban Water Supply Distribution System*: A Case Study of Debre Markos, East Gojjam Zone, Amhara Region, Ethiopia*** is my original work and has not been presented for a degree in Jimma University or any other university and that all sources of materials used for the thesis have been fully acknowledged.

Name	Signature	Date
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This thesis has been submitted for examination with my approval as a university supervisor.

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ABSTRACT

Water supply distribution systems are 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. Debre Markos town, which is found in East Gojjam zone Amhara Region, is rapidly grown commercial town. To assure this growth a potable, reliable and adequate quantity of water supply is needed. Then evaluating the hydraulic performance of water supply distribution system of the town was paramount importance 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. Primary and secondary data were collected. Hydraulic network Simulation with Bentley WaterCAD was carried out to track water supply, water demand, flow velocity and pressure at each node. Both single and extended period simulation were carried out. According to the research result, the following were found. Debre Markos population in 2011, 2012, 2013, 2014 and 2015 was 70910,73959,77140,80457 and 83916 respectively. Debre Markos 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 14.4, 17.5, 19.4, 29.6, and 24.2 respectively. During steady state simulation water supply failed to meet nodal demand. Moreover, at extend period pressures at 39 out of 69 junctions were higher than recommended pressure (60m) and the flow velocity 78 out of 97 links were below minimum allowance velocity (0.6m/s).

Key words: *Water Distribution System, Water Demand, Water Loss, Water Supply, Bentley WaterCAD V8i, Hydraulic Performance and Hydraulic Parameters.*

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First and for most, Praise and thanks be to the **Almighty God** for giving me health and strength to pass through all confronts during my life. I would like to thank my advisor **Dr.Brook Abate (Ph.D**) and my co-advisor **Mr.Zeinu Ahmed (Ph.d candidate)** for their kind academic advice, continuous support during the data preparation and proposal writing and provided me to had fruitful knowledge about research writing methods. Last but not the least; I am especially thankful to my families for their continuous support.

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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
DMWSS	Debre markos Water Supply Service
DMFEDO	Debre markos Finance and Economics Development Office
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 North western Switzerland
WB	World Bank
WDS	Water Distribution System
WHO	World Health Organization

ABBREVIATION

App.Figure	Appendix Figure
App.Table	Appendix Table
D	Diameter
DI	Ductile Iron
DCI	Ductile Cast Iron
DN	Nominal Diameter
E.C	Ethiopian Calendar
GI	Galvanized Iron
gpm	gallon per minute
HL	Head Loss
J	Junction
Km	Kilometer
L	Length
L/s	Liter per Second
Lpcd	Litter per Capital per Day
M	Meter
M^3	Cubic Meter
m/s	Meter per Second
mm	Millimeter
P	Pipe
P	Pump
Q	Discharge
R	Reservoir
St	Steel
T	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 and 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 were 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 pipe line was layed from Entoto 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 were 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 were used through out the country towns (Behailu, 2012). Debre markos town got its first piped water service in 1980. However, this project currently is not giving service for Debre markos town. The water supply distribution system, which give service for Debre markos town at the moment 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, a 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 to deliver potable water to satisfy combination of domestic, commercial, industrial, and fire fighting demands at required time with sufficient hydraulic performance (Zyoud, 2003; M. and 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 at intermittent systems (Gottipati and 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 and 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 and 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 which 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 allmost the entire water distribution network gave birth to water distribution network (Oyelowo, 2013; Henshaw and Nwaogazie, 2015).

In this study the hydraulic model used is Bentley WaterCAD V8i for evaluating the hydraulic performance of urban water supply system to the studied area. Because Bentley WaterCAD was open-structured, 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. Bentley WaterCAD 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 recommend solution. Generally, adopting computer models to design water distribution networks such as Bentley WaterCAD, one will have enough results including number of graphs, tables and comparison 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) was 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 and 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 for 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 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 and Nanduri, 2014).

Debre markos is attractive, it becames home of different banks, education sector, government and non-governmental organizations. Moreover, in Debre markos many people immigrated for searching job and many regional conference are being conducted. This increase in population brought stress and exposed the town for sever water shortage. On the other hand, Debre markos town was considered as one of the rapidly grown commercial towns and has good prospect for development. Hence, to assureits progress a potable, reliable and adequate water supply system is 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 Debre markos existing 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 Objectives

- To evaluate water supply versus water demand of Debre markos
- To assess water loss in the distribution system
- To study the hydraulic parameters of the water distribution system of Debre markos Town using Bentley Water CAD V8i software

1.4 Research Questions

- Is the existing water supply satisfied current demand of Debre markos town?
- Is there water loss in water distribution system before reaching customer?
- Are Debre markos water supply system hydraulic parameters good?

1.5 Scope of the Study

The scope of this research include: evaluating water supply versus demand, water loss and hydraulic network modeling of 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 necessary 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 Debre markos water supply system, the demand of livestock assumes from river or other source and un-meter water loss and illegal water connection are considered as loss.

1.6 Significance of the Study

The significance of this research was to evaluate hydraulic performance of Debre markos 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 fill 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 recommend solution to solve the problem. Therefore,the performance of water distribution system will be improved and fill the gap of water supply to meet current and future demand.

1.7 Limitation of the Study

The limitations of data collection completely depend on the municipality. There were some sorts of limitation while this document was prepared. Shortage of relevant data for the compilation of literature review, data would not organized in computerized system. A municipality must consider how it would be collected, store and evaluate the data to allow it to make the most informed decisions. There were challenges to hydraulic networks and allocated nodal demand. Pressure gages in distribution system were not functional to measure pressure. It was difficult to compare simulated and field survey results. The other problem was nodal demand allocation, because there were no clearly known population from each node.

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 get access to 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 socio-economic 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 of 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 vastgeographical areas to millions of consumers. These systems consist of elements, such aspipes, pumps, valves, storage tanks, reservoirs, meters, fittings, and other hydraulic appurtenances needed to carry water from source of potable water to the various point ofuse (Laura, 2006; Leirens *et al*., 2010; Elsheikh *et al*., 2013).

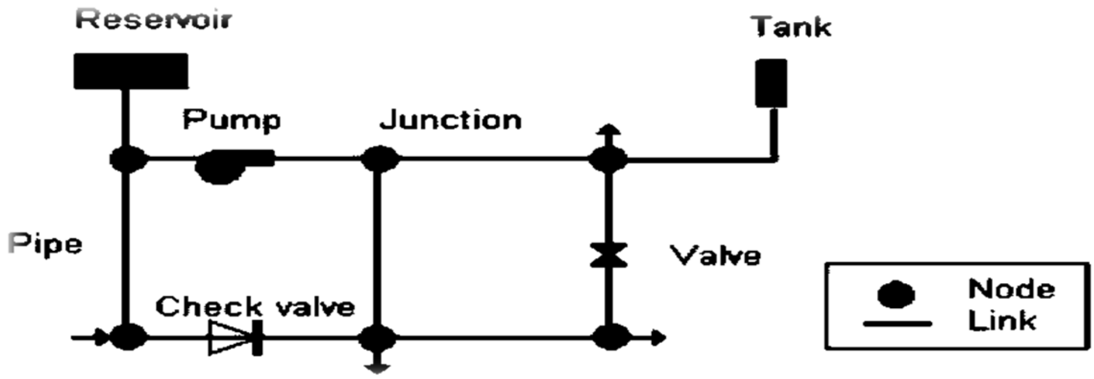

Figure 2.1 Physical components of water distribution system

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 instead 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 provide or isolate portions of the distribution system during emergencies. Ground level storage can consist of steel stand pipes 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

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 determined by the upstream and downstream conditions. Types of valve are listed 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 reducer 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 valve is used 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.

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 implemented 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 be used to represent reduced pressure back flow 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, man made reservoir and ground water sources. The intake structures and pumping stations constructed to extract water from these sources (Swammee *et al*., 2008).

The raw water is transported 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 maintained 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 maintained either by pumping or by gravitational potential (Laura, 2006). Transmission main and distribution network are calling water supply delivery system. The distribution network delivers water to consumers through service connections (Leirens *et al*., 2010).

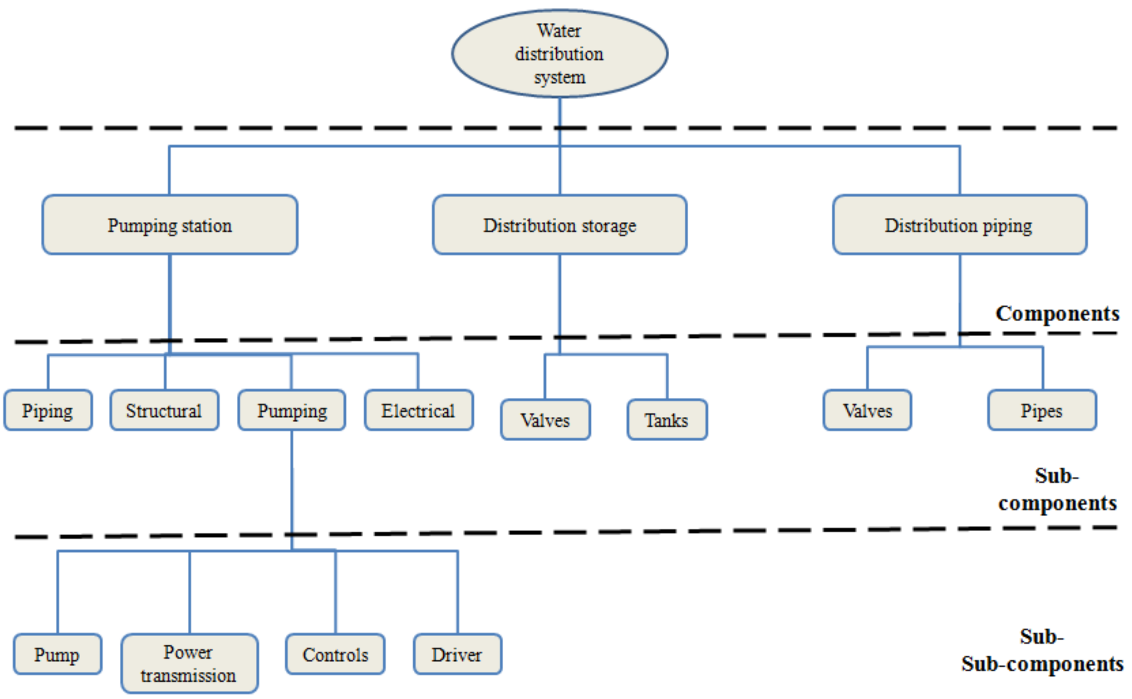

Figure 2.2 Water distribution system components (Swammee *et al*., 2008).

2.4 Methods of Water Supply System

Water can be delivered to customer in continuous supply system or intermittent supply system.

2.4.1 Continuous System

Continuous water supply system is the best system and water is supplied 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 fire fighting. 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 supplied after intervals, it is called intermittent system. The system has disadvantages such as pipe lines 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 and 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 pipe line is necessary, which should force the water to reach at every place. The methods of distribution system classified asgravity 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 be 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.

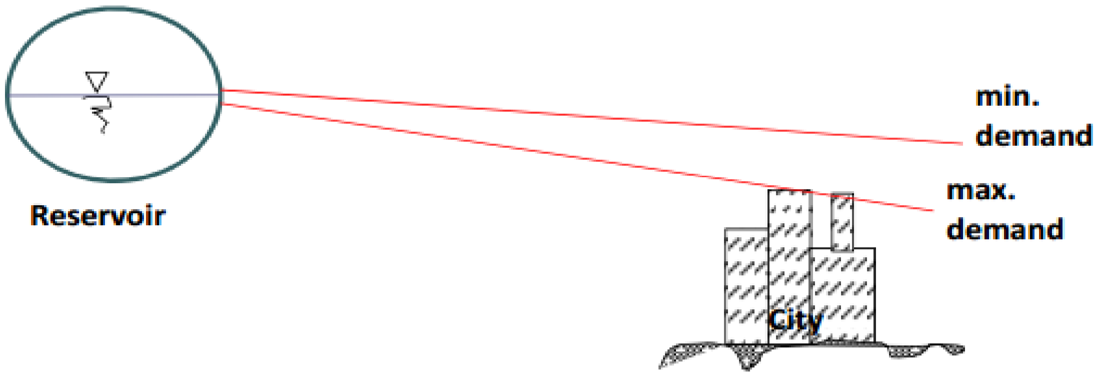

Figure 2.3 Water distribution system by gravity (Sharma, 2008).

2.5.2 Pumping System

Constant pressure head can 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.

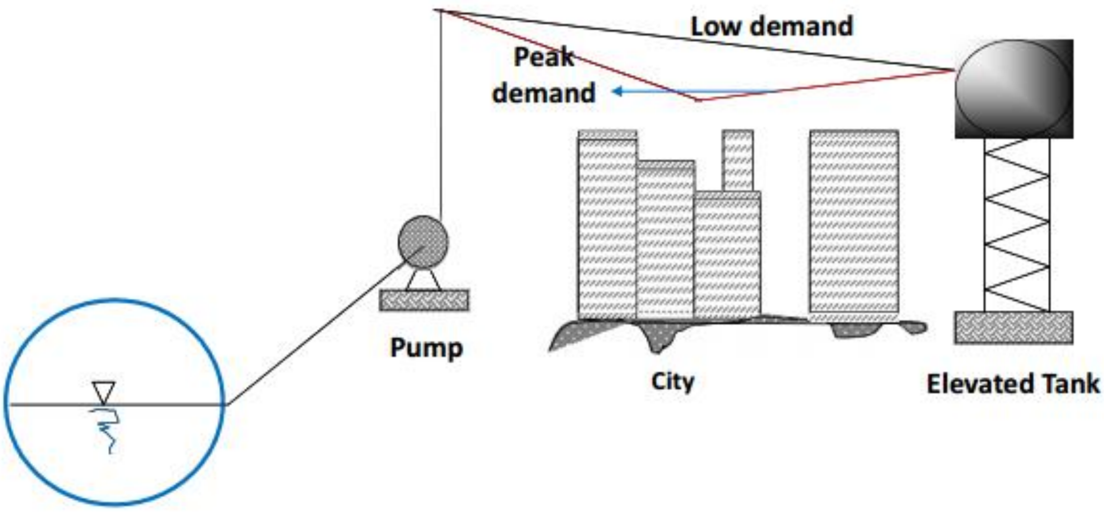

Figure 2.4 Water distribution system by pumping (Leirens *et al*., 2010).

2.5.3 Combined System

The pump connected to the mains as well as elevated reservoir 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 both the pumping station as well as elevated reservoir. This system is more reliable and economical, because it requires uniform rate of pumping but meet slow as well as maximum demand. The water stored in the elevated reservoir meets the requirements of demand during breakdown of pumps and for fire fighting (Venkateswara, 2005).

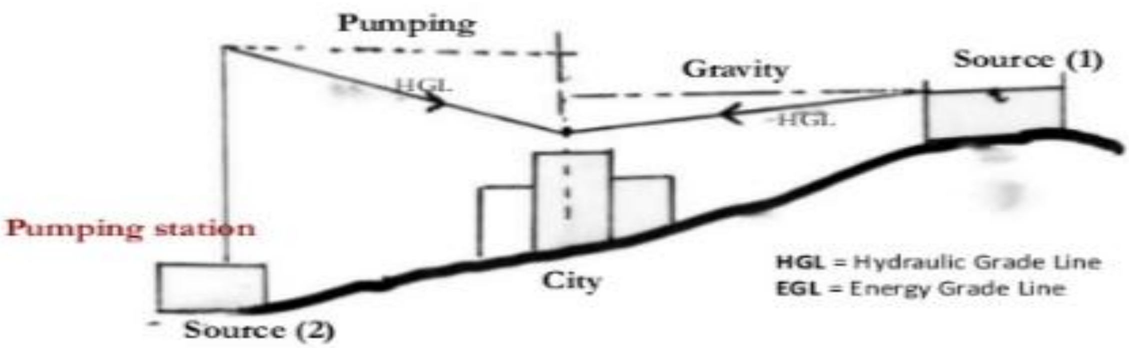

Figure 2.5 Water distribution system by combined system

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 sub divided into unauthorized consumption, meter inaccuracies and data handling errors. Real losses are made 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 accepted that real water losses cannot be eliminated. Nevertheless, there has been a large increase in the knowledge and development of state-of-the-art equipment, allowing us to manage water losses with in 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 Branched Or Dead End System

Branched configurations are also provided depending upon the general layout plan of thecity roads and streets. Rural water networks have branched configurations.

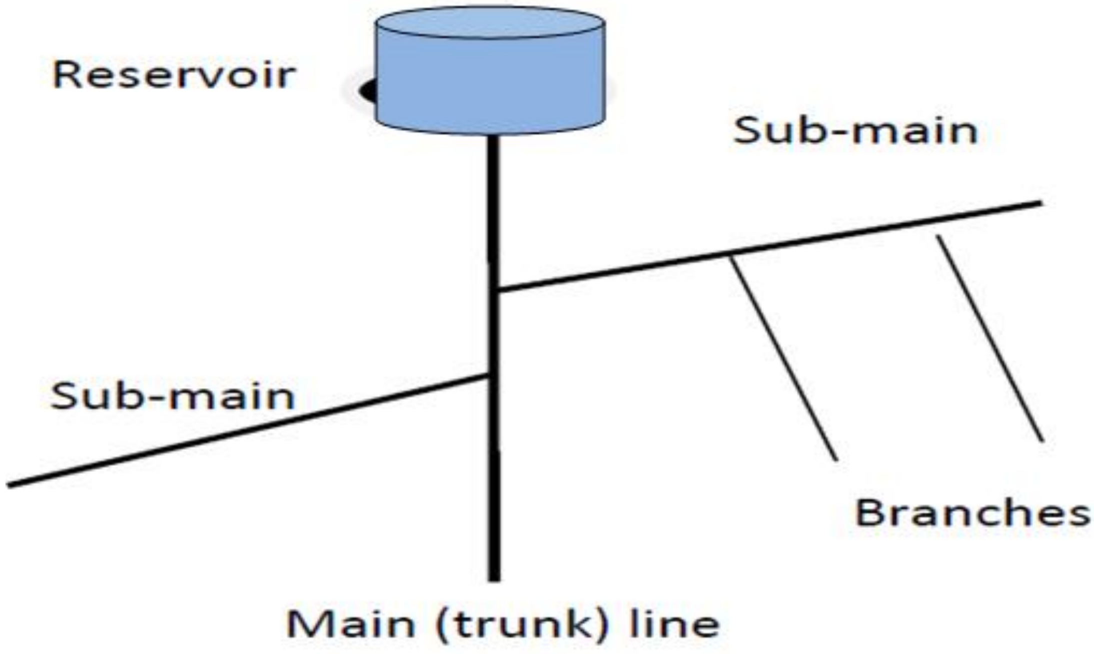

Figure 2.6 Branched system (HQDoA, 1986).

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).

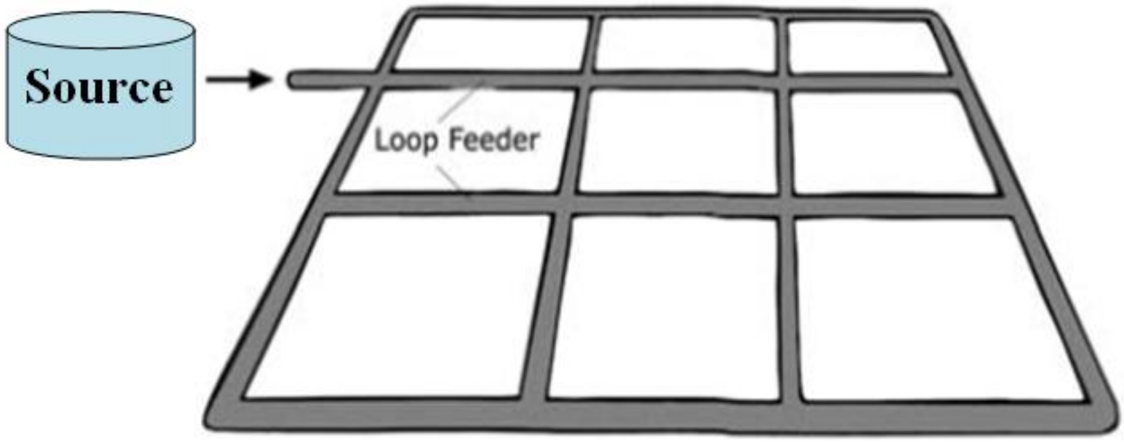

Figure 2.7 Looped system

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 used for water distribution systems whenever practicable. Water distribution into different areas is divided into zones, and each zone serving with a separate distribution reservoir and a separate distribution main (Gottipati and Nanduri, 2014; WHO, 2014).

2.8 Design of Hydraulic Network

Water distributions systems are designed adequately satisfying the water requirements for a combination of a 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 been 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:

$$P_n=P_O*(1+R)^n \tag{2.1}$$

where:

R=annual growth rate of the population

P_n=population at time n in the future

P_o = 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, ware houses, 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 fire fighting 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).

Fire fighting Demand: The quantity of water required for fire fighting can be generally calculated by using different empirical formulae (Alemayehu, 2010). Ethiopia National Board of Fire fighting calculates fire demand:

$$Q_F = 231.6\sqrt{\boldsymbol{P}}(1\text{-}0.01\sqrt{\boldsymbol{P}}) \qquad (2.2)$$

Where:

Q_F = 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):

$$\mathbf{Pcd = Q/(P * 365)} \qquad (2.3)$$

Where:

P_{cd}=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 and size of the city. The rate of water consumption increase in the pressure of the building and evenwith 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 calculated 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 can be calculated with the formula shown below:

$$P_f=P_{hd}/A_{hd} \tag{2.4}$$

Where:

P_f = peak factor

P_{hd}= Peak hour demand

A_{hd} = Average hour demand

$$P_{df}=P_{dd}/A_{dd} \tag{2.5}$$

Where;

P_{df}= Peak day factor

P_{dd}= Peak day demand

A_{dd}= Average daily demand

$$P_{hd}=A_{hd}*P_{hf} \tag{2.6}$$

Where:

P_{hd}= peak hour demand

A_{hd}= average hour demand

P_{hf}= peak hour factor

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

$$Q_{\text{day-avg}} = P_d \text{ x } p \tag{2.7}$$

Where:

P_d=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.

$$Q_{\text{day-max}} = Q_{\text{day-avg}} * M_{df} \tag{2.8}$$

Where:

$Q_{\text{day-max}}$=maximum day demand

$Q_{\text{day-avg}}$=average day demand

M_{df}=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.

$$Q_{\text{peak-hour}} = Q_{\text{day-avg}} \text{ x } P_{hf} \tag{2.9}$$

Fire Flow Rate*:* fire fighting calculated from the following formula

$$Q_F = 231.6\sqrt{\boldsymbol{P}}(1\text{-}0.01\sqrt{\boldsymbol{P}}) \tag{2.10}$$

Where:

Q_F=fire flow rate

P=population

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

$$Q_{cd}= 231.6\sqrt{\boldsymbol{P}}(1\text{-}0.01\sqrt{\boldsymbol{P}}) + Q_{\text{day-avg}} * M_{df} \tag{2.11}$$

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.

Table 2.1 Water Losses Percentage each Design Period (MoWR 2006).

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

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 base line 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 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.

$$\boldsymbol{coverage}\ \% = \left[\frac{\left(\boldsymbol{B}+(\boldsymbol{C}*\boldsymbol{D})\right)*\boldsymbol{E}}{\boldsymbol{A}} * \mathbf{100}\right] \tag{2.12}$$

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 designed 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 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 is based on population served, types of dwellings in the area, and fire fighting 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 pipe lines 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:

Table 2.2 Pressure Limit (MoWR, 2006).

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

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 be 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 be observed from the following equation:

$$\boldsymbol{V = \sqrt{\frac{4Q}{\pi D^2}}} \tag{2.13}$$

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 (<100 m) 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 therefore be governed by the maximum velocity criterion. Any internationally recognized formula may be used in the hydraulic computations, with coefficients taken as follows: For Hazen-Williams (C-value):

Table 2.3 Hazen William C-Factor (Amdework, 2012).

Type of pipe	Upvc	Steel	DCI/GI
New	130	110	120
Existing	100-110*	90-110*	100-110*

Table 2.4 Colebrook-White and Darcy-Weisbach f-Value (Amdework, 2012).

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*

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).

$$h_L = \frac{8fLQ^2}{gD^5\pi^2} \qquad \textbf{2.14}$$

Where:

f = Darcy-Weisbach friction factor

g = Gravitational acceleration constant (m/s^2)

L = Pipe length (mm)

D = Pipe diameter (mm)

Q = Pipe Flow (m^3/s)

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, fire fighting demands if to allow for, and for a limited emergency volume in case of power breakdown, repairs or O and M activities. In order to provide for security of supplies above the need for balancing purposes it is recommended 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 inthe 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 short falls 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 pipe fittings (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 continuity equation for steady flow in a circular pipe of diameter D is

$$\boldsymbol{Q} = \frac{\boldsymbol{\pi}}{\mathbf{4}}\boldsymbol{D}^{\mathbf{2}}\mathbf{x}\ \mathbf{V} \qquad (2.15)$$

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 and 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 micro computer (Ormsbee and 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 and Sagir, 2013).

Hydraulic modeling of water distribution systems can allow determining system pressure and flowing 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 acomputer 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 Hydraulic Modeling of Water Distribution Network

Bentley WaterCAD V8i is a powerful, easy-to-use program that helps hydraulic engineers design and analyzes water distribution systems. Bentley WaterCAD V8i provides intuitive access to the tools you need to model complex hydraulic situations. It can be used for many different kinds of applications in distribution system analysis. In this study, it was used to carry out the hydraulic analysis of the distribution networks in the study area. Bentley WaterCAD V8i can be analyze complex distribution systems under a variety of conditions for a typical Bentley WaterCAD V8i project, it may be interested in determining system pressure, velocity and flow rates under average loading conditions, head loss or under fire flow conditions.

- Perform steady state and extended period simulation
- Analyze multiple time variable demands at any junction node

A simulation of the network was carried out using the Bentley WaterCAD V8i. Water CAD V8i views the water distribution system as a network containing nodes and links, where the nodes are connected by links. Data used for Bentley WaterCAD V8i has x, y coordinate and elevation, junction or node and demand values. Bentley WaterCAD 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 Bentley WaterCAD 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.

$$\boldsymbol{Inflow\ at\ nodes = Outflow\ at\ nodes} \qquad (2.16)$$

$$A_1*V_1=A_2*V_2+A_3*V_3$$

Where:

$Q_a=A_1*V_1$ =inflow into node

Q_b = outflow $=A_2*V_2$

Q_C = external flow into the system or withdrawal$=A_3*V_3$

Conservation of Energy: The conservation of 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.

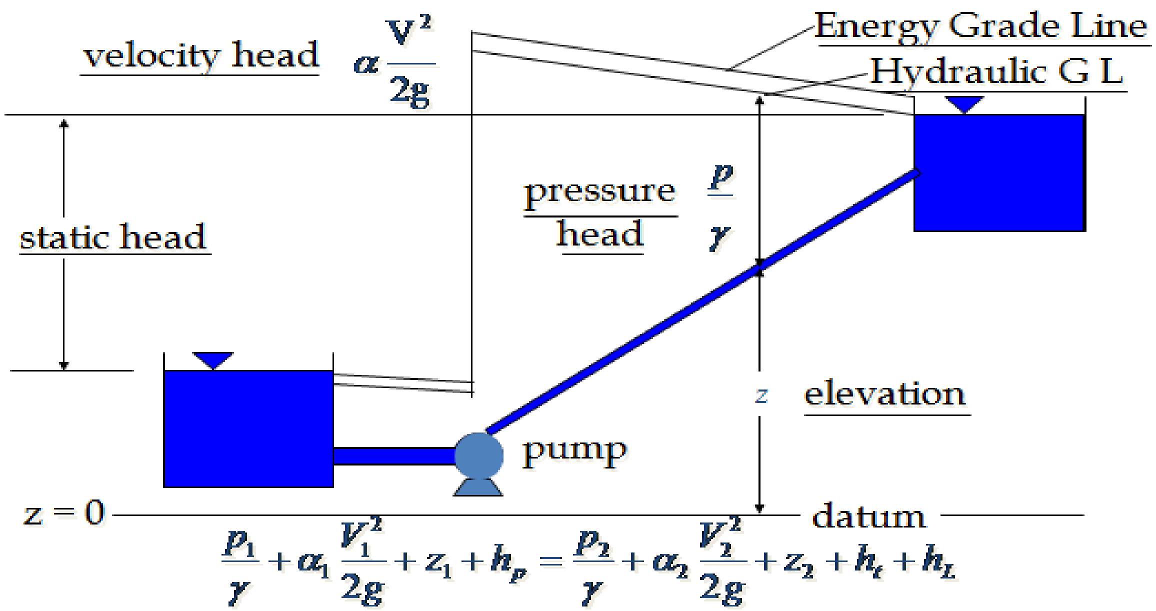

Figure 2.8 Conservation of energy (Rosmann, 2000).

$$\frac{P1}{\gamma}+\frac{v1^2}{2g}+z_1+h_p=\frac{P2}{\gamma}+\frac{v2^2}{2g}+z_2+h_l \qquad (2.17)$$

Where

$\frac{P1}{\gamma}$ and $\frac{P2}{\gamma}$ =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

$$\boldsymbol{hf}=10.69\left[\frac{Q}{C_{HW}}\right]^{1.852}*\frac{L}{D^{4.87}} \qquad (2.18)$$

Where:

h_f= head loss (m),

L = pipe length (m),

D = pipe diameter (m),

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

C_{HW} = Hazen-William Coefficient (Dawe, 2000b).

3 Materials and Methods

3.1 Description of Study area

Debre Markos town is located in north western Ethiopia, in Amhara National Regional State, East Gojjam zone, at a distance 300 km from Addis Ababa, and 265 km from Bahirdar the regional capital. Its location is 10º 21” North Latitude and 37º 43’ East Longitude and an elevation of 2,446 meters above sea level. The annual average temperature is 18.5^{0}C and its annual average rainfall is 1380 mm. It’s climate is classified as “*woiynadega*”. Based on the 2007 national census conducted by the **Central Statistical Agency** of Ethiopia (CSA), this town had a total population of 62,497, of whom 29,921 are men and 32,576 women. This is registered by Ethiopian central statistical Authority (CSA) in 2007 and other stakeholder. Unknown populations will not be considered in the analysis.

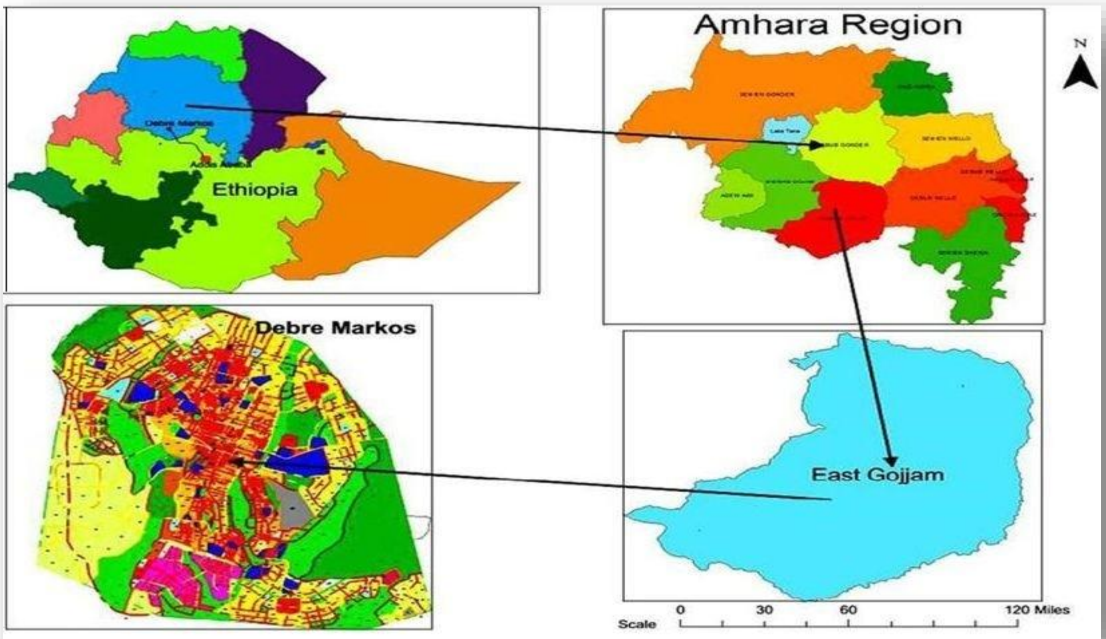

Figure 3.1 Map Of Debre markos Town

3.1.1 History of Establishment of Debremarkos town

Debre Markos town was founded in 1852 by Dej azmach Tedla Gualu who was the then administrator of the town. Its name was initially called Menkorer. The name of the town was changed to Debre Markos when due to the establishment of Saint Markos church, King Teklehaimanot who came to power in 1879 proclaimed that the town shall be named Debre Markos instead of Menkorer. Debre Markos is one of the reform towns in the region and has a town administration, municipality and 7 kebeles. The town has a structure plan which was prepared in 2009.

3.2 Materials

This research has evaluated hydraulic performance of Debremarkos water supply delivery system. To conduct the goal of the research, it is required to review of applicable practices, research findings, information on impact and cause of hydraulic performance loss in water delivery system. Additionally, it was aimed to analyze and interpret information to provide a recommendation to the research findings and to make the work load easy materials play vital role. The materials, which were used for this research, are listed below:

- Computer
- Stationary
- Garmin GPS
- Software (WATER CAD, Arc GIS 9.3, word and excel)

3.3 Methodology

In this research the survey was conducted in three sub cities out of 7 sub cities by using structural interviews, the statistical software SPSS Version 20 was used for data analysis. The analysis work of evaluation of hydraulic performance of urban water supply distribution system in Debre markos town, both secondary and primary data used. Based on the research objectives and questions how the research was carried out are discussed here.

3.3.1 Questionnaire

The questionnaire consists of 25 main questions with various sub questions dealing with different issues related to water service such as water supply coverage, water losses, water supply service and causes of water loss.

After successful completion of data collected from the study area data were analyzed to evaluate water supply versus water demand, to assess water loss and to identify deficiency of hydraulic parameters as shown below using flow chart Figure 3.2.

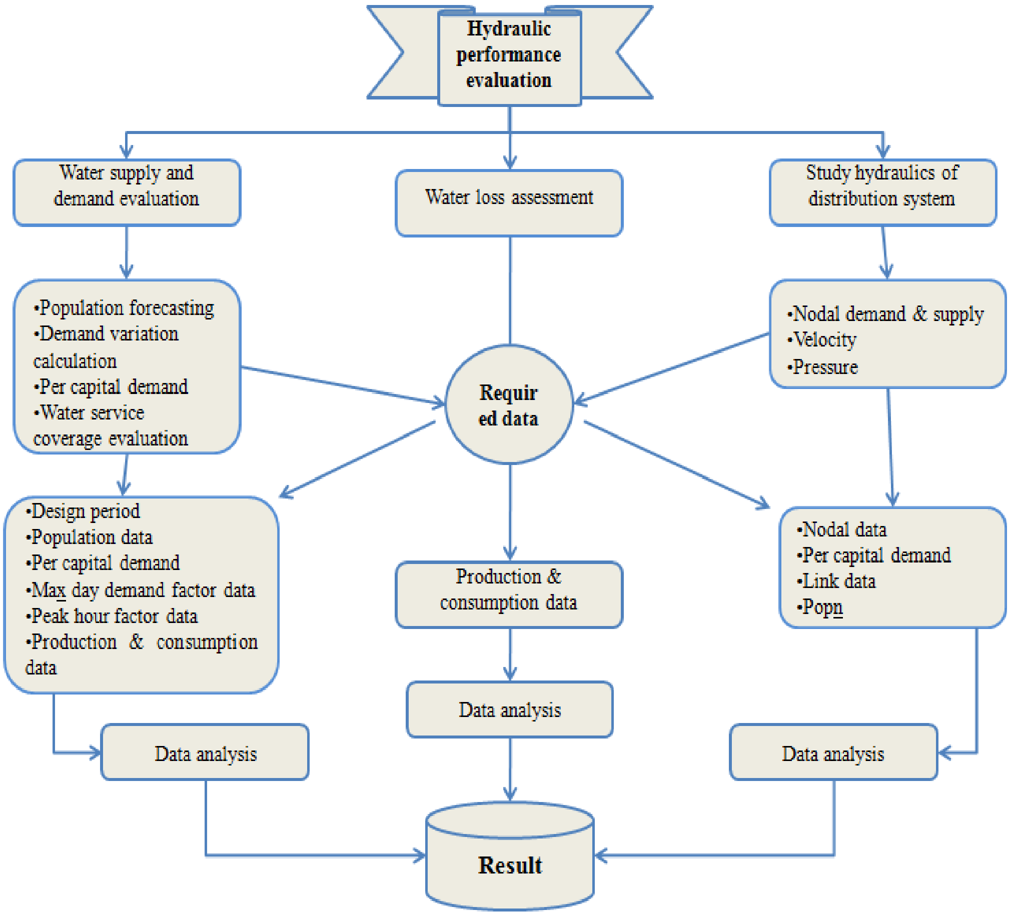

Figure 3.2 Flow chart of activities to evaluate hydraulic performance

3.3.2 Selection of Hydraulic Modeling software

Computer models are significant investments for water companies.to ensure a good investmeant 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 determination 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).

Bentley WaterCAD is the best software to hydraulic network modeling than other commercial (Rossman, 2000) due to its full featured and accurate hydraulic modeling, open-structured, accessible modeling due to simple operation and worldwide.

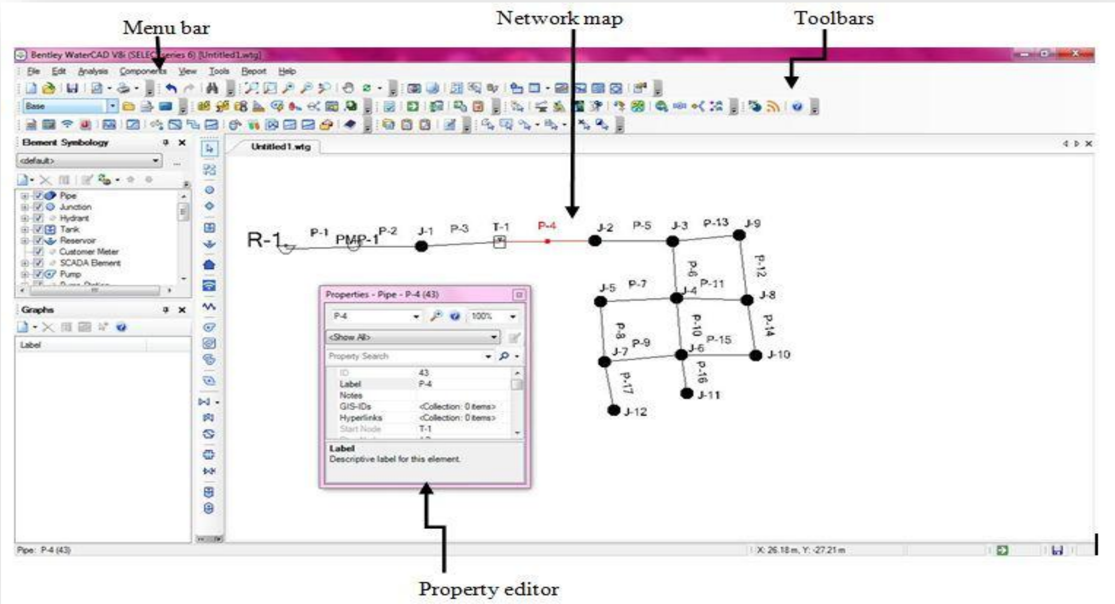

Figure 3.3 Bentley Watercad V8i workspace

When hydraulic network modeling of steady and extend period simulation with Bentley WaterCAD 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 and Lingireddy, 1997; Sarbu and Valea, 2011; Datwyler, 2014).

3.3.3 Input Data Collection

In order to carry out water supply versus demand analysis and simulation of hydraulic parameters in Debre markos 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 includes 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 obtained to that describe all tanks, reservoirs pumps and valves. Physical data power or data for use in describing the pump flow- head characteristics curve.

3.3.4 Data Source and Method of Collection

Both primary and secondary data were used. Primary data were collected from field survey and a random sample of 50 households that 20 of 50 from elevated area, 30 of 50 from lower elevated area who are water user from Debre markos water supply system while secondary data was collected from documents. Table 3.1 below shows data source.

Table 3.1 Data source and data type

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

3.4 Methods of Data Analysis

3.4.1 Analysis of Water Supply and Water Demand

In order to estimate total water demand that quantities of water produced to meet all water needs (residential, Institution and commercial, industrial, public use, fire fighting 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 Debre markos 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 are presented in Table3.2.

Table 3.2 Demand Factor (MoWR, 2006)

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

3.4.1.1 Population Data

Historical population data collected by projected Ethiopian urban CSA 2007. Population data of 2011-2015 were forecasted by geometric increase method. In 2007 population size of Debre markos town and annual growth rate was 62,497 and 4.3% respectively (CSA, 2008). The following formula was used for projection of the population of the town:

$$P_n = P_o * (1+R)^n \quad (3.1)$$

Where:

R=annual growth rate of the population

P_n=population at time n in the future

P_o = 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.

$$Q_{day\text{-}avg} = p_{cd} * p \quad (3.2)$$

Where:

$Q_{day\text{-}avg}$=average demand

P_{cd}=per capital demand

P=population

3.4.1.3 Maximum Day Demand

This shows the amount of water required during the day of maximum consumption in a year. The water supply, treatment plant and transmission lines should be designed 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.

$$Q_{\text{day-max}} = Q_{\text{day-avg}} * M_d \tag{3.3}$$

Where:

$Q_{day\text{-}max}$= maximum day demand

M_{df}= 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/day. It is an important parameter for design of distribution systems. Additionally, distribution system is design based on maximum daily demand plus fire fighting demand.

$$Q\text{peak-hour} = Q \text{ day-avg} * P_{HF} \tag{3.4}$$

Where:

$Q_{peak\text{-}hour}$= peak hour demand

P_{HF}=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).

$$\boldsymbol{coverage}\ \% = \left[\frac{(\boldsymbol{B}+(\boldsymbol{C}*\boldsymbol{D}))*\boldsymbol{E}}{\boldsymbol{A}} * \mathbf{100}\right] \tag{3.5}$$

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 Hydraulics Analysis

The model would be constructed in Bentley water cad software due to its hydraulic and water quality modeling capabilities. Bentley water cad 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 ground water 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 refilled by pumping from either a reservoir or a groundwater well. In Bentley WaterCAD, ground water well pump should model the same as a pumped reservoir. As pumping of the ground water occurs, drawdown of the water table elevation at the ground water well can occur. At higher pumping rates, the ground water well may not be able to recharge fast enough to maintain the pumping rate specified by the defined ground water 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 modeled 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 Department of Energy, 2005).

$$Q_{p1}+Q_{p2}=Q_{pm} \tag{3.6}$$

$$C_{p1}\cong C_{p2}=C_{cm} \tag{3.7}$$

Where:

Q_{p1}= discharge of p1,

Q_{p2}= discharge of p2,

Q_{pm}= discharge of pm,

p=pump, pm= model pump,

C_{p1}=curve of pump1,

C_{p2}= curve of pump2 and

C_{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).

$$C_{p1}+C_{p2}=C_{pm} \quad (3.8)$$

$$Q_{p1}\cong Q_{p2}=Q_{pm} \quad (3.9)$$

Where:

C_{p1}= Curve of pump 1,

C_{p2}=Curve of pump 2 and

C_{pm}=curve of modeling pump

Q_{p1}= discharge of pump 1

Q_{p2}= discharge of pump 2

Q_{pm}= 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, caused 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 recommended that hydraulically adjacent tanks be modeled 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).

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

Where:

V_{T1}=Volume of tank1

V_{T2}=Volume of tank2

V_{TM}=Volume of modeling tank

3.4.2.4 Skeletonization of Water Network

The next step in using Bentley watercad was to skeletonize the network and assign node numbers to the nodal points. Figure 3.4 shows below the schematic distribution of the network on the Debre markos town map. The skeletonization of water distribution layout based on master plan of the town, and then placed as a map on Bentley waterCAD platform as shown on.

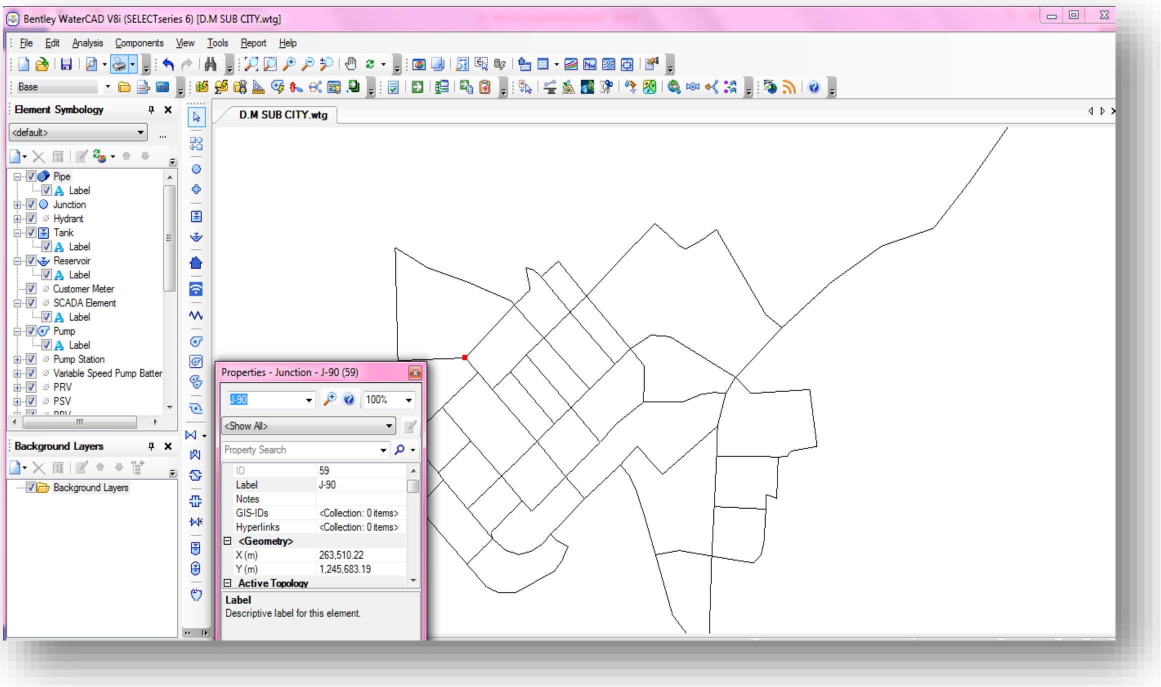

Figure 3.4 Schematic distribution of water networks of Debre markos town

3.4.2.5 Assign Network Parameters

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

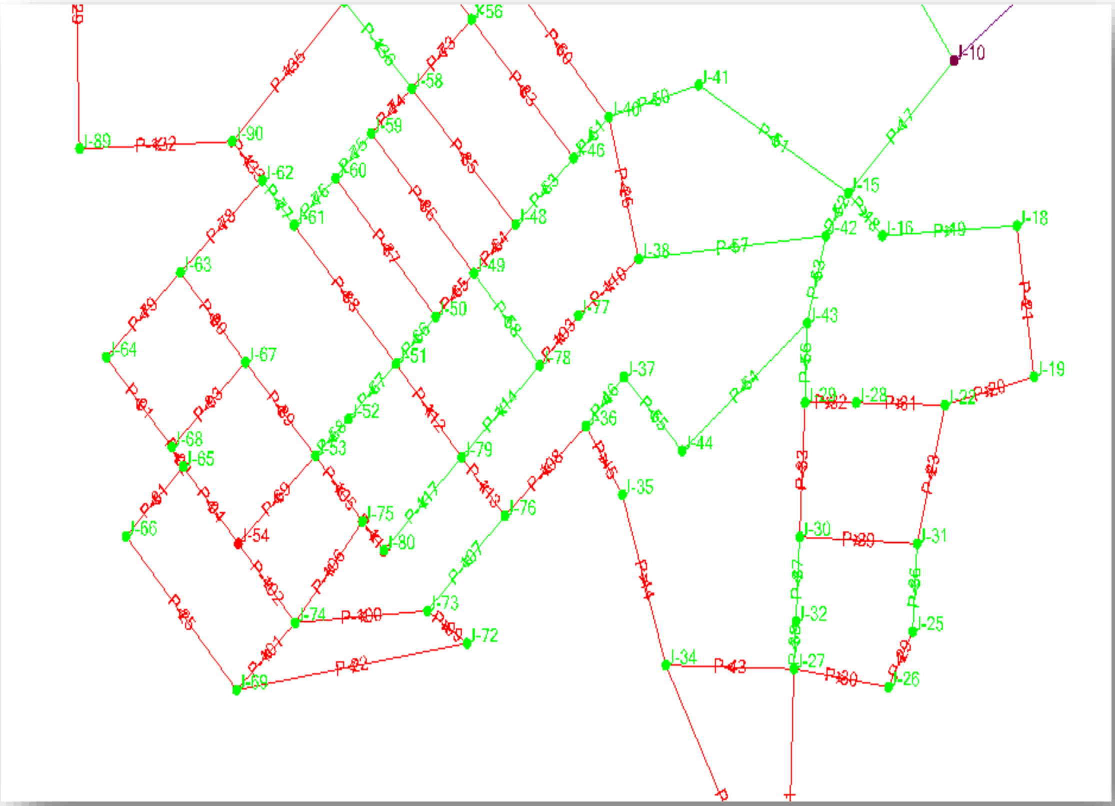

Figure 3.5 Assign Network Parameters

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 Debre markos and a daily per capita water consumption expected to be 20 l/c/d, 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.

$$N_d = \Sigma p_i . D_j \qquad (3.11)$$

Where:

N_d=Nodal demand

P_i=population in each service area

D_j = 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 entered 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 Ethiopian 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 and Energy put standard for urban water supply design maximum and minimum allowance pressure head. It is tabulated as follows.

Table 3.3 Maximum and Minimum Pressure Limit (MoWR, 2006).

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

3.5 Water loss analysis

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

$$\boldsymbol{Total\ water\ loss}(\%) = \frac{\boldsymbol{Total\ water\ production - Total\ water\ consumption}}{\boldsymbol{Total\ water\ production}} * \mathbf{100} \qquad (3.12)$$

4 RESULTS AND DISCUSSIONS

The Debre markos water supply design period is 20 years (ADSWE, 2010). It was constructed in 2008 and served the last 9 years. The distribution network of Debre markos town is looped type distribution pipes system, which covers all part of the town and it get power source from the national grid line. Debre markos water supply method is combination of pumping and gravitational system. Now a day, Debre markos water supply system feeds about 83,916 populations. In the distribution system, there are many public fountains in number, some of them are not functional. Those public fountains lactated at lower elevation 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 precounted 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 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 Debre markos town. In order to forecast 2011-2015 of the Debre markos town based on Ethiopian urban rank. The growth rate and population in 2008 was 4.3 %, and 62,497 used for the current projection (CSA, 2008). Debre markos town water service office current population recorded data show total population in 2015 was 83,417. However, this research was done based on 2008 Ethiopian urban rank.

Table 4.1Projection of Debre markos Population 2011-2015

Year	Population
2011	70,910
2012	73,959
2013	77,140
2014	80,457
2015	83,916

4.2 Water Supply and Demand in Debre markos 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 and UNICEF, 2012). Water supply schemes designed to supply 24-hour a day and 7 days in a week but they operated intermittently. 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 then 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 Debre markos town water supply system, which was designed for period 20 years‟ to give service for continuous 24 hour a day and 7 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 to many factors, water demand in Debre markos town was high and shown severe shortage of water in the town. During the study time, Debre markos used intermittent water supply system. Because, in Debre markos 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 and 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 Debre markos town is associated with erratic power supply that humped continued operation ofthe water supply system. In Debre markos 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 Debre markos 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 Debre markos population number ranged >50,000 then calculated by taking maximum day demand factor and peak hour factor was 1.20 and 1.7 respectively (Alemayehu, 2010), and per capital demand, 20 l/c/day (MoWE, 2015).

Table 4.2 Water Demand in Debre markos Town 2011-2015

Year	Population	Average day demand(m^3/day)	Maximum day demand (m^3/day)	Peak hour demand (m^3/day	Per capital demand (l/c/day)
2011	70,910	3545.5	4254.6	7232.82	20
2012	73,959	3697.95	4437.54	7543.818	20
2013	77,140	3857	4628.4	7868.28	20
2014	80,457	4022.85	4827.42	8206.614	20
2015	83,916	4195.8	5034.96	8559.432	20

Table 4.3 Water Supply in Debre markos 2011-2015

Year	Population	Production (m^3/year)	Average supply (m^3/day)	Water Supply (l/p/d)
2011	70,910	769,540	2108.32	19.73
2012	73,959	854,538	2341.2	18.65
2013	77,140	987,746	2706.15	18.08
2014	80,457	1,164,171	3189.5	19.64
2015	83,916	1,240,376	3398.3	18.49

As shown the above Table 4.2 and Table 4.3 the distribution system could not capable to deliver expected enough water for satisfied different demand of Debre markos town. The design period of Debre markos water supply system will end the next eight years at that time Debre markos total population will be 127,846. The new standard for urban water supply access with GTP-2 category minimum service level a town whose population is >100,000 will be >50 l/c/d (MoWIE, 2015). Therefore Debre markos water distribution system at the end of next five years forced to deliver 7,670.8 m^3/day and 10,866.9 m^3/day to satisfy maximum day demand and peak hour demand respectively. It is impossible to achieve this demand because it fails to satisfy current demand, even within 20 l/c/d per capital demand. During the research time, Debre markos town was served by 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 Figure 4.1.

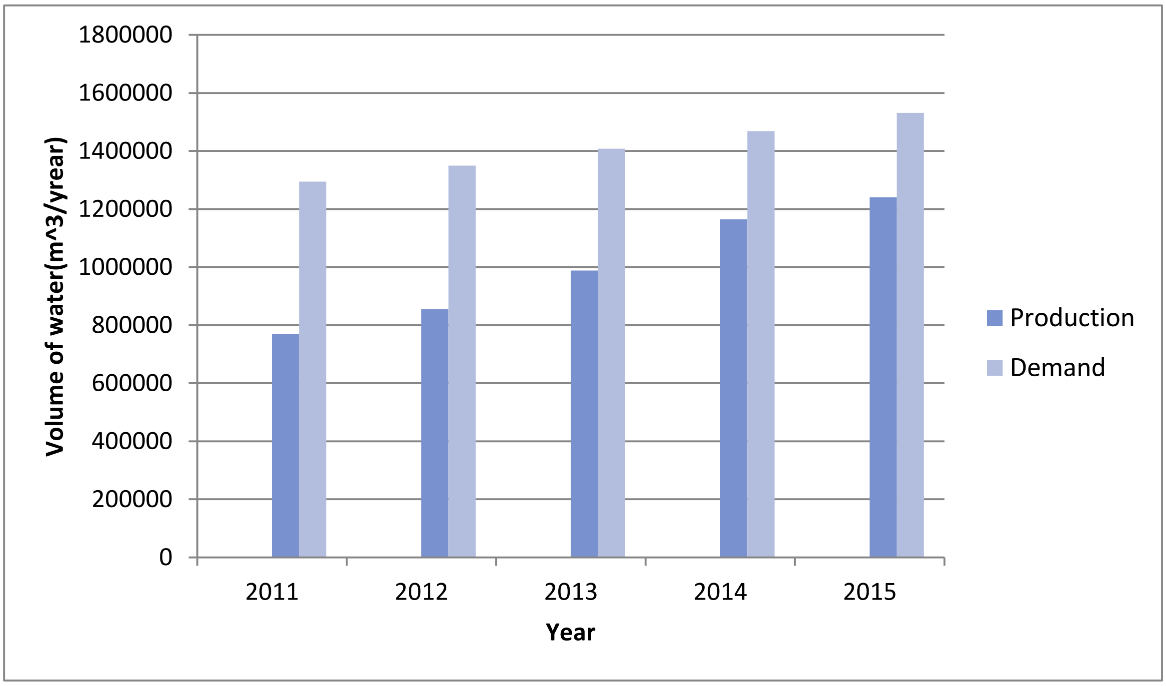

Figure 4.1 Water Supply versus Demand

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 was 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 and Tuna, 2014b; Nigam *et al*., 2015). In this research, real water loss was 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 Debre markos town was a 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 Figure 4.2 below.

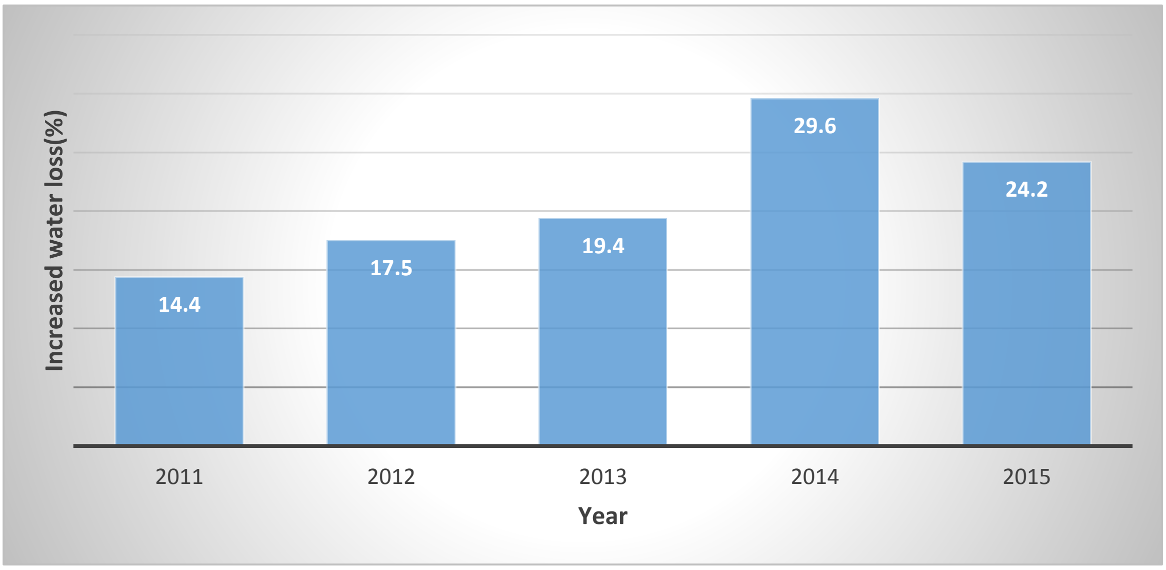

Figure 4.2 Water loses in Debre markos town water distribution system

The other problems, in Debre markos water distribution network was insufficient maintenance, poor rehabilitation measures and intermittent water supply that increase water loss from 2011-2015. The long served pipe and intermittent water supply in Debre markos 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 Debre markosTown

Service coverage means a piped connection to each household, ideally with water available 24 hours a day and 7 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. Contaminant may enter when the pipes are empty (WHO and UNICEF, 2012; WHO, 2014). In Debre markos, 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. In Debre markos, water supply service coverage was 77 % of from population 83,916.then 23% people had no access connection of water supply. Figure 4.3shows that Debre markos town water supply service coverage.

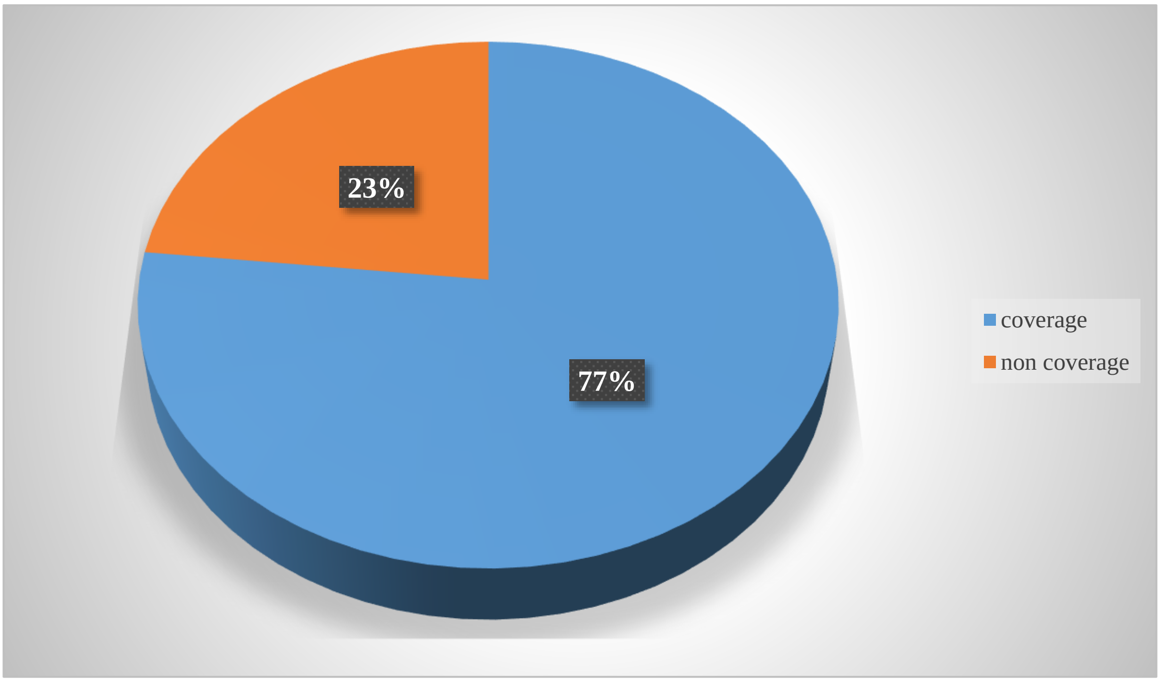

Figure 4.3 Water Supply Service Coverage

4.5 Hydraulic Analysis of Water Supply Network

When the water distribution system being modeled 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 created and used to evaluate system performance over time. Bentley WaterCAD 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 was used to calculate the friction losses in the pipes. After application of the software simulation steady state and 24 hour extended period, results obtained viewed on the network map. The analysis and modeling of distribution network in Debre markos water supplywas consisting of 98 pipes of different material, 69 nodes, 1 tank, 1service reservoir and 2 source reservoir from which water was pumped to the elevated tank as shown in Figure 4.4.

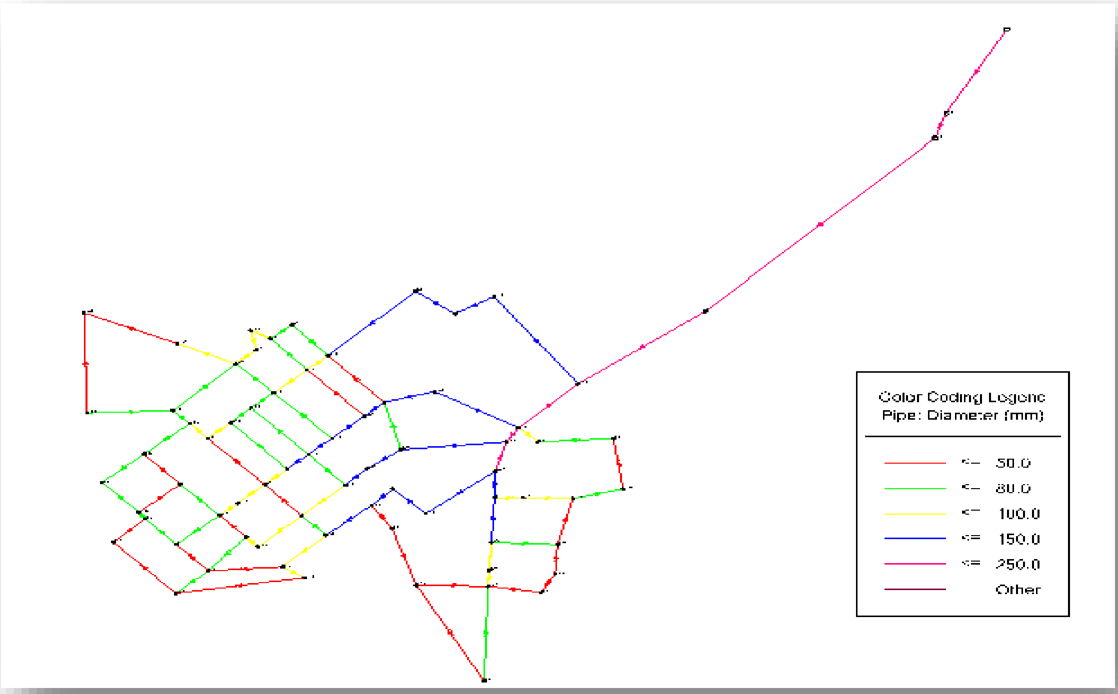

Figure 4.4 Hydraulic modeling of water distribution network

4.5.1 pump

The most common input of energy into a system was through pumping. Pumps were crucial to any distribution system that cannot supply acceptable pressure to consumer through the sole use of gravity flow.

4.5.2 Pump capacity curve

A pump curve represents the relationship between the head and flow rate that a pump can deliver at it nominal speed setting. Pump head is the head gain imparted to the water by the pump and plotted on the vertical of the curve in meter. Flow rate is plotted on the horizontal in litter per second. A valid pump curve must have decreasing head with increasing flow. An efficiency curve determines pump efficiency in vertical percent as a function of pump flow rate in horizontal flow.

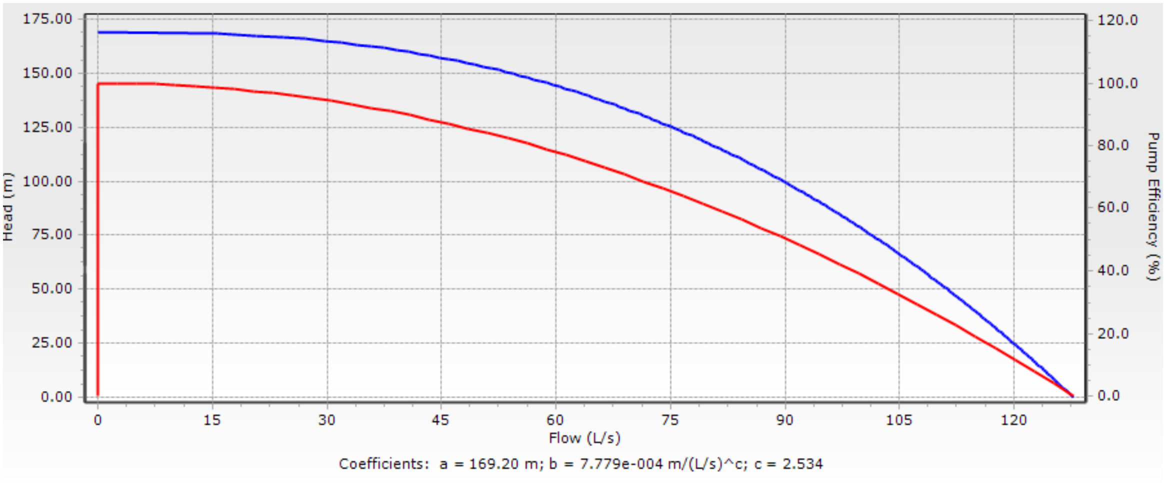

Figure 4.5 Pump capacity curve

4.5.3 Nodal Demand and Nodal Supply

Water demand in a distribution system fluctuates over time during simulation as shown from Figure 4.5 where as water product was constant. Not only water demand and also flow in Figure 4.6, pressure, velocity and hydraulic head varyfrom 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 become low and people at the morning wake up to perform their activity water is needed then water demand become high. When demand increase flow rate and velocity increase where as pressure head decreased at each demand pattern.

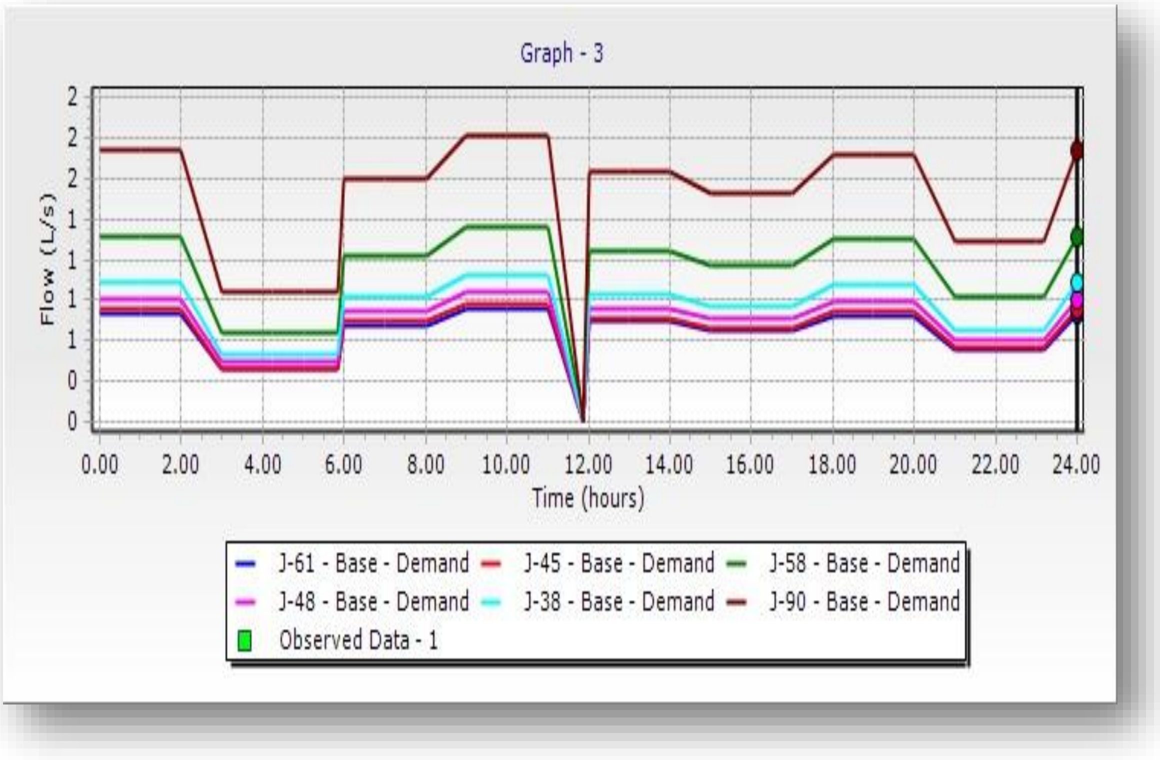

Figure 4.6 Flow variation with time

The 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 interrns 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) as analyzed by the software that current supplies at the nodal points failed.

Debre markos water distribution system feed 83,916 populations and standard per capital demand was 20 liters/day. Then the distribution system forced to be delivered 18 l/sec peak hour demand to this population. Because water distribution is designed to meet peak hour demand. Therefore, to meet current and future demand the sources of water must be improved and additional new source should be introduced to the system.

4.5.4 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.

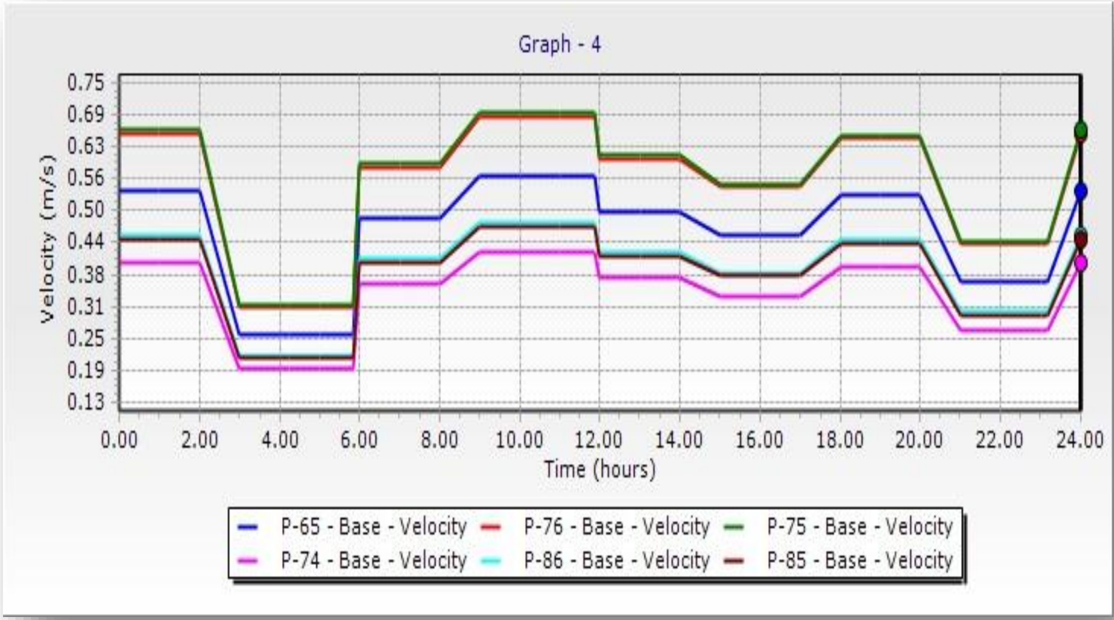

Figure 4.7 Velocity Variation for Selected Links

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).

Table 4.4 Velocity range description (MoWR, 2006).

Velocity range (m/s)	**Effect**
0.0 – 0.6	Sedimentation occur
0.6 – 2	Acceptable
>2	Head loss occur

Most links those 30 out of 36 velocities in Debre markos town water supply distribution system below 0.6 m/s during peak hour demand as shown in below Figure 4.8. This also exposed the water network for high sedimentation and poor water quality problem.

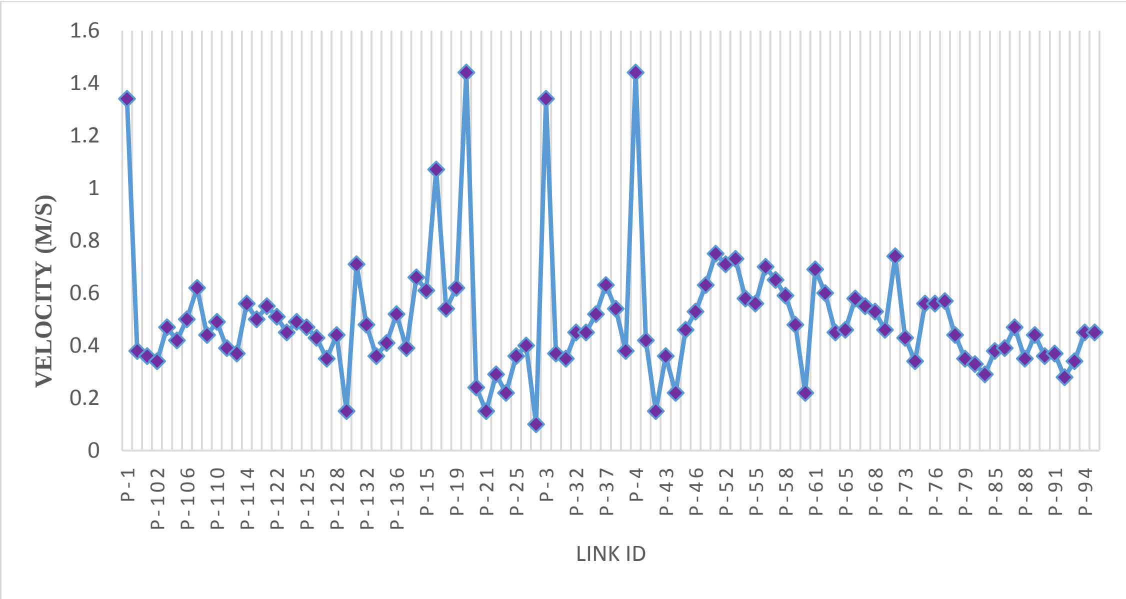

Figure 4.8 Nodal velocity at Peak hour demand

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 build up can result in loss of carrying capacity, reduced pressures and poorer water quality (Walski *et al*., 2003). Then corrosion pipe also can not resist high pressure then become pipe burst and water leakage

4.5.5 Pressure Head in Water Distribution System

Pressure modeled at each node on the distribution mains as shown below Figure 4.9 on Bentley WaterCAD V8i workspace.

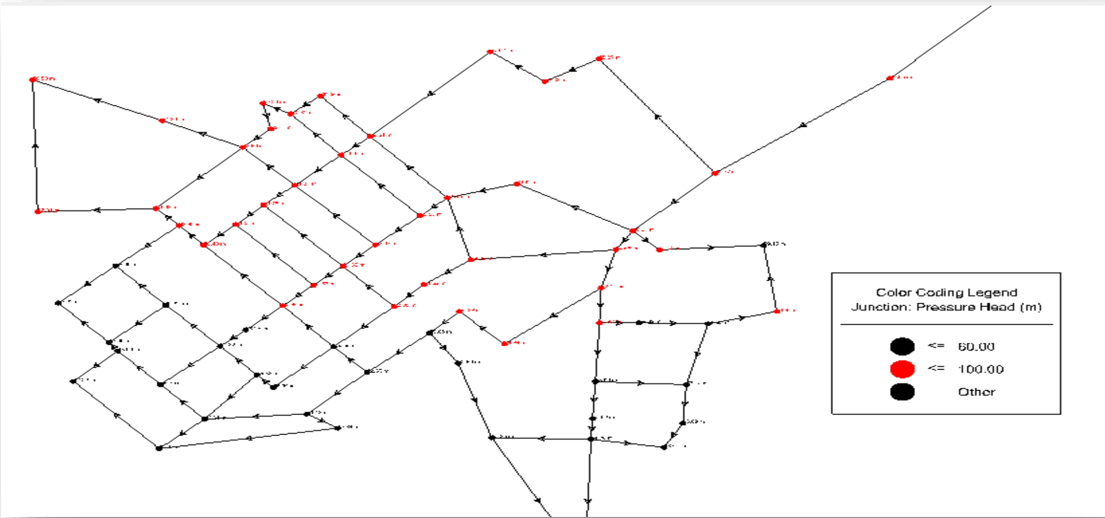

Figure 4.9 Simulation of Pressure Head

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.

A query was done using Bentley WaterCAD V8i software for all nodes with Bentley WaterCAD workspace 30 out of the 69 nodes have pressure heads above the required minimum pressure head of 15.00m and below maximum pressure require head 60m. However, 39 of nodes had maximum pressure head above maximum limited of pressure head. Off course at node J-13, pressure head 77.62 m 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 (77.62m) break or flow block on the transmission pipe will completely cease the water supply system.

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 Figure 4.9 above shown that the output had good pressure head between 15 and 70 m were J-18,J-22,J-25,J-26,J-27,J-28,J-30,J-31,J-32,J-33,J-34,J-35,J-52,J-53,J-54,J-63,J-64,J-65,J-66,J-67,J-68,J-72,J-73,J-74,J-75,J-76,J-79 and J-80 Simulation of hydraulic network shows that the lower elevation area was exposed higher pressure head as shown below from the contour map Figure 4.10, 4.11 and Table 4.4.

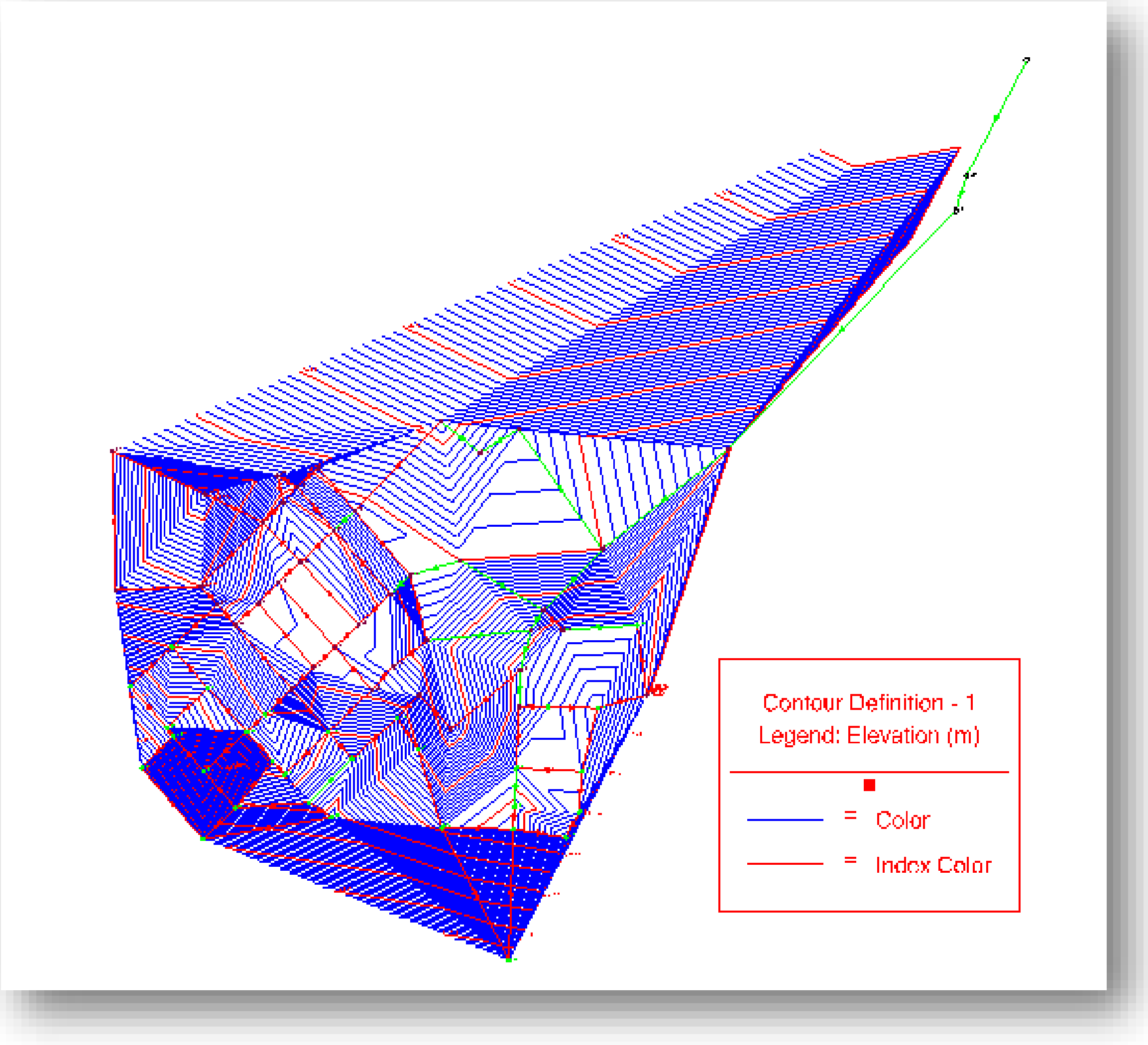

Figure 4.10 Conotur Map of Elevation

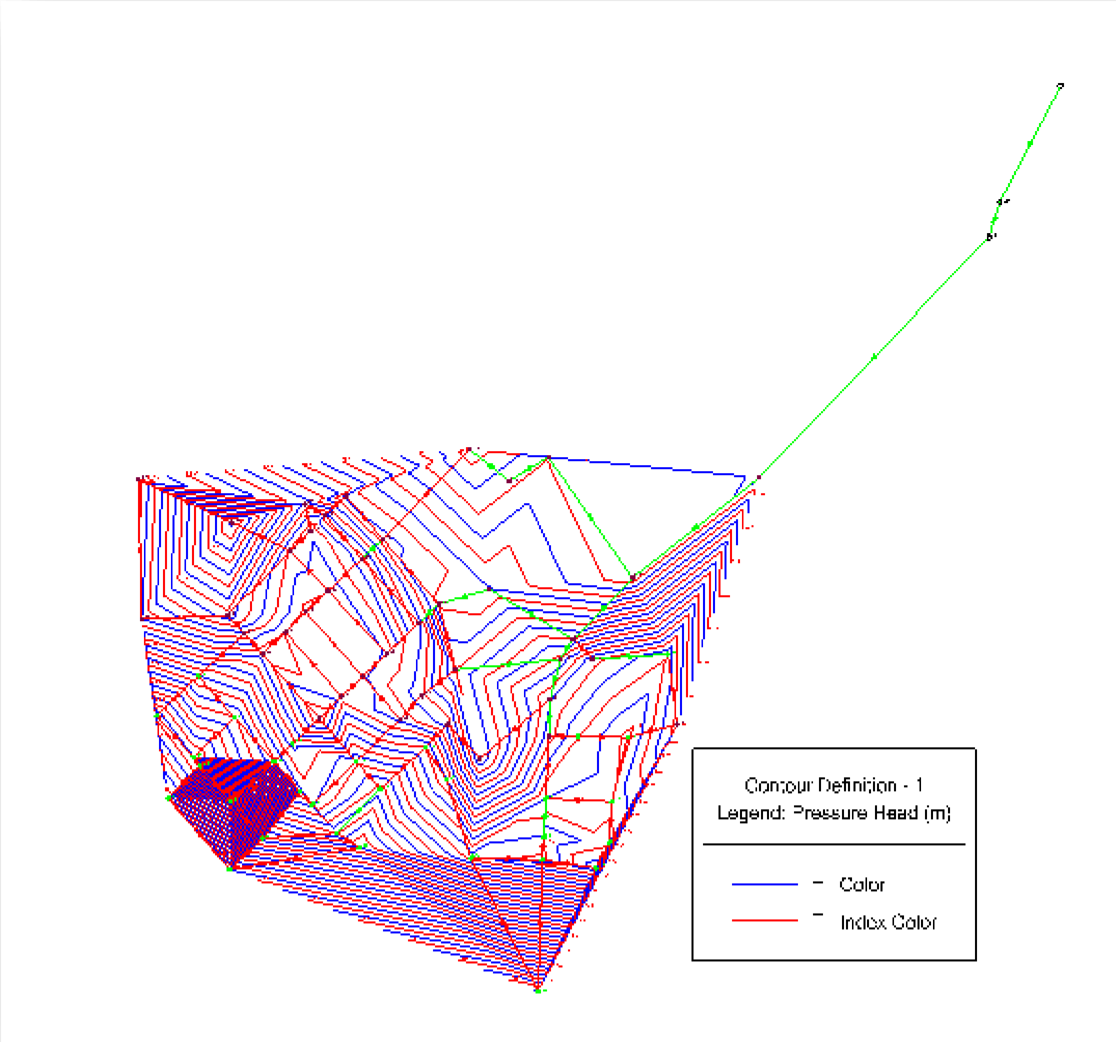

Figure 4.11 Contour Map of Pressure Head

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 Debre markos 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 Debre markos 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 discussed above 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 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The main goal of this research was to evaluating hydraulic performance of urban water supply in Debre markos 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 found:

❖ In Debre markos 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 Debre markos town was erratic of power supply that hampered continuous operation of the water supply system. Additionally, water supply service coverage in Debre markos town was 77%. Due to limited source capacity, rapid population growth, expansion of new area and illegal settlement around the town 23% of Debre markos town population was lived without water supply service connection.

❖ Water loss was high in Debre markos town and increasing in 2011-2014 and decreased in 2015. The increment was due to expansion road expansion, insufficient maintenance, poor rehabilitation measures and intermittent water supply.

❖ In this research hydraulic network, modeling was constructing with Bentley WaterCAD software due to its hydraulic and water quality modeling capabilities. Moreover, and has ability to perform steady and extend period simulation. During perform steady state simulation nodal supply was failed to satisfy 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 78 out of 97 links were below minimum allowance velocity (0.6m/s). Low velocity exposed water age, sedimentation and hygiene problem. In other direction, Debre markos 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 39 out of 69 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 Debre markos town was a big challenge to determine the gap between demand and supply. Then population of Debre markos 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 failed 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 increased with time for the past 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 done.

❖ 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.

In general, city water supply authority have to look seriously into water demand management. To satisfy continuous rising of water demand several measures will be take.
Such as:

- Increase source
- Awareness creation to the community
- Updating existing network data, measuring and evaluating water loss
- Operation and Maintenance strategy in order to decrease leakage

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APPENDIX

APP.Table 1 Water balance of Debre markos distribution system

Year	water production (m^3/year)	Water consumption (m^3/year)	water loss (m^3/year)	Water loss (%)
2011	769,540	658,659	110,881	14.4
2012	854,538	705,246	149,292	17.5
2013	987,746	796,428	191,318	19.4
2014	1,164,171	818,775	345,396	29.6
2015	1,240,376	940,375	300,001	24.2

APP.Table 2 Network links

ID	Length (Scaled) (m)	Start Node	Stop Node	Diameter (mm)	Flow (L/s)	Velocity (m/s)	Headloss (m)
P-1	739	J-1	J-10	250	66	1.34	5.3
P-100	345	J-73	J-74	50	1	0.38	1.57
P-101	206	J-74	J-69	80	2	0.36	0.5
P-102	221	J-54	J-74	50	-1	0.34	0.82
P-103	144	J-77	J-78	150	8	0.47	0.27
P-105	183	J-75	J-53	50	1	0.42	0.99
P-106	273	J-75	J-74	80	3	0.5	1.21
P-107	282	J-73	J-76	100	-5	0.62	1.42
P-108	281	J-76	J-36	150	-8	0.44	0.46
P-110	196	J-38	J-77	150	9	0.49	0.4
P-112	258	J-51	J-79	50	-1	0.39	1.26
P-113	165	J-79	J-76	80	-2	0.37	0.4
P-114	279	J-78	J-79	100	4	0.56	1.16
P-116	82	J-80	J-75	100	4	0.5	0.27
P-117	279	J-79	J-80	100	4	0.55	1.1

P-122	254	J-45	J-81	80	3	0.51	1.14
P-123	132	J-81	J-82	80	2	0.45	0.47
P-124	105	J-82	J-83	100	4	0.49	0.34
P-125	124	J-83	J-84	100	4	0.47	0.38
P-126	128	J-84	J-85	100	3	0.43	0.32
P-127	298	J-85	J-86	100	3	0.35	0.52
P-128	475	J-86	J-87	50	1	0.44	2.84
P-129	623	J-87	J-89	50	0	0.15	0.51
P-13	665	J-10	J-11	150	13	0.71	2.69
P-132	397	J-89	J-90	80	-2	0.48	1.62
P-133	112	J-90	J-62	80	-2	0.36	0.27
P-135	411	J-90	J-85	80	-2	0.41	1.23
P-136	251	J-85	J-58	80	-3	0.52	1.19
P-137	259	J-82	J-56	80	-2	0.39	0.7
P-14	210	J-11	J-12	150	12	0.66	0.74
P-15	231	J-12	J-13	150	11	0.61	0.71
P-17	388	J-15	J-10	250	-53	1.07	1.85
P-18	125	J-15	J-16	100	4	0.54	0.5
P-19	351	J-16	J-18	80	3	0.62	2.26
P-2	592	PMP-1	R-1	250	-66	1.34	4.29
P-20	238	J-19	J-22	80	-1	0.24	0.28
P-21	316	J-18	J-19	50	0	0.15	0.25
P-22	607	J-72	J-69	50	1	0.29	1.72
P-23	296	J-22	J-31	50	0	0.22	0.51
P-25	429	J-69	J-66	50	1	0.36	1.79
P-26	303	J-40	J-38	80	-2	0.4	0.88
P-29	131	J-25	J-26	50	0	0.1	0.05
P-3	1,514	J-1	T-1	250	-66	1.34	10.96
P-30	249	J-26	J-27	50	-1	0.37	1.06
P-31	231	J-22	J-28	100	-3	0.35	0.41
P-32	133	J-28	J-29	100	-4	0.45	0.38

P-33	278	J-29	J-30	150	8	0.45	0.47
P-36	182	J-31	J-25	50	1	0.52	1.48
P-37	175	J-30	J-32	100	5	0.63	0.91
P-38	98	J-32	J-27	100	4	0.54	0.38
P-39	306	J-31	J-30	80	-2	0.38	0.82
P-4	159	T-1	PMP-1	250	-66	1.34	1.15
P-41	588	J-27	J-33	80	2	0.42	1.88
P-42	673	J-33	J-34	50	0	0.15	0.53
P-43	333	J-34	J-27	50	-1	0.36	1.35
P-44	370	J-34	J-35	50	0	0.22	0.61
P-45	171	J-35	J-36	50	-1	0.46	1.1
P-46	142	J-36	J-37	150	-9	0.53	0.33
P-50	243	J-40	J-41	150	-11	0.63	0.79
P-51	448	J-41	J-15	150	-13	0.75	1.98
P-52	106	J-15	J-42	250	35	0.71	0.23
P-53	187	J-42	J-43	200	23	0.73	0.57
P-54	419	J-43	J-44	150	10	0.58	1.17
P-55	215	J-44	J-37	150	10	0.56	0.55
P-56	165	J-43	J-29	150	12	0.7	0.64
P-57	489	J-42	J-38	150	11	0.65	1.66
P-58	257	J-49	J-78	80	-3	0.59	1.54
P-59	569	J-13	J-45	150	9	0.48	1.12
P-60	389	J-45	J-40	50	0	0.22	0.64
P-61	125	J-40	J-46	150	12	0.69	0.48
P-63	204	J-46	J-48	150	11	0.6	0.59
P-64	148	J-48	J-49	150	8	0.45	0.26
P-65	134	J-49	J-50	150	8	0.46	0.24
P-66	142	J-50	J-51	100	5	0.58	0.64
P-67	169	J-51	J-52	100	4	0.55	0.69
P-68	115	J-52	J-53	100	4	0.53	0.43
P-69	270	J-53	J-54	80	2	0.46	1.03

P-71	131	J-45	J-56	100	6	0.74	0.91
P-73	211	J-56	J-58	100	3	0.43	0.55
P-74	140	J-58	J-59	80	2	0.34	0.3
P-75	131	J-59	J-60	80	3	0.56	0.72
P-76	145	J-60	J-61	100	4	0.56	0.6
P-77	124	J-61	J-62	100	4	0.57	0.54
P-78	286	J-62	J-63	80	2	0.44	0.99
P-79	261	J-63	J-64	80	2	0.35	0.59
P-81	207	J-65	J-66	50	1	0.33	0.71
P-83	391	J-46	J-56	50	1	0.29	1.07
P-85	390	J-58	J-48	80	-2	0.38	1.02
P-86	394	J-59	J-49	80	-2	0.39	1.07
P-87	387	J-60	J-50	80	-2	0.47	1.54
P-88	390	J-61	J-51	50	-1	0.35	1.51
P-89	265	J-53	J-67	80	2	0.44	0.91
P-90	251	J-67	J-63	50	1	0.36	1.01
P-91	252	J-64	J-68	80	-2	0.37	0.64
P-92	50	J-68	J-65	80	-1	0.28	0.08
P-93	259	J-67	J-68	50	1	0.34	0.96
P-94	214	J-65	J-54	80	-2	0.45	0.76
P-99	123	J-72	J-73	100	-4	0.45	0.34

APP.Table 3 Network Nodes

Node ID	**X (m)**	**Y (m)**	**Elevation (m)**	**Demand (Base) (L/s)**	**Pressure (m)**	**Head**
J-1	265,974.36	1,246,298.32	2,435.00	0.35	76.06	
J-10	265,386.70	1,245,850.30	2,430.00	0.35	75.77	
J-11	264,997.43	1,246,389.88	2,428.00	0.88	75.07	

J-12	264,816.09	1,246,283.54	2,427.00	2.31	75.33
J-13	264,633.28	1,246,424.03	2,424.00	0.51	77.62
J-15	265,111.24	1,245,576.69	2,437.00	1.18	66.91
J-16	265,199.92	1,245,488.01	2,442.00	3.39	61.42
J-18	265,549.98	1,245,509.01	2,442.00	0.94	59.15
J-19	265,594.32	1,245,196.29	2,440.00	1.10	61.41
J-22	265,363.28	1,245,137.95	2,445.00	0.82	56.68
J-25	265,279.27	1,244,668.87	2,443.00	0.92	56.69
J-26	265,216.26	1,244,554.52	2,441.00	0.71	58.64
J-27	264,970.42	1,244,591.86	2,446.00	0.80	54.7
J-28	265,132.25	1,245,142.62	2,443.00	0.90	59.09
J-29	264,999.22	1,245,142.62	2,441.00	0.98	61.47
J-30	264,985.22	1,244,864.90	2,444.00	1.35	58
J-31	265,290.94	1,244,850.90	2,444.00	0.71	57.18
J-32	264,975.89	1,244,689.88	2,446.00	2.41	55.09
J-33	264,950.77	1,244,004.28	2,473.00	0.84	25.82
J-34	264,637.58	1,244,599.64	2,444.00	0.47	55.35
J-35	264,524.44	1,244,952.19	2,440.00	0.73	59.96
J-36	264,429.73	1,245,094.27	2,442.00	0.47	59.06
J-37	264,528.63	1,245,196.18	2,437.00	0.73	64.39
J-38	264,566.93	1,245,440.55	2,433.00	0.49	69.02
J-40	264,489.05	1,245,733.62	2,430.00	2.02	71.15
J-41	264,722.79	1,245,800.06	2,430.00	0.41	71.94
J-42	265,053.85	1,245,487.97	2,439.00	0.27	64.68
J-43	265,004.61	1,245,307.42	2,441.00	0.47	62.11
J-44	264,679.99	1,245,042.99	2,433.00	0.59	68.94
J-45	264,229.25	1,246,023.46	2,431.00	1.16	69.51
J-46	264,397.58	1,245,648.79	2,437.00	0.65	63.67
J-48	264,246.90	1,245,511.46	2,438.00	0.86	62.07
J-49	264,138.18	1,245,410.37	2,438.00	1.18	61.82
J-50	264,039.00	1,245,320.73	2,438.00	0.33	61.57

J-51	263,936.00	1,245,223.45	2,435.00	0.20	63.94
J-52	263,812.03	1,245,109.01	2,440.00	0.43	58.25
J-53	263,726.20	1,245,032.72	2,443.00	0.75	54.82
J-54	263,525.92	1,244,851.52	2,480.00	1.02	16.79
J-56	264,132.14	1,245,935.69	2,435.00	0.98	64.6
J-58	263,976.77	1,245,792.86	2,438.00	0.82	61.05
J-59	263,871.52	1,245,700.14	2,438.00	0.84	60.75
J-60	263,778.81	1,245,607.42	2,437.00	0.57	61.03
J-61	263,671.05	1,245,509.69	2,437.00	0.47	60.43
J-62	263,588.36	1,245,602.41	2,434.00	1.16	62.89
J-63	263,375.36	1,245,411.96	2,440.00	3.63	55.9
J-64	263,182.41	1,245,236.55	2,448.00	0.20	47.32
J-65	263,382.49	1,245,010.39	2,448.00	1.35	48.03
J-66	263,233.78	1,244,866.31	2,453.00	0.84	42.32
J-67	263,544.47	1,245,225.92	2,444.00	0.20	52.91
J-68	263,352.98	1,245,050.80	2,448.00	1.69	47.96
J-69	263,521.56	1,244,548.34	2,459.00	2.96	38.11
J-72	264,120.59	1,244,644.90	2,451.00	0.59	47.84
J-73	264,017.47	1,244,712.01	2,448.00	0.78	51.18
J-74	263,673.77	1,244,687.46	2,451.00	0.57	46.61
J-75	263,848.90	1,244,896.95	2,443.00	1.02	55.82
J-76	264,219.09	1,244,908.92	2,448.00	0.43	52.6
J-77	264,409.86	1,245,323.40	2,437.00	0.88	64.62
J-78	264,309.74	1,245,219.55	2,439.00	1.18	62.36
J-79	264,106.11	1,245,029.37	2,445.00	0.37	55.19
J-80	263,904.40	1,244,837.26	2,443.00	0.31	56.09
J-81	264,062.41	1,246,215.38	2,429.00	0.35	70.36
J-82	263,961.68	1,246,130.15	2,435.00	0.10	63.9
J-83	263,869.99	1,246,180.51	2,433.00	0.37	65.56
J-84	263,896.05	1,246,059.12	2,436.00	1.18	62.18
J-85	263,801.55	1,245,972.60	2,435.00	1.90	62.86

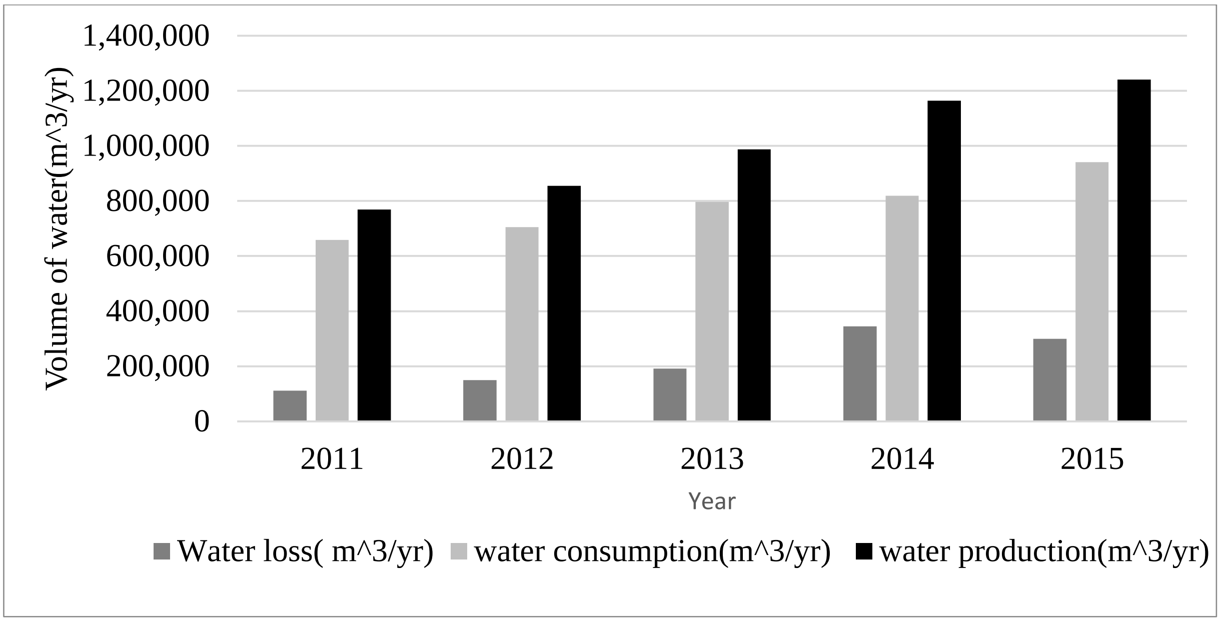
J-86	263,531.57	1,246,098.61	2,423.00	1.16	74.33
J-87	263,097.31	1,246,290.53	2,434.00	2.12	60.5
J-89	263,113.08	1,245,667.84	2,433.00	1.43	62.01
J-90	263,510.22	1,245,683.19	2,430.00	0.88	66.63

APP.Table 4 Pressure Head and Elevation

Node ID	Elevation (m)	Pressure Head (m)
J-1	2,435.00	76.06
J-10	2,430.00	75.77
J-11	2,428.00	75.07
J-12	2,427.00	75.33
J-13	2,424.00	77.62
J-15	2,437.00	66.91
J-16	2,442.00	61.42
J-18	2,442.00	59.15
J-19	2,440.00	61.41
J-22	2,445.00	56.68
J-25	2,443.00	56.69
J-26	2,441.00	58.64
J-27	2,446.00	54.7
J-28	2,443.00	59.09
J-29	2,441.00	61.47
J-30	2,444.00	58
J-31	2,444.00	57.18
J-32	2,446.00	55.09

J-33	2,473.00	25.82
J-34	2,444.00	55.35
J-35	2,440.00	59.96
J-36	2,442.00	59.06
J-37	2,437.00	64.39
J-38	2,433.00	69.02
J-40	2,430.00	71.15
J-41	2,430.00	71.94
J-42	2,439.00	64.68
J-43	2,441.00	62.11
J-44	2,433.00	68.94
J-45	2,431.00	69.51
J-46	2,437.00	63.67
J-48	2,438.00	62.07
J-49	2,438.00	61.82
J-50	2,438.00	61.57
J-51	2,435.00	63.94
J-52	2,440.00	58.25
J-53	2,443.00	54.82
J-54	2,480.00	16.79
J-56	2,435.00	64.6
J-58	2,438.00	61.05
J-59	2,438.00	60.75
J-60	2,437.00	61.03
J-61	2,437.00	60.43
J-62	2,434.00	62.89
J-63	2,440.00	55.9
J-64	2,448.00	47.32
J-65	2,448.00	48.03
J-66	2,453.00	42.32
J-67	2,444.00	52.91

J-68	2,448.00	47.96
J-69	2,459.00	38.11
J-72	2,451.00	47.84
J-73	2,448.00	51.18
J-74	2,451.00	46.61
J-75	2,443.00	55.82
J-76	2,448.00	52.6
J-77	2,437.00	64.62
J-78	2,439.00	62.36
J-79	2,445.00	55.19
J-80	2,443.00	56.09
J-81	2,429.00	70.36
J-82	2,435.00	63.9
J-83	2,433.00	65.56
J-84	2,436.00	62.18
J-85	2,435.00	62.86
J-86	2,423.00	74.33
J-87	2,434.00	60.5
J-89	2,433.00	62.01
J-90	2,430.00	66.63



APP.Figure 1 Water Balance in Debre markos Distribution System 2011-2015

Pump Definition Detailed Report: Pump - 1

Element Details

ID	201	Notes	
Label	Pump - 1		

Pump Definition Type

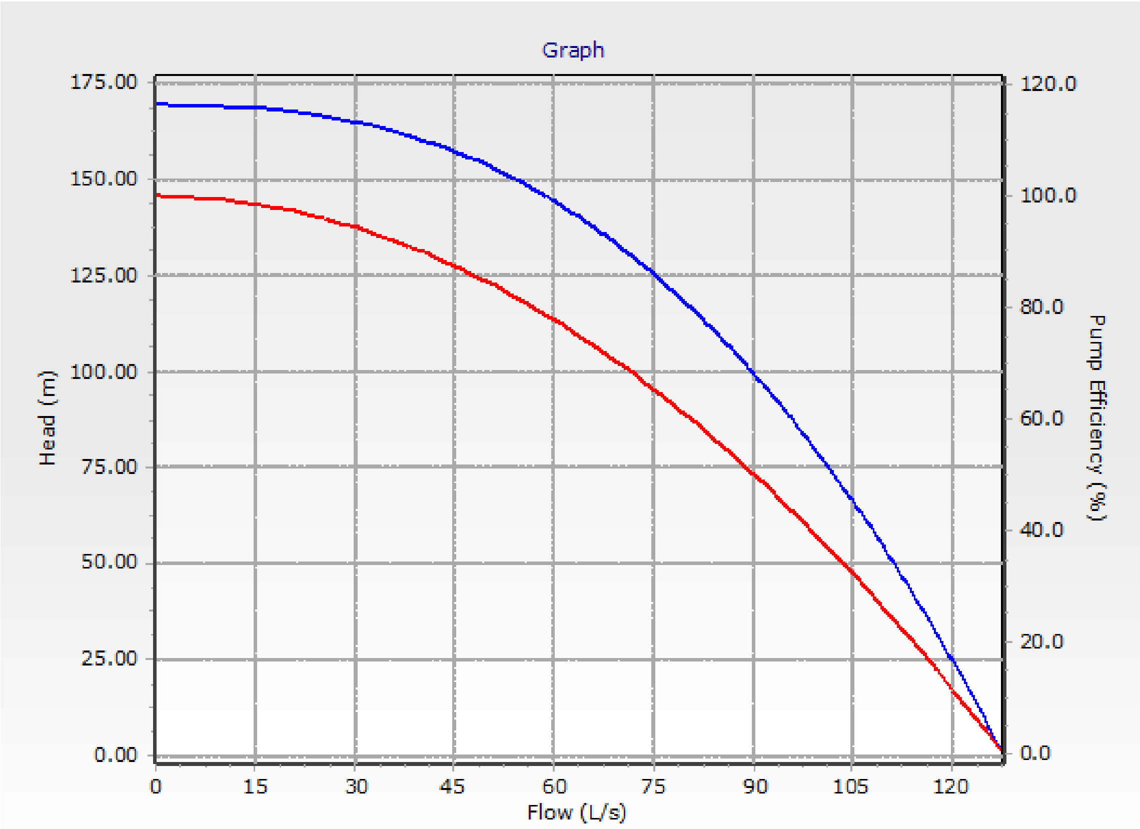
Pump Definition Type	Standard (3 Point)	Design Head	141.00 m
Shutoff Flow	0 L/s	Maximum Operating Flow	91 L/s
Shutoff Head	169.20 m	Maximum Operating Head	97.60 m
Design Flow	63 L/s		

Pump Efficiency Type

Pump Efficiency Type	Best Efficiency Point	Motor Efficiency	100.0 %
BEP Efficiency	100.0 %	Is Variable Speed Drive?	False
BEP Flow	0 L/s		

Transient (Physical)

Inertia (Pump and Motor)	0.000 kg·m²	Specific Speed	SI=25, US=1280
Speed (Full)	0 rpm	Reverse Spin Allowed?	True

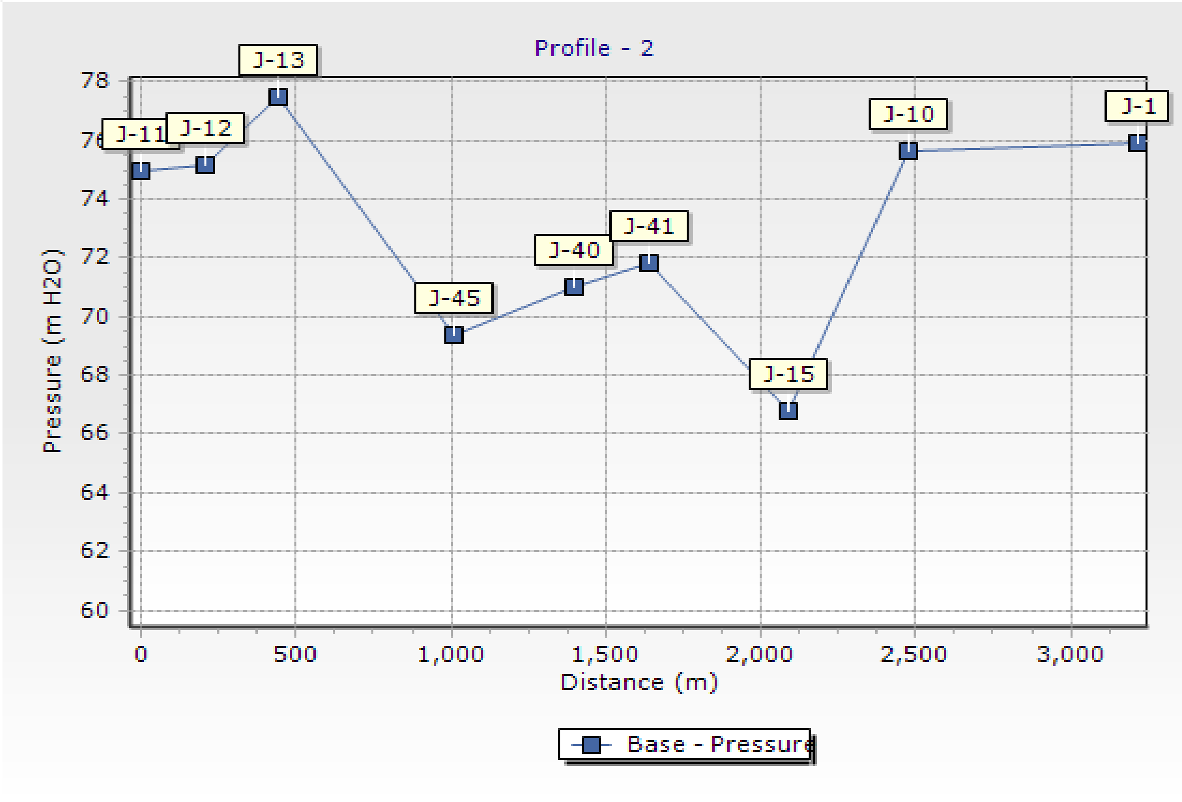


debre markos sub city layout.wtg
12/6/2016

Bentley Systems, Inc. Haestad Methods Solution Center
27 Siemon Company Drive Suite 200 W
Watertown, CT 06795 USA +1-203-755-1666

Bentley WaterCAD V8i (SELECTseries 6)
[08.11.06.58]
Page 1 of 1

APP.Figure 2 Pump report



APP.Figure 3 Pressure Profile at Each selected Node

QUESTIONNAIRES

Questionnaire No.:____ Name of Interviewer: _______________ Date of interview: ___________

I. Personal Information of Respondents

1. Name of Kebele _____

2. House No: _____

3. Sex: (1) Male (2) Female

4. Age: (1) Under 14 years (2) 15-39 years (3) 40-64 years (4) above 65 years

5. Educational attainment

    (1) None (2) Read- Write (3) Elementary school (4) Secondary school

    (5) High School (6) College (7) Graduated (8) Higher education (9) Others_____

6. Occupation: (1) Government Sector (2) Retired (3) Private Sector (4) Housekeeper

    (5) Other (specify) ____________

7. How many persons live in your household? __________________

    Infants (less than 1 year old) ( ) persons

    Children (1-18 years old) ( ) persons

    Adults (more than 18 year old) ( ) persons

8. House holding: (1) Private (2) Rent (3) Government (4) private (5) other (specify) ________

II. Local Administrative (DMWSS)

1. Is your Unaccounted-for-Water below 10%?

    1) Yes 2) No

2. Are there problems with the water source (s)?

    1) Yes 2) No

3. Is water available year round?

    1) Yes 2) No

4. What causes water loss in distribution system?

    1) Pipe damage 2) Heavy track 3) Illegal connection 4) Ageing pipe

5) I don’t know

5. Do you conduct an annual water audit of your system?

1) Yes 2) No

6. Do think your current method of calculating water loss fairly and accurately reflects the amount of water loss in the system?

1) Yes 2) No 3) I don’t know

7. Is your system 100% metered?

1) Yes 2) No 3) I don’t know

8. Are all public-sector facilities billed for their water use?

1) Yes 2) No

9. Do you bill based on actual meter readings, not estimated use?

1) Yes 2) No 3) I don’t know

10. Is the volume of water used stated in gallons on the bill?

1) Yes 2) No

11. Are water supply system operations fully funded by water supply system revenues?

1) Yes 2) No

12. Was your rate structure developed to promote water conservation and/or control demand (that is, do you charge more for water when demand is higher – for example, in the summer)?

1) Yes 2) No

13. Do you have a public education plan on water conservation?

1) Yes 2) No

III. Local Community

14. Do you have secondary source of water for drinking?

1) Yes 2) No 3) I don’t know

15. Are there any leaks in the household pipes?

1) Yes 2) No

16. Is water point secure?

1) Yes 2) No

17. Time required to the nearest alternative source of water

1) Minutes 2) Hours

18. How many times do you get water in a period?

1) Once in a day 2) Once a week 3) Twice a week 4) Monthly

5) Never through a year

19. Does the water line distribute in your home?

1) Yes 2) No

20. Where do you get water?

1) Borehole 2) Public tap/ stand pipe 3) Water point

21. Do you think shortage of water is occurred in Debre markos?

1) Yes 2) No 3) I don't know

22. What causes shortage of water?

1) There is no enough source of water 2) Problem of distribution 3) I don't know

23. Do you think pipe damage causes water loss?

1) Yes 2) No 3) I don't know

24. Do you think illegal connection occur in your area?

1) Yes 2) No 3) I don't know

25. Does the water line distribute in your home?

1) Yes 2) No