



**PERFORMANCE EVALUATION OF HALABA KULITO TOWN  
WATER SUPPLY SYSTEM, SNNPR, ETHIOPIA**

**MASTERS OF SCIENCE IN WATER RESOURCES ENGINEERING  
AND MANAGEMENT**

**BY**

**BEREKET KEBEDE**

**HAWASSA UNIVERSITY, HAWASSA, ETHIOPIA**

**JUNE, 2020  
HAWASSA, ETHIOPIA**

**PERFORMANCE EVALUATION OF HALABA KULITO TOWN  
WATER SUPPLY SYSTEM, SNNPR, ETHIOPIA**

**BY**

**BEREKET KEBEDE**

**M.SC THESIS SUBMITTED TO DEPERTEMENT OF WATER  
RESOURCES AND IRRIGATION ENGINEERING, SCHOOL OF  
GRADUATE STUDIES, HAWASSA UNIVERSITY HAWASSA**

**IN PARTIAL FULFILLMENT OF THE REQUIREMENT FOR THE  
DEGREE OF MASTERS OF SCIENCE IN WATER RESOURCES  
ENGINEERING AND MANAGEMENT**

**MAJOR ADVISOR: - Dr. MIHRET DANANTO (PhD)  
Co- ADVISOR: - Dr. ABRAHAM WOLDEMICHAEL (PhD)**

**JUNE, 2020  
HAWASSA, ETHIOPIA**

## DECLARATION

I, the undersigned, declare that this thesis entitled with **Performance Evaluation of Halaba Kulito Town Water Supply System**; In case study of Halaba Kulito Town is my original work, and has not been presented and submitted for any degree in any other university. It is being submitted for the degree of Master of Science in Water Resources Engineering and Management, and all sources of material used for this thesis have been fully acknowledged by means of complete references.

Bereket Kebede  
Candidate

-----  
Signature

-----  
Date

**APPROVAL SHEET**  
**HAWASSA UNIVERSITY**  
**HAWASSA INSTITUTE OF TECHNOLOGY**  
**DEPARTEMENT OF WATER RESOURCES AND IRRIGATION**  
**ENGINEERING**

This thesis entitled with **Performance Evaluation of Halaba Kulito Town Water Supply System**; prepared for partial fulfilment of the requirement for the degree of Master of Science in Water Resources Engineering and Management is produced by student Bereket Kebede, under my or our supervision. Therefore, I /or we recommend that he can submit the thesis to the department for defense.

Submitted by

Bereket Kebede)

-----

-----

Signature

Date

Approved by

Dr. Mihret Dananto (PhD)

-----

-----

Major Advisor

Signature

Date

Dr. Abraham Woldemichael (PhD)

-----

-----

Co-Advisor

Signature

Date

**HAWASSA UNIVERSITY**  
**HAWASSA INSTITUTE OF TECHNOLOGY**  
**DEPARTEMENT OF WATER RESOURCES AND IRRIGATION**  
**ENGINEERING**  
**EXAMINERS' APPROVAL SHEET**

We, the under signed members of the board of examiners of the final open defense by Bereket Kebede, have read and evaluated his thesis entitled **Performance Evaluation of Halaba Kulito Town Water Supply System**; and examined the candidate. This is therefore to certify that the thesis has been accepted in partial fulfilment of the requirements for the degree of Master of Science in Water Resources Engineering and Management.

Dr. Mihret Dananto (PhD)	_____	_____
Name of Major advisor	Signature	Date
Dr. Abraham Woldemichael (PhD)	_____	_____
Name of Co-advisor	Signature	Date
Mr. Gonse Amalo (MSc)	_____	_____
Name of Internal Examiner - 1	Signature	Date
Dr. Moltot Zewdie (PhD)	_____	_____
Name of Internal Examiner -2	Signature	Date
Dr. Beshah Mogessie	_____	_____
Name of External Examiner	Signature	Date

**Stamp of SGS**

Date\_\_\_\_\_

Final approval and acceptance of the thesis is contingent upon the submission of the final three copy of the thesis to School of Graduate Studies (SGS) through the Department Committee (DC) of the candidate's department acknowledgements.

## **ACKNOWLEDGEMENTS**

First and for most I am so grateful to the Almighty God for giving me health and strength to pass through all confronts during my life.

I am grateful to express my sincere thanks to my Major Advisor Dr. Mihret Dananto (PhD), and my Co-Advisor Dr. Abraham Woldemichael (PhD) for their academic advice, continuous support during thesis writing, for rendering their immense and valuable supervision and guidance and valuable comments for the accomplishment of my research paper. Thank you!

I would like to extend my sincere gratitude to Hawassa University to elaboration and giving this chance to me. I would also like to acknowledge Halaba Kulito Town water supply and sewerage service enterprise, which was my source of data to begin with my study, and my specific gratitude extends to Mr. Geremewu, Manager of HKTWSSSE, Mr. Mansur Jemal and Mr. Daniel Taba for helping me during field GPS survey and data collection for his technical support and consultation when I begin my thesis. I am indebted to my uncle Mr. Tekle Dobocho for providing me the basic tools in life to recognize the importance of education, and the support necessary to help me realize my academic goals.

I would like to take this opportunity to special thanks for my father Kebede Kendolo and my mother Mulu Sedebo for encouraging and helping me in sharing of my burden and my deepest gratitude to all my friends for those share supportive idea in doing the research.

## TABLE OF CONTENTS

Contents	Page
DECLARATION .....	i
ACKNOWLEDGEMENTS .....	iv
TABLE OF CONTENTS .....	xi
LIST OF TABLES .....	xv
LIST OF FIGURES.....	xvi
LIST OF ACRONYMS and ABBREVIATIONS .....	xvii
ABSTRACT.....	xix
1. INTRODUCTION.....	1
1.1 Background.....	1
1.2 Statement of the problem.....	2
1.3 Objectives of the study .....	3
1.3.1 General Objective .....	3
1.3.2 Specific Objectives .....	3
1.4 Significance and rational of the study .....	3
1.5 Scope of the study.....	4
2. LITERATURE REVIEW .....	5
2.1 Urban Water Supply .....	5
2.1.1 Performance indicators of urban water supply systems.....	5
2.1.2 Urban Water Demand and Coverage .....	6
2.1.3 Urban Water Supply Distribution Network Analysis Problem.....	6
2.2 Water supply distribution network .....	7
2.2.1 Problems of water distribution network.....	8
2.3 Performance evaluation of urban water supply systems.....	8
2.3.1 Hydraulic performance .....	9
2.3.2 Structural Performance .....	9
2.3.3 Customer Perception .....	10
2.4 Water supply mode in distribution system .....	10
2.4.1 Continuous system .....	10
2.4.2 Intermittent supply system .....	11

2.5	Layouts of Pipe Networks .....	11
2.6	Methods of Water Distribution.....	12
2.7	Components of water distribution network .....	12
2.8	Water Distribution Network Hydraulic Modelling.....	16
2.8.1	History of Hydraulic Modelling.....	16
2.8.2	Modeling Theory.....	16
2.8.3	Needs for Hydraulic Modeling .....	17
2.8.4	Gathering a Model .....	17
2.8.5	WaterGEMsv8i Modeling Capabilities .....	17
2.8.5.1	Data Requirements for Modeling of Water Distribution Network.....	18
2.8.5.1.1	Basic Hydraulic Model Inputs.....	19
2.8.6	Principles of pipe network hydraulics and Modeling Concept.....	19
2.8.6.1	Conservation of mass or continuity principle.....	19
2.8.6.2	Conservation of energy or energy principle .....	20
2.8.7	Head losses.....	21
2.8.7.1	Head loss equations and area of application .....	21
2.8.7.2	Friction losses .....	22
2.8	Hydraulic design parameters .....	22
2.8.1	Pressure .....	22
2.8.2	Flow Rate .....	23
2.9	Water distribution network simulation .....	24
2.9.1	Steady-state simulations.....	24
2.9.2	Extended- period simulations.....	24
2.10	Water Loss .....	25
2.10.1	Some Definitions of UFW .....	27
2.10.2	Non- Revenue Water (NRW).....	27
2.10.3	Causes of water losses.....	27
2.10.4	Factors causing loss of hydraulic integrity in water distribution network.....	29
2.11	Urban water demand.....	30
2.11.1	Types of urban water demand.....	31
2.11.1.1	Domestic water demand .....	31
2.11.1.2	Non-domestic demand.....	31
3.	MATERIALS AND METHODS.....	32

3.1. Descriptions of the study area.....	32
3.1.1 Location .....	32
3.1.2 Climate of the Study area.....	32
3.1.3 Demographic characteristics .....	33
3.2 Materials .....	33
3.4 Methodology.....	35
3.4.1 Framework of the Study.....	36
3.4.1.2 Sample Size and Sampling Techniques.....	36
3.4.2 Data Sources and Methods of Data Collection .....	37
3.4.2.1 Sources of Primary Data Collection.....	38
3.4.2.2 Secondary Data Collection .....	39
3.4.2.3 Data Quality Control .....	39
3.5.2 Analysis of Water Supply and Water Demand of Halaba Kulito Town.....	42
3.5.2.1 Estimation of domestic water consumption/coverage.....	43
3.5.2.2 Water Loss (Unaccounted for Water) Analysis .....	43
3.5.3 Distribution Network Simulation.....	43
3.5.3.1 Steady-State Simulation .....	44
3.5.3.2 Extended Period Simulation .....	44
3.5.4 Hydraulic Parameters .....	45
3.5.4.1 Pressure in the Distribution Network .....	45
3.5.4.2 Flow Velocity in the Distribution Network.....	46
3.5.4.3 Model Building and Data Entry .....	46
3.5.4.3.1 Modeling the existing water distribution network .....	46
3.5.4.3.2 Importing the Network .....	48
3.5.4.3.3 Model Skeletonization.....	49
3.5.4.3.4 Data Entering and Data Proofing .....	50
3.5.4.4 Hydraulics Analysis .....	51
3.5.4.4.1 Source Analysis .....	51
3.5.4.4.2 Modeling of Pumps .....	52
3.5.4.4.3 Tank Analysis.....	53
3.5.4.4.4 Assign Network Parameters .....	53
3.5.4.4.5 Hydraulic Calculation.....	54
3.5.4.4.6 Model representation.....	55
3.6 History and Description about Halaba Kulito Town Water Supply Distribution Network....	57

4. RESULTS AND DISCUSSIONS .....	59
4.1 Assessing the current status of water supply system of Halaba Kulito Town.....	59
4.1.1 Water Sources and Production .....	59
4.1.2 Coverage of Potable Water .....	59
4.2 Water Losses .....	60
4.3. Model Representation .....	61
4.3.1 Simulation Results .....	62
4.3.1.1 Steady State Simulation .....	62
4.3.1.2 Extended-Period Simulation .....	62
4.3.1.2.1 Pressure .....	63
4.3.1.2.2 Pipe Flow Velocity.....	68
4.3.2.3 Demand Pattern.....	71
4.3.3 Behavior of Storage tank at different consumption hours of a day .....	72
4.3.4 Pump.....	73
4.3.4.1 Pump capacity curve .....	73
5. CONCLUSIONS AND RECOMMENDATIONS .....	75
5.1 Conclusions.....	75
5.2 Recommendations.....	77
REFERENCES .....	78
APPENDIXES .....	84
Appendix 1.1.....	84
Appendix 1.2.....	85
Appendix 2.1.....	86
Appendix 2.2.....	88
Appendix 2.3.....	89
Appendix 2.4.....	90
Appendix 3 .....	92

## LIST OF TABLES

Table 2. 1 Input parameters and primary purposes of waterGEMS tools (Source; Bentley,2008)..	18
Table 2. 2 Head loss equations and area of application (Dighade, et al., 2014). .....	22
Table 2. 3 Water balance showing NRW components; IWA water loss task force (Source, Farley and Trow, 2003) .....	26
Table 3. 1 Total households and sampled respondents/households.....	37
Table 3. 2 Halaba Kulito Town water sources/reservoirs and their short description (HKTWSSSE, 2019).....	40
Table 3. 3 Pipe diameter and corresponding length used as software inputs (HKTWSSSE, 2019).	41
Table 3. 4 Location and capacity of existing storage tanks (HKTWSSSE, 2019).....	42
Table 3. 5 Demand Factor (MoWR, 2006) .....	42
Table 3. 6 The allowable operating pressures in the distribution network according to MOWR, (2006) .....	46
Table 3. 7 Pipe velocity range from various sources .....	46
Table 3. 8 Hazen-William roughness coefficients for pipe material (Chase <i>et al.</i> , 2003) .....	56
Table 4. 1 Annual water consumption of Halaba Kulito Town.....	59
Table 4. 2 Pipe diameter and corresponding lengths with their coverage (HKTWSSSE, 2019).....	61
Table 4. 3 Simulated Pressure distribution in system at peak hour consumption (8:00) .....	64
Table 4. 4 Nodes/Junctions were selected in the network modeling process.....	64
Table 4. 5 Simulated results of velocity range in distribution network .....	69

## LIST OF FIGURES

Figure 2. 1 Performance Classification of WDS (Jalal, 2008).....	9
Figure 2. 2 Physical Components of Water Distribution System.....	13
Figure 2. 3 Algebraic sum of flow rates entering and withdrawing from the node (Almasri, 2010).....	20
Figure 2. 4 Conservation of energy (Sharma, 2008) .....	20
Figure 3. 1 Map of study area.....	33
Figure 3. 2 Framework of the Study.....	36
Figure 3. 3 Flow chart for steady state simulation .....	44
Figure 3. 4 Schematic distribution of water networks of Halaba Kulito Town .....	50
Figure 3. 5 Data entering on flex table .....	51
Figure 3. 6 Assign network parameters .....	54
Figure 4. 1 Steady State Simulated pressures and velocities.....	62
Figure 4. 2 Pressures at 7:00 AM to 8:00 AM in the peak-hour consumption .....	63
Figure 4. 3 Graph showing nodes with negative pressure at peak hour consumption .....	65
Figure 4. 4 Profiles of nodes showing distance from storage Tank - 1 (T- 1) with elevation...	67
Figure 4. 5 Pressure contour at peak hour demand displayed with elevation .....	68
Figure 4. 6 Velocity distribution at peak hour consumption.....	69
Figure 4. 7 Main transmission line showing velocity verses time graph (P - 58, P - 75, P - 33, P - 96, P - 95, P-69).....	70
Figure 4. 8 Main transmission line showing flow verses time graph.....	70
Figure 4. 9 Demand pattern in Water distribution of Halaba Kulito Town within 24 hours ....	71
Figure 4. 10 The distribution network demand in Halaba Kulito Town .....	72
Figure 4. 11 Tank - 1 Water volume fluctuation over 24 hour periods.....	73
Figure 4. 12 Pump – head verses flow curve .....	74

## **LIST OF ACRONYMS and ABBREVIATIONS**

AWWA	American Water Works Association
BH	Borehole
CSA	Central Statistical Agency
DCI	Ductile Iron
EPA	Environmental protection Agency
EPS	Extended Period Simulation
GIS	Geographic Information System
GPS	Global Position System
HC	House Connection
HDPE	High Density Polyethylene
HKTWSSSE	Halaba Kulito Town Water Supply and Sewerage Service Enterprise
HTU	House Tab User
ISO	Insurance Service Organization
KPa	Kilo Paskal
M.a.s.l	Mean above sea level
MDGR	Millennium Development Goal Report
MoWIE	Ministry of water Irrigation and Electricity
MoWR	Ministry of Water Resource
MSc	Master of Science
NGOS	Non-Governmental Organizations
NTU	Neighbour Tab Users
OWWDSE	Oromia Water Woks Design and Supervision Enterprise
Ph.D.	Philosophy of Doctorate
PTU	Public Tab User
PVC	Polyvinyl Chloride
SSR	Summary and Statistical Report
SSS	Study State Simulation
TRex	Terrain Extractor
UFW	Unaccounted for Water
UNICEF	United Nations Children's Fund
WDN	Water Distribution Network
WDS	Water Distribution System

WHO	World Health Organization
YC	Yard Connection
YCS	Yard connection Share
YTU	Yard Tap Users

## ABSTRACT

*Safe and adequate delivery of water to a consumption node is an essential function of water distribution network. However, throughout the world especially in developing countries, urban water supply distribution network evaluation is inadequate to transfer available water to a consumption node. Therefore, this study aims at the study of evaluation of urban water supply distribution network with hydraulic parameters performance of Halaba Kulito Town water supply distribution network project as the case study. The methods used to carry out this research were desk study to evaluate previous designs, field data collection and data analysis. For conducting this study, both primary and secondary data were collected and hydraulic modeling software such as Bentley WaterGEMSv8i, ArcGIS version10.1, Geographic positioning system Garmin72 (GPS), EndNote program and other relevant materials were used. The study was carried out by selecting pipes having diameter greater or equal to 50 mm in diameter. The simulated result for both steady state and extended period simulation showed that the performance of distribution system related to pressure 48% for pressure value ( $< 15$  mH<sub>2</sub>O), 48% for pressure value between (15 - 60) mH<sub>2</sub>O and 4% for pressure value ( $> 60$  mH<sub>2</sub>O) pressure head and the velocity of pipe flow showed that 50% for velocity ( $< 0.6$  m/s), 48.8% for velocity range (0.6 - 2 m/s) and 1.2% for velocity ( $> 2$  m/s). Those problems are resulted from incorrect nodal placement and improper pipe connection during designing the system and when expanding the network to the newly established settlement area. The low pressure zone areas around Halaba Kulito Poly Technic College and below and above Halaba Kulito Hospital did not get water at these junctions during peak hour demand. The per capita domestic water consumption of study area was found to be 16 l/c/d in the year 2019. The minimum quantity of domestic water required in urban area of developing country in the radius 0.5 km taken as 20 l/c/day. Regarding to this value, the domestic water supply of Halaba Kulito Town only satisfies 80% of the standard value and the quantity of domestic water required in urban areas of Ethiopia is taken as 50 l/c/d. According to this value, the domestic water supply of Halaba Kulito Town satisfies only 32% of the standard value. The total loss of water in the Town for the year was 219409 m<sup>3</sup> and approximately 31.4% of the total production. This figure is lower compared with the average for developing countries (35%). The average the amount of water, which actually reached the consumers (the system efficiency) accounts for only 68.6% of the total water produced. Finally, the researcher recommends preventive measures of interruption of water supply network using control valves, scheduling and installing storage distribution balancing tank above the Halaba Hospital and installing additional water sources for delivering enough and sufficient amount of water to the customers.*

**Key words:** demand, evaluation hydraulic performance, analysis urban water supply network, water losses.

# 1. INTRODUCTION

## 1.1 Background

Access to safe and adequate water supply is universally recognized human right, which has special significance to the survival of humanity. Provision of safe and adequate water supply services is necessary components for sustainable development. The provision of adequate supplies potable water for use in urban areas in developing countries is crucial for the well-being of the people. The demand for such supplies in the developing countries has been on the increase over time as a result of rising standards of living that occur with economic progress and population increase resulting from natural growth, and rural urban migration and rising per capital income (Rewata and Sampath, 2007).

The prerequisite of adequate and reliable water supply in developing countries is becoming a challenge for most water utilities especially public service providers (Khatrik, 2007). Water demand has been increasing drastically in developing nation due to population growth as a result of rural to urban migration. As a consequence, in many countries public water service utilities have failed to provide consumers with adequate water supply and sanitation services. A partly from service coverage, there are other problems that affect public service providers such as high unaccounted for water (UfW) and financial problems due to a combination of low tariff, poor services, poor consumer records and inefficient billing practices (Kimey, 2008).

According to Ethiopian Water Sector Strategy (2001), condition of sustainable, professional, reliable, and reasonable and user's acceptable water supply to the urban population is a major concern in Ethiopia in general and in south nation nationality and people region in particular. The need on water supply increases due to the population growth rate, increasing standards of living and the increase in per capita consumption. As the result of demand increase in water supply, the additional water resources and infrastructure is growing.

The main purpose of design of water distribution network (WDN) is to supply the required quantity of water at required time with sufficient pressure. But, in many of the developing countries, drinking water considered as probability of a node being connected to supplies are inadequate to meet consumers' demands. Water supply systems designed and operated as intermittent systems. Water supply and distribution systems serve many critical functions and play a large part in achieving human and economic health. Despite this, the performance of these systems often goes unnoticed until there is a major disruption or operational failure (Prasad and Nanduri, 2014).

WDN needs great economic, social and environmental burdens. Performance measurement is a key issue in engineering the behavior and control of any WDN. The most common challenges in water distribution networks include water quality degradation, capacity shortages, infrastructure aging and deterioration, demand increases, and their ever-increasing energy consumption coupled to the global energy crisis (Jalal, 2008). In water utility systems, significant amount of water is lost as leakage while in transport from source up to consumers. Water loss represents inefficiency in water delivery and measurement operations in rising main and distribution networks. By acquiring a continuous water supply, cities in the developing nations must ensure that their water systems become more efficient and effective by reducing water losses, gradually increasing water tariffs, improving revenue Collection, increasing staff productivity, and securing safe and reliable water supplies. When the productivity increased, investments in new infrastructure will lead to more effective and efficient water services (Dighade et al., 2014).

The estimated water supply service level of Ethiopia in terms of coverage, quantity, quality and reliability is very low. A well performing urban water supply system should provide water supply for human being and livestock consumption, for industrial and other uses in terms of coverage, quantity, reliability and acceptable quality taking the existing and future realities of the city in to consideration (Dighade et al., 2014). This research paper will assess and evaluate the performance of Halaba Kulito Town water supply system in terms of four main performance indicators such as Hydraulic performance, Structural performance, Water loss versus water supply coverage and customer satisfaction and recommend solutions for improving the water supply service.

## **1.2 Statement of the problem**

Mostly problems that occurred in developing countries are intermittent, erratic pressure is not acceptable, inequalities in service provision between the rich and the poor, high rate of water losses from the distribution systems, Population growth and urbanization, growing urban water demand, Infrastructure is aging and deteriorating (Jalal, 2008).

Even though all water supply distribution systems are designed to supply adequate potable water to the community whom they are constructed for, potable water shortage is a big issue, especially in developing countries including Ethiopia.

Halaba Kulito is one of the medium level town in SNNPR State of Ethiopia that passed through various socioeconomic progresses. The study area has been experiencing frequent and

regular disruption of water supplies for days to a week. Although the town water supply and sewerage service enterprise trying to curb the problem, delivering sufficient water without any interruption to the dwellers remains dream. Unprecedented rate of population growth, urbanization and the need for repair and maintenance of very old and outdated structures widen the gap between demand and supply of water in the town.

Earlier studies conducted on water supply both at international and national levels focused mainly on rural areas and big cities. However, in small and medium towns like Halaba Kulito no adequate research yet carried out. Furthermore, the town is one of the medium level towns in the country with recent rapid urbanization and high population growth. This clearly calls for the expansion and improvement of water supply system service through research based estimation of water demand and appropriate hydraulic networking. Therefore, this research attempts to evaluate the current performance, to identify problems related to flow velocity and pressure, to improve the current performance of the distribution system by formulating and suggesting solutions to the identified problems and its outlook to provide base line information for decision makers and further research.

### **1.3 Objectives of the study**

#### **1.3.1 General Objective**

The General objective of the study was evaluating performance of Halaba Kulito Town water supply network, using hydraulic modeling software (WaterGEMSV8i).

#### **1.3.2 Specific Objectives**

The Specific objectives of this research includes

- To assess the current status of water supply system of Halaba Kulito Town
- To assess the water supply system efficiency of Halaba Kulito Town
- To evaluate the hydraulic performance of the existing water supply network of Halaba Kulito Town.

### **1.4 Significance and rational of the study**

According to Binerley (2006), models are defined as representations of a complex reality. Modeling and simulation are aimed at providing valuable insights in the problem structure instead of giving precise answers. With the advances of this technology, water utilities and engineers have been able to analyze the status and operations of the existing system as well as to investigate the impact of proposed changes. Poor performance of water supply distribution network is the main problems in whole world predominantly in poor countries like Ethiopia.

The study result shows the existing water supply situation of the urban dwellers by investigating the water demand, water losses and identifying factors that affect the performance of water distribution network. It also provides insight to policy makers, NGOs, community based organizations and other stakeholders who are concerned with urban water supply problems. The paper also adds to the literature of urban water supply issue, which are currently the global challenge. Despite of these efforts good water supply distribution system in Halaba Kulito Town is still characterises with inadequate distribution, insufficient coverage of services and water losses. This is due to increasing number of population and town expansion in different infra structures. Evaluating the town water distribution network with WaterGEMsv8i software was the great importance in solving problems related to hydraulic performance in water supply distribution network of the Halaba Kulito Town. Little has been done to review and evaluate the performance of the urban water supply utilities. It also serves as the base line for other researchers who will be interested in the area.

### **1.5 Scope of the study**

The primary goal of the study was to assessing and evaluating the water supply network performance of the existing distribution network of the Halaba Kulito Town, which have aimed to help the enterprise to understand its distribution system needs and assist them in long-term planning of water distribution network assets. Thus, the scope of this study was to evaluate the performance of the existing drinking water distribution network using hydraulic simulation software by running the model for steady state and extended period simulation analysis to identify system hydraulic performances related to pressure and velocity in the urban residences of Ethiopia particularly in the Halaba Kulito Town and recommend any changes in the existing system.

## **2. LITERATURE REVIEW**

### **2.1 Urban Water Supply**

Safe drinking water is the birth right of all humankind as much a birth right as clean air (Rao, 2002) while access to clean water can be considered as one of the basic needs and rights of a human being. Health of people and dignified life is based on access to clean water (Korkeakoski, 2006).

Water is important in a number of ways; these include domestic and productive uses. Domestic water use takes the form of drinking, washing, cooking and sanitation, while productive water uses includes those for agriculture, Beer brewing, brick making, etc. Safe drinking water matched with improved sanitation contributes to the overall well-being of people; it has significant bearing on infant mortality rate, longevity and productivity. However, the majority of the world's population in both rural and urban settlements does not have access to safe drinking water (Alaci, 2009).

According to (WHO, 2006), only 16 % of people in sub-Saharan Africa had access to drinking water through a household connection (an indoor tap or a tap in the yard). The primary goal of all water supply utilities is to provide customers with a private connection to the piped water supply network. For many public officials, policy makers and politicians a household or yard connection (here after referred to as a private connection) is considered the most satisfactory way to meet the following key objectives; Public health objectives: by ensuring better quality and access. Commercial objectives: by facilitating cost recovery and revenue generation. Social objectives: by improving access for the poorest and enhancing security and safety. Environmental objectives: by enabling better demand management and water conservation (Khatrik, 2007).

#### **2.1.1 Performance indicators of urban water supply systems**

Before evaluating the performance of urban water supply system, it is important to develop appropriate performance indicators. The following are suggested performance indicators for evaluating urban distribution systems (McKibben, 2008). The indicators of urban water supply system are grouped under water resources performance, physical performance, and operational performance. Water resources availability and the availability of own water is mainly categorized under the water resource performance indicator. The capacity of storages, quality of the transmission and distribution lines and the density of the metered customers are taken as

physical performance. On the other hand, loss management, the operation and maintenance as well as quality of water supplied fall under operational performance.

### **2.1.2 Urban Water Demand and Coverage**

Water supply coverage provides a picture of the water supply situation of one specific country or city and helps to compare one country with others and the inter and intra city distribution within a specific country. The percentages of population with or without pipe water connection are a relevant indicator to compare the coverage of water supply in urban areas. Although the water supply coverage is better in urban areas while compared with the rural, the actual water supply coverage in cities of developing countries in general and African cities in particular is very low while compared to the demand.

According to the Global Water Supply and Sanitation Assessment 2015 Report, the African largest cities are having 45% house connection or yard tap, 22 % served by public tap while 33% of the population are un-served (WHO, 2009). A household is considered to have access to improved drinking water if it has sufficient amount of water (20liters/person/day) for family use, at an affordable price (less than 10% of the total household income), available to household members without being subject to extreme effort (less than one hour) a day for the minimum sufficient quantity), especially to women and children) (Hunter, 2010). On the other hand, a minimum quantity of 25 litres of potable water per person per day provided at a minimum flow rate of not less than 10 litres per minute with the source being available within 200 meters from a household and the supply not interrupted for more than seven days per year (i.e. water should be available 98% of the time) is considered as a basic service for southern African cities" domestic water supply (Wallingford HR., 2003)). Ethiopia has long been characterized by limited access to safe drinking water services. In 1990, for instance, only 19 percent of the country's population had access to a safe drinking water supply (Berhe, 2005). By 2007 this figure had reached 52 percent.

### **2.1.3 Urban Water Supply Distribution Network Analysis Problem**

The water distribution network analysis problem is defined as the selection of the most desirable configuration of circulation network considering the allowable pipe diameter and water demand in each point while satisfying various possible objectives such as network reliability, redundancy, water quality. One of the most common and favourable objective function of water distribution system is considered as minimizing the network arrangement cost, by suitable selection of pipe diameters and lengths. This can be expressed as:

$$c = \sum_{i=1}^N f(D_i, L_i) \quad 2.1$$

where  $f(D_i, L_i)$  is the cost of  $i_{th}$  pipe, with diameter  $D_i$  and length  $L_i$ , and  $N$  is the number of pipes in the network configuration.

In each engineering problem different phases should be performed to achieve a goal, analysis and design. In the water distribution systems problem, which is a complex system of pipes, the goal is defined as the length and diameters of the pipes forming a complex configuration while obtain the required water demands at certain points of the network (Dighade, *et al.*, 2014).

**Analysis phase:** In the analysis phase, the goal is to achieve a distribution of water for the postulated configuration of pipe lengths and diameters among an infinite number of distributions. This is achieved in the light of the fact that only our proposed distribution should satisfy the continuity equation in each nodes, and satisfy the hydraulic head loss principle in the system loops. In other word, only a few distributions can assure the continuity equation in each node and through these distributions, only one distribution can satisfy the hydraulic head loss equations (Dighade, *et al.*, 2014).

## 2.2 Water supply distribution network

Water supply distribution networks are very important lifeline infrastructure systems, where failures are inevitable. A typical WDNs consists of network of pipes, nodes linking the pipes, storage tanks, reservoirs, pumps, additional appurtenances like valves (Belay, 2012). Water distribution systems represent a major portion of the investment in urban infrastructure and a critical component of public works. The main goal is to design water distribution systems to deliver potable water over spatially extensive areas in required quantities and under satisfactory pressures. Therefore, hydraulic models for water distribution networks have become indispensable tools for understanding system behavior by simulating pressures and flows at different locations and times in the networks (Nyende *et al.*, 2012). The design of water distribution systems in general based on the assumption of continuous supply. However, in most of the developing countries, the water supply system is not continuous but intermittent (Khatrik, 2007). A well-planned water distribution network is very essential in the development of urban areas. The network is built to satisfy various consumer demands while meeting minimum pressure requirements at certain nodes. For lower pressures, there cannot be a water delivery and for higher pressures, there can be excessive amount of leakage. To provide this, the service area is divided into different pressure zones. One of the main criteria

determining the number of zones is the topology. A system serving to a highly elevated hilly area has more pressure zones than a relatively flat area (Misirdali, 2003).

### **2.2.1 Problems of water distribution network**

Water flow is a function of several things, including the size and shape of the opening, and the pressure at the opening (Rossman, *et al.*, 2003). Typically, city water supplies are at 40 to 70 m, (static pressure). Older private systems are set to maintain water pressure between 20 m and 40 m, which is too low for some lifestyles; plumbers can set systems higher if the pump is capable of delivering higher pressure (MOWR, 2006).

**Water pressure drops due to gravity:** Gravity is another source of pressure loss in a residential plumbing system. Energy is required to push the water uphill. For every 0.305 m of elevation increase in a pipe, approximately 0.434 m is lost. With no water flowing, the static pressure available at the street main may be 60 psi, but the static pressure at the second floor basin would be 52 m (Ilesenim, 2006).

**Water pressure drops due to corrosion:** When the water pressure is poor in the distribution system, the most common cause is corroded galvanized steel piping. The common 12.7 mm diameter piping can closedown so that the opening is only 3.18 mm diameter or even less. The only solution is to replace this pipe typically with copper. It is wise to replace with a larger diameter pipe on the main feeds at least to improve pressure. When galvanized steel pipe is present, and pressure is low, it is common for accessible pipes running across the basement ceiling to be replaced first (Hutton *et al.*, 2007).

**Water pressure drops due to distance from the source:** If more water is flowing, the pressure drops more at each point along the pipe (Hutton, *et al.*, 2007). The more fixtures flowing at once, the greater the pressure drop at all fixtures and the lower the flow at each fixture (Rossman, *et al.*, 2003).

**Other Causes of Poor Water Pressure:** The supply line from the street to the house may be undersized, damaged or leaking. Long runs of relatively small (13 mm diameter) pipe within a house will result in considerable pressure drop. Closed pipe within the house will adversely affect pressure. In addition, defective, undersized or poorly adjusted pump will result in poor pressure (Rossman *et al.*, 2003).

### **2.3 Performance evaluation of urban water supply systems**

Evaluating the performance of water supply systems is an important for water industry to deliver competent levels of service. A good distribution system should be a capable of

supplying water at all intended place within the city with reasonably sufficient pressure head and the requisite amount of water for various types of demand (Garg, 2010). The performance of urban water supply scheme can be evaluating based on four performance measures: hydraulic, structural, water quality and customers’ observation. The performance of urban water distribution network can be categorized in Figure 2.1.

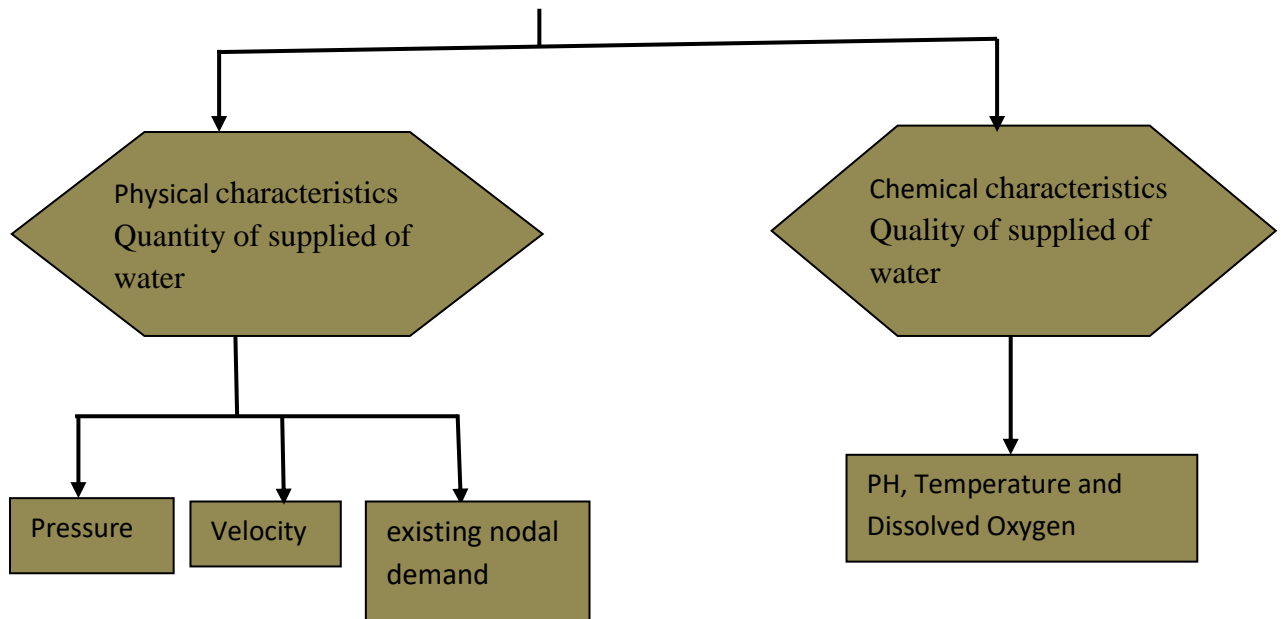


Figure 2. 1 Performance Classification of WDS (Jalal, 2008)

### 2.3.1 Hydraulic performance

The hydraulic performance of a water distribution system is the ability to provide liable water supply at an acceptable level of service that is, meeting all demands placed upon the system with provisions for adequate pressure, fire protection, and reliability of uninterrupted supply (Zyoud, 2003). Thus, hydraulic simulation modeling is now a day the most common tool used by water supply engineers and managers, as a complement to their experience and insight, at the process of establishing a diagnosis, defining the remedies and implementing them (Tabesh *et al.*, 2011).

### 2.3.2 Structural Performance

Water mains generally consist of a variety of pipe work and fittings, and which over time are subject to various episodes of augmentation, refurbishment, renewal, replacement, repair and extension. Physical performance of water supply system is the ability of the distribution

system to act as a physical barrier that prevents external contamination from affecting the quality of the internal, drinking water supply (Tabeshe and Dolakhahi, 2006).

The most obvious indication of the physical deterioration and failure of the pipe network is leakage. Analysis of a pipe network is essential to evaluate a physical system of water supply systems. The annual volume of water lost is an important indicator of water distribution efficiency, both in individual years, and as a trend over a period of years. High and increasing water losses are an indicator of ineffective planning and construction, and of low operational maintenance activities (Mckenzie, Hamilton and Seago, 2006). The other indicator is the volumetric efficiency which is the ratio of the registered volume and the total supplied volume during a certain reference period of time a value above 75% is considered to be acceptable.

### **2.3.3 Customer Perception**

It is important to maintain the public's confidence in the quality of drinking water and the services provided by a utility. Satisfied customers will pay their bills promptly and will provide political support for necessary rate increases or bond issues. In order to evaluate a WDS, it would be ideal to identify all major customers with their preferences, expectations, needs and requirements and then to explore the ways of meeting their expectations with consideration to associated consequences. Major customers may need those facilities that constitute significant portion of supply demand in a region (e.g., residential, Industrial, and firefighting users, public health officials). An ideal approach might be to investigate the quantity of water needed for each Individual customer, the period they need water for, and the appropriate level of water quality that is suitable for their need. The estimation of the quantity of water should reflect customer preferences and expectations efficiently. The more closely customer needs are met, the higher the level of satisfaction for customers and the better the water utility is managed (Jalal, 2008).

## **2.4 Water supply mode in distribution 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 firefighting. In addition, due to continuous circulation, water always remains fresh. In this system less diameter of pipes are required and rusting of pipes will be less. Losses will be more if there are leakages in the system (Sharma, 2008; Venkateswara, 2005)

### **2.4.2 Intermittent supply system**

The distribution system usually designed as a continuous system based on the assumption of continuous supply. However, in most developing countries water supply is not continuous but intermittent. 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, polluted water may enter the supply mains through leaking joints, pollute the supplies, increases the maintenance cost, polluted water through leaks during non-flow periods and more wastage to collect fresh water at each supply time and create the health related problem. Intermittent systems, which require frequent valve operations, are likely to affect equitable distribution of water mostly due to operator negligence. 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 Layouts of Pipe Networks**

**Branched or Dead End System:** In this system, a main line is taken from the reservoir along the main road. The sub-mains are taken suitably from the main line. Cut-off valves are provided at the entry of sub-mains. From the sub mains, the branch lines are taken from which service connections are given to consumer through the ferrule. The end of the sub-mains and branch lines are stopped by scour valves which are known as dead – ends. Due to the dead – ends, there is no free circulation of water and the water remains stagnant within the pipe line. This system is suitable for regular developing town or city.

**Grid Iron or Loop System:** This system has pipes that are interconnected throughout such that water can move through the entire system back and forth, depending on the points of largest demand. In this system, the main line, the sub-main lines, and the branch lines are interconnected. So, there is free circulation of water through the pipe lines Jeffrey and Gilbert (2012).

**Circular or Ring System:** Supply to the inner pipes is from the mains around the boundary. It has the same advantages as the grid-Iron system. Smaller diameter pipes are needed. The advantages and disadvantages are same as that of grid-Iron system.

**Radial System:** This is a zoned system. Water is pumped to the distribution reservoirs and from the reservoirs it flows by gravity to the tree system of pipes. The pressure calculations are easy in this system. Layout of roads needs to be radial to eliminate loss of head in bends.

This is most economical system also if combined pumping and gravity flow is adopted. It is suitable when the town or city can have oriented with radial roads and streets. In this system, the water from the main reservoir is allowed to flow through the main pipe and sub-main pipe and get collected at distribution reservoir of each zone. The water is supplied to consumers through the distributor pipe lines.

## **2.6 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 as gravity system, pumping system and combined system (Behailu, 2012).

**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.

**Pumping System:** Constant pressure head can have 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.

**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 firefighting (Venkateswara, 2005).

## **2.7 Components of water distribution network**

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 groundwater

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. 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).

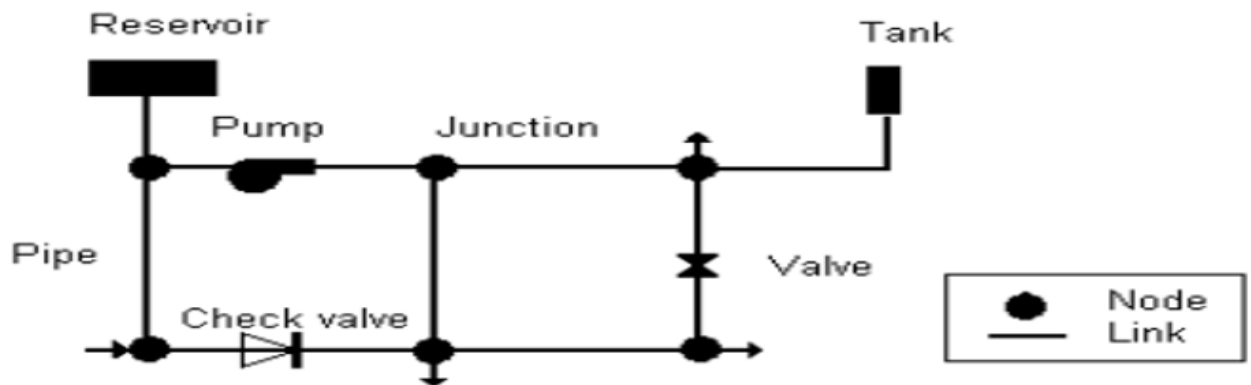


Figure 2. 2 Physical Components of Water Distribution System

**Distribution Reservoir:** Water is collected for use in distribution reservoirs which may be natural or artificial. The primary water sources of water supply system are distribution reservoirs. Dams, water wells, spring collections and water treatment plant storages are some examples to the distribution reservoirs. Distribution reservoirs store large volumes of water to let the water supply system to run continually.

**Storage Tank:** Storage Tanks are artificial structures that store water and provide water to the system when needed. Equalizing and emergency storage are the two basic task of storage tanks. The variation in flow can be dealt with by operating pumps in parallel and /or building balancing storage in the system. Moreover, in low demand hours when the water consumptions of consumers are almost zero, amount of pumped water is higher than system demand and extra water coming from pumps are stored at storage tank and equilibrium of water distribution system is satisfied again. This equilibrium purpose of storage tank is called

as equalizing storage. In addition, storage tanks help water utility to easily manage pressure distribution by prevention pressure fluctuations. Emergency storage ability of storage tanks provide required water to perform fire-fighting operations or maintenance operations. For instance, if the pump of distribution network is turned off due to power cut, distribution network continues to serve to the customers by using water stored in the storage tank till the end of power outages (Al-Rayess, 2015).

**Pipe Network:** The piping system contains transmission main/trunk, distribution mains and service mains to convey water from the source to the consumers. Transmission mains convey large amount of water over a great distance, mostly from the treatment plant to storage tanks, whereas distribution mains are smaller in diameter than transmission mains and deliver water to the end customers following the general topography and the alignment of main streets.

**Pipe length:** The length assigned to a pipe should represent the full distance that water flows from one node to the next, not necessarily the straight- line distance between the nodes of the pipe. Scaled versus schematic length. Most simulation software enables the user to indicate either a scaled length or a use-defined length for pipes. Scaled length are automatically determined by the software, or scaled from the alignment along the electronic background map. User-defined lengths, applied when scaled electronic maps are not available, require the user enter pipe length. Even in some scaled models, there may be areas where there are simply too many nodes in close proximity to work with them easily at the model scale (such as at a pump station) (New bold, 2009). In this case, the modeler may want selectively the portion of the system schematically.

**Pipe diameter:** A pipe's nominal diameter refers to its common name, such as a 4 inch (100 millimeter) pipe. The pipe's internal diameter, the distance from one inner wall of the pipe to the opposite wall may differ from the nominal diameter because of manufacturing standards. Most new pipes have internal diameter that are actually larger than the nominal diameter.

**Junctions:** The primary function of junction node is to provide a location for two or more pipes to meet. The other is to provide a location to withdraw water demand from the system or inject inflows (sometimes refers to as negative demands) into the system. Junction demands typically do not directly relate to real-world components since pipes are usually joined with fittings, and flows are extracted from the system at any number of customer connections along a pipe (Al-Rayess, 2015).

**Pump:** A pump is an element that adds energy to the system in the form an increased hydraulic grade. Since water flows “downhill” (that is, from higher energy to lower energy), pumps are used to boost the head at desired locations to overcome piping head loses and physical elevation difference (Kaychamber, 2004).

A three-point pump curve can be developed based on our static and hydrant tests, a range of demand and/or tank levels in the proposed system. The formulae can be used to develop a 3-pont pump curve (Kaychamper, 2004).

$$Q_o = Q_t \left( \frac{P_s - P_o}{P_s - P_t} \right)^{0.5} \quad 2.2$$

Where

$Q_o$  = Flow available at the chosen pressure ( $m^3/s$ );  $Q_t$  = Residual flow during hydrant test ( $m^3/s$ );  $p_s$  = Static pressure during hydrant test (kpa);  $p_o$  = Chosen pressure, at which  $Q_o$  is to be calculated (psi, kpa);  $p_t$  =Residual pressure during hydrant test (psi, kpa).

Pumps are energy devices which provide pressure and head to the water. The graph of head vs. flow for a particular pump is called the ‘pump curve’. Generally, there are three parameters that define the pump operation; the shut off head, the design point, and the maximum point. The system curve is an important curve necessary to decide the best operating point of pump. The pump should be able to overcome the elevations differences, which is dependent on the topography of the system. The head added on the pump to overcome these differences is called the static head. Friction and minor losses also affect the discharge through the pump. “When these losses are added to the static head for different discharge rates, the plot obtained is called system head curve” Walski, Thomas (2007).

**Valves:** The two general valves in water distribution systems are isolating and control valves. Isolating valves are installed to isolate part of a distribution system for maintenance and repair and they should be located so that the area isolated for repair receive the lowest inconvenience throughout the repair and maintenance program. Many utilities have valve turning programs on a regular basis even though it is difficult to implement on large systems. But these valves have to be turned at least once per year to reduce the likelihood of the valves to become inoperative due to corrosion. The other category of valves is the control valves group which consists of pressure reducing valves, pressure sustaining valves, throttling valves, check valves and flow rate control valves to control the flow and pressure in the distribution system.

**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).

**Hydrants and other appurtenances/blow-off and air release valves/:** The primary purpose of hydrants is firefighting. Although water utilities are not legally responsible for firefighting, distribution systems are designed to support needed fire flow since developmental requirements often include fire flow (Almasri, 2010). Fire hydrants are exercised and tested periodically by fire department personnel or by water utilities to satisfy the requirement of the Insurance Service office (ISO) or as water distribution system calibration programs (ISO, 2003).

## **2.8 Water Distribution Network Hydraulic Modelling**

### **2.8.1 History of Hydraulic Modelling**

The use of mathematical methods to calculate flows through a complex pipe network was first proposed by Hardy Cross. For utilizing the Hardy Cross methodology, improved solution methods were developed with the advent of computers and computer based modelling. Initially hydraulic models were simulating flows and pressures in a distribution system under steady state conditions assuming all demands and operations remained constant, but since demands and flows vary over the course of a day, Extended Period Simulation Models which can simulate distribution systems behavior under time-varying conditions were developed (EPA, 2005a). Steady state simulations were advanced to EPS using the technique developed by Rao and Bree in the late 1970's (Laura Baumberger *et al.*, 2007).

### **2.8.2 Modeling Theory**

In order to effectively utilize the capabilities of WDN simulation software's, it is must to understand the mathematical principles involved and the principles of hydraulics related to fluid properties. Specific weight, fluid viscosity and compressibility are the most important fluid properties to be considered in WDN simulations. As thoroughly discussed by different references like (AWWA, 2012) and (Paula, 2000). Models essentially use two types of relations to calculate flows in a complex pipe network system: Conservation of Mass and Conservation of Energy principles.

### **2.8.3 Needs for Hydraulic Modeling**

Most small and medium towns do not have very complex networks as compared to Cities; however, they have poor data and records regarding their systems. In such cases, when one has to evaluate the hydraulics and the water quality of the distribution systems, it is advantageous to use computer models. Computer models making use of hydraulic simulation software are capable of representing the behavior of a real time system and have the capability of predicting the performance of the same system for future 'what if 'scenarios (Walski Hasted, 2003). Simulation can be used for analysis of the existing system to improve the supply in terms of pressure / flows/ and minimize leakage. Simulation of a network is also important to make decisions about the network augmentation requirements due to increase in water demand or expansion of a water servicing area. The understanding of pipe network flows and pressures is important for making such decisions for a water supply system (Swamnee and Sharma, 2008).

### **2.8.4 Gathering a Model**

Gathering information describing the WDN is necessary before building the model. System maps, recordings, topographic maps, as-build drawings, electronic maps and recordings, non-graphical data and Computer-Aided drafting are potential sources of data. Then model skeletonization and the level of detail to be included should be decided by both the modeler and the utility which administrates the water supply system. Water distribution networks contain both nodes and links. The WDN nodes are grouped by sources, control and distribution nodes and demand nodes. On the other hand, links are capacitated as transmission and distribution pipes with specified length, diameter and other attributes. Due to operational flow and pressure requirement, pumping cost considerations, flow redirections following failure of major supply path, links in WDN are subjected to occasional changes except pipes attached to a source or sink. To establish realistic correlations between the topology of the network and operational aspects, a comprehensive assessment of WDNs resilience should be taken in to account the non-topological specifications of the network components (Yazdani and Jeffrey, 2011).

### **2.8.5 WaterGEMSv8i Modeling Capabilities**

WaterGEMSv8i provides and allowing modeling practically for any distribution system aspect. Therefore, working with WaterGEMSv8i used as for decision-support tool for water infrastructures and were help to assess and/or operate (Dawe, 2000a; Water (Bentley, 2008); The hydraulic analysis at a steady-state or an extended-period simulation, Pressure, flow and

demands in the system and to see how behaves over time, The size of pipes, pump and computer system head curves, Tank, pump and valve behavior in the system, Leakage and water loss from the network, Calibration the model either manually or use the Darwin Calibrator methods, And, generate fully customizable in graphs, charts and reports form.

**Input data for assembling the model:** In practice, pipe networks consist not only of pipes, but composed of vary fittings, services, storage tanks and reservoirs, meters, regulating valves, pumps, and electronic and mechanical controls. For modeling purposes, these system elements were organized into the following categories (Bentley,2008).

Table 2. 1 Input parameters and primary purposes of waterGEMS tools (Source; Bentley,2008).

Element	Type	Primary modeling purpose	Input data
Reservoir	Node	Provides water to the system	Hydraulic grade line (water surface elevation)
Tank	Node	Stores excess water within the system and releases that water at times of high usage	Base Elevation, Max. Elevation, Min. Elevation, and Diameter
Junction	Node	Discharge the demand required or recharge the inflow water from/to the system	Elevation
Pipe	Link	transport water from one node to another	Elevation, Diameter, Material and Roughness coefficient
Pump	Node /Link	provide energy to the system and raise the water pressure to overcome elevation differences and friction losses	Elevation, Pump definition (Characteristics of max. operation and design discharge and head efficiency)
Valves	Node /Link	Controls flow or pressure through a pipe and results in a loss of energy in the system	Elevation, Diameter, Valve type

### 2.8.5.1 Data Requirements for Modeling of Water Distribution Network

**Sources of Data:** Electronic maps, recordings and CAD drawings are common target data sources. Non-graphical data such as tracking and inventory data base or text based models can

also be used, but with great care; because simple topographic errors in a non-graphical network is very difficult to detect (Walski, 2003).

#### **2.8.5.1.1 Basic Hydraulic Model Inputs**

**Pipe network inputs:** The WaterGEMSv8i software package requires information on pipe diameters, pipe lengths, pipe roughness factor 'C', pump curves, different valve settings, tank cross-section information, tank elevations, nodal elevations, zonal boundaries and many other information (Walski, 2003).

**Water demand inputs:** Data concerning existing demand from water billing systems, spatial allocation of data from billing and GIS, time varying factors, projected future demands and their allocations from the water utility and regional planning documents can be collected.

**Operational and model control inputs:** Information on source nodes, pump stations, reservoirs, control valves and zonal valves can be collected from the operational staff. It is necessary to define a set of rules that tells how the water system operates in an EPS model. These operation rules maybe a set of 'logical controls' in which operations such as pump on/off, valve status, pump speeds, tank water levels, node pressures, demands etc. are controlled using 'what-if...then else' logical operators (Grayman and Rossman,1994).

#### **2.8.6 Principles of pipe network hydraulics and Modeling Concept**

In the network of inter connected hydraulic elements, every element is influenced by each of its neighbours; the entire system in such a way that the condition of one element must be consistent with condition of all other element. The Main principle of network analysis with Bentley WaterGEMSv8i 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). Therefore, the two basic equations that govern in waterGEMSv8i modeling network of these inter connections presented by (USEPA, 2005a) as follows:

##### **2.8.6.1 Conservation of mass or continuity principle**

For steady state condition, conservation of mass states that a fluid mass entering any pipe system will be equal to the mass leaving at any pipe system. It assumed that water is incompressible (Dawe, 2000b).

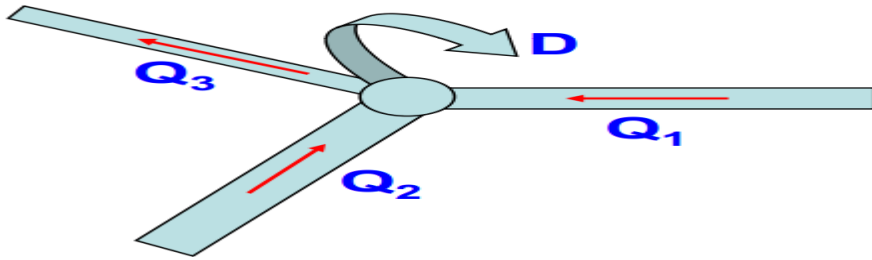


Figure 2. 3 Algebraic sum of flow rates entering and withdrawing from the node (Almasri, 2010).

$$Q_1 + Q_2 = Q_3 + D \quad 2.3$$

$$D = Q_1 + Q_2 - Q_3 \quad 2.4$$

Where,  $Q_1$  and  $Q_2$  inflows to node,  $Q_3$  = out flows from the node,

$D$  = external demand withdrawn from the node.

Mass of fluid at section 1 = Mass of fluid at section 2, Principle of conservation of mass which states that inflow at Nodes = out flow at Nodes

### 2.8.6.2 Conservation of energy or energy principle

The principle of conservation of energy states energy can neither created nor destroyed. Thus, the energy difference between two points is the same regardless of the path taken. The energy in pipe flow typically described in terms of head. The energy at any point in a distribution system is the sum of three components, pressure head, velocity head, and elevation head.

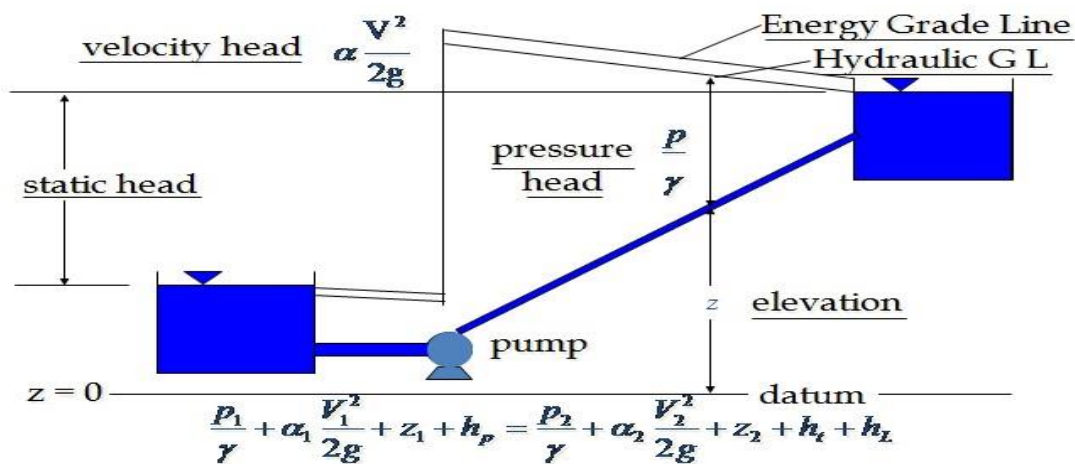


Figure 2. 4 Conservation of energy (Sharma, 2008)

$$\frac{p_1}{\gamma} + \frac{v_1^2}{2g} + Z_1 + h_p = \frac{p_2}{\gamma} + \frac{v_2^2}{2g} + Z_2 + h_L \quad 2.5$$

Where:

$\frac{P_1}{\gamma}$  and  $\frac{P_2}{\gamma}$  = pressure head;  $\frac{v_1^2}{2g}$  and  $\frac{v_2^2}{2g}$  = velocity head;  $Z_1$  and  $Z_2$  = elevation head at two section ;  $h_p$  = energy gains due to pumps;  $h_L$  = major loss and minor loss

**Hydraulic Grade Line/HGL/:** is the sum of elevation and pressure heads. In open water sources, the HGL is the water surface, but for piped pressurized flow condition, the HGL is the height to which water will rise in a piezometer or stand pipe if tapping is made.

**Hydraulic Gradient:** is the slope of the HGL and fluid flows normally occur from high pressure points to low pressure points in the direction of the hydraulic gradient. Any pipe lying above the HGL will cause negative pressure and this adverse pressure gradient results pushing the fluid back, against the direction of flow.

### 2.8.7 Head losses

There are different factors that cause the energy losses. The main reason of the energy loss is due to internal friction between fluid particles traveling at different velocities (Zyoud, 2003). Head loss calculated with famous Hazen-Williams equation:

$$h_f = \frac{10.7LQ^{1.852}}{C^{1.852}D^{4.87}} \quad 2.6$$

Where:  $h_f$  = head loss;  $L$  = pipe length (m);  $D$  = pipe diameter (mm);  $Q$  = flow rate in the pipe ( $m^3/h_r$ ) and  $C$  = Hazen-William roughness coefficient (Dawe, 2000b).

#### 2.8.7.1 Head loss equations and area of application

Hazen – William equation is most frequently used equation in the design and analysis of water distribution networks, it was developed by the experiment and used only for water within temperatures normally experienced in potable water systems (Zyoud, 2003). Manning's equation Commonly used for open channel flow, Chezy's (Cutter's) Widely used in sanitary and sewer design and analysis, Hazen-Williams Commonly used in the design and analysis of pressurized pipe systems and Darcy-Weisbach Can be used for pressurized pipe systems and open channel flows.

Table 2. 2 Head loss equations and area of application (Dighade, et al., 2014).

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

### 2.8.7.2 Friction losses

Hazen-Williams equation and the Darcy-Weisbach equation are the most commonly methods used for determining head losses in pressure piping systems. The assumptions for a pressure pipe system can describe as the following: Pressure piping is usually circular, so the area of flow, wetted perimeter, and the hydraulic radius can directly related to diameter. Through a given length of a pipe in a pressure piping system, flow is full, so the friction slope is constant for a certain flow rate.

## 2.8 Hydraulic design parameters

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 (EPA, 2005b).

### 2.8.1 Pressure

The pressure at nodes depends on the adopted minimum and maximum pressures within the network, topographic circumstances, and the size of the network (Zyoud, 2003) The minimum pressure should be maintained to avoid water column separation and to ensure that consumers' demands are provided at all times. The maximum pressure constraints result from service performance requirements such fire needs or the pressure bearing capacity of the pipes, also limit the leakage in the distribution system, especially that there is a direct relationship between the high pressure and the increasing of leakage value in the system. Pressure in the distribution network as a rule, a minimum of 15 m manometric head is considered adequate during Peak Hour Demands. However, in exceptional and rural water supply cases, depending

on the topography of the area, lower pressure levels may be permitted, but not less than 5 m. A maximum of 100 m manometric head, to avoid risking leaks and bursts in the distribution system, particularly during minimum flow conditions and when the static pressure would be dominant. If necessary, the distribution system is divided into separate pressure zones so that the maximum possible pressure does not exceed 100m. Pipe pressure classes are chosen for the maximum pressure head that may occur under no or minimum consumption condition which is set at 10 percent of the average day demand and the service reservoir at maximum water level. Pipelines in the distribution system shall withstand a maximum operating pressure of 100 m manometric head and hence UPVC and/or PN10 class pipes can be used. In exceptional cases PN16 UPVC class pipes are used. These high pressure class pipes will only be applied where the water supply to certain areas require crossing of specific low lying valleys, etc. Any connection to these pipes will require pressure-reducing valves. Other pipe materials: GS pipes Class B and DCI pipes, shall be used for transmission and distribution mains OWWDSE (2010).

### 2.8.2 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:

$$V = \frac{4Q}{\pi D^2} \quad 2.7$$

$$D = \sqrt{\frac{4Q}{\pi V}} \quad 2.8$$

Where D = diameter of pipe, Q = discharge (m<sup>3</sup>/s) and V = velocity (m/s). From the above equation it is clear that the velocity increasing should decrease the diameter value. When the velocity increases the head loss increases. Ethiopian guideline criteria for the minimum and maximum operating pressure value in the distribution network were 10 m to 70 m respectively (MoWR, 2006). Maximum pressure limitation is required to reduce the additional cost of the

pipe, strengthening necessary due to the high pressure. The guideline further states that water velocity shall be maintained at 0.6 to 2 m/s. Maximum and minimum velocity limitation was necessary because: Velocity is not being lower than 0.6m/s to prevent sedimentation and Velocity is not being more than 2m/s to prevent high head loss

According to Oromia Water Works and Design Supervision Enterprise (2010), design guideline for water supply projects experience shows that in many cases pipes designed to flow velocities of, say, 0.8 to 1.2 m/sec are quite at optimum conditions for long lines; however, the following flow velocities will be acceptable: Minimum 0.5 m/s and Maximum: 2 to 2.5 m/s. but, short sections, particularly at special cases, e.g. at inlet and outlet of pumps, may be designed for different values.

## **2.9 Water distribution network simulation**

Simulation refers to the process of imitating the behavior of one system through the functions another. In our case, the term simulation refers to the process of using a mathematical representation or real system, called a model (Bentley, 2008). Simulation can be used to predict system responses to under a wide range of conditions without disrupting the actual system, and solutions can be evaluated before time, money, and materials are invested in a real-world project. There are two most basic types of simulations that a model may perform, depending on what the modeler is trying to observe or predict in water distribution networks the most basic type of model simulations are either steady-state or extended-period simulation (Tomas, *et al.*, 2003).

### **2.9.1 Steady-state simulations**

Represent a particular view of point in time and are used to determine the operating behavior of a system under static conditions. It computes the hydraulic parameters such as flows, pressures, pump operating characteristics, and others by assuming that demands and boundary conditions were not change with respect to time. In general, this type of analysis is used to determining the short-term effect of demand conditions on the system (Tomas, *et al.*, 2003).

### **2.9.2 Extended- period simulations**

Determine the dynamic behavior of a system over a period of time, and it analyze the system on assumption that the hydraulic demands and boundary conditions were change with respect to time. Hence, extended period analysis used to evaluate system performance over time and allows the user to model pressures and flow rates changing, tanks filling and draining, and regulating valves opening and closing throughout the system in response to varying demand

conditions and automatic control strategies formulated by the modeler. Therefore, regardless of project size, model-based simulation can provide valuable information to assist an engineer in making well-informed decisions (Tomas, et al., 2003).

## **2.10 Water Loss**

Water losses occur in all water distribution networks, even new one and it is only the volume that varies. Thereby, the volume of these losses reflects the capacity of water authorities to manage their distribution networks (Dighade, *et al.*, 2014). In general, „water losses consist of real and apparent losses. And to most water utilities, the level of Non-Revenue Water (NRW) is a key performance indicator of efficiency. Utility managers should use the water balance to calculate each component and determine where water losses are occurring. By quantifying NRW from the water balance concept, volumes of lost water into system can be calculate and they will then prioritize and implement the required policy changes and operational practices which lead to the proper understood and take the required actions“ (Farley, *et al.*, 2008). Therefore, the water balance can guide water loss estimation in the distribution system while also indicating the level of accuracy of the Non- Revenue Water calculation.

Table 2. 3 Water balance showing NRW components; IWA water loss task force (Source, Farley and Trow, 2003)

System Input Volume for known	Authorized consumption	Billed Authorized Consumption	Billed Metered Consumption (including water exported)	Revenue Water
			Billed Unmetered Consumption	
		Unbilled Authorized Consumption	Unbilled Metered Consumption	Non- Revenue Water (NRW)
			Unbilled Unmetered Consumption	
	Water losses	Apparent Losses	Unauthorized Consumption	
			Customer Metering Inaccuracies	
		Real Losses	Leakage on Transmission and/or Distribution Mains	
			Leakage and Overflows at Utility's Storage Tanks	
		Leakage on Service Connections up to point of Customer metering		

Water utilities around the world have always established water balances but unfortunately a wide diversity of formats and definition is used, often with in the same country so it was (and still is) virtually impossible to compare UFW, NRW leakage or water losses of different utility. Being aware of the problem of different water balance formats and methods, IWA established standard water balance as shown in table 2.3 above. According to IWA the above abbreviated terminologies are defined as below: System input volume is the annual volume input to that part of the water supply system, Authorized consumption is the annual volume of metered and/or non-metered water taken by registered customers, the water supplier and other who are explicitly or implicitly authorized to do so. It includes water exported, and leaks and overflows after the point of customer metering, Non-revenue water (NRW) is the difference between system input volumes and billed authorized consumption, Water losses are the difference between systems in put volume and authorize consumption, and consist of apparent losses and real losses, Apparent losses consist of unauthorized consumption and all types of metering in accuracies, Real losses are the annual volumes lost through all types of leaks,

bursts and over flows on mains service reservoirs and service connection up to the point customer metering.

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

### **2.10.1 Some Definitions of UFW**

There is no universally applied or accepted definition of unaccounted-for water. In general, unaccounted-for water (UFW) is the difference between the water supplied to a distribution system and the water that leaves the system through its intended use (Richard G. et al., 2000). UFW may be defined as percentage of the water produced from the raw water source which is not accounted for (Dighade *et al.*, 2014). UFW is defined as the difference between water delivered to the distribution system and water sold. The term Unaccounted-for Water (UFW) refers to an accumulated range of losses that will be experienced by a Water Utility when comparing the system demand of a hydraulic water network with the quantity of water that is acknowledged as consumed by the water consumers residing within the network (UNEP, 2000). If the term UFW is used at all, it should be defined and calculated in the same way as non-revenue water (NRW) (Farley and Trow, 2003).

### **2.10.2 Non- Revenue Water (NRW)**

Non-revenue water (NRW) is the total amount of water losses in the system from the water treatment plant outlet meter to the customer's meter and it consists of real loss and apparent losses. Thus, it is described as the difference of total amount of water production and authorized consumption figure.

NRW = System Input Volume - Billed Authorized Consumption. Unaccounted-for-water also expressed as a percentage and, has generally evaluated as the amount of water produced minus the metered customer use divided by the amount of water produced and multiplied by 100 (EPA, 2010).

$$\text{Unaccounted for Water} = \frac{(\text{Total water produced} - \text{Total water billed})(\text{m}^3) \times 100}{\text{Total water produced} (\text{m}^3)} \quad 2.9$$

### **2.10.3 Causes of water losses**

Leakage is usually the major component of water loss in developed countries, but this is not always the case in developing or partially developed countries, where illegal connections, meter error, or an accounting error are often more significant (Farley and Trow, 2003). The other components of total water loss are non-physical losses, e.g. meter under registration, illegal connections and illegal and unknown use (WHO, 2008).

### **Leakage from transmission and distribution mains**

Leakages occurring from transmission and distribution mains are usually large in volume. Thus, considerable volume of water is lost through bursts, leaking pipes, joints, valves and fittings of distribution system components. These causes are usually as result of age of the installations, bad quality of materials used, and poor workmanship. Although this factors were lead to reduction of pressure in the distribution network and intermittent in water supply (Dighade, *et al.*, 2014).

### **Leakages from reservoirs and storage tanks**

Leakage and overflows from reservoirs and storage tanks are easily quantified. By observing overflows, utility experts can estimate the duration and flow rate of the events. While, most overflows occur at night when demands are low, therefore it is essential to undertake regularly night observations. Observations can be undertaken either physically or by installing a data logger which record reservoir levels automatically at preset intervals. Also, leakage from tanks is calculated using a drop test were the utility closes all inflow and outflow valves, measures the rate of water level drop, and then calculates the volume of water lost (Farley, *et al.*, 2008).

### **Leakage on service connections up to the customer's meter**

This leakage is more difficult to identify and it covers the greatest volume of physical losses. So that, utility experts can calculate the approximate volume of leakage in service connections by deducting the mains leakage and storage tank leakage from the total volume of physical losses (Farley, *et al.*, 2008).

### **Commercial Loss and Real loss**

Commercial loss is also referring to as apparent losses, and it consist of unauthorized consumption, all types of metering inaccuracies and data handling errors. It also includes water that is consumed but not paid by the users (Farley, *et al.*, 2008). In the developing countries, metering inaccuracies (mainly under recorded problem) and illegal users of water within the distribution system is the common problem of water losses. Whereby, they contribute large coverage to apparent losses, so the levels of these losses were one of the significant concerns in developing country water distribution systems (Dighade, *et al.*, 2014). Therefore, apparent losses can amount to a large volume of water than physical losses and often have a greater value, since reducing apparent losses increases revenue, whereas physical losses reduce production costs. For any profitable utility, the water tariff will be higher than the variable production cost and sometimes up to four times higher. Thus, even a small volume of apparent loss will have a large financial impact" (Farley, *et al.*, 2008).

**Physical losses:** on the other hand, sometimes called „real losses“, are the annual volumes lost through all types of leaks, bursts, and overflows on mains, service reservoirs and service connections up to the point of customer metering. So, utility managers must verify the physical loss assessment of towns water distribution system (Farley, *et al.*, 2008).

#### **2.10.4 Factors causing loss of hydraulic integrity in water distribution network**

In most of the developing regions, the design of water distribution systems is based on the assumption of direct supply, although most of these systems are intermittent systems which result in severe supply, insufficient pressure in the distribution system (pressure losses in several areas in the network), inequitable distribution of the available water and very short duration of supply (Zyoud, 2003). However, the purpose of hydraulic integrity in the water distribution system is to supply water at adequate/acceptable pressure and flow. But, according to (Dighade, *et al.*, 2014) the most common factors for intermittent water supply and loss of hydraulic integrity in the distribution system are;

**Low pressure:** However, there is pressure loss by the action of friction at the pipe wall and its magnitude also dependent on the water demand, properties of the fluid that is passing through the pipe, the speed at which it is moving, and the internal roughness of the pipe, pipe length, gradient and diameter of the pipe. Such situations may occur where there are: properties on high ground, remote properties at the end of long lengths of pipe, demands that are greater than the design demand, pipes of inadequate capacity (too small diameter), rough pipes (e.g. corroding iron pipes or pipes with a build-up of sediment) and equipment failures such as pumps and valves.

In general, poor pressures tend to be caused by inadequate capacity in a pipe or pump, high elevations, or some combination of the two (Khatrik, 2007). Therefore, one of the most hydraulic integrity is maintaining adequate water pressure inside the pipe. Hence, the water utilities should achieve a high degree of hydraulic integrity through a combination of proper system design, operation, and maintenance along with good monitoring.

**High pressure during low demand conditions:** High pressure during low demand conditions can cause pipe bursting, leakage and large amount of water losses through the distribution networks. Therefore, when dealing with high pressures, PRVs should be used to reduce and regulate pressure in the system (Tomas, *et al.*, 2003). Accordingly, pipes and pumps must be sized to overcome these problem and to provide acceptable pressure in the system. Although, sizing of control valves based on the desired flow conditions and pressure differential is vital (NRC, 2006).

**Pump Capacity:** A pump is device in which mechanical energy is applied and transferred to the water as total head, and these head is a function of flow rate through the pump (Tomas, *et al.*, 2003). While, the failures, location, size and capacity of pumps in water distribution are the major impacts for low flow or negative pressures arise in the system, and this can lead to intermittent water supply in the distribution system (Khatrik, 2007).

There are many reasons and factors why a pump is not performing well in a certain situation of water distribution system. But, as per (Misirdali, 2003); the important and possible reasons to less performing of pumps were identified as below: When the pump is of poor design and quality, if it is not suitable for the given situation and does not work in its optimal range, If the pump is not being used properly and maintained regularly (cleaning, greasing, etc.), If the pump is excessively exposed to sun, rain, dust, If it is overused and was not repaired properly after a break-down and If supply of spare parts is difficult.

**Demand Increase:** Rising water demand as a result of population growth and urbanization has an effect on the availability and reliability of existing water distribution system. Therefore, water demands need to be assessed on the basis of considering the year and date supplying water through the distribution system. The primary objective is to make sure that the community is being serviced adequately. If there are deficiencies in meeting current or future goals because of population growth, this needs to be identified for the areas of the community where there may be inadequate flows to meet customers' consumption during peak hour water demand of the day (Tabeshe and Dolakhahi, 2006).

### **2.11 Urban water demand**

Water demand is the volume of water requested by users to satisfy their needs. In a simplified way, it is often considered equal to water consumption, although the two terms conceptually do not have the same meaning (Wallingford, 2003). In most developing countries, the theoretical water demand considerably exceeds the actual consumptive water use (Berhe, 2005). (Maher and Trifunovic, 2013) noted that, water demand is the algebraic sum of the quantity of water utilized by consumer (consumption) and the amount of water physically lost from the system (leakage). It usually expressed as per capita demand. Per capita water usage varies widely due to the differences in climatic conditions, standard of living, population growth, type of commercial and industrial activity and water pricing. Water demand increases with time due mainly to population growth. Therefore, new water resources ought to be developed in order to meet the increasing water demand at present and in future (Kimey, 2008).

### **2.11.1 Types of urban water demand**

It is usual to classify water demand in various sorts depending on the characteristics of the consumers'. The most common types are domestic, commercial, industrial, firefighting and unaccounted water demand.

#### **2.11.1.1 Domestic water demand**

Domestic demand includes the water required in private building for drinking, cooking, bathing, flushing and washing clothes (WHO, 2008). Garg (2010) indicated that the domestic consumption varies according to the living conditions of the consumers, economic status of the community, climatic condition, mode of service and affordability and accessibility of the service. Daily per capita water consumption in Ethiopia is generally very low throughout the country. DWD is suppressed in almost all towns in the country because of supply shortages. Actual demand is expected to be greater than present consumption if greater supplies were available to the community (MOWE, 2012).

#### **2.11.1.2 Non-domestic demand**

Non-domestic demand comprises Industrial, Commercial, and Institutional, Firefighting demands, Unaccounted Water Demand (UWD).

**Industrial water demand:** represents the amounts of water demand required by industries and factories in the cities. According to Garg (2010), the ordinary per capita consumption of industries is 50l/c/d. but due to the modernization of technology in reusing waste water the amount of water requires for industry getting reduced.

**Commercial and institutional water demand (CIWD):** In addition to those of household consumers, the water requirements of towns include the needs of such commercial and institutional consumers as public schools, clinics, hospitals, offices, shops, bars, restaurants, and hotels. CIWD is usually linked directly to population size. For medium town, the CIWD estimate is taken as 10 per cent of DWD (MOWE, 2012).

**Unaccounted water demand:** is the amount of water physically lost from the system and theft (Motiee *et al.*, 2007). Losses from water supply systems vary considerably according to diverse factors. According to MoWR (2011), water losses is a function of the quality of construction, the type and age of the pipes in the distribution network, and pressure within the system. Losses can also originate in treatment plants. Loss for urban scheme is taken as 25 per cent of the total domestic, commercial and institutional, and industrial water demand. It can be obtained as the difference between the supplied volume and the metered volume.

### **3. MATERIALS AND METHODS**

#### **3.1. Descriptions of the study area**

##### **3.1.1 Location**

The study conducted in Halaba Kulito Town, which is the administrative capital of the Halaba Zone of the Southern Nations, Nationalities and Peoples Regional State (SNNPR) of Ethiopia, it is located north of the main road between Shashamene and Wolayita Sodo, on a gently sloping bank of the Bilate River, The location of the town is 7° 17' 15" to 7° 19' 21" N latitude and 38° 04' 21" to 38° 06' 27" E longitude, elevation ranging from 1750 to 1900 m.a.s.l showing a difference of 150 m within the boundary and is accessible by asphalt road from Addis Ababa 310 km and from the regional capital Hawassa 85 km and covering a total area of the town is 48.6 sq km.

##### **3.1.2 Climate of the Study area**

According to the traditional temperature zone classification of Ethiopia, (which is based on altitude) the town is found within the 'Woina-Dega' Agro-ecological zone. Consequently, it experiences mean annual temperature between 17 °C and 27 °C with mean value of 22 °C (NUPI, 1999, and AG. *et al.*, 2010). The highest temperature is experienced between January and March and the lowest temperature record is between July and September, the area receives a bimodal rainfall where the small rains are between March and April while the main rains are from July to September (IPMS, July 15, 2005).

The air is usually humid as a result of abundant vegetation cover and enough rainfall. Halaba Kulito gets rainfall almost throughout the year. The annual rainfall is The annual rainfall varies from 857 to 1085 mm and reaches up to 1500 mm, (SNNPRS, 2011). The highest rainfall is recorded between July and September and the lowest rainfall occurs between November and February which are relatively dry months of the year.

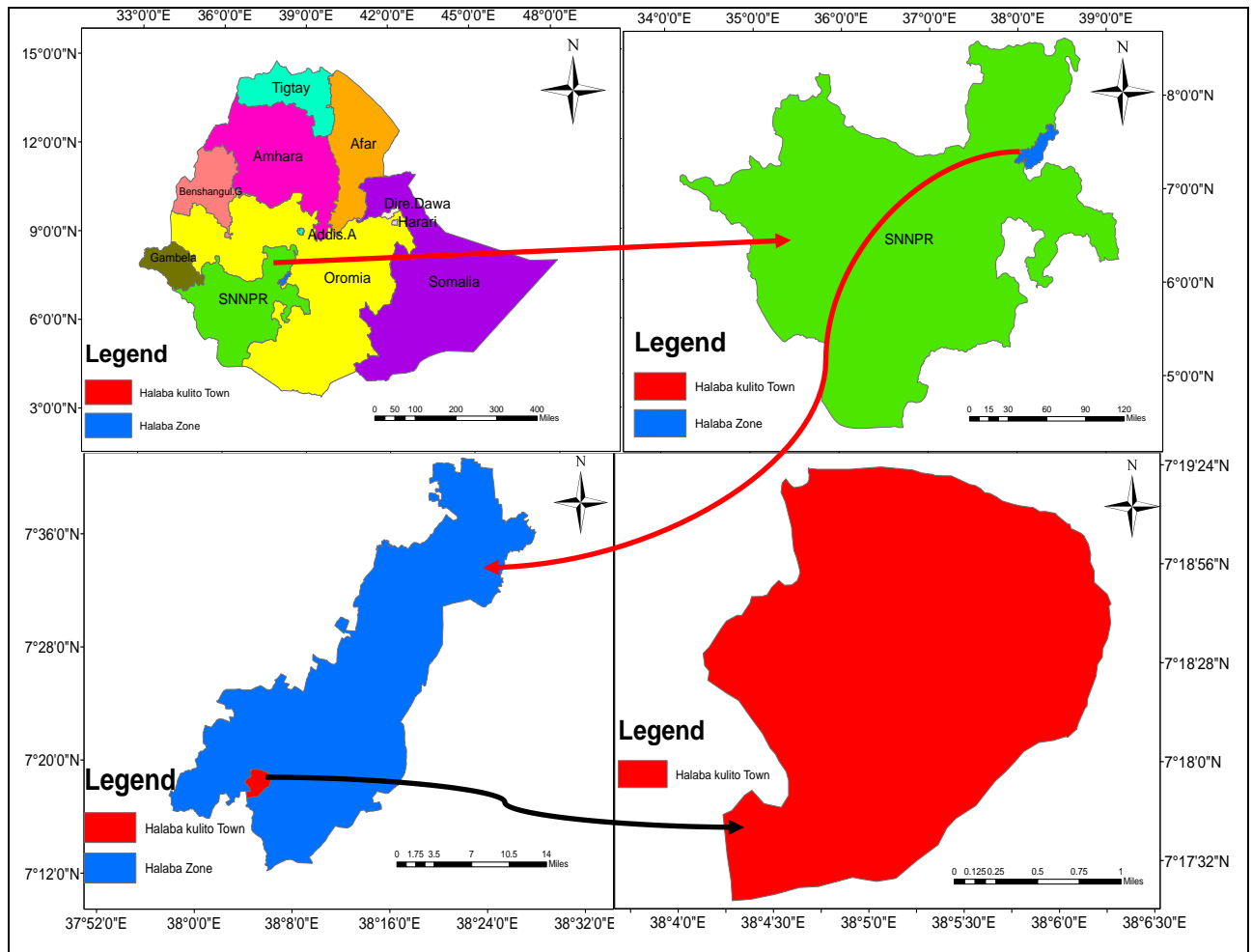


Figure 3. 1 Map of study area

### 3.1.3 Demographic characteristics

According to the last National Population and Housing Census carried out in 2007, Halaba Kulito Town has the total number of population was 26,867 out of which 13,959 were males and the remaining 12,908 were females. Although, the population of the town become increasing from time to time in relation with the town development in investment, trade and due to expansion of town and town expansion was rural to urban migration as result of remittance send from South Africa in the town. The current population of the town is projected and approving to report of Administration of Town Finance and Economic Development, (2019), Halaba Kulito Town has a total number of population is 82,099 of which 41,172 is males and 40,927 is females.

### 3.2 Materials

This research was conducted on Urban water supply network analysis and evaluation in distribution system. To complete the goal of the research the materials that were used Bentley

WaterGEMSv8i, Arc GIS version 10.1, AutoCAD 2017, GPS Garmin72, EndNote program and other relevant materials.

**Bentley WaterGEMSv8i:** was used to model the behavior of water distribution systems. Bentley WaterGEMSv8i has a capability of modeling water distribution behavior at steady-state and time varying situation. To design and evaluate hydraulic performance of the existing water distribution system, a model was developed using Bentley WaterGEMSv8i. Bentley WaterGEMSv8i is selected for this study because of the following reason: Used to analyze potable water networks, fire protection, allows calibration of large distribution networks, performing cost analysis and operational studies and many more functions. It also has AutoCAD interface, GIS and Microsoft excel integration features, requires less effort and shorter time to build a model than others and It is aided with good quality of manual. Creating the network within AutoCAD by adding the figure directly to the drawing is possible, Graphical user interferences and latest as camper to Epanet and WaterCAD 6.5 software (Swammee and Sharma).

WaterGEMSv8i is a powerful, easy-to-use program that helps hydraulic engineers design and analyzes water distribution systems. It provides spontaneous access to the tools you need to model complex hydraulic parameters performance. 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 parameters performances analysis of the distribution networks in the Halaba Kulito Town. The data sorting and color coding capability of WaterGEMSv8i software was used to identify very small and very large values and to focus on these values when testing and calibrating the hydraulic model. The other advantages of WaterGEMSv8i supported several methods of exchanging data with external applications, preventing duplication of effort and allowing us to save time by re using data already present in other locations. For this study, Heastad standalone WaterCAD software played the major role to import the AutoCAD drawing to waterGEMS software in the form of .mdb sub model file extension.

**ArcGIS version 10.1:** was used to delineate the study area and used to display the overlapped shape file of the distribution network on the topographic map of the town. While, Microsoft Excel sheet were used to organize elevation data, to calculate a repeated work of nodal base water demand requirement of distribution network simulation and for manual pressure validation work.

**GPS Garmin 72:** was used to take with accuracy  $\pm 4\text{m}$  coordinates and elevation data for cross-checking with the coordinates of the boundary nodes obtained from the AutoCAD source and elevations generated by TRex and Pressure readings were done using pressure gauge which is commonly taken in the selected points of distribution system. It was also used to take coordinates of some components of the distribution system which have no spatial data on the AutoCAD network. A combination of waterCAD 6.5, AutoCAD, waterGEMS and Arc GIS software's were used for the model development. Finally, Endnote program used to place citation and references in the document.

### **3.3 Study parameters**

The hydraulic parameters were nodal pressure and velocity. The type of supply (continuous or intermittent), water pressure, total non-revenue water, per capita supply and extent of metering could be considered as performance indicators for water supply systems. Overall, system performance is function of many parameters some of which are independent (such as physical and chemical characteristics of the water), and some which are closely linked such as pressure levels and flow rates.

### **3.4 Methodology**

In this study the survey was conducted in the study town in five kebele's by using structural interviews, the statistical software SPSS Version 24 was used for data analysis. The analysis work of hydraulic performance of urban water supply distribution network in Halaba Kulito Town, both secondary and primary data used. Based on the research objectives and questions how the research was carried out were discussed here.

### 3.4.1 Framework of the Study

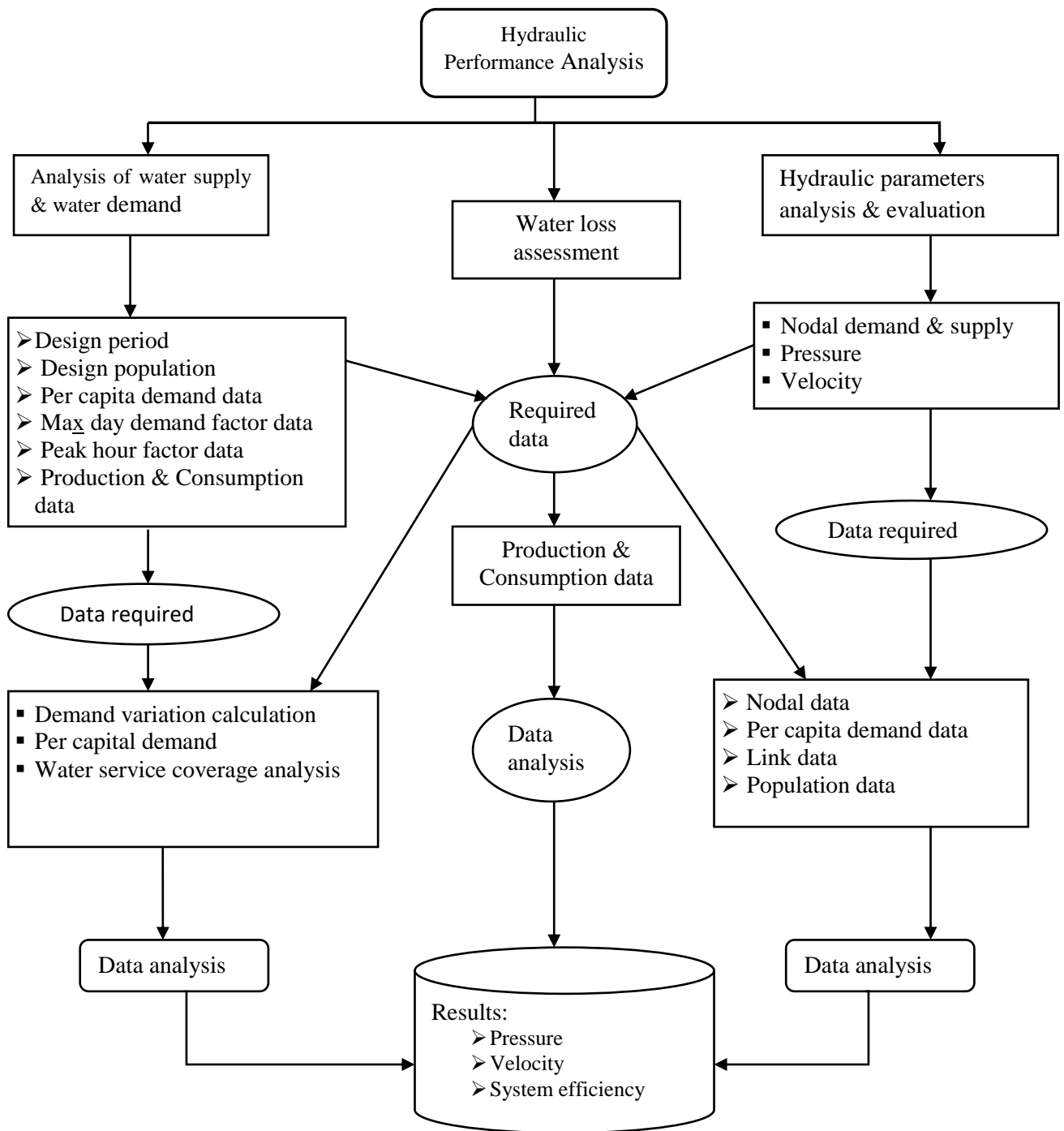


Figure 3. 2 Framework of the Study

#### 3.4.1.2 Sample Size and Sampling Techniques

To get reliable data providing the necessary information required to answer the research questions of the study and for the achievement of the intended objectives of the study, both probability and purposive sampling techniques were employed in the study, Purposive sampling was used to gather general information with the issues understudy to provide the

general information required to realize the objectives of the study. And probability sampling technique was used to select sample households/respondents of the survey from the target population.

To this end to get the representative population and the necessary information accordingly, this research used systematic sampling techniques to select household respondents, officials and stakeholders. From the total residents in the town five kebele's such as Mahal arada, Wanja ber, Danabe Fama, Murasa ber and Lenda ber, respondents were selected using systematic sampling techniques to select the respondent from each five kebele's taken based on proportion described as equation 3.1. Accordingly; 390 participants were selected.

$$n_r = \frac{4pq}{d^2} \quad 3.1$$

Where:

$n_r$  is required sample size;  $p$  is Proportion of water supply coverage of the existing water supply coverage of the town reported to 58 % in 2019 (HKTWSSO, 2019);  $q = (1-p)$  and  $d$  is Proportion of sampling error tolerated which is 5%.

Table 3. 1 Total households and sampled respondents/households

No	Name of kebele	Total households (N)	Sampled households (n)
1	Mahal arada	4459	98
2	Wanja ber	4028	92
3	Danabe Fama	3674	80
4	Murasa ber	2135	62
5	Lenda ber	2120	58
Total		16416	390

Key: N- represents census size; and n- Represents sample size.

### 3.4.2 Data Sources and Methods of Data Collection

There were two major categories done for data gathering techniques in building the model of the distribution network, the data were first gathered regarding all the distribution system parameters. The collection process was performed using both primary and secondary data collection techniques, secondary data from legal agencies and organizations and primary data from field survey of the town.

### 3.4.2.1 Sources of Primary Data Collection

**Questionnaire and Household Survey:** In this study, to generate first-hand and additional information from sampled households/respondents, the questionnaire with both open ended and close ended questions was distributed. The questionnaire was prepared in English language and to avoid language barriers it was translated to Amharic language.

Then, the questionnaire was distributed to randomly sampled households/respondents to gather the relevant information required for the study to achieve its general and specific objectives, data concerning all relevant variables such as problems of urban water supply, parameters performance misuses house hold connections and physical condition of water supply points under study, other problems with technical and institutional issues (Appendix - 4).

**Interview:** The interview conducted with purposively selected key informant interview was conducted with the town's residents from different offices, Kebele leaders, persons have been different responsibilities, knowledge and experience about the town's accessibility/availability of water with required amount, water coverage, the balance between demand and supply of water in the town, major challenges facing in the water consumption the service. These key informants were purposively selected from different offices assuming that they have deep and relevant information from their official responsibilities and continue involvement about the issues.

**Field Observation:** It was mainly employed to gather data related to the presence of pipe lines in selected households, to check the presence of water at any time, the areal coverage of water pipe lines and the factors behind some varieties on miss connection of pipes, location and altitude. It was carried out through the help of checklists according to the objectives of the study to get sufficient pressure in the distribution network. Primary data were collected the background information about the status of urban water supply and distribution system through field observation, researcher has conducted with the selected individuals, who were believed to have good information about the area and that of the subject matter, kebele's administration officials as well as with local administrative of HKTWSSSE and Zonal Water, Mineral and Energy Office professionals and Geographical coordinates system (GPS) Garmin 72 has been used in during field visit taking the location of the selected main node, water sources, service reservoirs in the distribution network with HKTWSSSE experts, assumed for future expansion and tanks elevation points.

### 3.4.2.2 Secondary Data Collection

The secondary data which was collected from the recognized or legal organizations, agencies and literatures for the development of network & analysis were the electronic copy (softcopy) from Halaba Kulito Town water supply and sewerage service enterprise, Municipality, Zonal water, Mineral and energy office, South water, irrigation and energy bureau the following archived/recorded data has been collected.

- ✚ The existed layout of the distribution network of the town,
- ✚ Halaba Kulito Town master plan including settlement map of the town and road route,
- ✚ Water production and consumption,
- ✚ Pump capacity,
- ✚ Tank data (size & its section),
- ✚ Pipe data like material type, size or diameter and length,
- ✚ Base demand of the nodes,
- ✚ CSA data (CSA, 2010), and other necessary data from Journals, internet and books.

From list of population and housing census report (CSA, 2010), the growth rates and population for Halaba Kulito Town has been collected in order to forecast the future population.

### 3.4.2.3 Data Quality Control

Checking the validity and reliability of data collecting instruments before providing to the actual study issue is the central part to assure the quality of data. To ensure internal and external validities of the study, the researcher to analyze and evaluate the collected data without any bias and vagueness and determined proper sample size from target population. To make the data more reliable, the researcher avoid ambiguity in the measurement scale by selecting fitting, consistent, dependable, and clear statements to the respondents by preparing three to four alternatives to each closed-ended question. The researcher stays away from the subjectivity of the omitted questions. In addition to this, to increase the quality of the data, prepared a fieldwork manual to check every day progress the data handling good. Finally, has checked the reliability and the accuracy of the data, the questionnaires checked by my advisors.

### 3.5 Data Analysis

#### 3.5.1 Existing Water Supply System in Halaba Kulito Town

##### 3.5.1.1 Water Sources and Water Production with their Short Description

The water supply source of the entire town is groundwater sources from different areas of groundwater/deep bore-holes which is located in the distance of the town. The water production has been evaluated as a total annual water supplied to the water distribution system (WDS). In the study year the production of water depends on four-supply sources, which are administrated by Halaba kulito Town water supply and sewerage service enterprise. The designed gross water production capacity of these boreholes is 38.5 liters per second (l/s) or (2,635.2 m<sup>3</sup>/day) average working for 19 hours per day.

Table 3. 2 Halaba Kulito Town water sources/reservoirs and their short description (HKTWSSSE, 2019).

Item No	Name of the Sources/Local names	Easting (X)	Northing (Y)	Elevation (m)	Discharge l/sec	Daily working hour	Daily water production (m <sup>3</sup> /day)
BH2	Kera	398746	807208	1760	4.4	19	300
BH4	Maramiya	398421	808767	1773	8.3	19	568
BH5	Hantame	398425	810281	1771	12.9	19	882
BH6	Debancho	399437	812069	1768	12.9	19	882
	Total				38.5		2,632

As shown in the Table 3.2 the total daily water production of the wells was calculated by multiplying the discharging capacity of wells by pump operating hours (19) hours a day which indicates total daily production was 2,632 m<sup>3</sup>/day.

##### 3.5.1.2 Existing Distribution Network

The existing water distribution system of the town is both pump and gravity system. The water from the four sources is taken to storage tank by pump and the water stored in the storage tank, Then, the stored water is distributed to the town by the gravity (uPVC, PVC, HDPE and DCI). The water distribution network of the town consists of about 49.8 km. The existing distribution system consists of a variety of pipe types: ductile iron (DCI), Galvanized Iron (GI), uPVC (u-polyvinyl chloride), PVC (Polyvinyl Chloride) and HDPE (high-density polyethylene). The town supplied by water intermittently by water staff who is managing the

system mainly by using controlling valves in order to supply all customers at least three times a week.

### 3.5.1.3 Distribution Network Pipes

Pipes are the essential elements of a water distribution system. All the elements of distribution system, such as junction (nodes), pumps, reservoirs, valves and tanks are linked to each other by pipes (Dighade, *et al.*, 2014).

Table 3. 3 Pipe diameter and corresponding length used as software inputs (HKTWSSSE, 2019).

Diameter (mm)	Material type				Total Length (m)
	DCI	PVC	HDPE	UPVC	
50	4,179	6,701	5,491	-	16,371
63	-	-	10,746	-	10,246
75	547	4,444	1,326	-	6,317
100	3,088	369	1,917	1,137	6,511
150	4,119	489	-	1,256	5,894
250	1,856	-	-	2,598	4,454
Total	13,789	12,003	19,480	4,991	49,793

To deals performances of hydraulic parameters for all pipe individual up to individual house hold is difficult and time consumed to represent by software. So, skeletonization was needed. Skeletonization is the process of selecting for inclusion in the model for enabling quicker calculation (Walski, Thomas, 2007). Using skeletonization pipes having diameter greater or equal to 50 mm were selected for modeling the distribution system. As indicated in Table 3.3, the total length of pipes in the water distribution network was 49,793 m.

### 3.5.1.4 Storage Tank

The study area has two ground storage tanks which functions for storing water and equalizing flow to each service area. The municipality uses these storage tanks as a pressure zone boundary based on the topography to manage the distribution.

Table 3. 4 Location and capacity of existing storage tanks (HKTWSSSE, 2019)

Tank label	Elevation (m)	X (m)	Y (m)	Diameter (m)	Capacity (m <sup>3</sup> )
Choroko T-1	1795.14	399544	810680	12	800
Choroko T-2	1795.14	399549	810687	10	200
Aroge biro	1769	399089	807657	4	25
Total					1,025

### 3.5.2 Analysis of Water Supply and Water Demand of Halaba Kulito Town

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 Borehole/reservoir data, Tank data and Pump data. The water production and consumption found from Halaba Kulito town water supply and sewerage service enterprise that recorded in the year 2019. The distribution system was 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 Table 3.8.

Table 3. 5 Demand Factor (MoWR, 2006)

Population Range	Maximum day demand Factor	Peak hour factor
< 20000	1.3	2
20001 to 50000	1.25	1.9
50001 to 100000	1.2	1.8
X > 100000	1	1.6

### 3.5.2.1 Estimation of domestic water consumption/coverage

The coverage of water supply for the town has been evaluated based on the average per capita consumption and by mode of service. Water demand is the daily water requirement for use by human being for different domestic purposes. The annual total volume of water consumed for domestic purpose has been converted to average daily per capita consumption using the total number of population (OWWDSE, 2010). The volume of water consumed for domestic purpose has been aggregated to all kebeles of the town so as to analysis the distribution of the water supply coverage among different localities. The annual consumption data has been converted to average daily per capita consumption using the number of population. The average daily per capita consumption/(liter/person/day) was derived using the following expressions:

$$\text{Domestic per capita consumption (l/p/d)} = \frac{\text{Annual Consumption (m}^3\text{)} \times 1000\text{L/m}^3}{\text{Population number of the town} \times 365} \quad 3.2$$

Data on individual domestic water consumptions, total water consumption (m<sup>3</sup>) and total production (m<sup>3</sup>) were collected from Halaba Kulito water supply and sewerage service enterprise billed documents for analysing average per capita consumption.

### 3.5.2.2 Water Loss (Unaccounted for Water) Analysis

Water losses in the water supply distribution system, illegal connections, overflow from reservoirs and improper metering etc. referred to as unaccounted for water. Unaccounted for water (losses) is expressed as percentage of domestic plus public water demands and is calculated the difference between water produced and water consumed or sold. The total annual water produced and distributed to the distribution system and the water billed that was collected from the individual customer meter readings were used to quantify the total water loss for the entire town. All the water consumptions in the town were metered except reservoir cleaning in the study area. The annual water production and consumption was derived using the following expressions (EPA, 2010).

$$\text{Total water loss (\%)} = \frac{(\text{Total water produced} - \text{Total water billed})(\text{m}^3) \times 100}{\text{Total water produced (m}^3\text{)}} \quad 3.3$$

### 3.5.3 Distribution Network Simulation

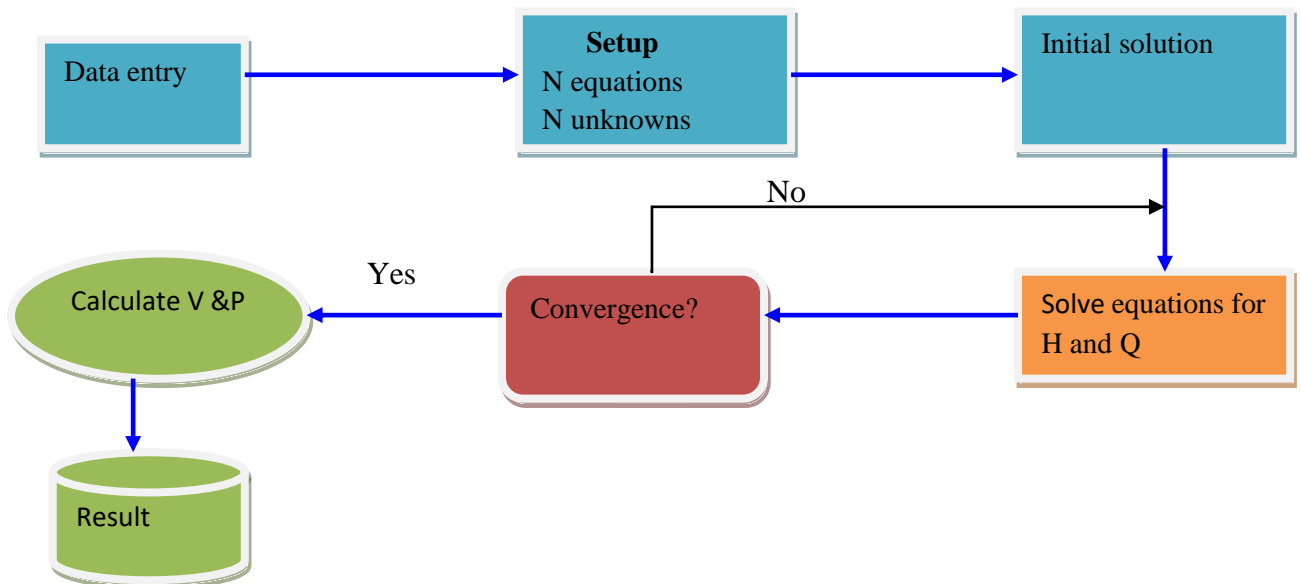
WaterGEMsv8i is capable of performing two types of simulations, steady-state and extended period simulation. Analysis of the model of existing systems has been done by running the

model at current year daily average at peaking and temporal variation of demand with different scenarios.

### 3.5.3.1 Steady-State Simulation

It computes the state of the system (flows, pressures, pump operating attributes, valve position, and so on) assuming that hydraulic demands and boundary conditions do not change with respect to time. A steady-state simulation provides information regarding the equilibrium flows, pressures, and other variables defining the state of the network for a unique set of hydraulic demands and boundary conditions.

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



Source: (Bentley WaterCAD/GEMs ,2008)

Figure 3. 3 Flow chart for steady state simulation

### 3.5.3.2 Extended Period Simulation

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

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

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

### **3.5.4 Hydraulic Parameters**

The main hydraulic parameters in water distribution networks are the pressure, velocity and the flow rate, other relevant design factors are the pipe diameters, and the hydraulic gradients (Zyoud, 2003).

#### **3.5.4.1 Pressure in the Distribution Network**

The pressure at nodes depends on the adopted minimum and maximum pressures within the network, topographic circumstances, and the size of the network. The minimum pressure should have maintained to ensure that consumers' demand provided at all times. The maximum pressure also contains limitation of leakage and lead to water losses in distribution system. The static pressure in the distribution piping system is the pressure head with no water flowing in the network is equal to the height to which the column of liquid could be raise. The general consideration is the water should reach up to the upper stories of low-rise buildings in sufficient quantity and pressure, considering firefighting requirements. In case of high- rise buildings, booster pumps are installed in the water supply system to water for pressure head requirements. The MoWR water supply design criteria (2006) recommended the pressure range in distribution system to be 15 - 60 m water head. However, there is no defined maximum and minimum pressure ranges set by the office, regarding to this literature based recommendation for optimum operating pressure was used to asses system hydraulic performances.

- ✚ The minimum static pressure at peak hour demand 30 m of water column (30 mwc) would be required to serve up to three stories high,
- ✚ Maximum static pressure during low demand periods was limited to 60 m of water column (60 mwc),
- ✚ Minimum dynamic head was established at 15 m meter of water column (15 mwc).

The operating pressure in the distribution network is given in Table 3.6.

Table 3. 6 The allowable operating pressures in the distribution network according to MOWR, (2006)

Pressure	At normal condition (m)	Exceptional conditions (m)
Minimum	15	10
Maximum	60	70

### 3.5.4.2 Flow Velocity in the Distribution Network

It is the quantity of water passes within a certain time through certain section. Velocity is directly proportional to the flow rate. For a known pipe diameter and a known velocity, the flow rate through a section can estimated. Low velocities affect water consumption and severe to diseases problem.

Different design guide line has been developed by different researchers for the standard velocity in pipe flows. They recommended optimum velocities for pipe flow in transfer and distribution mains are presented in Table 3.7.

Table 3. 7 Pipe velocity range from various sources

Distribution type	MoWR (2006)	World Bank (2012)	OWWDSE (2010)
Maximum transfer main velocity	2 m/s	3 m/s	2.5 m/s
Maximum velocity in distribution	2 m/s	1.5 m/s	0.8 – 1.2 m/s
Minimum velocity in distribution	0.6 m/s	0.4 m/s	0.5 m/s

### 3.5.4.3 Model Building and Data Entry

#### 3.5.4.3.1 Modeling the existing water distribution network

Computer models are significant investments for water companies to ensure a good investment 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 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). Bentley WaterGEMS/CAD 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.

**Bentley WaterGEMSV8i:** is a powerful tool for design, analysis and evaluating the existing urban water distribution system. Bentley WaterGEMSV8i 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 WaterGEMAv8i can be analyze complex distribution systems under a variety of conditions for a typically Bentley WaterGEMSV8i 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 WaterGEMSV8i. WaterGEMSV8i views the water distribution system as a network containing nodes and links, where the nodes are connected by links. Data used for Bentley WaterGEMSV8i has x, y coordinate and elevation, junction or node and demand values. Bentley WaterGEMSV8i 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).

A model was developed utilizing WaterGEMS software (WaterCAD 6.5 for AutoCAD 2007 software). WaterGEMS is selected for this study because of it is aided with good quality of manual, integration with other external software's, like AutoCAD, GIS background support and Microsoft excel, requires less effort and shorter time to build a model than others do, rule-based controls and ground elevation extraction from shape files and AutoCAD drawings. The other capabilities of the WaterGEMS software are evaluate the hydraulic performance for different demands at a single node with varying time patterns, solve for different frictional head losses using Hazen-William, Darcy-Weisbach, Chezy's(Cutter's) or Manning's equations, determine fire flow capacities for hydrants, model tanks, including those, which are not circular and model various valve operations (Bhadbhade, 2004). In order to analyze the distribution network system, the available data and plan of the distribution network of water supply system was reviewed. The modeling process are input data collection, model building in Bentley WaterGEMSV8i, data entry (Elevations, XY coordinates, base demand, pump data,

tank data and pipe data), model testing and hydraulic modelling and problem analysis. Input data for the analysis of distribution system included: -

- Nodes: Elevations and base demand
- Pipes: Pipe diameters, lengths, material type and the friction coefficient factors
- Tanks: Base, minimum and maximum elevation and diameter of the tank
- Pumps: The most important parameter defining the pump operation is the pump curve  
Other input needed is the elevation of the pump
- Reservoir: Elevation

After all the parameters require to run, the out puts of the simulation model are: -

- ✚ Flows at every point of time in the system
- ✚ Velocities in the pipes
- ✚ Levels in the tanks
- ✚ Pump curve
- ✚ Water age and quality
- ✚ Pressure head at node
- ✚ Head losses

#### **3.5.4.3.2 Importing the Network**

To import the AutoCAD drawing in to the waterGEMS software, the AutoCAD drawing was first converted to .dxf file extension after correcting all types of errors on the AutoCAD drawing. Then the dxf file network was imported in to Heastade standalone waterCAD software using the shape file link wizard. However, since the Heastade standalone waterCAD software has many limitations as compared to the WaterGEMsv8i software during integrations with AutoCAD, Excel and Microsoft office, re-importing the drawing from the standalone waterCAD in to the waterGEMsv8i software was used for this paper even though there may be different methods of importing network data. The modeling was performed using the following steps:

1. Input data arrangement and checked
2. Initial setup (the unit was set to SI unit)
3. Network schematic (connect un-connected junction and pump by pipe)
4. Data entering model builder and flex table
5. Nodal demand calculation
6. Validate and run process
7. Problems analysis based on result

### 3.5.4.3.3 Model Skeletonization

Skeletonization is the process of selecting parts of the distribution system, which have significant impact on the behavior of the whole distribution system when included in the model for the model preparation. Even though preparing an all-pipes model requires less skilled labor to collect the data, it was not financially feasible and since it is time consuming, a moderately skeletonized network model was prepared by taking pipes with nominal diameter above 50mm, as exactly situated on the AutoCAD drawing. But the demand nodes were created using Darwin Skelebrator wizard of waterGEMS software by merging series pipes with the same diameter in to an equivalent pipe. The nodes were created at diameter changes, at end of pipes and at intersection of more than two pipes. The level of skeletonization depends on the purpose of the study. This study is intended to be used for general purpose, especially for improving the operation and management of Halaba Kulito Town water supply distribution network. So the skeletonization technique will not affect the purpose of the study as long as components with significant impacts on the whole system are included in the model. It is possible to identify segments with significant impact on the performance of the network by undertaking criticality analysis using waterGEMS software, but with great professional decision. So as a fresh researcher, to take care of those errors due to wrong professional decision, the whole AutoCAD network pipes were imported as they are on the AutoCAD drawing with slight addition for missing pipes and deletion of some miss-placed pipes. But by obeying the skeletonization rules, significant changes were made on the AutoCAD nodes to create the model nodes for modelling the distribution network. The level of skeletonization is also still within the moderate range.

To perform the model building process, data for modeling the system which were missed during the data collection, especially the elevations of demand nodes were generated using TRex wizard of waterGEMS software. Then grouping of pipes with similar characteristics (diameter), merging of series pipes and replacing them with equivalent pipes and other modifications on the network were made. After building the distribution layout using the combination of standalone waterCAD, AutoCAD and waterGEMS software's, all the collected data and generated input data were entered in to the waterGEMS model and running the model was performed to test the reasonableness of the simulated parameters and to check topological errors with in the network. Figure 3.4 shows below the schematic distribution of the network on the Halaba Kulito Town map. The skeletonization of water distribution layout based on master plan of the town, and then placed as a map on Bentley waterGEMS platform as shown on.

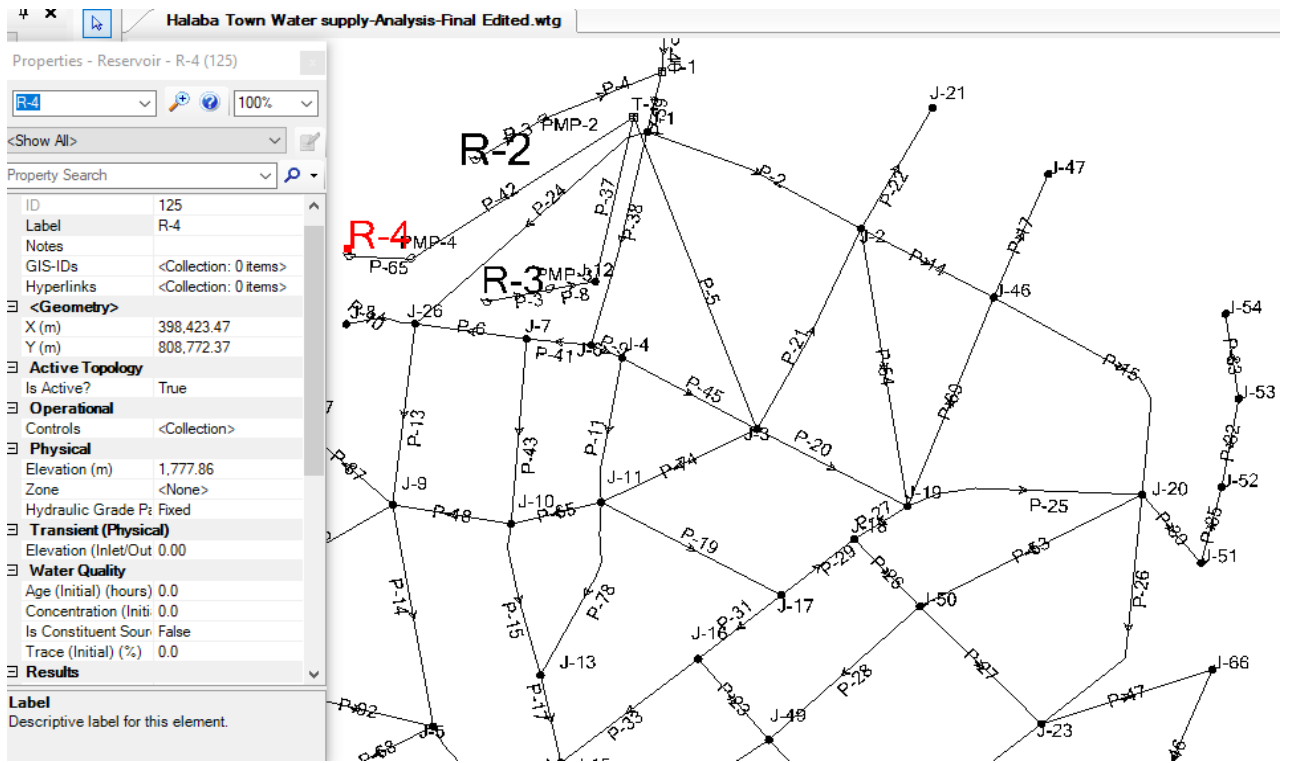


Figure 3. 4 Schematic distribution of water networks of Halaba Kulito Town

#### 3.5.4.3.4 Data Entering and Data Proofing

The distinguishing and correcting data errors related to network data, demand data and operational data, which occur during data gathering process, data preparation and data analysis processes were undertaken. The input data should be entered into the software using different techniques these were model builder from dxf. File to software, use the properties editor for each element by individually opening the properties editor or used flex table for similar element data used by model builder so that the total input data for the analysis of distribution system included:- Nodes (Elevations and base demand), Pipes (Pipe diameters, lengths, material type), Tanks(Base, minimum, initial and maximum elevation and diameter of the tank), Pumps (The most important parameter defining the pump operation is the pump curve, Other input needed is the elevation of the pump), Reservoir (Elevation) and Hazen –Williams pipe coefficient values, and other necessary values used by flex table. Throughout the process, International System Unit (SI) has been used to request the use of these units in WaterGEMsv8i, the user chooses SI flow unit under the hydraulics option. In this study, it was selected liters per second for flow in this model, which also defines all other units using the SI system. Hence, lengths, pressure, head, elevations are taken in meters, and diameters of pipes are defined as millimetres.

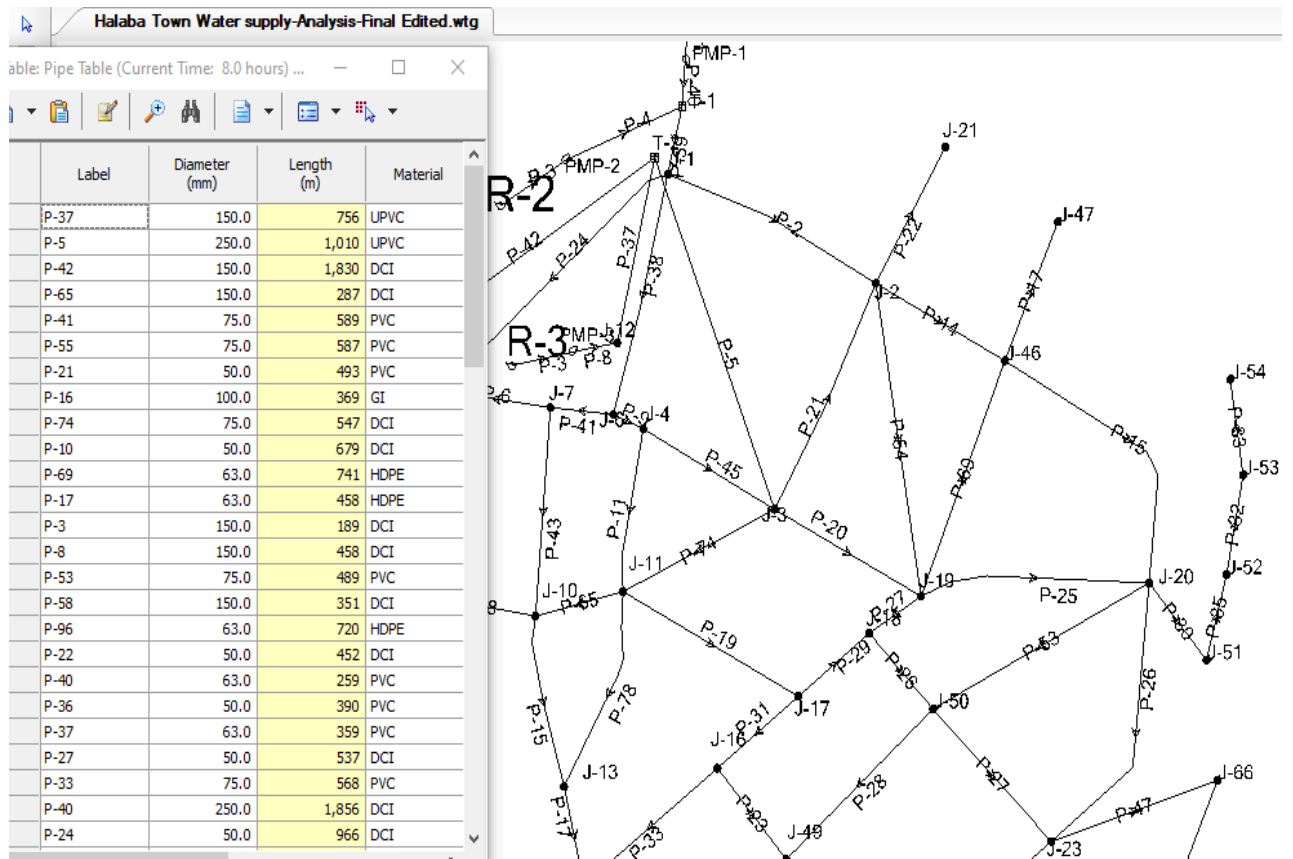


Figure 3. 5 Data entering on flex table

### 3.5.4.4 Hydraulics Analysis

The model would be constructed in Bentley waterGEMSv8i software due to its hydraulic and water quality modeling capabilities. Bentley waterGEMSv8i 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).

#### 3.5.4.4.1 Source Analysis

When analyzing any water system, it is critical to understand the sources supplying water to the system. Without adequate source, even the best-designed water systems will fail to deliver the required flow to water users (Datwyler, 2014). Typical water supply sources include reservoirs, storage tanks, and external water supply at junction nodes such as groundwater wells. Reservoirs and storage tanks furnish the water supply to the water distribution network.

Reservoirs treated as inexhaustible sources of water, and as such, their water level never varies. However, as a storage tank empties, its water level lowers and it has to be refilled by pumping from either a reservoir or a groundwater well. In Bentley WaterGEMS, groundwater well pump should model the same as a pumped reservoir. As pumping of the groundwater occurs, drawdown of the water table elevation at the groundwater well can occur. At higher pumping rates, the groundwater well may not be able to recharge fast enough to maintain the pumping rate specified by the defined groundwater well pump curve. So during simulation assume that the source is not fluctuated (Rossman, 2000).

#### 3.5.4.4.2 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).

$$Q_{P1} + Q_{P2} = Q_{Pm} \quad 3.4$$

$$C_{P1} \cong C_{P2} = C_{cm} \quad 3.5$$

Where:

$Q_{P1}$  = discharge of  $p_1$ ;  $Q_{P2}$  = discharge of  $p_2$ ;  $Q_{Pm}$  = discharge of  $p_m$ ;  $P$  = pump and  $P_m$  = 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).

$$C_{P1} + C_{P2} = C_{cm} \quad 3.6$$

$$Q_{P1} \cong Q_{P2} = Q_{aPm} \quad 3.7$$

Where:

$C_{P1}$  = curve of pump1;  $C_{P2}$  = curve of pump2 and  $C_{cm}$  = modeling pump curve;  $Q_{P1}$  = discharge of  $p_1$ ;  $Q_{P2}$  = discharge of  $p_2$ ;  $Q_{Pm}$  = discharge of modeling pump;  $P$  = pump and  $P_m$  = model pump;

#### **3.5.4.4.3 Tank Analysis**

When performing an extended period simulation, the two 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 caused 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).

$$V_{T1} + V_{T2} = V_{Tm} \quad 3.8$$

Where:

$V_{T1}$  Volume of Tank1;  $V_{T2}$  = Volume of Tank2;  $V_{Tm}$  = Volume of modeling tank.

#### **3.5.4.4.4 Assign Network Parameters**

After the skeletonization of the network on Bentley waterGEMS 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.

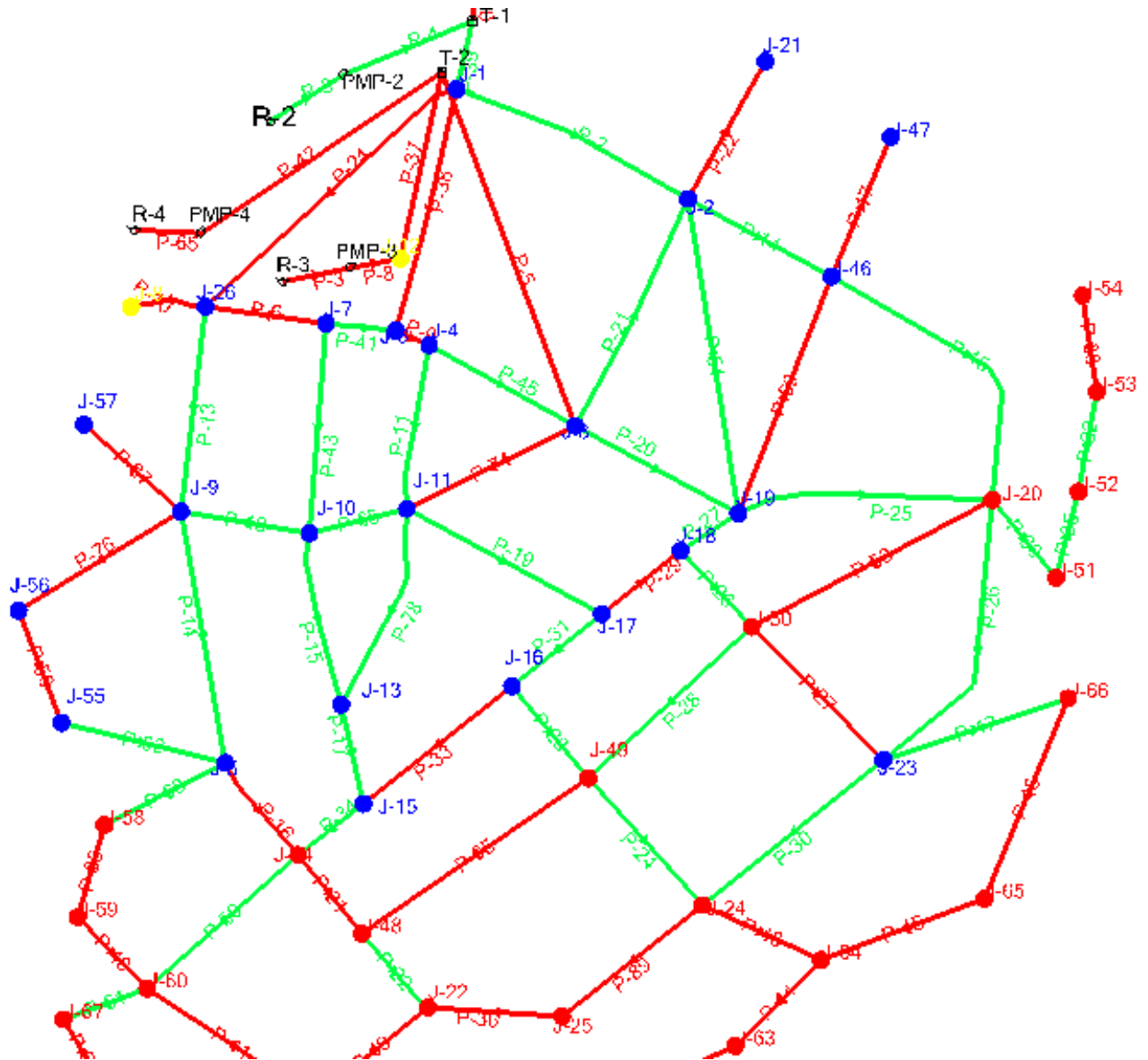


Figure 3. 6 Assign network parameters

### 3.5.4.4.5 Hydraulic Calculation

**Nodal Demand Allocation:** A special aspect of the model building process is the determination of nodal demands. Bentley WaterGEMSv8i enables to allocate demand to the model of water distribution system. Load Builder greatly facilitates the tasks of demand allocation and projection. In the study the nodal demand was allocated using unit line demand allocation method. Demand for each node was calculated and analyzed based on the number of population point load data selection nearest node based on available load builder methods for each consumption node, and the period of supplying water to calculate the peak factor of demand for each node. 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. Population around the node was identified, and the people served by the node were multiplied by per capita demand. 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:

$$N_d = \sum p_i D_j \quad 3.9$$

Where:

$N_d$  = nodal demand;  $p_i$  = population supplied by the nearest node of the service area;  $D_j$  = per capita demand assigned for the study area;  $i$  = subscript referring to the  $i$ -th node in the service area;  $j$  = subscript referring to the  $j$ -th pressure zone in the service area.

#### 3.5.4.4.6 Model representation

The town water supply distribution network analysed and optimised using application model of WaterGEMsv8i. 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. The analysis of the data which were simulated by models were hydraulic design of pressure line from four reservoir sources to the service tanks and tank outlet to pipe line network of the town. System distribution networks were drawn as a combination of various system components. The model was commonly, in water distribution system represented by system elements, such as reservoir, tank, pipe, node, pump and valves.

**Reservoir:** Reservoir is a node that represent an infinite external source or sink of water to the network. The primary input property for a reservoir is its hydraulic head. Because a reservoir is a boundary point to a network, its head cannot be affected by what happens within the network. Therefore, it has no computed output properties.

**Pump:** A three-point pump curve can be developed based on our static and hydrant tests, a range of demand and/or tank levels in the proposed system. The formulae can be used to develop a 3-pont pump curve (Kaychamper, 2004).

**Pipes:** Pipes are links that convey water from one point in the network to another. Flow direction is from the end at higher hydraulic head to that at lower head. The principal hydraulic input parameters for pipes during analysis were start and end nodes, diameter, length, roughness coefficient and status (open or closed). The computed outputs for pipes

included head loss, velocity and flow. To compute friction head losses, Hazen-Williams equation were used with the assumption that viscosity is constant. The Hazen-William formulae was selected and its roughness coefficient (C-value) of 130 was selected for the uPVC and DCI, pipes,150 for the PVC,120 for steel and GI pipes. The formula was selected and used for this study Hazen-Williams formulas (OWWDSE, 2010).

$$H_f = \frac{10.7 L Q^{1.852}}{C^{1.852} D^{4.87}} \quad 3.10$$

Where,  $H_f$  = Head friction;  $Q$  = discharge (m<sup>3</sup>/s);  $L$  = Length of the pipe (m);  $D$  = Diameter (mm) and  $C$  = Roughness coefficient which varies for different pipe materials and age. The pipe roughness coefficient refers to a value that defines the roughness of the interior of a pipe. Two common roughness coefficients are the Hazen-Williams C-value and the Darcy-Weisbach f-value.

Although the Darcy-Weisbach term is generally considered more accurate and flexible by giving information about flow regime, it is also more complicated and difficult to determine. Therefore, the Hazen-Williams C-value is commonly used in network modelling as in this study. The higher the value, the smoother the interior surface of the pipe and the greater the carrying capacity of the pipe. Since the determination of C-values at the site is very difficult, generally the approximate values in literature are used by knowing the material type and installation year of each pipe. The Hazen William roughness coefficients value of different pipe materials are given in Table 3.7.

Table 3. 8 Hazen-William roughness coefficients for pipe material (Chase *et al.*, 2003)

Pipe type	Pipe Age (years)		
	New	Old (10 - 20)	> 20 years
HDPE, PVC, UPVC	150	125	105
DCI	130	105	96

**Junctions:** Junctions are points in the network where links join and where water enters or leaves the network (Rossman, 2000). The basic input data required for junctions are elevation above some reference (usually mean sea level), location (X-coordinate, Y-coordinate) and water demand (rate of withdrawal from the network), besides representing the connection point between pipes, can represent the following components in a network:

- ✚ Points of water consumption (Demand nodes)

- ✚ Points of water input (Source nodes)
- ✚ Location of Tanks or Reservoirs (Storage nodes)

**Storage Tank:** Storage tank is a structure used to store water and provide water to the system when needed. Storage tank is crucial to continuously supply during a pump turned off and equalize water during peak demand hours. The study area has two storage tanks which functions for storing water and equalizing flow to each service area. The municipality uses these storage tanks as a pressure zone boundary based on the topography to manage the distribution see Table 3.4.

### **3.6 History and Description about Halaba Kulito Town Water Supply Distribution Network**

Halaba Kulito Town has been through a problem of water supply in the past years. The current water supply source of the town consists of six Groundwater deep borehole locally named as Aroge biro BH, Kera BH, Tena tabiya BH, Maramiya BH, Hantame BH and Debancho BH. During the study year two of them are non-functional, which are Aroge biro BH, Kera BH, with average estimated yield of 38.5 lit/sec. During the study 2(two) concrete circular tanks and one old plastic cylindrical tanks with capacity of total volume of 1,025 m<sup>3</sup> and functional and all of them are used for storing and balancing water supply distribution system in Halaba Kulito Town. They located in different sites: The two tanks are located in Choroko area and covers all part of the town. The oldest one tank is located near to the oldest water supply office (Aroge biro). The total number of customers in 2019 was described in the demand projection section.

Even though, Water demand in Halaba Kulito Town was high and shown severe shortage of water in the town. During the study time, Halaba Kulito Town was used intermitted water supply system. Because of, in Halaba Kulito Town the quantity of water that wells produced not enough to meet the needs of consumers and system flushing, and other needs. The problem arises due to limited source capacity, high population growth in town, poor operation and maintenance, inequity of water in distribution due to the shortage of the sources. Moreover, another problem of water supply in Halaba Kulito Town was associated with unpredictable/erratic power supply that humped continued operation of the water supply system. In Halaba Kulito following rapidly development of the town construction field such as buildings, expansion of road including five rural kebele's and population number increase rapidly. This also highly challenge and make stressed on water supply system. In other hand

Halaba Kulito 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.

## 4. RESULTS AND DISCUSSIONS

### 4.1 Assessing the current status of water supply system of Halaba Kulito Town

#### 4.1.1 Water Sources and Production

The main source of water supply for the study town is from four groundwater deep borehole with a total discharge capacity of the town was 38.5 l/s. According to the information found during discussion and field observation with the experts of Halaba Kulito Town water supply and sewerage service enterprise no other sources of water for potable drinking water except these sources.

As described in Table 3.2 the town water supply is a 100 % dependent on electric power with working (19 hours), the wells produce a total volume of discharge 38.5 l/s which is equal to 30.5 l/s in 24 hours. Therefore, the total volume of water entered to the storage tank within 24 hours is 2,635.2 m<sup>3</sup>. According to the information obtained during discussion with the experts of water supply and sewerage service enterprise, one problem of the water supply of town is the source of power fluctuation for pump motor. That means, the supply of water to the town was 100% electric power dependent.

#### 4.1.2 Coverage of Potable Water

Average per capita consumption was used to assess the domestic water supply coverage of the town. Data on individual domestic water consumptions, total water consumption (m<sup>3</sup>) and total production (m<sup>3</sup>) were collected from Halaba Kulito Town water supply and sewerage service enterprise billed documents for analysing average per capita consumption. As we have seen in equation 3.2, the potable water supply coverage, the quantity of per capita water consumption was used. The average per capita water consumption was derived from the yearly consumption of the town that have been aggregated from the individual water meter and public tap. Thus, the annual water consumption data was converted to average daily per capita consumption using the population data of Halaba Kulito Town.

Table 4. 1 Annual water consumption of Halaba Kulito Town

Year	Population	Annual billed Consumption (m <sup>3</sup> )	Per capita Consumption (l/c/d)
2019	82099	479858	16

The average per capita domestic water consumption was derived from the yearly consumption of the town that have been aggregated from the individual water meter and public tap. Thus, the annual water consumption data was converted to average daily per capita consumption using the population data of Halaba Kulito Town using equation 3.2. As shown in Table 4.1,

the per capita domestic water consumption of Halaba Kulito Town was found to be 16 l/c/d in the year 2019. According to WHO (2008), the minimum quantity of domestic water required in urban areas of developing country in the radius 0.5 km taken as 20 l/c/day. Regarding to this value, the domestic water supply of Halaba Kulito Town only satisfies 80% of the standard value and the quantity of domestic water required in urban areas of Ethiopia is taken as 50 l/c/day (MoWIE, 2015). According to this value, the domestic water supply of Halaba Kulito Town only satisfies 32% of the standard value. The per capita water consumption of town is viewing very low. The main reasons for reduction in the town's per capita water consumption as time goes is the increase in the population number of the town, pump failure and seasonal fluctuation of the source. The population number of the town is increasing from time to time with increasing demand on the existing water supply system of the town. As a result, the per capita domestic water consumption of the town gets lower and lower. Thus, it is advisable to develop the public preferred nearby source with supply and install new pump to improve the per capita water consumption of the town.

#### **4.2 Water Losses**

The monthly water produced and distributed to the distribution system and the water billed that was collected from the individual customer meter readings in the year 2019, used to quantify the total water loss for the entire town. The total water loss has been also evaluated based on percentage of system input volume, length of main and number of connections as explained under the performance indicators sub title later in this section.

The designed water production capacity of the town was 2,635.2 m<sup>3</sup>/day. However, the actual production of water has been lower than the maximum designed capacity due to pump failure and seasonal fluctuation of the source. Production data computed from all sources in (HKWSSSE) shows that actual average production of water at present from the system was 1,915.8 m<sup>3</sup>/day, which indicates 73% of its designed capacity (2,635.2 m<sup>3</sup>/day).

The total production of water recorded by the water supply and sewerage service enterprise of Halaba Kulito in the year 2019 was 699267 m<sup>3</sup> and the total billed consumption of the town was 479858 m<sup>3</sup>. The amount of total consumed water is less than the amount of water supplied. Water loss from water distribution systems (WDSs) has long been a feature of the WDN operations management. According to Motie.etal (2007), total water loss or unaccounted for water (UFW) is the difference between the volume of water produced, and the volume that is billed or consumed. Therefore, the total loss of water in the town for the year was 699267 m<sup>3</sup> – 479858 m<sup>3</sup> which gives 219409 m<sup>3</sup> and approximately 31.4% of the

total production. This figure is lower compared with the average for developing countries (35%) according to (Kingdom, 2006). It is different with the result of the study on Debre Markos town (Mebet, 2007) which is 39.68% and Axum town (Shimeles, 2011) which is 39.1%. The average the amount of water, which actually reached the consumers (system efficiency) of the entire town, accounts for only 68.6% of the total water produced. According to Mckenzie *et al* (2006), the system efficiency is good (acceptable) if above 75% of water produced reaches the consumer. Therefore, Halaba Kulito Town water supply distribution network is not good.

The main reasons for this high loss of water are the current water distribution system of the town. And also the average tariff for 1(one) m<sup>3</sup> of water in the town as 3.5 birr, the water loss is estimated to be 767931.5 birr every year. However, the real loss is beyond this as the water tariffs like other developing countries are usually subsidized.

### 4.3. Model Representation

Network data describes all physical components of the water distribution system and defines how those elements are interconnected. Distribution system networks are drowning as a combination of various system components. It commonly includes; reservoir, pipe, tank, pumps and valves. with little difference the real water distribution system represented as a combination of nodes and links. junctions, reservoir and tanks usually represented as nodes; Pipes, Pumps and valves represented as links.

Table 4. 2 Pipe diameter and corresponding lengths with their coverage (HKTWSSSE, 2019).

Diameter (mm)	Material type				Total Length (m)	Coverage (%)
	DCI	PVC	HDPE	UPVC		
50	4,179	6,701	5,491	-	16,371	33
63	-	-	10,746	-	10,246	20
75	547	4,444	1,326	-	6,317	13
100	3,088	369	1,917	1,137	6,511	13
150	4,119	489	-	1,256	5,894	12
250	1,856	-	-	2,598	4,454	9
Total	13,789	12,003	19,480	4,991	49,793	
Coverage (%)	27	24	39	10		

As described in Table 4.2 the total length of pipes represented in the modeled materials were DCI covers 27 %, PVC covers 24 %, HDPE covers 39 % and UPVC covers 10 %. In the model different diameter pipes are represented to contribute their function to water distribution network. In the model 50 mm, 63 mm and 100 mm pipes are used in high percentage compared to other diameter represented in the model.

### 4.3.1 Simulation Results

#### 4.3.1.1 Steady State Simulation

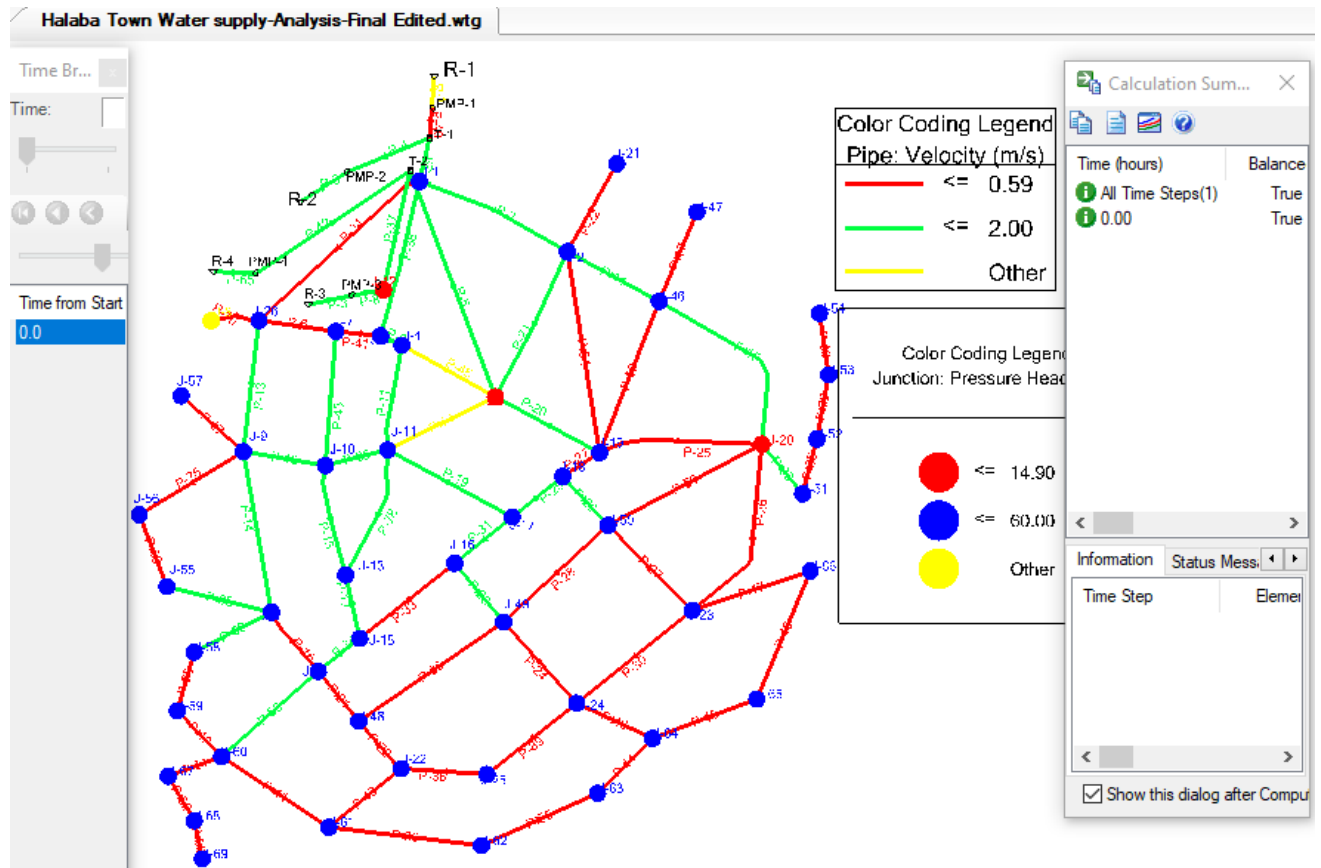


Figure 4. 1 Steady State Simulated pressures and velocities

The model has been performed in steady state simulation analysis for the average daily demand, which is the demand at every node not changing throughout 24 hours of a day. It is required to run single period at the beginning of the simulation as to observe the model under snapshot situation. The simulated result is presented in Appendix (1)

#### 4.3.1.2 Extended-Period Simulation

The system conditions have been computed over 24 hours with a specified time increment of one hours and starting model run at time 12:00 PM. The software simulates dynamic state hydraulic calculation based on mass and energy conservation principle. The model has been

simulated for every one-hour time setup in the twenty-four-hour duration. However, for the analysis the peak and minimum hours' demand has been simulated to identify the current performance of the system related to system parameter like pressure and velocity. The model has been performed 12:00 PM to 3: 00 AM for minimum hour consumption and 6:00 AM to 8:00 AM for the peak hour consumption. It is noted that minimum hour model run has been made at 0:00 hour from starting time and peak hour model has been made at 7:00 AM from the starting to 8:00 AM. The dynamic simulated results have been presented in Appendix (2).

#### 4.3.1.2.1 Pressure

Pressure in water distribution systems has to be maintained optimum; as to efficiently make water available to each demand category including at instances of firefighting (high withdrawal period) and as to reduce leakage as well as pipe breakage across the system. Swamnee and Sharma (2008) described that the minimum design nodal pressure is the pressure assigned to discharge flow on to the system.

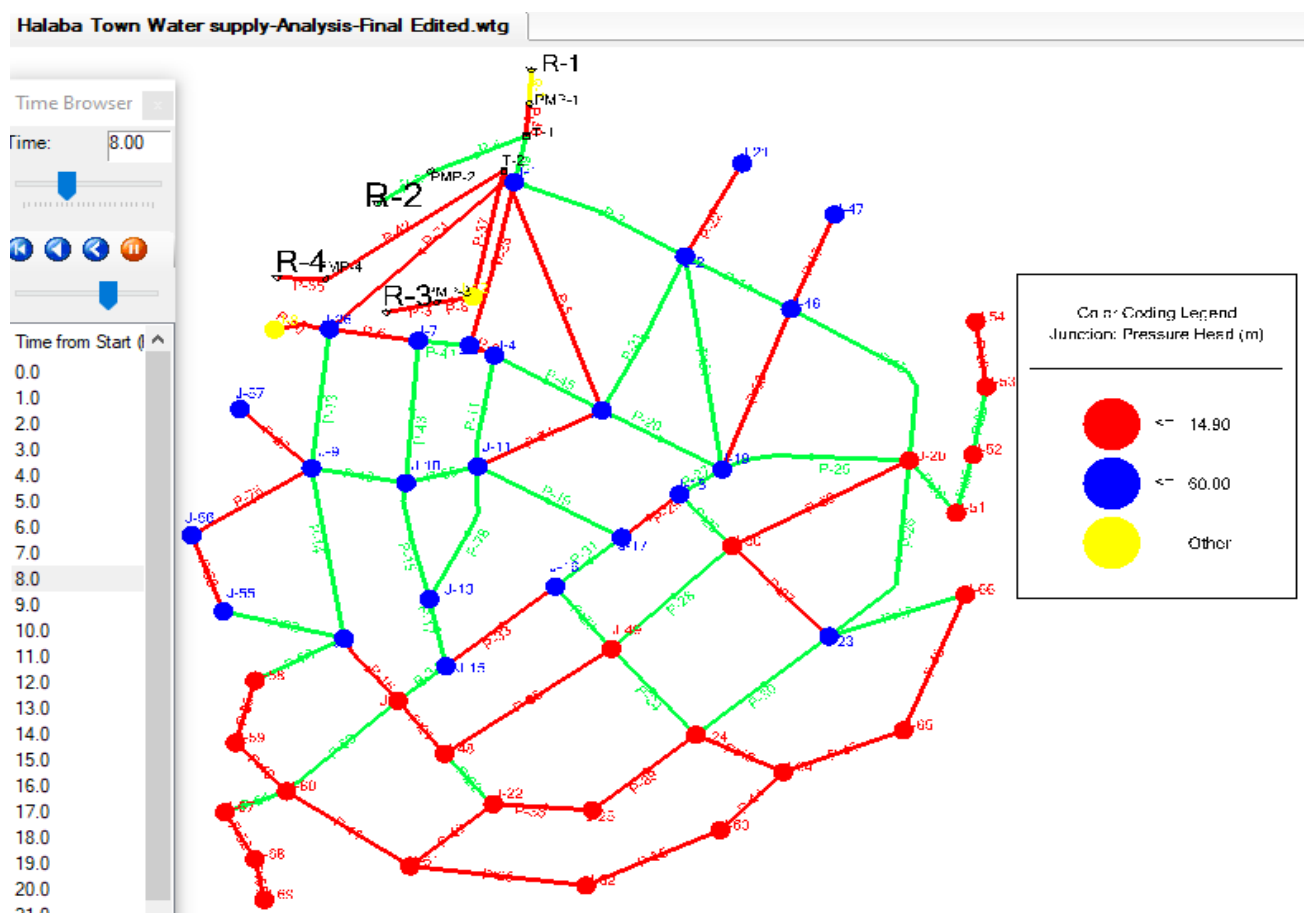


Figure 4. 2 Pressures at 7:00 AM to 8:00 AM in the peak-hour consumption

At minimum peaks through night hours the pressure in the system becomes high and the leakage losses expected to increase whereas at high peaks the pressure becomes small and the

leakage losses expected to decrease When the pressure exceeds the elevation of the storage tank, water can start to fill the storage tank. The higher the pressure is more water start to enter to the tank. In this study, the model run from the input of existing data a total node of 50 was reported from the project inventory dialog box. Based on Table 3.6 the results show that 26 nodes from a total 50 nodes have been observed out of the recommended serviceable pressure (15 mH<sub>2</sub>O to 60 mH<sub>2</sub>O). The steady state analysis describes the behavior of the system at a specific point in time with flow rate and hydraulic grade remains constant overtime. Extended period simulation indicates the performance of the distribution system better than steady state simulation during high consumption or at stress condition. The simulated result for extended period simulation at 24(twenty-four) junctions showed that changes from positive pressure to negative pressure during high consumption period. The negative pressure indicates that the area supplied by nodes should not gate water at maximum consumption hours. The pressure at 2(two) nodes greater than the recommended pressure 60 mH<sub>2</sub>O. High value of pressures affects adversely the hydraulic performance of the distribution network at night time during low consumption period, the pressure in the system become high and it causes pipe burst at the lower location which were presented in Table 4.4 below.

Table 4. 3 Simulated Pressure distribution in system at peak hour consumption (8:00)

Pressure range	< 15 mH <sub>2</sub> O	(15 - 60 mH <sub>2</sub> o)	> 60 mH <sub>2</sub> O
Number of Nodes	24	24	2
Percent (%)	48	48	4

As shown in the Table 4.3 from the total nodes 48% nodes had pressure below 15 mH<sub>2</sub>O, 48% had permissible pressures between 15 mH<sub>2</sub>O and 60 mH<sub>2</sub>O and 4% of the node had above 60 mH<sub>2</sub>O pressure.

Table 4. 4 Nodes/Junctions were selected in the network modeling process

Pressure ranges	Name of nodes
< 15 mH <sub>2</sub> O	J-68, J-59, J-64, J-67, J-58, J-66, J-62, J-63, J-60, J-69, J-25, J-22, J-61, J-54, J-24, J-52, J-65, J-53, J-20, J-49, J-48, J-51, J-50, J-14,
15 mH <sub>2</sub> o - 60 mH <sub>2</sub> O	J-9, J-23, J-55, J-57, J-5, J-16, J-18, J-46, J-56, J-21, J-19, J-17, J-15, J-2, J-47, J-13, J-26, J-1, J-10, J-3, J-7, J-4, J-6, J-11,
> 60 mH <sub>2</sub> O	J-8, J-12,

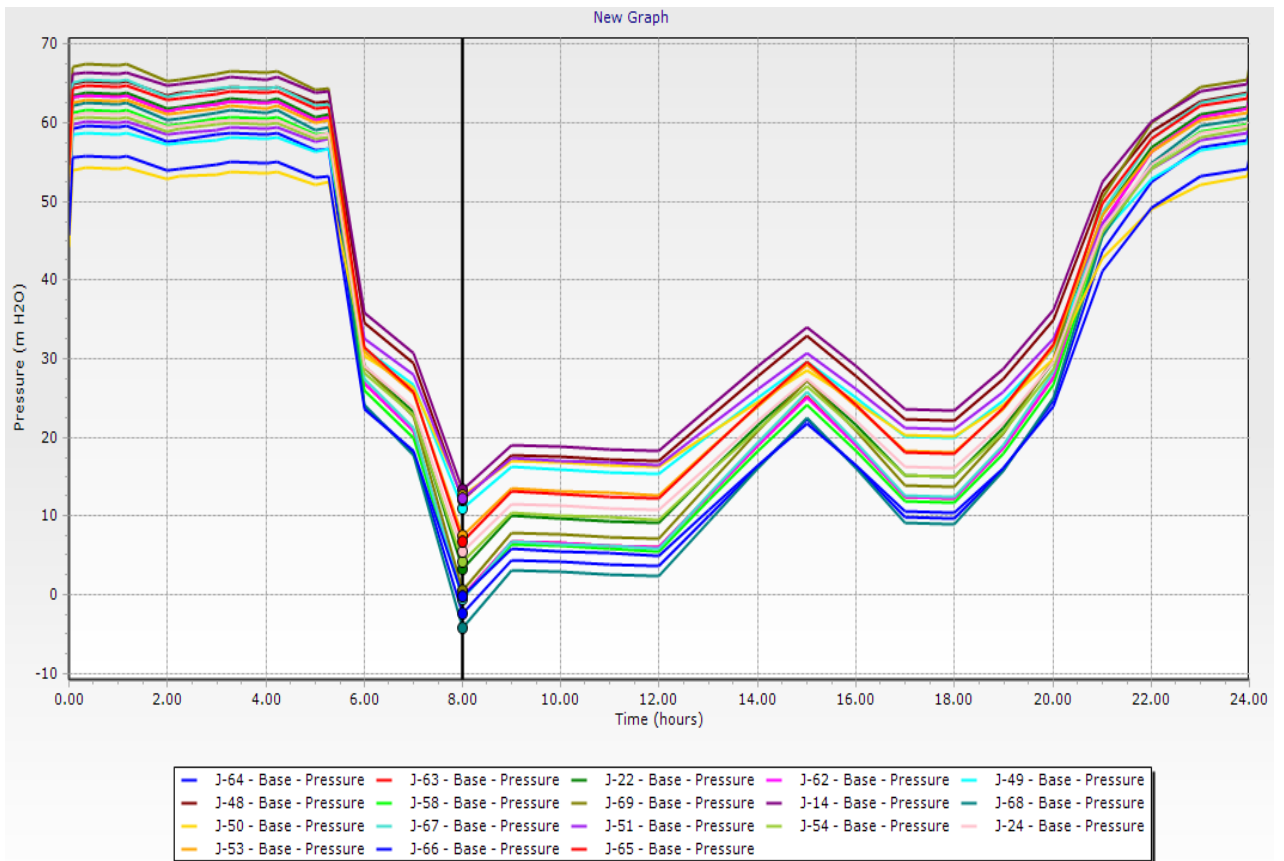


Figure 4. 3 Graph showing nodes with negative pressure at peak hour consumption

As shown in Figure 4.3 Pressures at nodes on the distribution network, which indicate unexpected terminal pressure at customer premises, are modeled and low pressure values at these points which show the presence of poor pressure condition at customer premises are identified using the color coding ability of waterGEMS software. when the system operated at steady state the demand at every node did not changing, the pressure for all the eighteen nodes was positive. However, as the demand changed to the peak hour demand the pressure decreased to negative (J – 64, J – 48, J – 50, J – 53, J – 63, J – 58, J – 67, J – 66, J – 22, J – 69, J – 51, J – 65, J - 62, J - 14, J - 54, J - 49, J - 68, J - 24) therefore, during this time water could not reach to consumers supplied by nodes as shown in the Table 4.3. The results of the WaterGEMsv8i model of pressure in the water distribution network nodes shows minimum hydraulic pressure which less than a minimum of the design criteria of 15 mH<sub>2</sub>O water column at the distribution system around Poly Technic College, Murasa, Gedeba and around above and below Halaba Hospital areas. This represents that the areas or village did not receive water from these junctions which shows negative pressure they produce low velocities which accelerate the deterioration and corrosion of the pipes in the distribution network. The pressure at nodes as shown Table 4.4 are greater than the recommended pressure 60 mH<sub>2</sub>O.

High value of pressures affects adversely the hydraulic performance of the distribution network at night time during low consumption period, the pressure in the system become high and it causes pipe burst at the lower location.

**Average-daily water consumption condition:**

In the average water consumption condition, all nodes except nodes around Poly Technic and Hospital, are receiving normal pressure in periods from 1:00 AM to 5:00 Am.

**Maximum-daily consumption condition:**

Maximum-daily consumption is the average water consumption rate at the maximum water use day. The maximum water consumption occurs at the weekend (Saturday and Sunday) in almost all water supply distribution systems all over the world. It is already mentioned as the maximum daily water use peaking factor is 1.15 times the average-daily water use. The EPS result showed as Storage Tank \_1 is receiving almost zero pressure in this water consumption condition.

**Peak-Hour Consumption Condition:**

Is the average rate of use during the maximum hour of usage and is assumed to be 1.8 times the average-daily water use. Since all water distribution systems are designed to meet the peak-hour demand condition, it was must to analyze the performance of the distribution system for two extreme periods, relatively high water use period and relatively low water use period. For the first case, all nodes in Poly Technic and around Hospital and Murasa, Gedeba receive negative pressures.

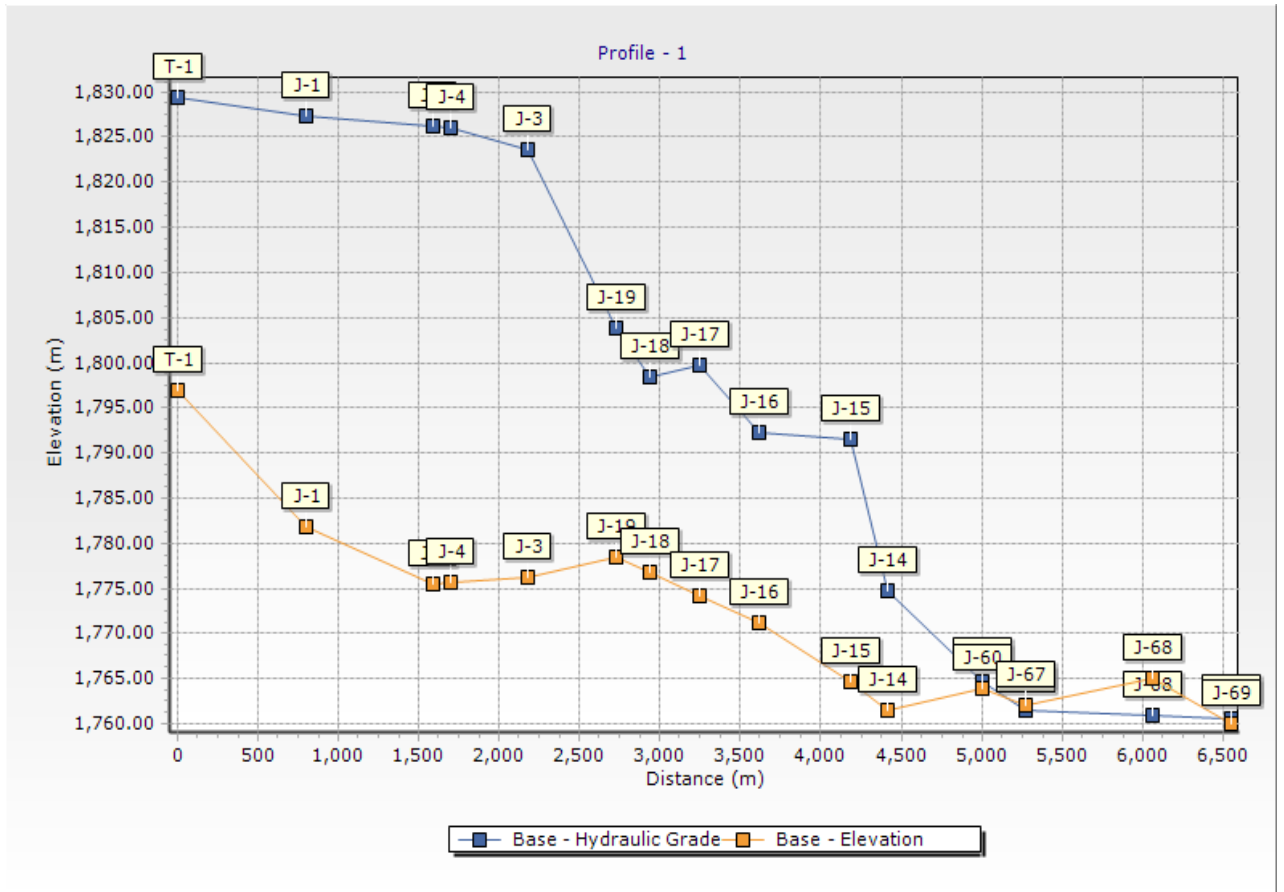


Figure 4. 4 Profiles of nodes showing distance from storage Tank - 1 (T- 1) with elevation Misirdali (2003), showed that as consumption nodes are furthest away from supply points Such as storage reservoirs will always receive less water than those nodes nearest to the source due to pressure losses in the network is increasing as far from the source. The Figure 4.4 shows how distance and elevation affect pressure distribution in selected nodes. In Halaba Kulito Town residents living around the choroko sefer and around Halaba Hospital get water at low pressure and low water pressure creates a low level of reliability of water users on a water supply system. The Figure shows the distance from storage Tank – 1 (T- 1) to the point of consumption nodes as described above.

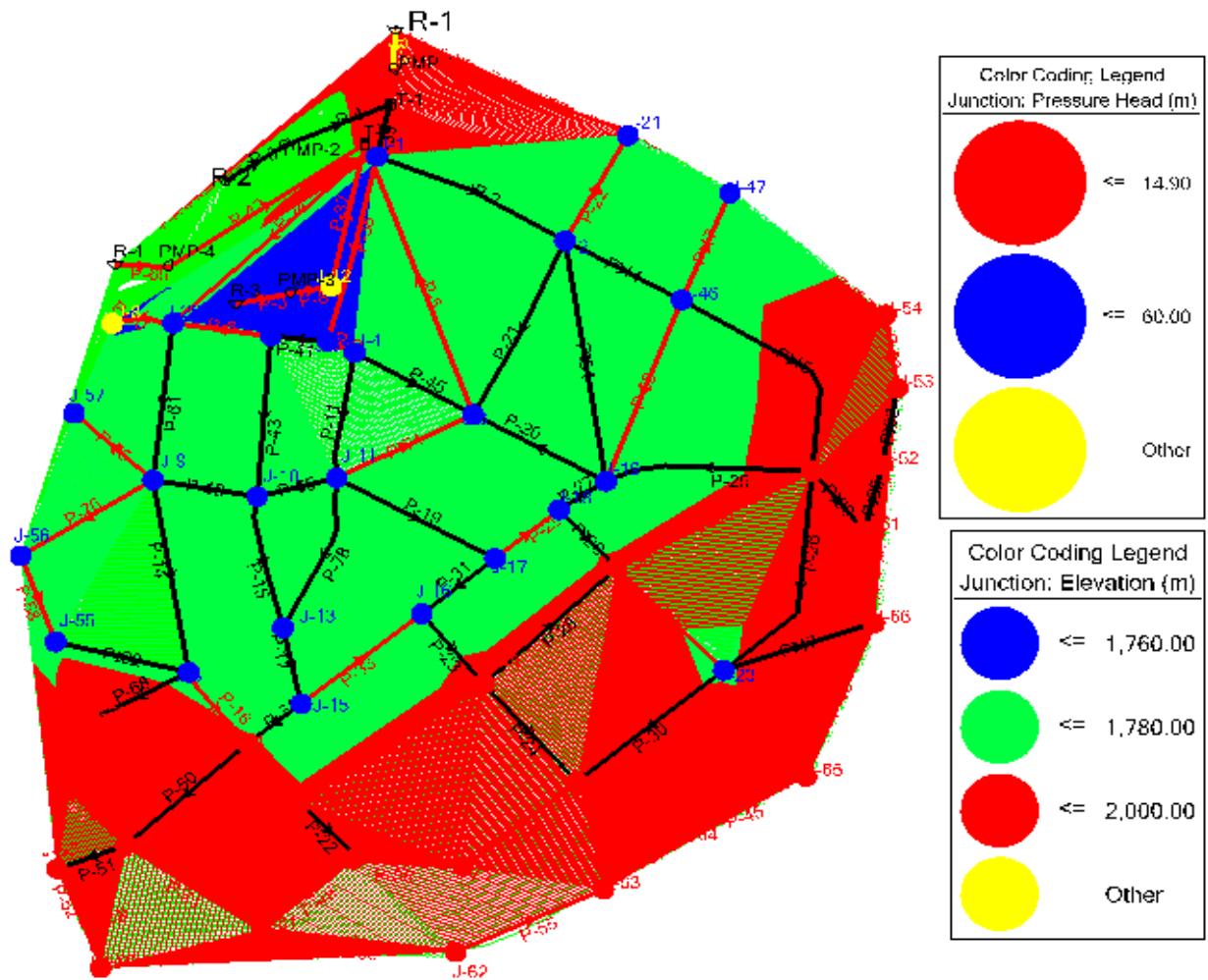


Figure 4. 5 Pressure contour at peak hour demand displayed with elevation

The pressures at nodes depend on the topography of the area and the performance of the input energy like pumps. The area highlighted by red color in the Figure 4.5 is pressure deficit area below 15 mH<sub>2</sub>O water head. It is based on the population served, types of dwellings in the area, and firefighting requirements. The pressure at node depends on the adopted minimum and maximum pressure within the network, topographic circumstances, and the size of the network. The contour map of pressure clearly shows the pressure difference in the whole systems in the study town.

#### 4.3.1.2.2 Pipe Flow Velocity

Different design guide line has been developed by different researchers for the standard velocity in pipe flows. According to Ethiopian Topographical Condition the allowable velocity in distribution system indicated by the MoWR (2006) water supply design criteria recommended pipe flow velocity to be a minimum of 0.6 m/s and maximum of 2 m/s.

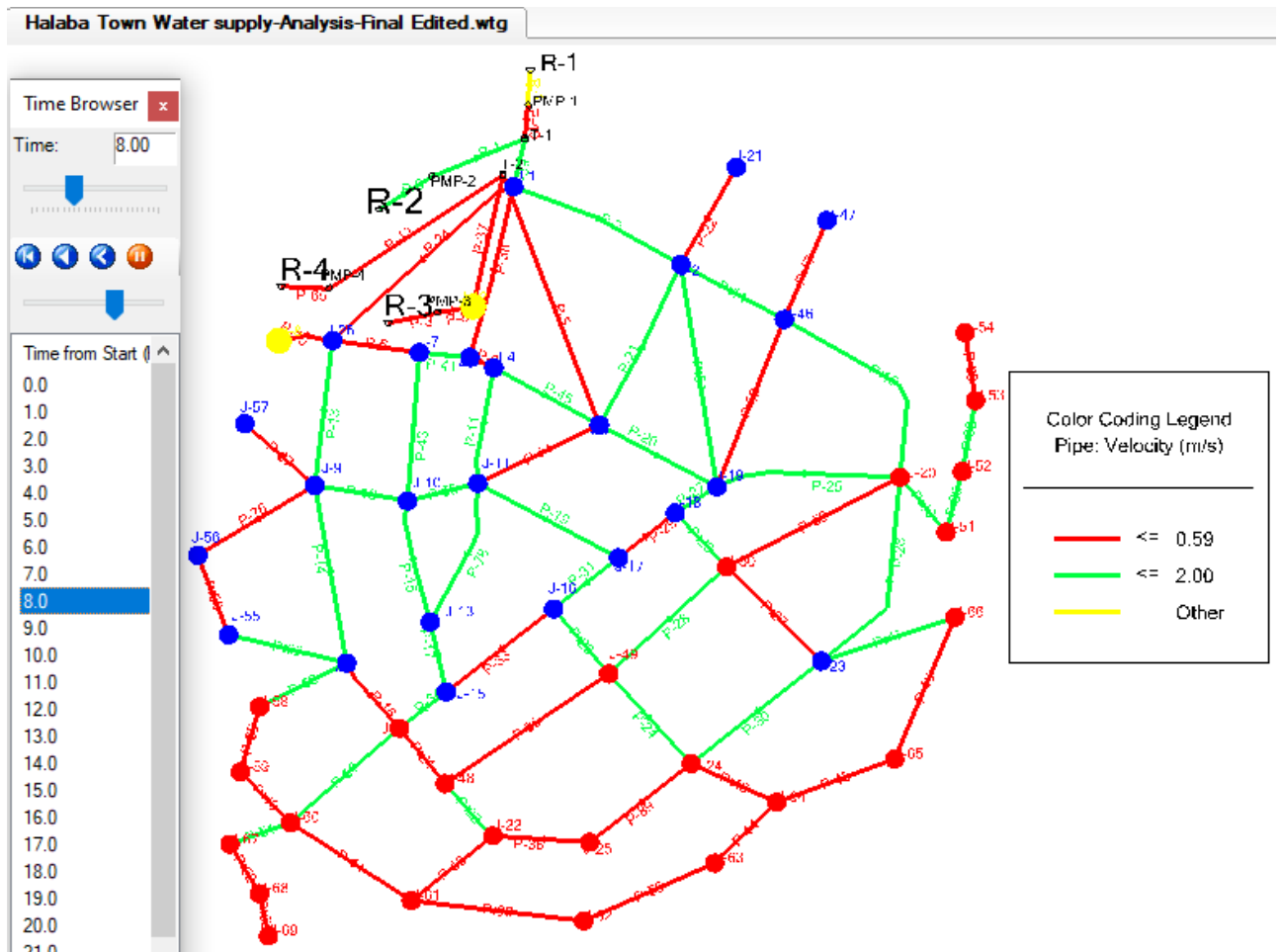


Figure 4. 6 Velocity distribution at peak hour consumption

Table 4. 5 Simulated results of velocity range in distribution network

Velocity range (m/s)	Count		Effect
	Number	(%)	
< 0.6	41	50	Sedimentation problem
0.6 - 2	40	48.8	Normal
> 2	1	1.2	High head loss occurred

As indicated in Table 4.5, 50 % of the pipes are below the permissible range of velocity; 48.8 % at permissible range; and 1.2 % above the range according to Table 3.7 criteria set by MoWR (2006). Low velocity in pipe flow affects the proper supply of water and undesirable for hygienic reason. When the diameter value of pipe increases the velocity decreases and long-time of retention causes sediment formation. The flow and pressure distributions across a network are affected by the arrangement and resizes of the pipes and the distribution of the demand flows. Since a change of diameter in one pipe length will affect the flow and pressure

distribution everywhere, network simulation is not an explicit process. Pipe network analysis involves the determination of the pipe flows and pressure heads that satisfy the continuity and energy conservation equations (Rossman, 2000).

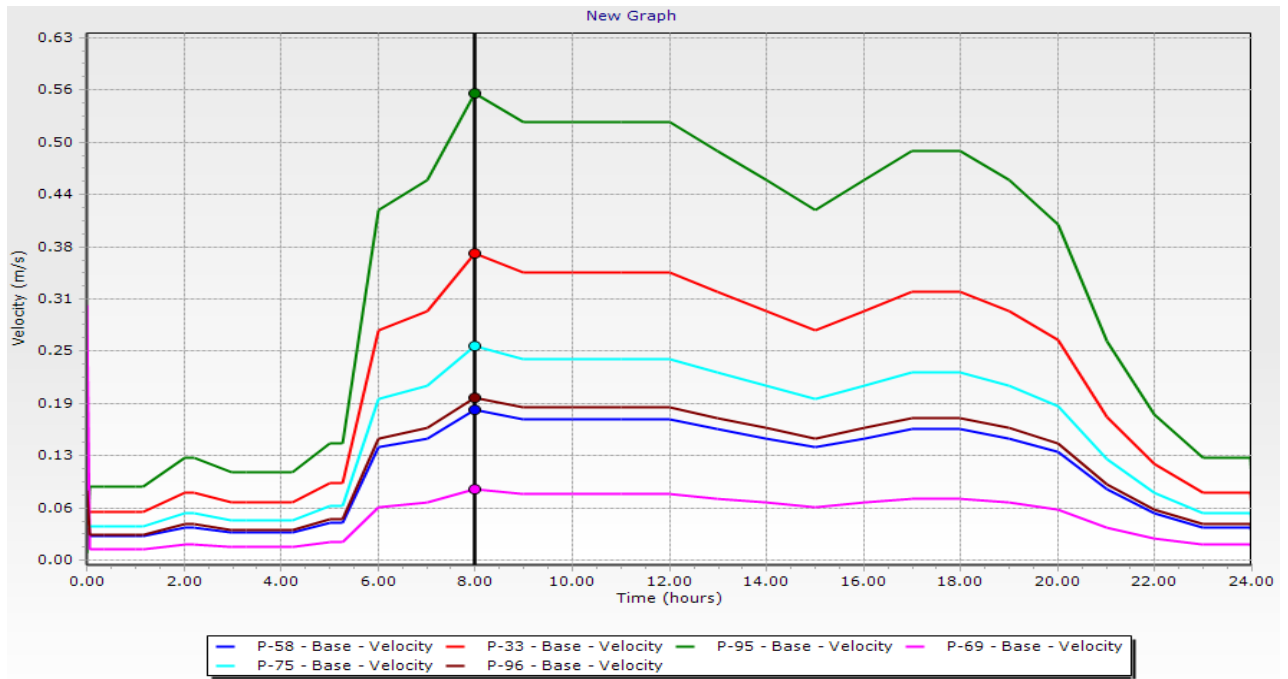


Figure 4. 7 Main transmission line showing velocity versus time graph (P - 58, P - 75, P - 33, P - 96, P - 95, P-69)

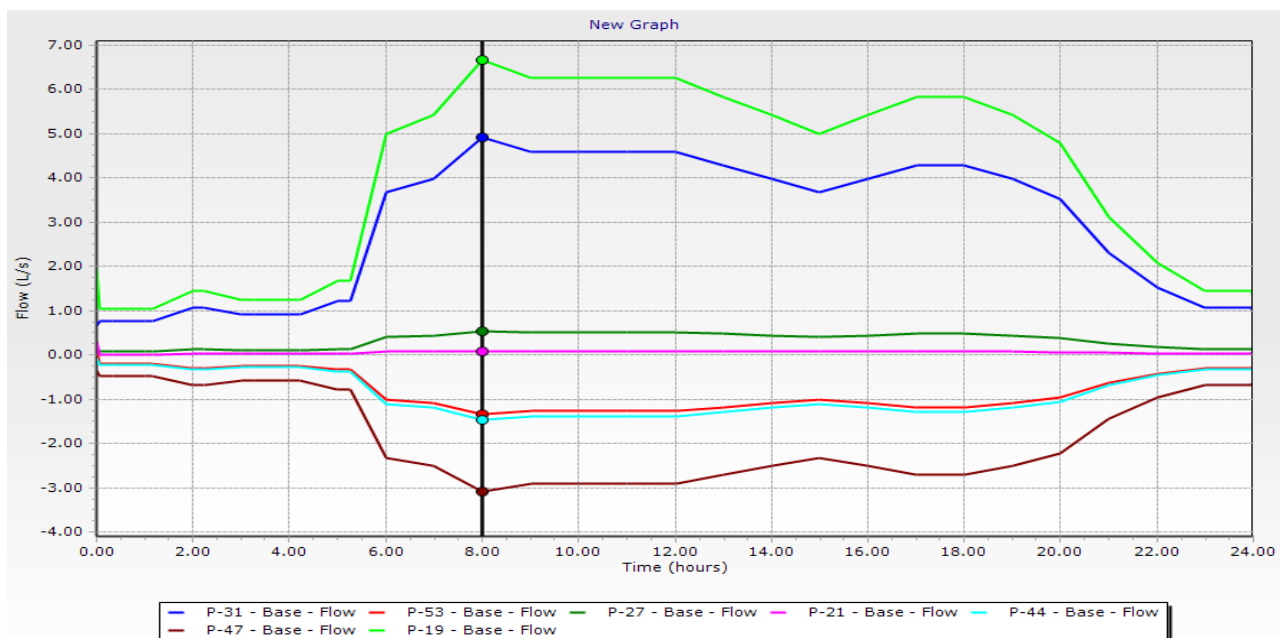


Figure 4. 8 Main transmission line showing flow versus time graph

The above Figures 4.7 and 4.8 shows velocity in distribution network is in high consumption time for selected pipes (P - 31, P - 47, P - 53, P - 19, P - 27, P - 21, and P - 44) the velocity and flow increase as the time increases from 2:00 hour up to 24:00 hour.

#### 4.3.2.3 Demand Pattern

Demand pattern is one of critical component at the system, from which is identified how much capitals consume to describe in graph. As far as distribution of water is concerned, the properties of hydraulic parameters in distribution network allowable limit was known. The driving force hydraulic parameters was demand of water consumptions.

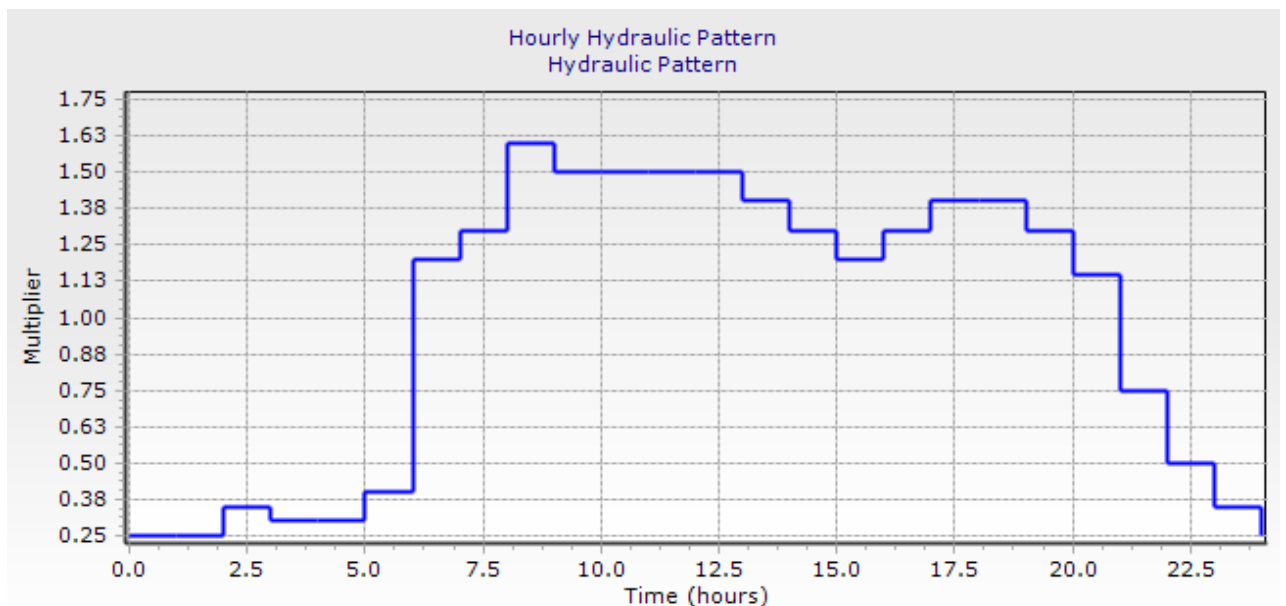


Figure 4. 9 Demand pattern in Water distribution of Halaba Kulito Town within 24 hours

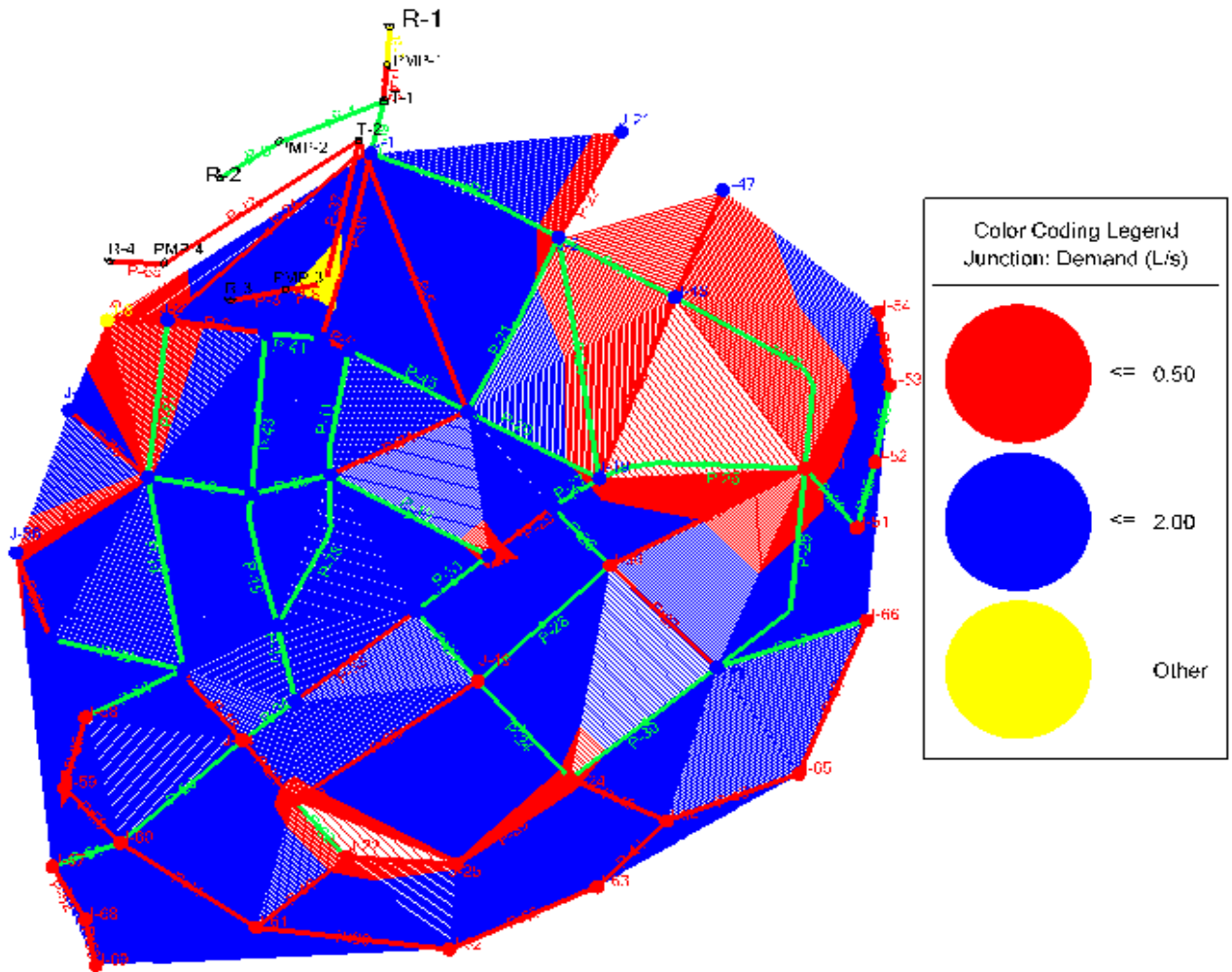


Figure 4. 10 The distribution network demand in Halaba Kulito Town

**4.3.3 Behavior of Storage tank at different consumption hours of a day**

The service reservoir (storage tank) is provided to balance (constant) supply rate from the water source or treatment plant with the fluctuating water demand in distribution area. Dynamic (EPS) simulation result was used to show the fluctuating storage volume with time increments during high and low consumption. Moreover, in low demand hours when the water consumption of consumer is almost zero, amount of pumped water is higher than system demand so that extra water coming from pumps are stored at storage tank and equilibrium of water distribution system satisfied again. The time varying simulation indicates that storage Tank- 1 starts to decrease its volume at 8:00 AM hour that means up to 8: hour AM the volume in the tank is full.

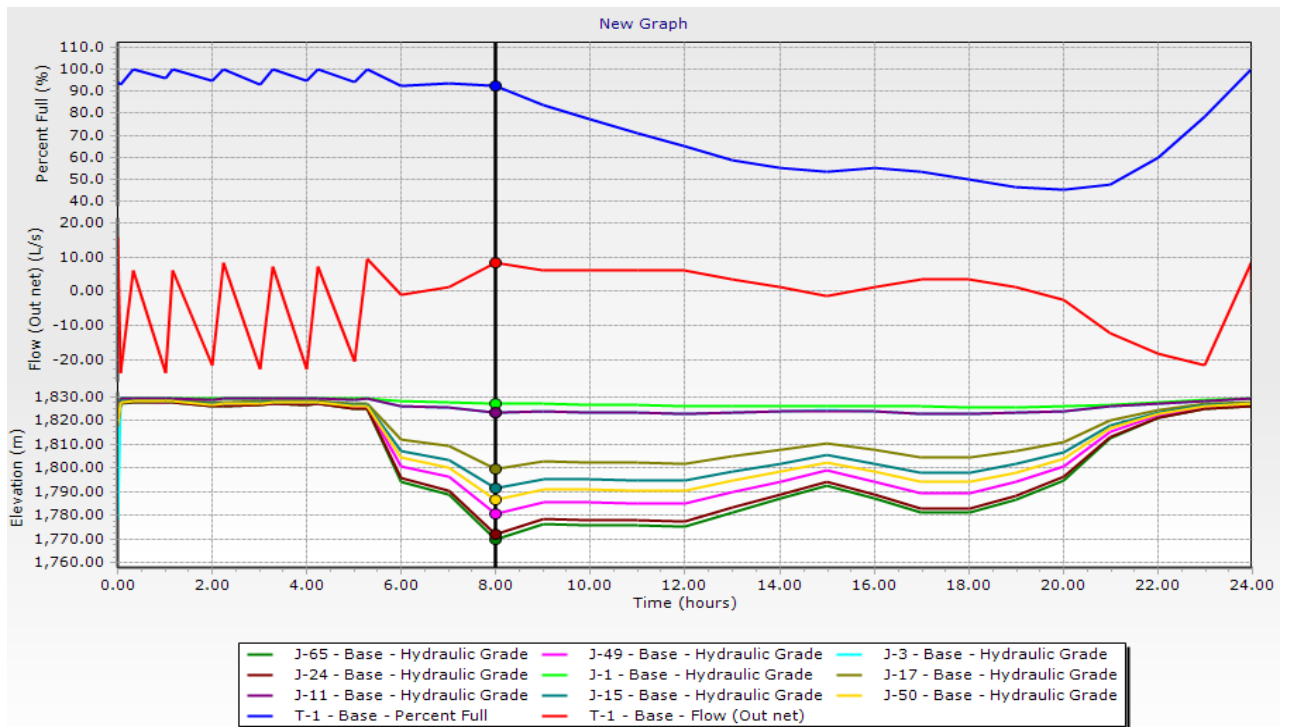


Figure 4. 11 Tank - 1 Water volume fluctuation over 24 hour periods

Figure 4.11 shows during the extended period simulation the storage level of the tank fluctuate for 24-hour period which shows the change in percent of full in different time interval and in different elevation ranges. When the simulation run begins the tank was full and then the volume starts to decrease up to 8 hours, so that the pumps should have to operate to replenish the volume of the tank starting from 8 hours.

#### 4.3.4 Pump

Pump is one of the important elements, which add energy to the system. Since water can flow from the higher energy location to the lower energy. Pumps used to boost the head at desired locations to overcome desired piping head losses physical elevation difference.

##### 4.3.4.1 Pump capacity curve

A pump curve represents the relationship between the head and flow rate can deliver water at nominal speed settings. Pump head is the head gain imported to the water by the pump and plotted on the vertical of the curve in meter.

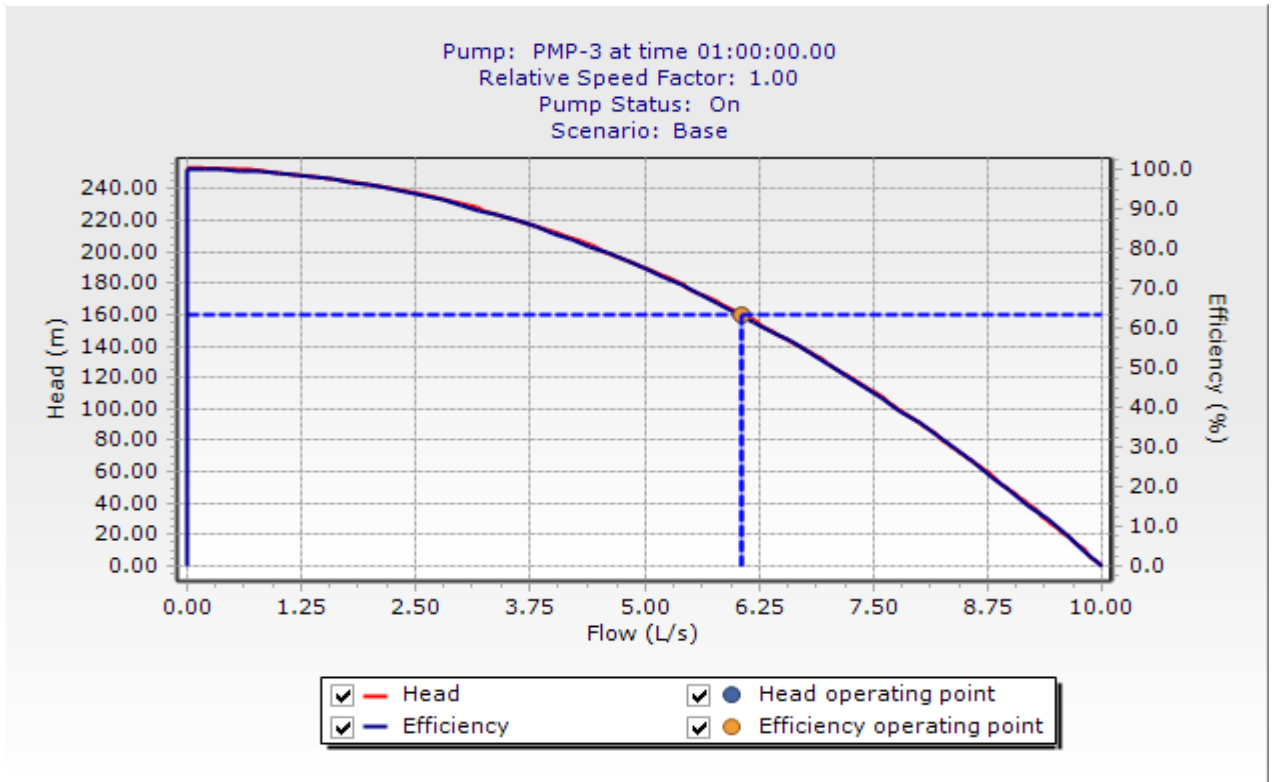


Figure 4. 12 Pump – head verses flow curve

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 4.12 shows as the blue line the head increases the amount of discharge pushed by the pump decreases. When the head decreases the pump can push high amount of discharge to a lower elevation so that pump curve indicates decreasing head with increasing flow illustrate by red line both of them had been overlapped.

## 5. CONCLUSIONS AND RECOMMENDATIONS

### 5.1 Conclusions

The total average per capita consumption of the Halaba Kulito Town in the year 2019 was 16 l/p/d which shows lower performance as compared to 20 l/p/d which is set by WHO (2008) within a radius of 0.5 km. For the study year which only satisfies 32 % of the minimum urban water consumption value set by (MoWIE, 2015) and 80 % value set by (WHO, 2008). The designed water production capacity of the town was 2,635.2 m<sup>3</sup>/day. However, the actual production of water has been lower than the maximum designed capacity due to pump failure and seasonal fluctuation of the source. Production data computed from all sources in (HKTWSSSE) shows that actual average production of water at present from the system was 1,915.8 m<sup>3</sup>/day, which indicates 73% of its designed capacity (2,635.2 m<sup>3</sup>/day). The total water loss was composed by subtracting the consumption (bill data) from the water supplied is 219409 m<sup>3</sup> that means 31.4% of the total production in 2019 at the town level. This figure is lower compared with the average for developing countries. The amount of water which actually reached the consumers (system efficiency) of the entire town, accounts for only 68.6% of the total water produced. Therefore, Halaba Kulito Town water supply distribution network is not good.

On the water distribution network hydraulic parameters performance has been evaluated using the existing water distribution layout, surveyed data of base demand, elevation and directions of northing and easting of junction, source, tanker and pipe data by using WaterGEMSv8i software.

The results of the WaterGEMSv8i model of pressure in the water distribution network nodes shows minimum hydraulic pressure which less than a minimum of the design criteria of 15 mH<sub>2</sub>O water column at the distribution system around Poly Technic College, Murasa, Gedeba areas and Halaba Hospital areas. Which represents that the areas or village did not receive water from these junctions which shows negative pressure they produce low velocities which accelerate the deterioration and corrosion of the pipes in the distribution network. Result shows during peak hour consumption, parts of the distribution system receive water with low pressure and under some conditions risk of obtaining no water is observed because of the pressure in the distribution system is below permissible minimum requirement. Which shows the analysis result performance of the distribution system has 52 % of consumption nodes have out of the indicated range of performance. 48 % nodes have acceptable pressure limits between (15 mH<sub>2</sub>O - 60 mH<sub>2</sub>O). The simulated result for the distribution system was resulted

48 % nodes has been found below the acceptable limits of pressure value ( $< 15 \text{ mH}_2\text{O}$ ), in pressure zone one due to high elevation difference and long distance from the source. The result for dynamic simulation for 24-hour period and during high consumption period 4% nodes have negative pressure.

The nodes are furthest from the source of supply point storage tank (T- 1 and T-2). As the distance increases the water pressure diminishes(reduces) in distribution system. The negative value of pressure indicates that the area supplied by those nodes could not get water in peak demand hours. Therefore, areas which need zoning or installing another new tank above the Halaba Hospital to enable the system to continuously supply the residents at all demanding time is known. Low velocity in pipe flow affects the proper supply of water and undesirable for hygienic reason. When the diameter value of pipe increases the velocity decreases and long-time of retention causes sediment formation. The flow and pressure distributions across a network are affected by the arrangement and sizes of the pipes and the distribution of the demand flows. The cause of sedimentations has been identified from 50 % of water distribution pipes with velocity below the acceptable limits ( $< 0.6 \text{ m/s}$ ). When the flow is moving very slow in pipes the very tinny materials remains on the inner wall of the pipes and causes water quality problems. The locations of pipes with high head loss are known from 1.2 % for velocity values ( $> 2 \text{ m/s}$ ).

## 5.2 Recommendations

In order to improve in terms of water supply analysis and hydraulic parameters performance in the distribution and water supply coverage of the town the following activity should be performed:

As this study, was specifically limited to evaluate and analyze hydraulic performance related to pressure and pipe flow velocity and to forecast the future population and demand a study should be undertaken related to spatial allocation of demand and leakage detection of the system. So that the administrative or other concerned body can use the findings and layout of this research to overcome the distribution network problems of the study town.

Halaba Kulito Town water supply and sewerage service enterprise should gather x, y, coordinates of all of the components of water supply distribution network, its customers water meter and prepare population layer by shape file in order to model the distribution system using WaterGEMSv8i software for future improvement of distribution network and expansion works with Technology technics to address the problems and to satisfy the customer needs.

To increase the reliability of water supply coverage, the groundwater sources has to be augmented by surface water and water losses control measures should be taken. The simulated result has shown low pressure at a nodes supplied by storage tank one; therefore, the water supply and sewerage service enterprise should have to construct additional reservoir or have to increase zoning around the above the Halaba Kulito Hospital area of the town to get high pressure head and install additional water sources. Necessary pressure supplementing valves should be installed to upraise pressure during peak demand time. More of the pipes are found under low flow velocity so that, the diameter of the pipes should be minimized to upgrade the quality and sufficient amount of water in the system. To reduce such problems in the Town distribution system should check every gate valves (flow control valves) and schedule water distribution to the customers as per their control status due to the shortage of the sources.

The Halaba Kulito town water supply and sewerage service should be organize to minimize the problem faced in the course of this study was associated with getting sufficient updated secondary data: shortage of well-documented data sources, unreadability of pressure gauges and adequate report especially in the study area from Halaba Kulito town water supply and sewerage service enterprise and due to resource constraints in terms of the research experiment materials coupled with cost to be incurred on the research to study water quality.

## REFERENCES

- Alaci & Alehegn (2009) Infrastructure Provision and the Attainment of Millennium Development Goals (MDG) in Decentralized Systems of Africa; Experiences from Ethiopia and Nigeria. Abuja, Nigeria.
- Almasri, N. (2010). *Simulation of water distribution networks*. The use of EPANET.
- Al-rayess, M. (2015) Performance evaluation and development of SDSS for Gaza city water network northern Nasser Area.
- Anden, S.P. and Kelkar, P.S., (2007) Performance of Water Distribution Systems During Intermittent Versus Continuous Water Supply. , 22 (4), Pp. 2001 – 2003.
- AWWA (2005). M32- Computer Modeling of Water Distribution Systems. Denver, CO., 159 pp. Cesario, L. (1995). Modeling, Analysis, and Design of Water Distribution Systems, American Water Works Association.
- AWWA (2012) Computer modeling of water distribution systems american water works associations manual M32.
- Behailu, S., (2012) Water Supply and Sanitation for Urban Engineers and Planners, Addis Ababa: Ethiopia: Ethiopia Civil Service College.
- Belay, A., (2012) School of Graduate Studies Addis Ababa Institute of Technology Hydraulic Network Modeling and Upgrading of Legedadi Subsystem Water Supply (Case Study of Addis Ababa City). , (March).
- Bentley. (2008). *Efficient Pressure Dependent Demand Model for Large Water Distribution System Analysis*. Haestad Methods Solution Center, Bentley Systems, Incorporated. Retrieved May 5, 2008 from [ftp://ftp2.bentley.com/dist/collateral/whitepaper/WDSA2006EfficientPDD\\_haestad\\_eng\\_lowres.pdf](ftp://ftp2.bentley.com/dist/collateral/whitepaper/WDSA2006EfficientPDD_haestad_eng_lowres.pdf).
- BERHE, W. (2005). *Water supply coverage and losses in distribution system*. The case of Addis Ababa, Addis Ababa University.
- Bhadbhade, N. M. (2004). *Performance evaluation of a drinking water distribution system using Hydraulic simulation software*. oilton, pune. Addis Ababa, Addis Ababa University.
- Birerley, G. (2006). The Use of System Dynamic Simulation in Integrated Water Resources Management. Geography and Environmental Science. Auckland, uckland University.
- BIRDIE, G.S. (1989). “Water Supply and Sanitary Engineering”, including Environmental Engineering and latest Water and Air pollution loss.

- Bouman, D., (2014) Hydraulic Design for Gravity Based Water Schemes. , (April). Chase,
- Dsavic, Grayman, Beckwith & Koelle (2003) Advanced water distribution modeling and management. Haestad Methods, Waterbury, CT.
- CSA (2010). and SSR (2007). *Population and Housing Census. IN ETHIOPIA, C.S.A.O.(Ed.) Population Size by Age and Sex*, Addis Ababa.
- Datwyler, T.T., (2014) Hydraulic Modeling : Pipe Network Analysis, 22(11), Pp. E391– E401.
- Dawe, P., (2000a) Hydraulic Network Modelling of the Rarotonga Water Supply System Cook Islands., (March).
- Dawe, P., (2000b) Workshop on Hydraulic Network Modelling With Watercad 16 – 20 October 2000., (October).
- DIGHADE, M.S. KADU & A.M. PANDE (2014). *Challenges in Water Loss Management of Water Distribution systems in Developing countries*, International Journal Innovative Research in Science Engineering Technology, vol.3, 1-2.
- EPA (2005a). Water Distribution System Analysis: Field Studies, Modeling and Management. Ohio, United States Environmental Protection Agency.
- EPA (2005b). Water distribution System analysis: field studies modeling and management. Ohio, United States Environmental Protection Agency.
- EPA (2010) Control and mitigation of drinking water losses in distribution systems.
- EPD (2007). Method for determining future water demand needs for public/private water systems, Georgia Environmental Protection Division.
- Farley, M., & Trow, S. (2003). Losses in water distribution networks: A practitioner's Guide to Assessment, Monitoring and control. . Caxton St, London,Uk: Alliance House,12.
- Farley, M., Wyeth, G., mdGhazali,Z. B., Istander,A.and Singh,S.(2008).The manager's Non revenue water Hand Book:Aguide to understanding water losses.UASD& Rahill utilities Berhad&USAID publication,report.
- Garg (2010). Water supply Engineering. Twentieth revised edition, Delhi, Mc grew.
- Grayman and Rossman (1994). Modelling chlorine residuals in drinking water distribution systems.
- Gupta, I., Khitoliya, R.K. and Kumar, S., 2013. Study of Water Distribution Network Using EPANET., (9), Pp.58–61.
- Hutton, Haller & Bartram (2007). Global cost benefit analysis of water supply and sanitation, journal of water and health. 5, 481-502.
- Hunter, P.R., Macdonald, A.M. and Carter, R.C., 2010. Water Supply and Health. Plos Medicine, 7 (11).

- Ilesenim (2006) Pre-feasibility assessment of onsite and decentralized sanitation systems for new satellite settlements in Abuja, Nigeria. PhD Thesis. Hamburg University of Technology.
- IPMS 2005; ALABA PILOT LEARNING SITE DIAGNOSIS AND PROGRAM DESIGN, Alaba.
- ISO (2003). Fire Supervision Rating Schedule.
- JALAL, M. M. (2008). *Performance Measurement Of Water Distribution Systems, Civil Engineering*. Toronto.
- Jeffrey A. Gilbert, P.E. (2012). Practical Design of Water Distribution System: Meadow Estates Drive.
- Joshi, M., Student, P.G. and Morbi, L.E.C., 2014. Design of Water Distribution Supply Network for Kuchhadi Village Assistant Engineer Narmda Water Resources Water Supply Depart-. , Pp. 94–97.
- KARAMOUZ.M, SZIDAROVSKY.F & ZAHRAIE.B (2003). *Water resources system analysis lewis publisher*. Boca Raton London, New York Washington, DC.
- Kaychamber, L. (2004). Design and operation of distribution networks.
- Khatrik & K, V. (2007). *Challenges for urban water supplies and sanitation in the developing countries. discussion draft paper for the session on urbanization delft*., Netherlands.
- Kingdom, B. (2006). The Challenge of Reducing Non Revenue Water (NRW) in Developing Countries, How the Private Sector Can Help. A Look at Performance-Based Service Contracting. *Water supply and sanitation board discussion paper series*., pp. 8, 40.
- Kimey, Hoko & Mashauri (2008) Performance of urban water supply utilities. Water Resource Engineering. Deressalaam, Deressalaam University.
- Korkeakoski (2006) A guide to sanitation and hygiene for those working in developing countries global dry toilet club.
- Laura Baumberger, Anderson and Jackwell (2007). Florida Water Resource Journal.
- Leirens, S. *et al.*, (2010). Coordination in Urban Water Supply Networks Using Distributed Model Predictive Control. American Control Conference (Acc), 2010, 19.
- Maher Abu-Madi & Trifunovic, (2013). *Impacts of supply duration on the design and performance of intermittent distribution systems in West Bank*.journal of water international, 38, 263-282.
- Mckenzie, Hamilton & Seago (2006) A review of performance indicators for real losses from water supply systems.

- McKibben. JALAL, M. M. (2008). *Performance Measurement of Water Distribution Systems, Civil Engineering*. Toronto (Tr) 21, 8, 2., (pp. pp. 54-66.)
- Mebet, A. (2007). Assessment of water loss in water supply network: a case of Debre Markos town. MSc Thesis. Arba Minch University, Arba Minch, Ethiopia.
- Ministry of Water Resources (2001). Ethiopian Water Sector Strategy, The Federal Democratic Republic of Ethiopia.
- Misirdali (2003) A methodology for calculating hydraulic system reliability of water distribution networks.
- Motiee, Emcbean & Amotiei (2007) Estimating physical unaccounted for water (UFW) in distribution networks using simulation models and GIS. *Urban Water Journal*, 4, 4342.
- MOWE (2012) Training Manual and Guideline for Performance Indicators and Benchmarking for Utilities, Water, Addis Ababa, Ethiopia: Federal Democratic Republic of Ethiopia Ministry of Water and Energy.
- MOWIE (2015) Federal Democratic Republic of Ethiopia Ministry of water irrigation and electric second growth and transformation national plan for the water supply and sanitation sub-sector (2015/16 - 2019/20).
- MOWR (2006). Urban water supply design criteria. IN ADMINISTRATION, W. R. (Ed.) Urban water supply and sanitation department. Addis Ababa, Ministry of Water Resource.
- National Meteorological Services Agency (NMSA) (2006). Report submitted to initial national communication of Ethiopia to the United Nations Framework Convention on Climate Change (UNFCCC), Addis Ababa, Ethiopia.
- Newbold, J.R., (2009). Comparison and simulation of a water distribution network in EPANET and a new Generic Graph Trace Analysis based model. *Environmental Engineering*. Blacksburg, VA.
- National Research Council (NRC). (2006). Drinking water distribution systems: Assessing and reducing risks. The national academics press, Washington, D.C.
- NUPI (1999). *Report On Development Plan of Halaba Kulito Town*. Addis Ababa.
- NYENDE-BYAKIKA, GADDI NGIRANE-KATASHAYE & M. NDAMBUKI, J. (2012). *Comparative analysis of approaches to modeling water distribution*. Civil Engineering and Environmental.
- OWWDSE (2010). Design Guideline for Water Supply Projects. FINFINE, GG Water Works Consultant.

- Paula (2000). Workshop on Hydraulic Network Modelling Using WaterCAD.
- PRASAD V.K.S.V. GOTTIPATI & V. NANDURI, U. (2014). Equity in water supply in intermittent water distribution networks. *Water and Environment Journal promoting sustainable solutions*.
- Pretoriou & Shutte (1997) *Water Demand and Population Growth*. 3, 127-133.
- Rao (2002) A guide to sanitation and hygiene for those working in developing countries global dry toilet club.
- Rossmann, L. (2000). *water supply and water resources division National Risk Management*, Research Laboratory Cincinnati. OH 45268.
- ROSSMAN.LA, BOULOS.PF & ALTMAN.T (2003). *The discrete volume element methods for modeling water quality in pipe networks*. Journal of water resource planning and management, vol.119, 56-67.
- Richard, G., K., A.nad L., R 2000.Monitoring and managing Unaccounted For Water Analysis.
- Sharma, 2008. Performance Indicators of water Losses in Distribution System, Delft: Netherlands: UNESCO-IHE.
- Shimeles, K. (2011). Water Supply Coverage and Water Loss in Distribution System with Mod. The Case Study of Addis Ababa.
- SNNPRS, 2011. *Potentials of Halaba Special Woreda*, SNNPRS Investment Expansion Core Process.
- Swamnee K. (2008) Design of Water Supply Pipe Networks, New Jersey, America: John Wiley and Sons, Inc.
- TABESH, M. JAMASB & R. MOEINI (2011). *Calibration of water distribution hydraulic model, a comparison between pressure dependent and demand driven analysis*, Urban water journal, no.8, 93-102.
- Tabeshe & A. Dolakhahi (2006). *Effect of pressure assessment of distribution network*. Iranian Journal of Science and Technology, vol.130, 119-127.
- Tomas, M.W., Donald, V.C., Dragan, A.S., Walter, G., Stephen, B. & Edmundo, K. (2003). (1<sup>st</sup> ed.) *Advanced water distribution modeling and management*: Haestad Press, USA Published.
- Trifunovic, N. 2006., reprint 2008. *Introduction to Urban Water Distribution*, 509 p., Taylor & Francis Group, London, UK.
- Twort, A., Crowley, D., & Ratnayaka. (2000). water supply hand book. United State Of Department Of Energy, 2005. Energy Tips – Pumping Systems., (October), Pp.1–2.

- UNEP. (200). Malta Water Service Corporation, Leakage Control and Unaccounted For Water Analysis.
- Venkateswara P., 2005. Vocational Course First Year Water Supply 1st Ed., New Delhi: India:Telugu Akademi.
- Wallingford (2003). *Hand book for assessment of catchment water demand and use.*
- Walski Hasted (2003). *Advanced Water Distribution Modeling and Management.*
- Walski, Thomas M. (2007). *Advanced Water Distribution Modeling and Management.* Bentley Institute Press. Exton, Pennsylvania.
- Walski, Donald V. Chase, Dragan A. Savic, Walter Grayman, Stephen Beckwith and Edmundo Koelle (2003). *Advanced Water Distribution Network Modelling and management* (first edition, second printing).
- WHO (2006): Guidelines for drinking water quality.3rd Edn.WHO, Geneva.
- WHO (2008). Guidelines for Drinking-Water Quality, incorporating the first and second addenda, World Health Organization, Geneva Switzerland. Volume 1.
- WHO (2009). Water System Design Manual N. Feagin Et Al., Eds., Washington:Amarica.
- WHO & UNICEF, (2012) Estimates for the use of improved drinking water sources. Ethiopia. Updated.
- World Bank Institute, Roland Liemberger, (2006). *Training Module 6, Performance Indicators.*
- World Bank (2012). Rural water supply design manual. Manila, Wter Partnership Program.
- Yazdani, P. Jeffrey (2011). Complex network analysis of water distribution systems. *Chaos; An interdisciplinary journal of non-linear time series methods,*
- Zyoud, S.H.A.R. (2003). *Hydraulic Performance of Palestinian Water Distribution System.* Nablus, An-Najah National University.3.4 Study period and design.

## APPENDIXES

### Appendix 1.1 Steady state simulation at 0:0-hour Analysis Table for pipes (Links)

Flex Table: Pipe Table  
Current Time: 0.00 hours

Label	Diameter (mm)	Length (m)	Material	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
P-36	50.0	390	PVC	0.04	0.02	0.000
P-41	75.0	589	PVC	0.22	0.05	0.000
P-55	75.0	1,087	PVC	-0.34	0.08	0.000
P-59	75.0	259	PVC	0.48	0.11	0.000
P-33	75.0	568	PVC	-0.49	0.11	0.000
P-75	75.0	359	PVC	0.50	0.11	0.000
P-58	150.0	351	DCI	-2.83	0.16	0.000
P-17	63.0	758	HDPE	0.50	0.16	0.001
P-96	63.0	720	HDPE	0.51	0.16	0.001
P-16	100.0	369	PVC	1.42	0.18	0.000
P-53	63.0	759	HDPE	-0.58	0.19	0.001
P-53	75.0	489	PVC	0.85	0.19	0.001
P-33	50.0	1,289	HDPE	0.45	0.23	0.002
P-49	63.0	489	HDPE	-0.85	0.27	0.002
P-27	50.0	537	DCI	0.58	0.30	0.003
P-40	250.0	1,856	DCI	15.45	0.31	0.000
P-21	50.0	493	PVC	0.65	0.33	0.003
P-27	50.0	212	PVC	-0.71	0.36	0.003
P-44	63.0	358	HDPE	-1.14	0.37	0.003
P-46	75.0	658	DCI	-1.62	0.37	0.003
P-48	63.0	839	HDPE	1.17	0.38	0.003
P-52	75.0	789	HDPE	1.70	0.38	0.003
P-89	50.0	572	HDPE	-0.76	0.39	0.005
P-45	50.0	589	PVC	-0.77	0.39	0.005
P-95	63.0	801	HDPE	-1.25	0.40	0.004
P-22	50.0	752	DCI	0.80	0.41	0.005
P-20	50.0	551	PVC	-0.82	0.42	0.005
P-85	63.0	259	HDPE	1.33	0.43	0.004
P-26	100.0	885	HDPE	3.36	0.43	0.003
P-10	50.0	679	DCI	-0.85	0.43	0.006
P-69	63.0	741	HDPE	-1.39	0.45	0.005
P-6	100.0	350	DCI	3.54	0.45	0.003
P-76	100.0	554	HDPE	-3.68	0.47	0.003
P-28	63.0	489	HDPE	-1.47	0.47	0.005
P-24	50.0	966	DCI	0.94	0.48	0.007
P-35	63.0	758	HDPE	1.86	0.60	0.008
P-32	50.0	297	PVC	1.30	0.66	0.013
P-30	50.0	632	HDPE	-1.33	0.68	0.013
P-29	63.0	313	HDPE	2.11	0.68	0.010
P-47	63.0	652	HDPE	-2.12	0.68	0.010
P-5	250.0	1,010	UPVC	-33.45	0.68	0.002
P-43	50.0	627	DCI	1.38	0.70	0.014
P-51	63.0	268	HDPE	2.20	0.71	0.011
P-22	50.0	486	HDPE	1.39	0.71	0.014
P-25	50.0	735	PVC	1.40	0.71	0.011
P-24	50.0	499	PVC	1.41	0.72	0.015
P-41	100.0	280	UPVC	5.72	0.73	0.007
P-54	63.0	895	HDPE	2.33	0.75	0.012
P-31	75.0	363	PVC	3.38	0.76	0.010
P-15	100.0	1,578	DCI	6.00	0.76	0.007

P-21	50.0	741	HDPE	-1.60	0.82	0.019
P-3	150.0	355	DCI	14.62	0.83	0.005
P-4	150.0	1,765	DCI	14.62	0.83	0.005
P-15	50.0	488	PVC	1.63	0.83	0.019
P-30	63.0	368	HDPE	2.61	0.84	0.015
P-50	63.0	587	HDPE	-2.74	0.88	0.016
P-42	150.0	1,830	DCI	15.76	0.89	0.006
P-65	150.0	287	DCI	-15.76	0.89	0.006
P-68	50.0	358	HDPE	1.83	0.93	0.024
P-9	250.0	108	UPVC	46.66	0.95	0.004
P-78	50.0	601	PVC	1.88	0.96	0.025
P-14	50.0	736	PVC	1.96	1.00	0.021
P-26	50.0	299	PVC	1.97	1.00	0.028
P-23	50.0	349	HDPE	2.04	1.04	0.029
P-92	50.0	459	PVC	-2.05	1.04	0.030
P-38	250.0	789	UPVC	53.13	1.08	0.005
P-65	75.0	278	HDPE	-4.91	1.11	0.021
P-14	100.0	502	DCI	8.74	1.11	0.015
P-37	150.0	1,256	UPVC	22.36	1.27	0.012
P-11	150.0	489	PVC	23.13	1.31	0.010
P-3	150.0	189	DCI	23.66	1.34	0.013
P-8	150.0	458	DCI	23.66	1.34	0.013
P-34	50.0	238	HDPE	-2.66	1.35	0.048
P-39	250.0	799	UPVC	69.15	1.41	0.008
P-19	75.0	949	PVC	6.29	1.42	0.033
P-61	50.0	618	DCI	2.83	1.44	0.054
P-17	50.0	326	HDPE	3.02	1.54	0.061
P-2	100.0	857	UPVC	14.28	1.82	0.037
P-18	50.0	352	PVC	-3.81	1.94	0.072
P-74	75.0	547	DCI	9.25	2.09	0.067
P-45	100.0	478	HDPE	22.62	2.88	0.086
P-1	63.0	456	HDPE	-15.45	4.96	0.405

Halaba Town Water supply-Analysis-Final  
 Edited.wtg  
 2020-06-22

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

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

## Appendix 1.2 Steady state simulation at 0:0-hour Analysis Table for Junctions (Nodes)

Flex Table: Junction Table  
 Current Time: 0.00 hours

Label	X (m)	Y (m)	Elevation (m)	Demand (L/s)	Pressure (m H2O)	Pressure Head (m)
J-54	401,187.32	808,581.64	1,767.00	0.45	-12	-12.27
J-68	398,335.35	806,271.49	1,765.00	0.85	-12	-12.00
J-52	401,175.41	808,010.06	1,771.00	0.56	-10	-10.18
J-59	398,257.95	806,771.63	1,768.00	0.85	-10	-9.70
J-64	400,425.21	806,646.59	1,768.00	0.80	-9	-9.03
J-66	401,145.64	807,408.71	1,772.00	0.50	-8	-8.46
J-53	401,228.99	808,301.81	1,765.00	0.85	-8	-7.96
J-20	400,924.21	807,986.33	1,780.19	0.85	-8	-7.75
J-69	398,371.82	806,097.34	1,760.00	0.85	-7	-7.40
J-67	398,216.27	806,473.93	1,762.00	0.50	-7	-6.71
J-25	399,670.33	806,482.30	1,765.71	0.80	-7	-6.60
J-58	398,335.35	807,039.56	1,766.00	0.50	-7	-6.59
J-62	399,645.23	806,158.37	1,764.00	0.85	-6	-6.34
J-63	400,175.14	806,396.53	1,764.00	0.80	-6	-6.18
J-60	398,460.39	806,563.24	1,764.00	0.80	-6	-5.77
J-24	400,079.68	806,806.41	1,766.65	0.80	-5	-4.81
J-22	399,278.82	806,508.46	1,763.85	0.50	-5	-4.73
J-61	398,948.61	806,241.72	1,761.00	0.56	-3	-2.81

J-50	400,222.77	807,617.10	1,773.96	0.50	-2	-2.22
J-65	400,901.52	806,825.21	1,763.00	0.85	-1	-1.20
J-51	401,109.91	807,759.99	1,768.00	0.75	-1	-1.09
J-49	399,746.45	807,176.50	1,769.30	0.85	0	-0.09
J-21	400,262.91	809,263.10	1,786.40	0.80	1	1.30
J-3	399,708.66	808,201.58	1,776.31	0.85	1	1.34
J-23	400,606.69	807,229.27	1,768.81	0.50	1	1.39
J-12	399,198.37	808,689.10	1,788.76	1.30	2	1.56
J-19	400,184.05	807,947.16	1,778.49	0.80	2	2.16
J-46	400,455.81	808,636.60	1,781.58	0.85	3	2.54
J-18	400,015.67	807,839.03	1,776.82	0.85	3	3.16
J-48	399,085.56	806,724.00	1,762.64	0.50	3	3.50
J-14	398,900.55	806,952.44	1,761.42	0.70	6	6.44
J-16	399,523.69	807,443.88	1,771.15	0.85	8	8.25
J-17	399,786.04	807,654.18	1,774.17	0.80	9	8.99
J-2	400,037.07	808,863.14	1,782.30	0.80	9	9.29
J-9	398,559.05	807,952.48	1,773.15	0.50	10	10.18
J-5	398,687.46	807,221.15	1,755.14	0.75	13	12.87
J-55	398,210.32	807,337.26	1,768.00	0.78	14	13.59
J-57	398,275.81	808,206.54	1,769.00	0.50	14	14.22
J-15	399,090.14	807,101.52	1,764.72	0.85	15	14.56
J-56	398,085.28	807,664.73	1,764.00	0.85	18	17.68
J-47	400,628.24	809,043.38	1,765.00	0.50	19	18.59
J-13	399,025.83	807,390.49	1,764.16	0.50	35	34.88
J-26	398,630.37	808,549.33	1,779.83	0.80	37	36.68
J-10	398,933.27	807,889.11	1,770.99	0.85	37	37.53
J-7	398,981.49	808,499.72	1,776.60	0.80	41	40.88
J-1	399,362.16	809,182.69	1,781.89	0.80	41	41.31
J-11	399,217.57	807,960.64	1,771.05	0.80	43	43.23
J-4	399,282.55	808,437.16	1,775.63	0.90	43	43.34
J-6	399,185.36	808,479.66	1,775.45	0.75	44	43.92
J-8	398,412.97	808,548.45	1,760.00	0.85	52	52.57

Halaba Town Water supply-Analysis-Final  
 Edited.wtg  
 2020-06-22

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

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

**Appendix 2.1** Extended period simulation for pipes (Links) at maximum consumption hour)

Flex Table: Pipe Table  
 Current Time: 8.00 hours

Label	Diameter (mm)	Length (m)	Material	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
P-37	150.0	1,256	UPVC	0.00	0.00	0.000
P-5	250.0	1,010	UPVC	0.00	0.00	0.000
P-42	150.0	1,830	DCI	0.00	0.00	0.000
P-65	150.0	287	DCI	0.00	0.00	0.000
P-41	75.0	589	PVC	-0.01	0.00	0.000
P-55	75.0	1,087	PVC	-0.19	0.04	0.000
P-21	50.0	493	PVC	0.09	0.05	0.000
P-16	100.0	369	PVC	0.44	0.06	0.000
P-74	75.0	547	DCI	-0.34	0.08	0.000
P-10	50.0	679	DCI	-0.16	0.08	0.000
P-69	63.0	741	HDPE	0.26	0.08	0.000
P-17	63.0	758	HDPE	0.40	0.13	0.000
P-3	150.0	189	DCI	2.48	0.14	0.000
P-8	150.0	458	DCI	2.48	0.14	0.000
P-53	75.0	489	PVC	0.72	0.16	0.001
P-58	150.0	351	DCI	-3.18	0.18	0.000
P-96	63.0	720	HDPE	0.61	0.19	0.001

P-22	50.0	752	DCI	0.40	0.20	0.001
P-59	75.0	259	PVC	0.69	0.22	0.001
P-36	50.0	390	PVC	-0.48	0.24	0.002
P-75	75.0	359	PVC	0.80	0.26	0.002
P-27	50.0	537	DCI	0.55	0.28	0.003
P-33	75.0	568	PVC	-1.34	0.30	0.001
P-40	250.0	1,856	DCI	15.45	0.31	0.000
P-24	50.0	966	DCI	0.62	0.32	0.003
P-52	75.0	789	HDPE	1.44	0.33	0.002
P-85	63.0	259	HDPE	1.09	0.35	0.003
P-6	100.0	350	DCI	2.87	0.37	0.002
P-33	50.0	1,289	HDPE	0.72	0.37	0.004
P-46	75.0	658	DCI	-1.73	0.39	0.003
P-9	250.0	108	UPVC	20.73	0.42	0.001
P-53	63.0	759	HDPE	-1.34	0.43	0.004
P-29	63.0	313	HDPE	1.36	0.44	0.005
P-76	100.0	554	HDPE	-3.58	0.46	0.003
P-44	63.0	358	HDPE	-1.47	0.47	0.005
P-45	50.0	589	PVC	-0.93	0.48	0.007
P-49	63.0	489	HDPE	-1.52	0.49	0.005
P-89	50.0	572	HDPE	-0.96	0.49	0.007
P-38	250.0	789	UPVC	26.79	0.55	0.001
P-95	63.0	801	HDPE	-1.74	0.56	0.007
P-48	63.0	839	HDPE	1.82	0.58	0.008
P-54	63.0	895	HDPE	1.88	0.60	0.008
P-41	100.0	280	UPVC	4.86	0.62	0.005
P-45	100.0	478	HDPE	4.93	0.63	0.005
P-26	100.0	885	HDPE	5.03	0.64	0.005
P-43	50.0	627	DCI	1.27	0.65	0.012
P-51	63.0	268	HDPE	2.24	0.72	0.011
P-22	50.0	486	HDPE	1.44	0.73	0.015
P-32	50.0	297	PVC	1.44	0.73	0.015
P-35	63.0	758	HDPE	2.34	0.75	0.012
P-24	50.0	499	PVC	1.49	0.76	0.016
P-28	63.0	489	HDPE	-2.39	0.77	0.013
P-21	50.0	741	HDPE	1.51	0.77	0.017
P-39	250.0	799	UPVC	38.62	0.79	0.003
P-11	150.0	489	PVC	14.40	0.82	0.005
P-14	50.0	736	PVC	1.60	0.82	0.019
P-25	50.0	735	PVC	1.61	0.82	0.019
P-15	50.0	488	PVC	1.61	0.82	0.019
P-3	150.0	355	DCI	14.63	0.83	0.005
P-4	150.0	1,765	DCI	14.63	0.83	0.005
P-30	50.0	632	HDPE	-1.69	0.86	0.021
P-50	63.0	587	HDPE	-2.82	0.90	0.017
P-68	50.0	358	HDPE	1.89	0.96	0.025
P-27	50.0	212	PVC	-1.89	0.96	0.026
P-78	50.0	601	PVC	1.92	0.98	0.026
P-92	50.0	459	PVC	-1.93	0.98	0.026
P-47	63.0	652	HDPE	-3.09	0.99	0.021
P-15	100.0	1,578	DCI	8.47	1.08	0.014
P-31	75.0	363	PVC	4.91	1.11	0.021
P-14	100.0	502	DCI	8.76	1.12	0.015
P-23	50.0	349	HDPE	2.21	1.13	0.034
P-30	63.0	368	HDPE	3.54	1.13	0.026
P-20	50.0	551	PVC	2.28	1.16	0.036
P-65	75.0	278	HDPE	-5.22	1.18	0.023
P-26	50.0	299	PVC	2.39	1.22	0.039
P-2	100.0	857	UPVC	9.93	1.26	0.019
P-17	50.0	326	HDPE	2.73	1.39	0.050

P-61	50.0	618	DCI	2.93	1.49	0.057
P-19	75.0	949	PVC	6.67	1.51	0.037
P-34	50.0	238	HDPE	-3.27	1.66	0.070
P-18	50.0	352	PVC	-3.53	1.80	0.081
P-1	63.0	456	HDPE	-15.45	4.96	0.405

Halaba Town Water supply-Analysis-Final  
 Edited.wtg  
 2020-06-22

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

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

**Appendix 2.2** Extended period simulation for Junctions (Nodes) at maximum consumption hour  
 Flex Table: Junction Table  
 Current Time: 8.00 hours

Label	X (m)	Y (m)	Elevation (m)	Demand (L/s)	Pressure (m H2O)	Pressure Head (m)
J-68	398,335.35	806,271.49	1,765.00	0.72	-4	-4.08
J-59	398,257.95	806,771.63	1,768.00	0.40	-3	-3.10
J-64	400,425.21	806,646.59	1,768.00	1.28	-2	-2.26
J-67	398,216.27	806,473.93	1,762.00	0.80	0	-0.46
J-58	398,335.35	807,039.56	1,766.00	0.80	0	-0.33
J-66	401,145.64	807,408.71	1,772.00	1.36	0	-0.20
J-62	399,645.23	806,158.37	1,764.00	0.80	0	-0.15
J-63	400,175.14	806,396.53	1,764.00	1.28	0	-0.12
J-60	398,460.39	806,563.24	1,764.00	1.28	1	0.57
J-69	398,371.82	806,097.34	1,760.00	0.72	1	0.63
J-25	399,670.33	806,482.30	1,765.71	0.48	2	2.33
J-22	399,278.82	806,508.46	1,763.85	0.40	3	3.41
J-61	398,948.61	806,241.72	1,761.00	0.90	4	3.57
J-54	401,187.32	808,581.64	1,767.00	0.72	4	4.29
J-24	400,079.68	806,806.41	1,766.65	0.40	6	5.55
J-52	401,175.41	808,010.06	1,771.00	0.90	6	6.10
J-65	400,901.52	806,825.21	1,763.00	0.80	7	6.82
J-53	401,228.99	808,301.81	1,765.00	0.72	8	7.53
J-20	400,924.21	807,986.33	1,780.19	0.16	10	9.78
J-49	399,746.45	807,176.50	1,769.30	1.36	11	11.11
J-48	399,085.56	806,724.00	1,762.64	0.40	12	12.06
J-51	401,109.91	807,759.99	1,768.00	1.20	12	12.26
J-50	400,222.77	807,617.10	1,773.96	0.80	13	12.67
J-14	398,900.55	806,952.44	1,761.42	0.80	13	13.33
J-9	398,559.05	807,952.48	1,773.15	0.48	15	15.46
J-23	400,606.69	807,229.27	1,768.81	0.80	16	16.43
J-55	398,210.32	807,337.26	1,768.00	1.25	19	18.92
J-57	398,275.81	808,206.54	1,769.00	0.80	19	19.00
J-5	398,687.46	807,221.15	1,755.14	1.20	20	19.63
J-16	399,523.69	807,443.88	1,771.15	1.36	21	21.14
J-18	400,015.67	807,839.03	1,776.82	0.86	22	21.58
J-46	400,455.81	808,636.60	1,781.58	0.16	22	22.07
J-56	398,085.28	807,664.73	1,764.00	0.40	23	23.03
J-21	400,262.91	809,263.10	1,786.40	0.40	24	24.10
J-19	400,184.05	807,947.16	1,778.49	0.40	25	25.32
J-17	399,786.04	807,654.18	1,774.17	0.40	26	25.65
J-15	399,090.14	807,101.52	1,764.72	0.80	27	26.76
J-2	400,037.07	808,863.14	1,782.30	0.40	29	28.85
J-47	400,628.24	809,043.38	1,765.00	0.40	38	38.44
J-13	399,025.83	807,390.49	1,764.16	0.80	44	43.70
J-26	398,630.37	808,549.33	1,779.83	0.40	44	44.31
J-1	399,362.16	809,182.69	1,781.89	1.28	45	45.39
J-10	398,933.27	807,889.11	1,770.99	1.36	46	46.11
J-3	399,708.66	808,201.58	1,776.31	0.80	47	47.34
J-7	398,981.49	808,499.72	1,776.60	0.72	48	48.19

J-4	399,282.55	808,437.16	1,775.63	1.39	50	50.48
J-6	399,185.36	808,479.66	1,775.45	1.20	51	50.75
J-11	399,217.57	807,960.64	1,771.05	0.93	52	52.52
J-8	398,412.97	808,548.45	1,760.00	0.16	64	63.96
J-12	399,198.37	808,689.10	1,788.76	2.48	387	387.82

Halaba Town Water supply-Analysis-Final  
 Edited.wtg  
 2020-06-22

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

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

### Appendix 2.3 Extended period simulation Table for pipes (Links) at minimum consumption hour

Flex Table: Pipe Table  
 Current Time: 1.00 hours

Label	Diameter (mm)	Length (m)	Material	Flow (L/s)	Velocity (m/s)	Headloss Gradient (m/m)
P-37	150.0	1,256	UPVC	0.00	0.00	0.000
P-5	250.0	1,010	UPVC	0.00	0.00	0.000
P-42	150.0	1,830	DCI	0.00	0.00	0.000
P-65	150.0	287	DCI	0.00	0.00	0.000
P-41	75.0	589	PVC	-0.01	0.00	0.000
P-55	75.0	1,087	PVC	-0.16	0.04	0.000
P-21	50.0	493	PVC	0.08	0.04	0.000
P-16	100.0	369	PVC	0.36	0.05	0.000
P-74	75.0	547	DCI	-0.28	0.06	0.000
P-10	50.0	679	DCI	-0.13	0.07	0.000
P-69	63.0	741	HDPE	0.21	0.07	0.000
P-17	63.0	758	HDPE	0.32	0.10	0.000
P-3	150.0	189	DCI	2.02	0.11	0.000
P-8	150.0	458	DCI	2.02	0.11	0.000
P-53	75.0	489	PVC	0.59	0.13	0.000
P-58	150.0	351	DCI	-2.58	0.15	0.000
P-96	63.0	720	HDPE	0.49	0.16	0.001
P-22	50.0	752	DCI	0.32	0.17	0.001
P-59	75.0	259	PVC	0.56	0.18	0.001
P-36	50.0	390	PVC	-0.39	0.20	0.001
P-75	75.0	359	PVC	0.65	0.21	0.001
P-27	50.0	537	DCI	0.45	0.23	0.002
P-33	75.0	568	PVC	-1.09	0.25	0.001
P-24	50.0	966	DCI	0.51	0.26	0.002
P-52	75.0	789	HDPE	1.17	0.26	0.001
P-85	63.0	259	HDPE	0.88	0.28	0.002
P-6	100.0	350	DCI	2.33	0.30	0.001
P-33	50.0	1,289	HDPE	0.59	0.30	0.003
P-40	250.0	1,856	DCI	15.51	0.32	0.000
P-46	75.0	658	DCI	-1.41	0.32	0.002
P-9	250.0	108	UPVC	16.84	0.34	0.001
P-53	63.0	759	HDPE	-1.09	0.35	0.003
P-29	63.0	313	HDPE	1.11	0.36	0.003
P-76	100.0	554	HDPE	-2.91	0.37	0.002
P-44	63.0	358	HDPE	-1.20	0.38	0.004
P-45	50.0	589	PVC	-0.76	0.39	0.005
P-49	63.0	489	HDPE	-1.23	0.40	0.004
P-89	50.0	572	HDPE	-0.78	0.40	0.005
P-38	250.0	789	UPVC	21.77	0.44	0.001
P-95	63.0	801	HDPE	-1.42	0.45	0.005
P-48	63.0	839	HDPE	1.48	0.47	0.005
P-54	63.0	895	HDPE	1.53	0.49	0.006
P-41	100.0	280	UPVC	3.95	0.50	0.003
P-45	100.0	478	HDPE	4.01	0.51	0.004

P-26	100.0	885	HDPE	4.09	0.52	0.004
P-43	50.0	627	DCI	1.03	0.53	0.008
P-51	63.0	268	HDPE	1.82	0.58	0.008
P-22	50.0	486	HDPE	1.17	0.59	0.010
P-32	50.0	297	PVC	1.17	0.60	0.010
P-35	63.0	758	HDPE	1.90	0.61	0.008
P-24	50.0	499	PVC	1.21	0.62	0.011
P-28	63.0	489	HDPE	-1.94	0.62	0.009
P-21	50.0	741	HDPE	1.23	0.63	0.011
P-39	250.0	799	UPVC	31.38	0.64	0.002
P-11	150.0	489	PVC	11.70	0.66	0.004
P-14	50.0	736	PVC	1.30	0.66	0.013
P-25	50.0	735	PVC	1.30	0.66	0.013
P-15	50.0	488	PVC	1.31	0.67	0.013
P-30	50.0	632	HDPE	-1.37	0.70	0.014
P-50	63.0	587	HDPE	-2.29	0.73	0.012
P-68	50.0	358	HDPE	1.53	0.78	0.017
P-27	50.0	212	PVC	-1.54	0.78	0.017
P-78	50.0	601	PVC	1.56	0.79	0.018
P-92	50.0	459	PVC	-1.57	0.80	0.018
P-47	63.0	652	HDPE	-2.51	0.81	0.014
P-3	150.0	355	DCI	14.66	0.83	0.005
P-4	150.0	1,765	DCI	14.66	0.83	0.005
P-15	100.0	1,578	DCI	6.88	0.88	0.010
P-31	75.0	363	PVC	3.99	0.90	0.014
P-14	100.0	502	DCI	7.12	0.91	0.010
P-23	50.0	349	HDPE	1.80	0.91	0.023
P-30	63.0	368	HDPE	2.87	0.92	0.018
P-20	50.0	551	PVC	1.85	0.94	0.025
P-65	75.0	278	HDPE	-4.25	0.96	0.016
P-26	50.0	299	PVC	1.94	0.99	0.027
P-2	100.0	857	UPVC	8.07	1.03	0.013
P-17	50.0	326	HDPE	2.22	1.13	0.034
P-61	50.0	618	DCI	2.38	1.21	0.039
P-19	75.0	949	PVC	5.42	1.23	0.025
P-34	50.0	238	HDPE	-2.65	1.35	0.048
P-18	50.0	352	PVC	-2.87	1.46	0.055
P-1	63.0	456	HDPE	-15.51	4.97	0.408

Halaba Town Water supply-Analysis-Final  
 Edited.wtg  
 2020-06-22

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

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

**Appendix 2.4** Extended period state simulation for Junctions (Nodes) at minimum consumption hour

Flex Table: Junction Table

Current Time: 1.00 hours

Label	X (m)	Y (m)	Elevation (m)	Demand (L/s)	Pressure (m H2O)	Pressure Head (m)
J-59	398,257.95	806,771.63	1,768.00	0.32	16	15.82
J-68	398,335.35	806,271.49	1,765.00	0.59	16	16.11
J-64	400,425.21	806,646.59	1,768.00	1.04	16	16.40
J-66	401,145.64	807,408.71	1,772.00	1.10	16	16.52
J-58	398,335.35	807,039.56	1,766.00	0.65	18	18.35
J-62	399,645.23	806,158.37	1,764.00	0.65	19	19.10
J-63	400,175.14	806,396.53	1,764.00	1.04	19	19.13
J-67	398,216.27	806,473.93	1,762.00	0.65	19	19.53
J-60	398,460.39	806,563.24	1,764.00	1.04	20	19.60
J-25	399,670.33	806,482.30	1,765.71	0.39	20	20.25
J-20	400,924.21	807,986.33	1,780.19	0.13	21	20.70
J-69	398,371.82	806,097.34	1,760.00	0.59	21	20.91

J-52	401,175.41	808,010.06	1,771.00	0.73	21	21.13
J-54	401,187.32	808,581.64	1,767.00	0.59	21	21.17
J-22	399,278.82	806,508.46	1,763.85	0.32	22	21.58
J-24	400,079.68	806,806.41	1,766.65	0.32	22	22.15
J-61	398,948.61	806,241.72	1,761.00	0.73	23	22.60
J-53	401,228.99	808,301.81	1,765.00	0.59	24	24.01
J-65	400,901.52	806,825.21	1,763.00	0.65	24	24.17
J-50	400,222.77	807,617.10	1,773.96	0.65	25	24.66
J-49	399,746.45	807,176.50	1,769.30	1.10	25	25.08
J-51	401,109.91	807,759.99	1,768.00	0.98	26	26.28
J-9	398,559.05	807,952.48	1,773.15	0.39	27	26.81
J-48	399,085.56	806,724.00	1,762.64	0.32	28	27.86
J-21	400,262.91	809,263.10	1,786.40	0.32	28	28.47
J-46	400,455.81	808,636.60	1,781.58	0.13	29	28.62
J-23	400,606.69	807,229.27	1,768.81	0.65	29	28.86
J-14	398,900.55	806,952.44	1,761.42	0.65	29	29.11
J-18	400,015.67	807,839.03	1,776.82	0.70	30	29.81
J-57	398,275.81	808,206.54	1,769.00	0.65	30	30.55
J-55	398,210.32	807,337.26	1,768.00	1.01	31	30.81
J-16	399,523.69	807,443.88	1,771.15	1.10	31	31.32
J-19	400,184.05	807,947.16	1,778.49	0.32	32	31.82
J-2	400,037.07	808,863.14	1,782.30	0.32	33	33.01
J-17	399,786.04	807,654.18	1,774.17	0.32	33	33.42
J-56	398,085.28	807,664.73	1,764.00	0.32	35	34.89
J-5	398,687.46	807,221.15	1,755.14	0.98	35	35.40
J-15	399,090.14	807,101.52	1,764.72	0.65	37	37.20
J-26	398,630.37	808,549.33	1,779.83	0.32	44	44.32
J-1	399,362.16	809,182.69	1,781.89	1.04	44	44.40
J-47	400,628.24	809,043.38	1,765.00	0.32	45	45.06
J-3	399,708.66	808,201.58	1,776.31	0.65	47	47.51
J-7	398,981.49	808,499.72	1,776.60	0.59	48	47.99
J-10	398,933.27	807,889.11	1,770.99	1.10	48	48.37
J-13	399,025.83	807,390.49	1,764.16	0.65	49	48.90
J-4	399,282.55	808,437.16	1,775.63	1.13	50	49.87
J-6	399,185.36	808,479.66	1,775.45	0.98	50	50.10
J-11	399,217.57	807,960.64	1,771.05	0.75	53	52.71
J-8	398,412.97	808,548.45	1,760.00	0.13	64	64.03
J-12	399,198.37	808,689.10	1,788.76	2.01	389	389.29

Halaba Town Water supply-Analysis-Final  
 Edited.wtg  
 2020-06-22

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

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

### Appendix 3

#### Questionnaires for Domestic Household Survey in Halaba Kulito Town

My name is \_\_\_\_\_. I am assisting an on-going research by Bereket Kebede in partial fulfilment for his Master's degree at Hawassa University. We are talking to selected sample households in Halaba Kulito Town about the Urban water supply condition, hydraulic Performance of water supply and Water loss in distribution of the town. The information that will be collected from this questionnaires survey will be used for research purpose only. Please be honest and open-minded in your evaluations and opinions. All information obtained will be kept severely confidential. Your kind assistance is highly appreciated.

Questionnaire No: \_\_\_\_\_ Name of Interviewer: \_\_\_\_\_ Date of interview: \_\_\_\_\_

#### I. Personal Information of Respondents

1. Name of Kebeke/mender: \_\_\_\_\_

2. House No: \_\_\_\_\_

3. Sex: Male \_\_\_\_\_, Female \_\_\_\_\_

4. Age: Under 14 years \_\_\_\_\_, 15-39 years \_\_\_\_\_, 40-64years \_\_\_\_\_, above 65 years \_\_\_\_\_

5. Educational background: None \_\_\_\_\_, Read-Write \_\_\_\_\_, Elementary school \_\_\_\_\_, Secondary school \_\_\_\_\_, High School \_\_\_\_\_, College \_\_\_\_\_, Graduated \_\_\_\_\_, Higher education \_\_\_\_\_, Others \_\_\_\_\_,

6. Occupation: Government Sector \_\_\_\_\_, Private Sector \_\_\_\_\_, Retired \_\_\_\_\_, other (specify) \_\_\_\_\_

7. How many persons live in your household \_\_\_\_\_.

8. How long have you been living in this town? \_\_\_\_\_ years.

## Water Sources and Uses

1. What is the main water supply source for your household?

- A. Piped with Household taps                      D. River/pond  
B. Private dug well                                      E. Rainwater collection  
C. Private water seller                                      F. Other sources  
(specify)\_\_\_\_\_.

2. If it is Piped with Household taps, where is your source connected?

- A. In own house  
B. In own yard/plot  
C. In neighbor's house  
D. Other, specify\_\_\_\_\_.

3. Are you satisfied with the quantity of water you get from your piped/ improved source?

Yes                                      No

4. If your answer for question 3 is NO, what are the reasons?

- A. Low quantity of the water  
B. Low reliability of the scheme  
C. Long distance to the scheme  
D. Delay for maintenance  
E. Scheme is non-functional  
F. Other, specify\_\_\_\_\_.

5. How is the water pressure (speed of pouring, when fully opened) from the town water supply system at your tap?

- A. High (quick)                                      E. Low (slow)  
B. Generally high                                      F. Every time low  
C. Sometime high                                      G. Very low  
D. Sometime low                                      H. Every time no water

6. Does your household get steady supply of water (without interruptions) from the town water supply system?    (a) Yes      (b) No

7. If there are water supply interruptions, do you suggest the main reasons?

- A. Water source problem                      C. Reservoir fails

- B. Pipe break
- D. Pump failure
- E. Other/specify\_\_\_\_\_.

8. If pipes break, when do you think it happens?

- A. During road construction
- C. During electric line installation
- B. During telephone line installation
- D. During building construction
- E. When it gets old
- F. Other (specify)\_\_\_\_\_.

9. On average, how often do pipe water interruption?

- A. Once a day
- E. 4 to 5 days a week
- B. Twice a day
- F. 1 to 2 weeks a Month
- C. 1 to 2 days a week
- G. Other, specify\_\_\_\_\_.
- D. 2 to 3 days a week

10. In a condition where piped/ taps water is not available, what is your alternative source of water?

- A. Neighboring house
- C. Water vendors
- B. Pond/river
- D. Well/borehole
- E. Other, specify \_\_\_\_\_.

11. Is there water loss in distribution system in Halaba Kulito Town?

- Yes
- NO

a) Do you see Pipe breakage in distribution system?

- Yes
- NO

b) Did you see that the concerned body take measurement, when the water losses is occur? If you can Explain\_\_\_\_\_.