ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Hydraulic Performance of Gimbi Town Water Distribution Network

A Thesis in Water Supply and Environmental Engineering Stream

By

Lencho Legese

January, 2020

Addis Ababa, Ethiopia

A thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

The undersigned have examined the thesis entitled **'Hydraulic Performance of Gimbi Town Water Distribution Network'** presented by **Lencho Legese**, acandidate for the degree of Master of science and hereby certify that it is worthy of acceptance.

Dr. Fiseha Behulu	____________	____________
(Advisor)	Signature	Date
Dr. Daniel Fikresilase	____________	____________
(Internal examiner)	Signature	Date
Dr. Geremew Sahile	____________	____________
(External examiner)	Signature	Date
Dr. Mebruk Mohammed	____________	____________
(Chair person)	Signature	Date

DECLARATION

I confirm that research work titled “**Hydraulic Performance of Gimbi Town Water Distribution Network**” is my own work. The work has not been presented elsewhere. Where material has been used from other sources it has been properly acknowledged.

Lencho Legese

Signature ______

January, 2020

ABSTRACT

Assessment of hydraulic performance of water supply distribution system to address water distribution bottlenecks within an urban water supply system is important. This can be achieved through investigating the status of the existing distribution system of the network. The main objectives of this study is to assess sustainability and to identify the challenges of Gimbi town WDS. The major problem found in this area is bursting and leakage problem of the pipe system in the distribution network resulting the system to deliver insufficient amount of water to the consumer. To examine the hydraulic performance of the water distribution network, water GEMS modeling was used and the model result is compared with the allowable pressure value and velocity in the distribution system. From the hydraulic analysis all of the velocity in the system is less than 0.6 m/s. It is also identified that 48% of the Nodes in the distribution system has maximum pressure head above 70m and 1% of the Nodes has minimum pressure less than15m. From the detailed analysis, the estimated current (2019) maximum water demand of Gimbi town is 5351 m^3/d (62 l/s). However, the current (2019) existing surface horizontal centrifugal types of pumps has only maximum capacity 20 l/s. Further, the analyzed water losses result in Gimbi town indicates that about 55% of production is non-revenue water and 60% of non -revenue water is estimated as a real lose. In general, rising in water demand, small capacities of existing pump and large volume of water loss leads to intermittent water supply in Gimbi town. Hence, most of resident are not satisfied from the town water service. To solve this problem, it is significant to rehabilitate the old pipe from distribution network that are more vulnerable by the current pressure and providing pressure reducing valve in the link having high pressure in distribution network to minimize the pressure to an acceptable level that reduce water losses in the distribution network. Accordingly, installation of 4 PRV in the model at different location in the distribution network reduces the system pressure to an acceptable range of pressure value (15m-70 m). Providing water pump that deliver the current maximum day demand of the water (62 l/s) for the town solves water scarcity in the town. Finally, it was recommended for water utility to give more attention to water losses reduction policies and strategies to minimize water loss in the town.

Key Words: Hydraulic analysis, Water distribution network, WaterGEMS, Gimbi, Water loss.

ACKNOWLEDGEMENT

Frist of all I would like to express my deepest gratitude to Almighty God for giving me health and strength in all my study work.

Secondly, I would like to express my greatest thanks to Dr.Fiseha Behulu, Assistant Professor in Addis Ababa University Institute of Technology. Without his guidance and support this work would not have been possible. I wish to thank him for his valuable mentoring, encouragement and support in times when I was needed it.

Specific gratefulness is given to Ato Bikila (Manager and process owner of Gimbi town water service office) and all staffs of Gimbi town Water supply and Sewerage office, for providing me with the all necessary information, data and their valuable assistance in field data measurement.

Special thanks go to my family specially my mother and my father, for their unlimited support and encouragement that contribute great throughout my life.

Lastly, but not least, my very special thanks go to my wife, Sifan Dereje, as this is as much a result of my as her, and without her support and sacrifice I would not completed this thesis.

CONTENTS

LIST OF TABLE

LIST OF FIGURES

ABBREVIATIONS

AWWA ______________ American Water Works Association

CARL ________________ Current Annual Real Loss

EPS __________________ Extended Period Simulation

GWSSE _______________ Gimbi Water Supply and Sewerage Enterprise

HGL __________________ Hydraulic Grade line

ILI ____________________ Infrastructure leakage Index

MoWIE ________________ Ministry of Water, Irrigation and Electricity

NRW___________________ Non-revenue water

PRV ____________________ Pressure resisting valve

SIV _____________________ System Input Volume

UFW ___________________ Unaccounted for water

UARL __________________ Unavoidable Annual Real Losses

UNDP __________________ United Nations Development Program

UNICE __________________ United Nations Children's Educational Fund

WBI _____________________ World Bank Institute

WDN ____________________ Water Distribution Network

WDS ____________________ Water Distribution System

WHO ___________________ World Health Organization

1. INTRODUCTION

1.1. General

A water distribution network is an essential hydraulic infrastructure which is a part of the water supply system composed of a different set of pipes, hydraulic devices and storage reservoirs (Shinde, et al. 2018).

Hydraulic network analysis of water supply distribution system to address water distribution bottlenecks within an urban water supply system is important and this can be achieved through investigating the status of the existing distribution system of the network. Today, apart from supply and demand gap, water distribution modelling is a critical part of operating water distribution systems that are capable of serving communities reliably, efficiently, and safely, both now and in the future (Izinyon and Anyata 2011).

Therefore, analysis of a pipe network is essential to understand or evaluate a pipe network system that ensure sufficient pressure and flow at the point of supply within a range whereby the maximum pressure avoids pipe bursts and the minimum ensures that water is supplied at adequate flow rates for all expected demands.

The minimum pressure that should be observed at junctions throughout the system varies depending on the type of water consuming sector and regulations governing the distribution system, but a typical operating range is between 15-70m (MoWIE 2006). It is undesirable to have high pressures because it causes more leaks, breaks and causes water wastage and the pressure below the minimum will not afford sufficient amount water to the customer.

In water distribution network systems there are problems such as inequalities in service provision and water loss. These indicate that, water distribution inefficiencies to end users. Commonly, water loss in water supply systems ranges from 15% to 30% in the developed world but elsewhere it is likely to range from 30% to 60% (Bridges and MacDonald 1994). Since there exists water loss in distribution system, the loss has to be reduced, because revenue from water supply decreases and the water supply does not balance the demand.

Since the population in the Gimbi town is increasing and burden on the water distribution system, looking the hydraulic performance of the water distribution network of pipes is necessary to solve the problem of water distribution system.

1.2. Statement of the Problem

Gimbi town faces a serious deficit in the water supply due to increased population and expanded economic activities in and around the town. To overcome such issues, water supply projects were implemented in the town back dating to almost two decades and this newly implemented water supply project starts commissioning since October, 2018. However, the existing system is not sufficient to provide sustainable water to the town residents and this project has stopped its function more than two times due to high bursting of pipe in the main line. Especially, from the service reservoir to distribution pipe. According to the communities living around the area the pipe was burst with high sound and water drains on the road as well as to resident house in most of the area. After they have made many solutions, the water utility starts supplying water that are small in amount. But, still there are more leakage and pipe burst in the distribution system.

Generally, the basic hydraulic problem found in the distribution system is frequent pipe burst, high water leakage in the system, areas found at high elevation do not get water. Therefore, the current water supply service of the town doesn't satisfy demand for the whole town and this is why some people use springs and streams as water supply in the town (Figure1-1).

Figure 1-1: Existing situation of Gimbi town water supply

1.3. Research Objective

1.3.1. General Objective

The main objectives of this research is to assess the sustainability of Gimbi WDS, to identify the challenges (related to hydraulic performance) of WDS and to prepare guidance for the future WDS of the town.

1.3.2. Specific Objectives

The specific objectives of this study includes:-

- To examine water supply and demand the in the Gimbi town.
  - ✓ Service coverage of water supply in the town.
  - ✓ Consumption and accessibility states of the town water supply.
- To assess the major factors for water loss in the town.
- To suggest the improvement mechanisms that increase service capacity of WDN and sustainability of the town water supply system.

1.4. Research Questions

- What is water supply coverage in the town?
- What is per capita water consumption of the town?
- What is percentage of water losses and what is the major cause of water loss?
- What is the solution to satisfy the demand and what mechanism will protect the pipe from bursting?

1.5. Significance of the Study

The significance of the study is to identify the problem of Gimbi WDN and to suggest improvement mechanisms that improve hydraulic performance of the system based on model simulation results. The model can be used to solve ongoing problems, analyze proposed operational changes, and prepare for unusual events. By comparing model results with field operations, the operator can determine the cause of problems in the system and formulate solutions. In general, the research will be significant for Gimbi water supply and sanitation office to improve the performance of the existing subsystem and to reduce the deficit of supply.

2. LITRATURE REVIEW

2.1. The Demand for Public Water Supplies

Water is used for different purpose. based on their uses (Nemanja 2006) classified water supply demand in to domestic and non-domestic demand. According to him domestic demand is the water used within the household for drinking, personal hygiene cooking and cleaning, and non-domestic demand is industrial, commercial, institutional and agricultural demand legitimately drawn from the distribution mains.

2.2. Water Distribution System Component

Water distribution systems consist of a network of smaller pipes with numerous connections that supply water directly to the users (Walski, Chase et al. 2003) .In the water distribution system, piping system is often categorized as transmission/trunk mains and distribution mains (Benyam 2016). According to (Nemanja 2006) trunk main is a pipe for the transport of potable water from treatment plant to the distribution area depending on the maximum capacity and distribution mains convey water from the secondary mains towards various consumers. (Walski, Chase et al. 2003) also describe reservoir (clear water storage), pump and valves as water distribution system component in addition to the pipe. According to them clear water storage facilities are a part of any sizable water supply system and can be located at source (i.e. the treatment plant), at the end of the transport system or at any other favorable place in the distribution system, usually at higher elevations. They also describe the uses of pumps in the distribution system is to add energy to water and a valve as an element that can be opened and closed to different extents to vary its resistance to flow.

2.3. Methods of Water Distribution System

Water is dispersed throughout the distribution system in a number of different ways, depending on local conditions or on regulations and requirements that influence water system design. The common methods of water distribution system according to (E.Hickey 2008) is gravity distribution, distribution with the aid of pump and dual system.

Gravity distribution system: According to Hickey this is possible when the treated water source is a retention pond, clear well, or storage tank at some needed elevation above the community and in this type of system, sufficient pressure is available due to gravity to maintain water pressure in the mains for domestic consumption and fire service demand. He also states this method is the most reliable method of distribution if the piping leading from the treated water source to the community is adequate in size and safeguarded against accidental breaks.

Dual system: According to Hickey through the use of pumps and elevated storage, the excess water pumped during periods of low consumption is stored in elevated tanks or reservoirs. According to him during periods of high consumption, the stored water supplements the water that is being pumped. He also describe the flow rates and pressures throughout the water system in this method is uniform and this method generally economical because the pumps may be operated at their rated capacity.

Distribution with the aid of pumps: When stationary pumps are used to distribute water, and no storage is provided on the distribution system, the pumps force water at the required volume and pressure directly into the mains.

2.4. Hydraulic Analysis of Water Distribution System

Many of the scholars define what the hydraulic modeling mean. Among them (Thomas M. W alski, Donald V . Chase et al. 2003) defines hydraulic modeling as the process of building, verifying and operating network models of distribution systems, which provide valuable insights into operational practice . According to them water distribution modeling is a critical part of designing and operating water distribution systems that are capable of serving communities reliably, efficiently, and safely, both now and in the future.

The computation of flows and pressures in networks of pipes has been of great value and interest for those involved with the design, construction and maintenance of public water distribution systems and with the advent of computer models of water distribution systems, it has now become possible to analyze more complex network components (e.g. pumps, tanks, etc.) of the water distribution system as well as to investigate more complex issues associated with their design and operation (Ormsbell,2006).

The supply may be from reservoirs, storage tanks and / or pumps or specified as in flow or outflows at some points in the network and from the known flow rates the pressures or head losses through the system is computed (Izinyon and Anyata 2011). The continuity equation may be expressed as;

$$\sum Q_{in} - \sum Q_{out} = C_j \dots\dots\dots\dots\dots\dots\dots. 2.1$$

Where Q_{in} and Q_{out} are the flow rates into and out of the junctions, respectively, and C_j represents external consumption or input flow rates at the junction.

The energy principle provides that the head loss between any two points in the system is the algebraic sum of the head loss of all the elements along any route between the points and the total head loss is the same by all routes. The energy or loop equations are of the form

$$\sum h_{ij} = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 2.2$$

Additionally, for each loop, the head loss may be expressed by the power equation given as;

$$h_f = kQ^n \ldots\ldots\ldots\ldots\ldots\ldots . 2.3$$

The values of K and n depend on the friction head loss equation adopted.

To solve for the unknowns, the equations must be solved simultaneously but the direct solution of the large number of simultaneous equations involved in distribution systems is impracticable for all but the simplest systems, hence systematic methods which utilize computers are needed for solving this system of simultaneous equations (Izinyon and Anyata 2011).

According to (Bentley 2008) WaterGEMS is a software tool designed, developed and programmed by Haestad Methods Inc. of Cincinnati, Ohio, USA primarily for use in the modelling and analysis of water distribution systems and utilizes the gradient algorithm even during peak demand conditions and decreasing pipes sizes to determine the potential for cost savings without compromising velocity standards.

2.5. Model Skeletonization

Skeletonization is the process of selecting for inclusion in the model only the parts of the hydraulic network that have a significant impact on the behavior of the system (Rakesh Bahadur, Jeffrey Johnson et al. 2006). Skeletonized models of Water Distribution Networks (WDN) are generated by selecting and eliminating a certain number of pipes in such a way that the new model can still reproduce the prototype adequately (J.G. Saldarriaga April 2009). One reason for using simplified network representations of an all pipes network is to increase the computational speed for which results can be obtained when assessing intentional or unintentional attacks on a water distribution system (Lina Perelman, Morris L. Maslia et al. 2015).The need for network simplification through skeletonization and aggregation dates back to the days of manual methods such as the Hardy Cross procedure for balancing network flows (Cross, 1936). Skeletonization techniques have been developed based on hydraulic criteria that required the skeletonized or simplified system to resemble the configuration or connectivity of the original all pipes network and perform similarly in terms of hydraulic variables, i.e., pressures, water levels, pump operations, and demand thus distribution system modeling and analysis packages contain some type of skeletonization module or routine for rapid system simplification (Lina Perelman, Morris L. Maslia et al. 2015). In order to skeletonize the model to an acceptable degree, (Mahmoud A. Elsheikh, Hazem I. Saleh et al. 2013) reviewed the study adopted by (USEPA 2005) and use the fallowing conditions to skeletonize the network as guideline.

- At least 50% of total pipe length in the distribution network have to be included.
- At least 75% of the pipe volume in the distribution system have to be included.
- All 300 mm diameter and larger pipes in the distribution system have to be included.
- All 200 mm and larger pipes that connect pressure zones, influence zones from different sources, storage facilities, major demand areas, pumps, and control valves, or are known or expected to be significant conveyors of water have to be included.
- All 150 mm and larger pipes that connect remote areas of a distribution system to the main portion of the system have to be included.
- All storage facilities with controls or settings applied to govern the open/closed status of the facility that reflects standard operations;
- All active pump stations with realistic controls or settings applied to govern their on/off status that reflects standard operations.

2.6. Impact of Model skeletonization on Water Distribution Model Parameter

According to (Rakesh Bahadur, Jeffrey Johnson et al. 2006) model skeletonization has an impact on the velocity, water age and demand. According to them average velocity increases with each skeletonized model thus higher mean velocities in all the skeleton models with respect to the all pipe model, average water age is higher for the all pipe model and decreases as the skeleton level increases. They also state skeletonization of a distribution system model involves the re - allocation of system demands and consequently a change in flows and velocities within the model thus demand increases with an increasing level of model skeletonization but overall system demand must remain unchanged in skeleton models. And according to (Lina Perelman, Morris L. Maslia et al. 2015) simulated pressures for skeletonized networks were nearly equivalent to pressures simulated by the all pipes network thus model skeletonization has no more impact on pressure.

2.7. Applications of Water Distribution Models

(Thomas M. W alski, Donald V . Chase et al. 2003) describe the application of water distribution modeling are used for long-range master planning, fire protection studies, water quality investigations, energy management, system design and daily operational uses including operator training, emergency response, and troubleshooting.

2.8. Water Distribution Modeling Tools

The various modeling software are available in market are of freeware as well as commercial. The choice of design software is entirely depends upon the availability of the data, time, financial implications, resources, applicability, compatibility and overall purview of the project. The following are the basic tools for water distribution modeling (Nitin P. Sonaje and Joshi 2015).

WaterCAD V8i (2014) is a hydraulic modeling software package comprised of wide range of functionality includes graphical and profiling advancements, flexibility in data archiving and representations, advancements in Graphical User Interface and its customization, etc. Many features like hydraulic and water quality analysis, steady state and extended period simulations are also made to function with enhanced capabilities, strong data management along with AutoCAD and GIS integrations. According to Sonaje and Joshi the advantages of WaterCAD V8i over other softwares include simplified model building with geospatial modules and tools like Load Builder and TRex, water quality modeling, fire flow analysis, optimization and scenario management, etc.

WaterGEMS V8i (2014) is a versatile hydraulic modeling software package with the advancements in the interoperability, optimization of networks; model building supported with geospatial tools and asset management tools. According to Sonaje and Joshi this software is highly efficient and dynamic modeling software which provides the wide regime of analysis and solutions for fire-flow analysis, water quality modeling, energy and capital cost management, etc. Many of the features and functions are common in WaterCAD V8i and WaterGEMS V8i which are streamlined model building, integration with the GIS and AutoCAD functionalities, optimized model calibration, design and its operations. According to Sonaje and Joshi the best part in the WaterGEMS V8i is the presentation of obtained results which is very attractive and appealing and can be presented with variety of graphical tools include ArcMap visualization, thematic mapping, contouring, profiling with color coding and symbology. They also prove that WaterGEMS V8i is one of the most popular and user friendly hydraulic modeling and optimization software package that has strong design algorithm to meet the criteria of accuracy in design of water distribution networks, control of distribution network variables like flow, pressure, and velocity along with their optimization

DisNet (2014) is powerful and efficient water distribution software and offers great simplicity in building water distribution networks. Key strengths of DisNet include its simplicity and appealing user interface with maximum accuracy in output with optimum input details. According to Sonaje and Joshi it is used in the modeling of stream hydrology, generation of unit hydrographs and establishing interrelationships between them.

EPANET (2014) is public domain software which can be efficiently used to design any sort of network. According to Sonaje and Joshi this software provides variety of advantages like water quality analysis, extended period simulation, residual chlorine calculations for disinfection, etc.

HydrauliCAD (2014) is AutoCAD based water distribution software integrated with EPANET hydraulic analysis program. According to Sonaje and Joshi HydrauliCAD possesses a feature of building query for addition and editing of different hydraulic parameters like head-loss, pressure, flow of distribution networks. According to them HydrauliCAD provides inbuilt pipe catalogue comprises of detailed information about pipe material, classes and sizes. The also describe as it is used for fire-flow analysis.

WATSYS (2014) is water distribution simulation and modeling software based on Geographical Information System (GIS). According to Sonaje and Joshi this software is efficiently used for designing new water distribution as well as for up-gradation of existing distribution network. According to them it uses EPANET program as basis for water distribution and water quality analysis scenarios. They also state as it can easily integrated with AutoCAD to use CAD based drawings to develop and build hydraulic networks.

Pipe2014 is the recent version of KYPipe hydraulic modeling software package which has a strong computational algorithm for the fluids essentially water. According to Sonaje and Joshi this software can be used for the designing and selection of pumps, valves, tanks as well as pipes and it includes the features like sizing of pipes and optimization of pump operations. According to them it provides a very interactive and user friendly interface which offers extensive flexibility to users for the designing, optimization of distribution networks and it is compatible to integrate with GIS and variety of formats of images used for designing distribution networks.

Synergi Water (2014) is hydraulic modeling and simulation software package with the strong database management used for increasing the efficiency of existing distribution network as well as in the design and development of the newer one. According to Sonaje and Joshi Synergi Water provides the variety of advantages over other public domain softwares, as it provides versatile environment of tools for detailed and comprehensive modeling, performs speedy and accurate analysis of extremely large systems comprising of more than one lakh system components, water quality modeling and designing of complex systems with proper arrangements of pump, valves and tanks. They also state its integrity with the GIS and SCADA is extremely flexible which makes the remote operations simple and trouble free.

H2Onet and H2Omap (2015) are commercial softwares which are integrated with the GIS and used for design, analysis and optimization of different types of water distribution networks. According to Sonaje and Joshi it has advantageous for leakage detection and assessment, analysis of fire-flow and hydrant, cost optimization, etc. The most important feature of this software according to Sonaje and Joshi includes programmed and automated online SCADA interface, its integration with GIS software provides different vector and raster tools which supports wide range of spatial analysis, sampling, planning, evaluation and assessment of existing as well as newly developed water supply system.

HYDROFLO3 (2015) is advanced version of HYDROFLO series and used to design variety of the distribution systems includes pumped flow, gravity flow, flow through pipes as well as open channel flows. According to Sonaje and Joshi this software provides a unique feature as Pump base used for the calculation of pump hydraulic characteristics required in forced flow systems. According them it also used for simulation of treatment plants, chemical dosing systems, industrial applications, fire flow analysis.

2.9. Modeling a System Using WaterGEMS

WaterGEMS is hydraulic simulation software, distributed by Bentley Systems. According to (Bentley 2008) the parameters that need to be defined for each model components include:

- Nods: Elevations and the base demands
- Pipes: Pipe diameters, lengths and the friction coefficient factors. By default WaterGEMS considers the pipe material of having a Hazen William friction coefficient factor.
- Tanks: Base Elevation, the minimum and maximum levels, diameter of the tank
- Pumps: The most important parameter defining the pump operation is the pump curve, elevation of the pump, curve of the pump and its location.
- Reservoir: Elevation of reservoir, location of reservoir, its diameter, and etc.

According to (Bentley 2008) after all the parameters required to run the simulation are entered into the model, the successful simulation run provides solution for the Pressure at every single element in the system, flows at every point of time in the system , velocities in the pipes, levels in the tanks, Pump cycles, Water age and constituent concentration. Additionally (Bentley 2008) describe the capability waterGEMS software to perform the analysis of the system for the steady state scenarios and for an extended period of any length.

2.10. Types of Water Distribution Simulation

According to (Walski, Chase et al. 2003) the term simulation is the process of using a mathematical representation or real system, called a model. According to them there are two most basic types of simulations that a model may perform, depending on what the modeler is trying to observe or predict. These are Steady state simulation and extended period simulation (EPS).

According to (Bentley 2008) 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. Bentley also describe the 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. He also describe that the steady state 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. (Bentley 2008) also describe as the extended period simulation is appropriate when the variation of the system attributes over time.

(Walski, Chase et al. 2003) states simulation duration and hydraulic time step as follow.

- An extended-period simulation can be run for any length of time, depending on the purpose of the analysis. According to them 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.
- Hydraulic time step is important decision when running an extended period simulation and it is the length of time for one steady-state portion of an EPS, and it should be selected such that changes in system hydraulics from one increment to the next are gradual. They also states a time step, too large may cause abrupt hydraulic changes to occur, making it difficult for the model to give good results.

2.11. Principles of Network Hydraulics

In networks of interconnected hydraulic elements, every element is influenced by each of its neighbors; the entire system is interrelated in such a way that the condition of one element must be consistent with the condition of all other elements. According to (Thomas M. W alski, Donald V . Chase et al. 2003) the two basic equations that govern in WaterGEMS modeling network are:-

- Conservation of mass or continuity principle.
- Conservation of energy or energy principle.

2.11.1. Conservation of Mass

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

$$\sum(Q - U) = 0 \ldots\ldots\ldots\ldots\ldots\ldots.\ldots\ldots.2.4$$

Where

Q= Net flow into junction

U= Use at junction.

2.11.2. Conservation of Energy

The principle of conservation of energy dictates that the difference in energy between two points must be the same regardless of the path that is taken and for convenience within a hydraulic analysis, the equation is written in terms of head according to (Walski, Chase et al. 2003) which is stated in the equation below.

$$Z1 + \frac{p1}{\gamma} + \frac{v1^2}{2g} + \sum hp = Z2 + \frac{p2}{\gamma} + \frac{v2^2}{2g} + \sum hl + \sum hm \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 2.5$$

Where

Z= elevation (L)

P= pressure ($M/L/T^2$)

γ = fluid specific weight ($M/L^2/T^2$)

V= velocity (L/T)

g = gravitational acceleration constant (L/T2)

h_P = head added at pumps (L)

h_L= head loss in pipes (L)

h_m= head loss due to minor losses (L)

Thus the difference in energy at any two points connected in a network is equal to the energy gains from pumps and energy losses in pipes and fittings that occur in the path between them (Walski, Chase et al. 2003).

2.12. Water Loss Analysis

Annual water balance is one of the method used to assess Non-Revenue Water (NRW) and its components (Lambert 2003). The fallowing are basic principal of water balance component defined according to (Lambert 2003) and (José Sardinha 2017).

System Input Volume (SIV) - annual volume of water entering the distribution system.

Authorized Consumption - annual volume of water metered or unmetered but actually consumed by clients, the supplier itself or by those who are implicitly or explicitly authorized to consume water, such as social commitments and legitimate use by the fire service. In addition, the volume of water that is exported is also included and existing leaks present on client service connections.

Non-Revenue Water (NRW) - is the difference between the volume of water introduced into the system and the authorized consumption that is actually billed.

Water Losses – the difference between the volumes of water introduced into the system and authorized consumption, representing both real and apparent losses;

Apparent Losses – corresponds to theft and illicit consumption and can be estimated by checking the number of illegal connections, number of defective meters and using estimates of per capita consumption used to calculate the volume;

Real Losses – volume that is lost annually through all types of leaks, bursts and leaks in pipe lines, reservoirs and service connections to the point of the client meter.

Table 2-1: Water balance approach

System Input Volume	Authorized Consumption	Billed Authorized consumption	Billed metered consumption	Revenue water
			Billed unmetered consumption	
		Unbilled Authorized consumption	Unbilled metered consumption	Non-Revenue water
			Unbilled unmetered consumption	
	Water Losses	Apparent losses	Un authorized consumption	
			Metering in accuracy and data handling error	
		Real Losses	Leakage on transmission mains and Distribution main	
			Leakage and overflows at utility's storage tanks	
			Leakage on service connection up to point of customer metering	

Source: AWWA (2004)

NRW includes all losses, real and perceived.

$$(\text{NRW}) = \text{Water Losses} + \text{Authorized Unbilled Consumption} \ldots\ldots\ldots\ldots\ldots 2.6$$

$$\text{Water Losses} = \text{Real Losses} + \text{Apparent Losses} \dots\dots\dots\dots\dots\dots\dots 2.7$$

The NRW calculation may be presented by volume, as indicated above, or, as often happens, in percentages:

$$\%\ \text{NRW} = \frac{\text{Unbilled Volume}}{\text{System Input}}\, x100 \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots 2.8$$

2.13. Approaches of Determining Losses

2.13.1. Top-down Approach

According to (José Sardinha 2017) top-down approach is the approach starts the process with all available information, usually being undertaken on paper or as a desktop exercise, in which no field work but a number of estimates are made. According to him in this approach, losses are calculated from metering of the various system inputs, minus values obtained by client billing systems.

According to (Mamade, Loureiro et al. 2018) top-down approach requires minimum data (i.e., no hydraulic model is needed) and the required data include inlet water volumes and hydraulic heads at delivery points, storage tanks and pumping stations, and electric energy consumption in each pumping station for a preliminary calculation of the pumping station’s efficiency.

2.13.2. Bottom-Up Approach

According to (José Sardinha 2017) Bottom-Up Approach applied in cases of sectorised systems and equipped with continuous monitoring, means it is possible to calculate the volume of real losses from night flow values.

According to (Mamade, Loureiro et al. 2018) to adopt this approach, the network should be progressive and properly structured and equipped in accordance with its normal operation, being analyzed through permanently sectored District Meter Areas (DMA).

2.14. Water loss Performance Indicator Assessment

2.14.1. Infrastructure Leakage Index

According to (Vermersch, Carteado et al. 2016) it is defined as the ratio of the current annual real losses (CARL) to the unavoidable (technical minimum) annual real losses (UARL).

$$\text{ILI} = \frac{\text{CARL}}{UARL} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots 2.9$$

Where: -

- CARL - Is current annual real los
- UARL-Un avoidable real los

2.14.2. Unavoidable Annual Real Losses (URL)

Real losses cannot be eliminated totally. The lowest technically achievable annual volume of Real Losses for well-maintained and well-managed systems is known as Unavoidable Annual Real Losses (UARL) and the general equation for UARL is (Vermersch, Carteado et al. 2016) written in the equation below.

$$\text{UARL}\left(\frac{\text{L}}{\text{day}}\right) == (18 \text{ x Lm} + 0.80 \text{ x Nc} + 25 \text{ x Lp})\text{x P} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. \quad 2.10$$

Where

Lm = Length of mains (km)

Nc = Number of service connections

Lp = Total length (km) of underground connection pipes (between the edge of the street and customer meters)

P = Average operating pressure (m)

2.15. Commercial Loss Management Strategies

According to (Vermersch, Carteado et al. 2016) commercial losses can be broken down into customer meter inaccuracy**,** unauthorized consumption**,** meter reading errors and data handling errors. They also state commercial losses that are more than 4-6% of authorized consumption needs sustained management commitment, political will, and community support.

2.15.1. How to Address Customer Meter Inaccuracy

Inaccurate meters tend to under-register water consumption leading to reduced sales and therefore reduced revenue. (Vermersch, Carteado et al. 2016) state the common problems with customer meter accuracies and solutions for utilities are installing meter properly, monitoring water quality, monitoring intermittent water supply, sizing meters properly, using the appropriate class and type of meter, maintaining and replacing meters properly.

According to (Vermersch, Carteado et al. 2016) meters should be installed properly according to the manufacturer's specifications and should also be installed where meter readers can easily read them, and where it is easy to identify each property's meter. In addition, they also suggest management and staff responsible for meter installations should be trained on proper handling of meters.

According to (Vermersch, Carteado et al. 2016) poor water quality resulting from poor raw water, inadequate treatment processes, or dirt infiltration due to pipe shutdowns may cause sediments to form in the pipes and these sediments can also build up on the internal parts of meters, especially mechanical meters. Thus, utilities must regularly monitor water quality and clean mechanical meters to minimize sediment levels and promote accurate meter measurements.

According to (Vermersch, Carteado et al. 2016) where water supply is intermittent, customer meters will register a certain volume of air when the water supply is first turned on and the sudden large increase in pressure can damage the meter's components. Thus, intermittent supply should be avoided for a number of reasons, including the negative impact on customer meter accuracy.

According to (Vermersch, Carteado et al. 2016) customer meters work within a defined flow range, with the maximum and minimum flows specified by each manufacturer and large meters will not register low flows when the flow rate is lower than the specified minimum. Therefore, utilities should conduct customer surveys to understand the nature of each customer's water demand and their likely consumption to determine the proper meter size for households and businesses.

(Vermersch, Carteado et al. 2016) also explain how to choosing the appropriate meter to ensure the accuracy of customer consumption data. According to them Class B meters are a good choice where water quality is low, Class D meters are more preferable where roof tanks are used and water quality is good, Class C meters are a suitable compromise in most situations, since they can measure low flows better than Class B meters and are not as expensive as Class D meters. According to them the most common type of meter for domestic and small commercial installations is the 15 mm and 20 mm positive displacement (PD) meter, Single-jet and multi-jet meters for small commercial and industrial installations that require 20 mm to 50 mm sizes and Electromagnetic meters are the best choice for sizes 100 mm and above.

(Vermersch, Carteado et al. 2016) also state how the utility should replace the meters systematically, beginning with the oldest meters and those in the worst condition. According to them poor maintenance will not only encourage inaccuracy but may shorten the life span of the meter and a scheduled maintenance as well as replacement program should be in place to manage this problem.

2.15.2. Unauthorized Consumption

Unauthorized consumption includes illegal connections, meter bypassing, illegal use of hydrants, and poor billing collection systems. The common problems and possible solutions according to (Vermersch, Carteado et al. 2016) are finding and reducing illegal connections and avoiding corrupt meter readers.

(Vermersch, Carteado et al. 2016) defines illegal connections as the physical installation of a connection to water distribution pipelines without the knowledge and approval of the water utility and customer awareness programs, customers should be encouraged to report illegal connections, and regulations should be in place to penalize the water thieves.

2.15.3. Meter Reading Errors

Errors can be easily introduced through negligence, aging meters, or even corruption during the process of reading the meters and billing customers (GAWP 2016). According to (GAWP 2016) incompetent or inexperienced meter readers may read the meter incorrectly or make simple errors, such as placing a decimal in the wrong place and dirty dials, faulty meters, and jammed meters can also contribute to meter reading errors.

2.15.4. Data Handling and Accounting Errors

(GAWP 2016)defines data handling errors as the error due to the meter reader writes down incorrect data, the billing department transfers incorrect data into the billing system. According to (Vermersch, Carteado et al. 2016) training of meter readers promotes diligence, good customer meter maintenance, and decreased meter reading errors and also if financially viable, utilities should consider electronic meter-reading devices, which reduce data handling errors to a minimum since all data transfers to the billing system are done electronically.

2.16. Physical Loss Management Strategies

Physical losses, sometimes called 'real losses' or 'leakage', includes the total volume of water losses minus commercial losses. However, the water balance process indicates that commercial losses are estimated and therefore the resulting leakage volume may be incorrect.

The three main components of physical losses according to (Vermersch, Carteado et al. 2016) include :-

- Leakage from treatment plant, transmission and distribution mains
- Leakage and overflows from the service reservoirs and storage tanks
- Leakage on service connections up to the customer's meter

The assessment and management of real losses contains so many elements that it is useful to consider the simplified overview is shown in Figure 2-1 taken from (AWWA 2014).

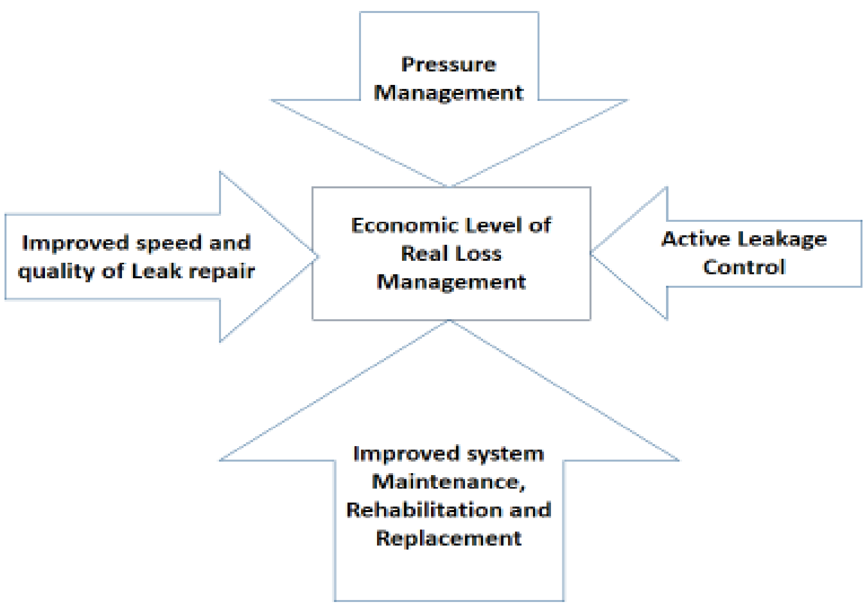


Figure 2-1: Real Losses Dynamic Scheme (Four Component Approach)

Source:(AWWA 2014)

2.16.1. Pressure Control

Pressure can affect the level of leakage from a system in several ways(GAWP 2016). According to (GAWP 2016) reducing system pressures wherever possible has advantageous and pressure is reduced by pressure zoning, reducing pump heads and pressure reducing valve.

Pressure zoning: isolating areas supplied by gravity from existing service reservoirs, and preventing flow to lower zones except with pressure reduction or under emergency conditions

Reduced pumping heads: where booster pumps are pumping to excessive pressures, reduction of pressure should reduce flow and also the cost of pumping. Pumps maybe automatically controlled to pump at lower pressure at night.

Pressure reducing valves: there are varieties to types: outlets pressures may be fixed; may be a proportion of inlet pressure, or may be varied at different times of the day. However, they require regular and specialized maintenance, careful design and sitting, and may require an air- valve downstream.

2.16.2. Active Leakage Control and Timely Leak Repair Programs

According to (GAWP 2016) examining the potential causes, evaluating potential activities for minimizing these causes, and implementing those activities deemed cost-effective management of real losses in a water distribution system that achieve its objective.

3. MATERIALS AND METHODS

3.1. The Study Area Description

3.1.1. Location

Gimbi Town is located in Western Oromia Regional State and 441 km far from Addis Ababa, the capital city of Ethiopia at latitude of 9^{0}10’ N and longitude of 35^{0}50’E. The elevation of the town ranges from 1600m to 2140m above sea level. Elevation varies between 1820 and 2140m at mountain peak, South of Gimbi town. The area receives an annual rainfall of 600 to 1400mm. The mean daily temperature also varies between 18^{0}C and 24^{0}C.The Gimbi town is located in the western part of the northwestern plateau of the Ethiopia physiographic subdivision. The town covers an area of 4950 hectare.

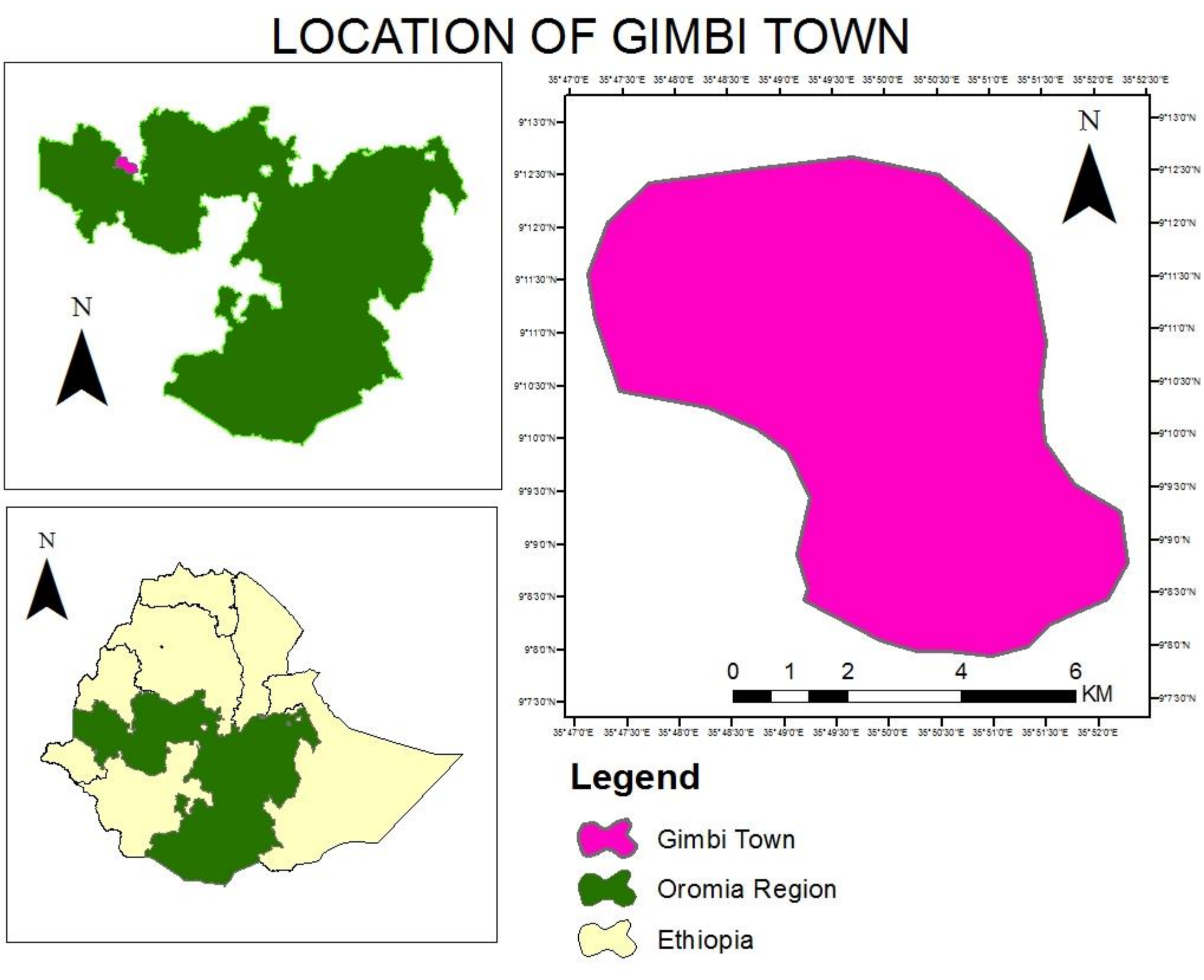


Figure 3-1: Location of Gimbi town

3.1.2. Land Uses and Land Cover

The existing land uses of Gimbi town is classified as the fallowing in figure (3-2).

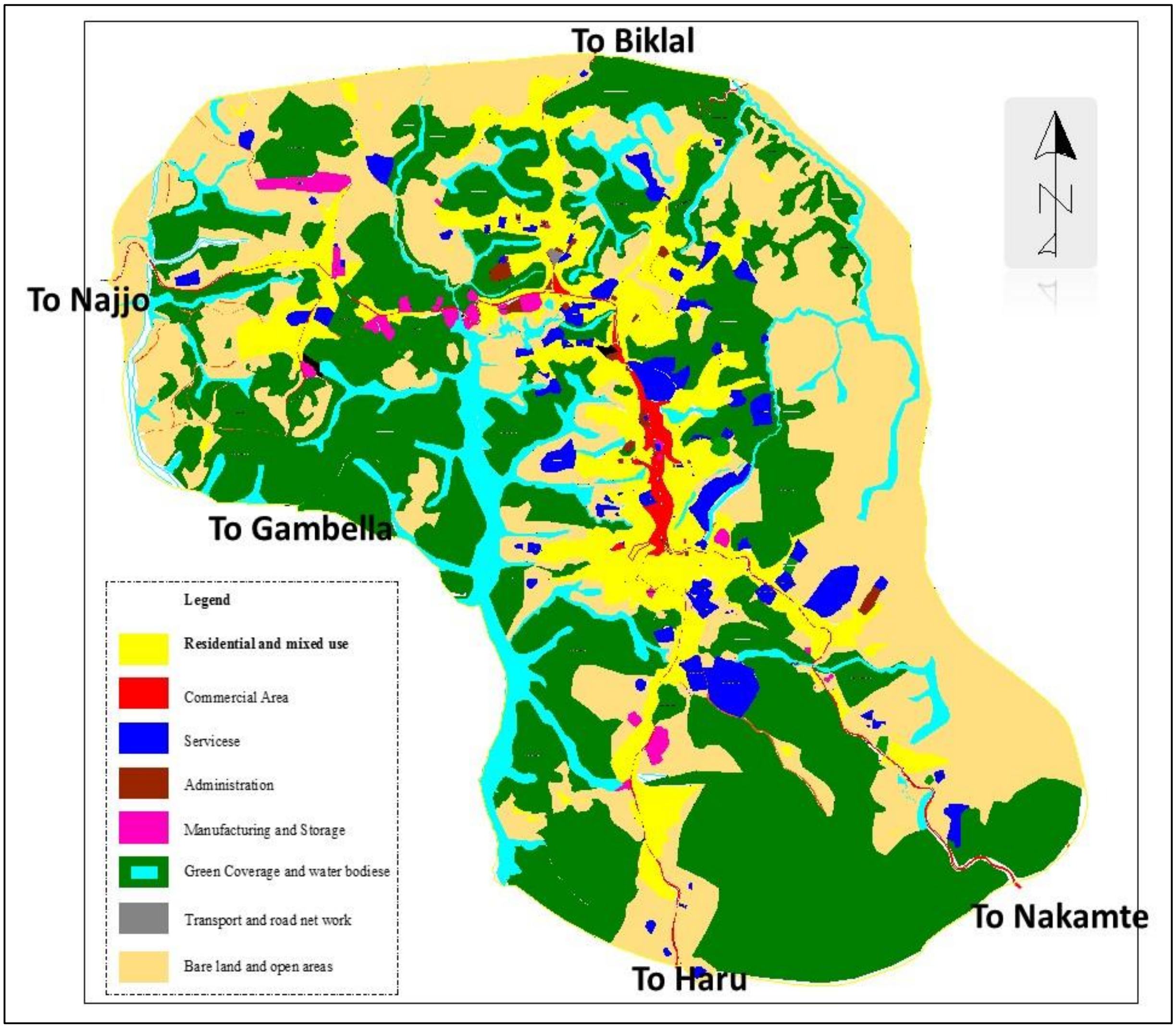


Figure 3-2: Existing Gimbi land uses

Source: Gimbi town administration office land management sector.

Table 3-1: Summary of the existing land uses.

No	Land use category	Area in hectare	Area in %tage
1	Existing residence and mixed	534.5	10.8
2	Commerce and business	22.3	0.45
3	Social and Municipal Services	65	1.3
4	Manufacturing and Storage	41.5	0.83
5	Green and open areas	4042	81.65
6	Road net work and Transport	230.2	4.6
7	Administration	14.5	0.3
8	Total	4950	100

3.1.3. Climate

Gimbi area has almost all year round precipitation with mean annual rainfall of 1835 mm and annual Potential evapotranspiration of 1268mm. The major rainfall occurs from May to October months. The movement of ITCZ in the northward direction brings moisture from the South Atlantic Ocean, which results in the high rainfall in the Gimbi area. The mean monthly maximum and minimum temperature is between 24.0^{o}c to 29^{o}c and 11^{o}c to 15^{o}c, respectively.

3.1.4. Economic Situation

Gimbi is a major transit center and it is located on the main road from Addis Ababa to Assosa. The town also serves as a major marketing center with thousands of rural people flocking into the town. The major economic activities according to the town's administration office are trading, hotel services and small-scale industries. Coffee is the main foreign currency earning cash crop, and it covers large area in and around the town. While, various categories of manufacturing and trade services at varying scales are the main sources of livelihood of the residents of the town. Ethiopian commodity exchange (ECX) has station (warehouse) in the area which will create great opportunity for agricultural industry. However, other industrial activities in Gimbi Town and its vicinity are limited to garages, coffee hulling, woodworks, metal works, grain mills, weaving, etc.

3.1.5. Basic Social Services

According to Gimbi town Educational office in Gimbi town, there are 15 regular schools (1 to 12 grades), 2 TVET, 1 NGO-owned college, and 1 University. There are also four private colleges. According to Gimbi town Health office, there are Two Hospital, one health station, 7 pharmacy and nine clinics are found in Gimbi town. The Hospital and health station has 79 beds. According to the information from the municipalities the overall sanitation of the town is poor and sanitation associated diseases are prevalent. There is no system for collecting, transporting, and dumping solid waste and liquid waste in the town. There are few garbage collection facilities located in the community, therefore, residents of the town dispose of domestic waste in any open spaces especially on the river courses, streets road verge and in drainage ditches.

3.1.6. Water Potential of the Area

The study area, Gimbi town, has variety of rivers (nearby the town), springs and streams of which are used as water supply, irrigation, and other ecosystem balancing for the town as well as for the rural area. The main rivers of the area are Gafare River (2km from the town), Seritti River (6km), Gelel River (9km), Melka Hora (4km), Huwwa River, Birbir River, Gabee River, Ganja River, Mexxi River, and Henna River are among the known potentials.

The first water supply project of the town that has been studied and designed by German Water Engineering and constructed in 1995 GC by the Ethiopian Water Works Construction Authority using Gafare River as source of water supply. Currently, the duration service of this project was outdated and the project has stopped its function completely due to increase in demand and the pollution of the Gafare water supply reservoir by waste generated from the town which requires expensive treatment units. This may occur due to topography of the town in which all the waste generated from the town was drained into the Gafare stream. As a result, the impounded stream has become converted in to a waste storage that deteriorated the raw water quality and increased treatment cost. Upstream usage of the river for irrigation has also increased significantly making the source in adequate for future water supply. To solve the above problem another project has been constructed on Gelel River for water supply of the town by Oromia Water Works Construction Enterprise and now the town has been getting water supply from Gelel River.

3.2. Existing Water Supply Description

Currently Gimbi town is getting water supply from Gelel River. The raw water at the source was collected through the intake weir along Gelel River and pumped to a treatment plant. The total length of the Raw Water Rising Main is 1.95km. Protected lined ferrous pipe, DN 400mm is used for the raw water rising main. The mean water level at the intake is taken as 1722.77 m and the top water level at the aerator in the Treatment Works is 1967.00 m. The Rising Main is sized in order to transfer the year 2037 maximum day demand plus losses and consumptions at treatment work 197.1 l/s (17,026 m^3/day)). The raw water from the source first enter aeration tank which used as preliminary treatment unit to remove the iron and bad odor from the incoming raw water. The water after passing through the aeration tank leads to coagulation and flocculation process. In this process the coagulant used is aluminium sulfate and they use magnesium sulfate to adjust the PH of the water. After the water has passed through coagulation and flocculation process the water enters the sedimentation tank. Water from sedimentation tank may enters the filtration tank in which rapid sand filtration (RSF) processes is carried out. Treated water that is passed through filtration tank enters to chlorination tank found in chlorination room in which disinfection process is carried out. After disinfection with chlorine, the treated water is drained into clear water storage tank (wet well) of 2500 m^3 and from the clear water tank the water is pumped to service reservoir of 1500 m^3 found at Gas hill mountain. In general the system consists of intake weir, raw water pumping station, aeration tank , sedimentation tank , rapid sand filtration unit, chlorination system, and clear water tank or wet-well, clear water pumping unit, Reinforced Concrete service/balancing reservoir , Booster station and distribution pipe network.

3.2.1. Mode of Services and Population Distribution by Mode of Services

According to the town water service office reports, there are four major modes of services for domestic water consumers of Gimbi town. These are house connections (HC), yard connections private (YCP), yard connections shared (YCS), and public fountains (PF). But, those populations not served from any of these modes of services are categorized as traditional source users (TSU).

The percentage of population served by each mode of service is varying with time. This variation is caused because of the changes in living standards, improvement of the service level, changes in building standards and capacity of the water supply service to expand. Census according to the central statics agency (CSA) 2007 states the overall domestic water supply coverage of Gimbi town was 74.9%. From this, only about 2.3% of the population has house tap connection users. The greater number of the populations was served their water need from both shared yard taps and public fountains found in different villages of the town are covers 21.6% and 34%, respectively. And 17% of the population were obtained water from their bounded privet yard connections. While, the remained 25.1% of the population was got water for their day to day activates from the spring and streams as water supply in the town. But, currently (2019) according to data from water supply office the total water supply coverage in the town is 61.7%. From this value the greatest number of population 38.3% uses spring and streams as water supply source and the left is stated in the table 3:2.

Table 3-2: Percentage of mode of service and population used.

Mode of service	2007 CSA report			2018 town water supply office report		
	Coverage (%)	served population	number of connections	Coverage (%)	number of connections	served population
HC	2.3	727	173	2.4	253	1265

YTP	17	5255	1251	26	2641	13205
YTS	21.6	6690	1076	22.3	2332	11660
PF	34	10546	27	11	11	5500
Total	74.9	23,218	2527	61.7	5734	31630

3.2.2. Components of Town Water Supply System

i) Water Supply Source

The Gimbi area is characterized by high rainfall regime about 1835 mm. Due to this there are perennial streams with adequate flow in the area that can be utilized as a source of water supply for the town. The potential surface water resource for the water supply of Gimbi town is Gelel River. The Gelel River is a perennial river, located south east of Gimbi town at about 6.6 km from the center in Melka Gasi Rural Kebele. Gelel is one of the tributary of the larger Didessa River. The stream has three major tributaries among which Melka Hola is the one. Based on the preliminary assessment by Oromia Water Works Design and Supervision Enterprise (OWWDSE), there are no potential pollutants along its catchment and there are no known potential users at upstream site.

According to the information obtained from Gimbi water supply project design report of 2011 by OWWDSE the Gelel stream is ungauged and its minimum and maximum flows are not known however, during their preliminary assessment elders leaving nearby the stream has been asked about the severe drought season. Their response is that the stream is perennial and they haven't seen any remarkable dry period flow during the recent drought that has occurred in the country

ii) Raw Water Pumping and Clear Water Pumping Station

As the design report, in the raw water pumping station there were five surface horizontal centrifugal types of pumps (four operational and one standby). These pumps were sucked water from the provided intake at the source to the treatment plant. But, currently only 3 pump was in operation.

In the clear water pumping station there were four surface horizontal centrifugal types of pumps (three operational and the other standby) as stated in the design report. These pumps were sucked water from the clear water reservoir which is adjacent to the pump station and pumping water to the service reservoir. But, currently only 2 pump was in operation. Each of these installed pumps has a maximum discharge of 10 l/s (36m^3/hr) and 154m of maximum head capacities. The pumps have a 22.5kW of shaft power and motor power of 37kW.

iii) Distribution System

The current distribution system of Gimbi town water supply is gravity system. In this system the water from treatment plant was pumped to service reservoir of 1500m^3. Then water flows in the mains due to gravitational forces from service reservoir of 1500m^3 to 500m^3 booster station reservoir. From this booster station reservoir water distributed for consumer found in zone 1 by gravity means. Water from booster station tank may also pumped to the tank (500m3) found at the radio station then distributed for consumer found in zone 2 by gravity means.

iv) Rising Main and Distribution Pipeline Network

The transmission and distribution main line consists of branching system with a total sum length of 29.4 km. The rising main transmit clear water to the service reservoir. It has a DCI pipe of DN 500mm from the treatment plant up to Gas hill reservoir and 400 mm from Gas hill reservoir up to Booster station reservoir which has a total length of 3.5km. from the booster station to the old tank it has DCI Pipe having DN 450mm which has length of 1812m and also from booster station to the radio station reservoir it contain PVC Pipe having DN 200mm which has total length of 743m.

The treated water is also further connected by distribution main line which serves the population with a total length of about 23km by gravity force. As per information obtained from the town water service office; the existing distribution pipe line was uPVC for pipes with DN 50 to 350 mm, DCI for pipes with DN250mm and above.

Table 3-3: Summarized quantity of pipe material in distribution system

Nominal diameter (mm)	Pipe materials	Length(m)	Average years	Coverage in system (%)
400-500	Ductile iron	5154	6	17.56
50-350	PVC	24203	6	82.44

Source: from Gimbi town water supply office

v) Service Reservoirs

Water shall be delivered to the reservoir in the system directly through the transmission main which is completely isolated from the distribution system. There are four circular RC type reservoirs which three of them Gas hill reservoir, booster station reservoir and radio station reservoir are for the new water supply system and one tank of diameter 400m3 is from the old water supply system were used in the town water distribution system, which used as service reservoir. Water is pumped directly into the service reservoir. From the service reservoir water is distributed for consumer by gravity means. Important information about the reservoir is described in table (3-4).

Table 3-4: Description of reservoir information

Description	Type	Function	capacity(m3)	location		
				Easting	Northing	Elevation(m)
Gas Hill reservoir	circular reinforced concrete	storage and balancing	1500	813079	1013654	2089.9
booster station reservoir	circular reinforced concrete	storage , balancing &wet well	500	812801	1013639	2034.84
radio station reservoir	circular reinforced concrete	storage, balancing &fire hydrant	500	814868	1014331	2124.36
Old system tank	circular reinforced concrete	storage, balancing &fire hydrant	400	812938.98	1013779.34	2003.3

3.2.3. Treatment System

The raw water from Gelel River is not potable without proper treatment. The treatment process adopted for Gimbi town water supply project design is Aeration for removal of iron, and manganese, chemical dosing of raw water to assist in flock formation and trapping of suspended impurities, Flocculation and clarification in up flow clarifier units, Filtration using Rapid Gravity Sand Filters and Disinfection by chlorination and pH correction of treated Water.

Figure 3-3: Existing reservoir

Figure 3-4: Gimbi water treatment plant

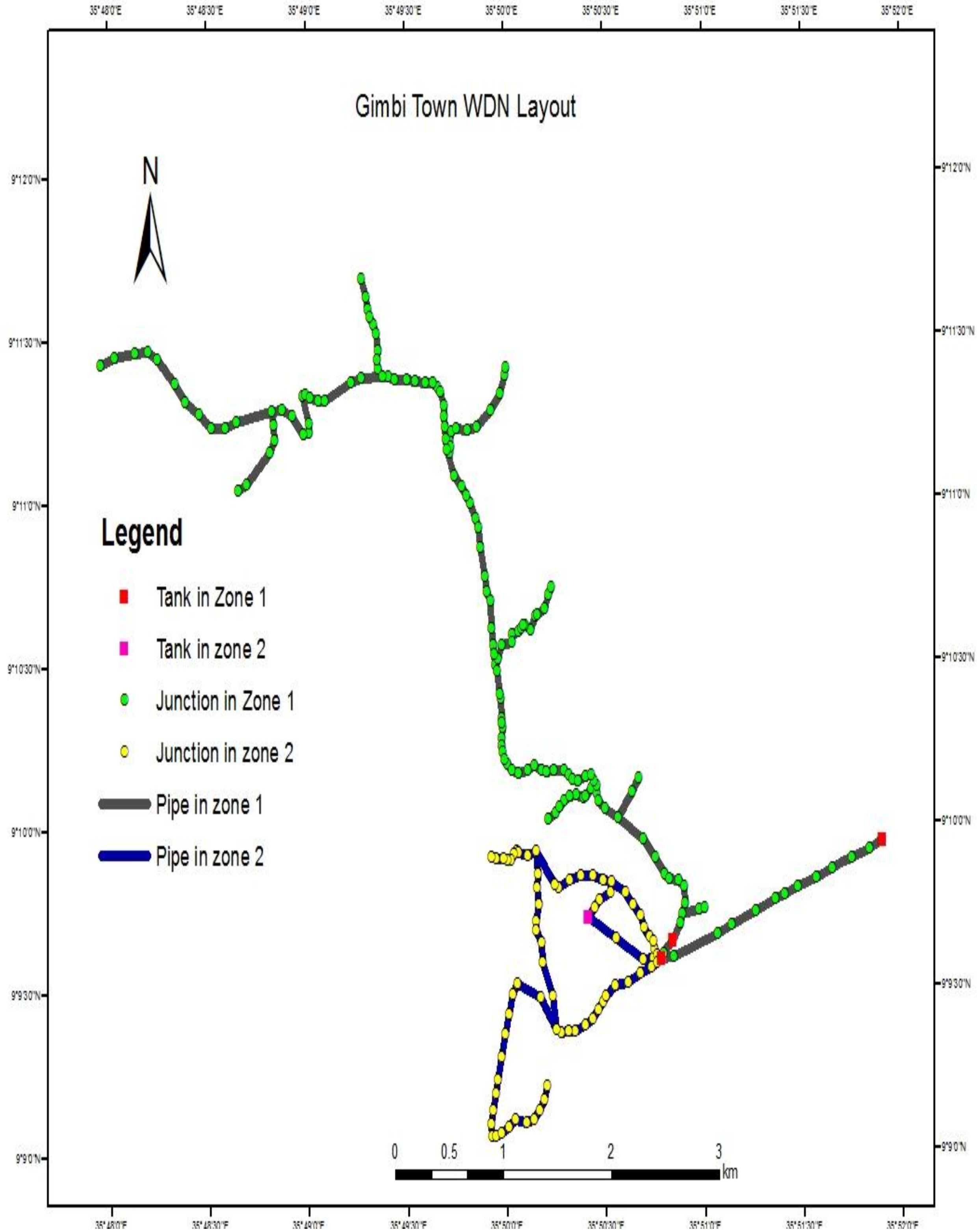


Figure 3-5: Gimbi water distribution system layout

3.2.4. Power Supply Units

There is power supply service in the town. The water utility was served power from hydroelectric power plant provided by EELPA and uses its own transformer having power of 100 kW. The water distribution system was operated for 24 hours when the hydroelectric power is available. And when the power is interrupted there is interruption of water supply as well. Because, no standby generator is used in distribution system during hydroelectric power failure.

3.3.Materials

Table 3-5: Materials used in the study

SN	Tools	Purpose	Source
1	WaterGEMS V8i software	For hydraulic analysis in the distribution system	Downloaded from https://bentley-watergems.software.com
2	ArcGIS 10.4 software	For mapping the study area and to visualize water distribution layout	Downloaded from www.ESRI.com
3	AutoCAD 2007 software	To extract AutoCAD drawing elements that combines graphics and data.	Downloaded from https://Softvela.com
4	Pressure gauge 2.5MPa (code: MC01830126)	To collect pressure data in distribution network	From Oromia Water Works Construction Enterprise
5	GPS	To collect geographical location of measured sample pressure point	From Gimbi town water supply and sanitation office

3.4.Methodology

The general procedures followed to perform this research was collection of important data required for the study, hydraulic analysis using hydraulic modeling process and Water loss analysis using water balance approach. The collected data are primary data and secondary data. Hydraulic modeling process include model skeletization, model setup, and model calibration and validation. Finally, from the model output results hydraulic problem of the water distribution is identified and remedial measures are suggested. Water loss in the distribution system is assessed using percentage of non-revenue water, Water loss per pipe length, Water loss per number of connection and Infrastructure Leakage Index (ILI).The above general process is shown in the figure (3-6) and briefly explained in the next section.

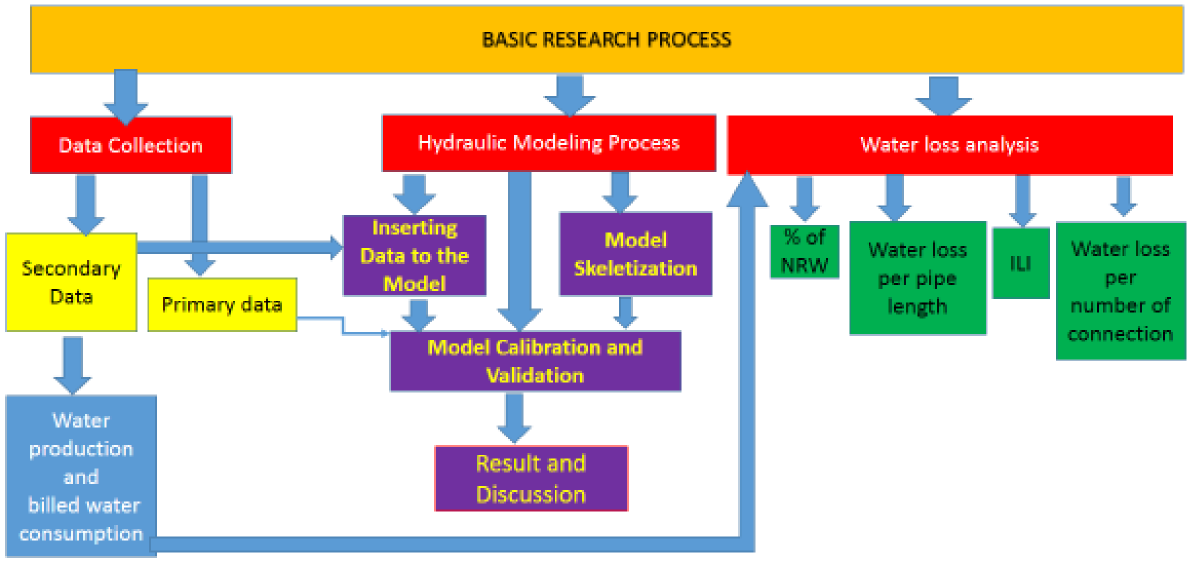


Figure 3-6: Framework of the research process

3.4.1. Data Collection

The data were involved both primary and secondary sources. For this study, the primary data were obtained from pressure reading, elevation surveying and by made of discussion with water utility staff members to obtain additional relevant information on the subject matter. Primary data collected are 30 pressure reading data taken from 15 different location in the distribution system using pressure gauge instrument. This pressure measurement is used to perform calibration and validation process.

While, secondary data were collected from different literature reviews, design report, the town water supply service office existing documents and annual reported papers. Secondary data used in this study is design document describing system skeleton, location and operating condition taken from Oromia water works design and super vision enterprise (OWWDSE). The boundary map, Topological map and master plan of the town Taken from town administration office and Ethiopian map agency. Total number of population to be served and total number of house has been taken, the number of customer data as per level of connection, Successive Water production and Consumption data was taken from town water supply service office.

3.4.2. Hydraulic Modeling Process

Hydraulic modeling is the process of building, verifying and operating network models of distribution systems, which provide valuable insights into operational practice .Water distribution modeling is a critical part of designing and operating water distribution systems that are capable of serving communities reliably, efficiently, and safely, both now and in the future hydraulic modeling process includes:

3.4.2.1.Model Skeletonization

Before an actual water distribution system may be modeled or simulated with a computer program, the physical system must be represented in a form that can be analyzed by a computer. In order to skeletonize the model to an acceptable degree, (Mahmoud A. Elsheikh, Hazem I. Saleh et al. 2013) reviewed the study adopted by (USEPA 2005) and used the fallowing conditions to skeletonize the network as guideline.

- At least 50% of total pipe length in the distribution network are to be included
- At least 75% of the pipe volume in the distribution system are to be included
- All 300 mm diameter and larger pipes in the distribution system are to be included
- All 200 mm and larger pipes that connect pressure zones, influence zones from different sources, storage facilities, major demand areas, pumps, and control valves, or are known or expected to be significant conveyors of water are to be included
- All 150 mm and larger pipes that connect remote areas of a distribution system to the main portion of the system are to be included
- All storage facilities with controls or settings applied to govern the open/closed status of the facility that reflects standard operations;
- All active pump stations with realistic controls or settings applied to govern their on/off status that reflects standard operations.

3.4.2.2.Model Input Preparation

To build the model all information required for modeling have to be fulfilled. This information was taken from secondary data from the different organization.

Input data to the model are:

- Pipe data: Pipe diameter, Pipe length, Hazen William C-factor,
- Node data: Nodal elevation, nodal demand,
- Pump data: pump head curve, pump efficiency, and pump location
- Valve data: valve setting and location

- Tank data: base elevation, initial elevation, maximum elevation, diameter and volume

Output data of the model are

- Flow in a pipe,
- velocity in a pipe, and
- Pressure head at any junction are the output of the model.

The basic task in model building is estimating the current water consumption at each node in the distribution network. The nodal demand may be obtained as the following steps below:-

1. Obtaining the total population of the town
2. Identification of number of houses around each supply node
3. Assigning number of peoples in each supply node
4. Assigning per capita consumption and average day water demand
5. Assigning base water demand in each supply node

1. Population of the town

Population is the important data to assess water demand in the distribution network. Facts show that there are different population forecasting methods which are used for estimating the current or future population of a given town, but the results of the methods are vary from one to the other due to considering parameters of each method. To predict the population of a town, it is necessary knowing factors affecting the population distribution, size and growth rate. In Ethiopia, the major factors that influences on the changes in population figure are births, death and migration. All these factors are influenced by family planning practice, war, natural disasters, development of the towns and the socio-economic activities in and around the towns.

2. Identification of number of houses around each supply node

For this study, Gimbi town topographic map was obtained and bought from Ethiopian Mapping Authority, with the scale of 1:50,000 and twenty meter contour interval. In ArcGIS this topographic map was displayed and the town distribution network map which was drawn in Water GEMS was exported in to ArcGIS shape file and overlapped it in the topographic map of the town. Therefore, the number of houses nearby each node were physically counted from the overlapped map and assigned to every node in the network by considering the actual condition of the residents in the town. The number of houses around each supply node obtained in the above process is stated in the second column of a table at Appendices A.

3. Assigning number of peoples in each supply node

The current average number of person in each house (person per housing unit) obtained from the town administration office is 5. The total number of houses in the town was identified by dividing the total population to the average number of person in the town, and it was estimated at 10,250.Therefore, on the Microsoft Excel sheet, all the 346 junctions in the system and the number of houses assigned for each node were entered respectively. Then, the number of assigned houses in each node was converted in to the number of people, by multiplying the average number of person in each house of the town using the equation below.

$$Number\ of\ people\ in\ suply\ node = no.of\ house\ used\ from\ the\ node\ x\ avg.no.of\ people\ in\ each\ house \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.1$$

As the above formula the obtained value is listed in the third column of table at Appendix -A.

4. Assigning per capita consumption

For assessing the average water demand of the town, deterministic water demand estimation method was used. Hence, the per capital water consumption of the town was calculated using the annual water consumption recorded data and projected total population figure of the year 2019 using the equation below.

$$Per\ capiatal\ consuption\left(\frac{\frac{l}{c}}{d}\right) = \frac{(\text{Annual consumption } (m^3)\ x\ 1000\ l/m^3)}{total\ population\ x\ 365} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.2$$

6. Assigning base water demand in each supply node

To calculate base water demand for the particular supply node the current consumption and NRW is also assigned.

Frist, the current billed consumption is added by multiplying the current per capita consumption with number of the user from the node using the equation below.

$$Nodal\ consumption = Population\ served\ from\ the\ node\ x\ percapita\ consumption \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.3$$

Then, non-revenue water must be loaded into the model just like any other demand. However, according to (Thomas M. W alski, Donald V . Chase et al. 2003) non-revenue means that the user does not know where to place it and usually, the user simply calculates total non-revenue water and divides that quantity equally among all nodes. If the modeler knows that one portion of a system has a greater likelihood of leakage because of age, then more unaccounted-for water can be placed within that section.

For each node, the demand must be corrected for non-revenue water. One approach is to assign unaccounted-for water in proportion to the consumption at a node using the formula below (Thomas M. W alski, Donald V . Chase et al. 2003).

$$\text{Base water flow} = (\text{Assigned consumption})\ x \left[\frac{(\text{Production})}{\text{billed consumption}}\right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..3.4$$

Therefore, nodal consumption which obtained (by the Equation 3.3) is listed in appendix -A under the fourth column and base water flow which obtained (by the equation 3.4) is listed in the fifth column of table at Appendix -A.

Institutional and commercial demand that is water demand of facilities such as schools, hospitals, hotels, etc. and small commercial enterprises, and also public demand where assessed based on the daily demands of different consumptions following as (MoWIE 2006).

Table 3-6: Institutional and commercial consumption figure

Institutional/commercial place	Consumption
Restaurants	10 l/seat
Boarding school	60 l/pupil
Day schools	5 l/pupil
Public offices	5 l/employee
Workshop/shops	5 l/employee
Mosques & Church	5 1/worshipper
Hospitals	50 - 75 l/bed
Public Bath	30l/visitor
Hotels	25 - 50 l/bed

Note: The data enumerated in the above table is obtained from the existing urban water supply design guideline. It is the only available document at national level and need to be taken as indicative figures.

But in this study due to due to lack of data that states consumption of every commercial and public separately, it was assumed that 10% ($82m^3$/d) of total water consumption of the town was used by public service and commercial services in the town and this consumption is assigned to the nodes with consideration of the number of the users and the above guideline.

Generally in this study, 15l/student is assigned for Gimbi University which has total 4000 student, 30l/bed for two hospitals having total 79 bed , 10l/bed for 13 hotels having average 25 bed and for bar and restaurant 2l/seat was used.

Peaking factors

For modeling, peak hour demand scenario was adopted. Demand for each supply node was performed by taken demand multiplier factors of 24 hour flow duration and computed with assessed base demand. Therefore, for this study the peaking factor is calculated as the ratio of discharges for the various conditions. For example, the peaking factor applied to average-day demands to obtain maximum day demands can be found by using Equation below.

$$PF = \frac{Q_{max}}{Q_{avg}} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \, 3.5$$

Where PF= peaking factor between maximum day and average-day demands

Q_{max} = maximum day demands (m^3/s)

Q_{avg} = average-day demands (m^3/s)

The total consumption of water is 262040 m^3. The average daily consumption was obtained by dividing the total consumption (262040 m^3) for 365 days which gives 717.9 m^3/day. The maximum water consumption was occur in the May which was 28030 m^3/month. Therefore, the average daily consumption in this month is 934.3 m^3/day.

Therefore, maximum day peaking factor was:-

$$MDF = \frac{934.3}{821.6} = 1.3$$

Therefore, maximum day factor adopted in this study is 1.3.

Peaking factors from average day to maximum day tend to range from 1.1 to 3.0, and factors from average day to peak hour are typically between 3.0 and 6.0. of course, these values are system-specific, so they must be determined based on the demand characteristics of the system at hand (Thomas M. W alski, Donald V . Chase et al. 2003).

Determining Peak hourly factor is difficult without important data. Therefore, it was adopted from the size of the consumer base according to (MoWIE 2006). Therefore, the proposed peak factor and patterns for demand multiplier factors were 1.8 based on the criteria listed in table below.

Table 3-7: Proposed hourly peak factor

Population	Peak hour factor
0-50,000	2
50,001-100,000	1.8
101,000 and above	1.6

Source:(MoWIE 2006)

3.4.2.3.Model calibration and validation

Model calibration

(Thomas M. W alski, Donald V . Chase et al. 2003) defines calibration as the process of comparing the model results to field observations and, if necessary, adjusting the data describing the system until model predicted performance reasonably agrees with measured system performance over a wide range of operating conditions.

In this study to calibrate the model result Hazen William's coefficient(C factor) and nodal demand is adjusted. The adopted the (C factor) for pipe is according to (MoWIE 2006) as stated in the table (3-8).

Table 3-8: Recommended C- values for various pipe material

C-Value for Hazen-Williams			
Type of Pipe	**uPVC**	Steal	DCI/GI
New	130	110	120
Existing	100-110*	90-110*	100-110*

Source: (MoWIE 2006)

Note: - * Depending on age and condition

As per pressure criteria according to (Walski,2003) 85% of the computed model results should become within ± 0.5m head of the observed field conditions and 100 % of the computed model results should become within ± 2m head of the observed field conditions. Hence to assure the acceptable level of calibration, the two most commonly used model inputs parameters; pipe roughness coefficients and junction demand data were adjusted. Therefore, during model calibration; C-factor was used 130 for PVC and value of 120 for DCI pipe. With regard to these, time series representations of the calibrated pressure head difference were presented in the table (4-4 and 4-5) of the chapter four.

Model validation

There are many ways to judge on the performance of model calibration, the calibration statistics used in this study was by calculating the squared relative difference between observed and simulated pressure for each test. The results and the observation data were entered to an excel sheet and the value of squared error was calculated for every test then the mean square error and standard deviation calculated from Excel sheet. The lower values of these parameters, the higher the accuracy of the calibration process.

$$R^2 = \frac{((X-\bar{X})(y-\bar{Y}))}{\sqrt{\sum(x-\bar{X})2\sum(y-\bar{Y})2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots .3.6$$

Where, R^2 is Correlation Coefficient,

X and Y are measured and simulated values respectively,

$\bar{X}$ and $\bar{Y}$ are average values of measured and simulated respectively.

The hydraulic analysis has been done under peak loading condition, and under low loading condition.

3.4.3. Water Loss Analysis

Water loss analysis in Gimbi was assessed at the town level based on the percentage of Non-Revenue Water, Water loss per pipe length, water loss per number of connection and Infrastructure Leakage Index (ILI).

3.4.3.1.Percentage of Non-revenue water

As per data obtained from the town water service office, the annual (October 2018, to September 2019 G.C) water production and consumption (billed water volume) in the system was identified. Using this data total percentage of water loss in the system is assessed using equation below.

$$\%NRW= \frac{(System\ Input\ volume - billed\ Authorised\ consumption)}{System\ Input\ volume} x100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.7$$

During field visit, it was also observed that the utility do not have any recorded data related with average leak flow, number of reported bursts and average leak duration; due to these physical loss in the main was assessed based on some assumption and the available data.

3.4.3.2.Water loss as per number of service connection

One of the appropriate indicators of water loss in the distribution system is describing it as per number of service connection (liters per service connection per day, l/c/d), and it gives more precise figure than NRW as a percentage of inputs volume. Based on this, the volume of water loss as per number of connection were analyzed from the total unbilled volume.

$$\text{Loss per connection} = \left(\frac{\text{water losses} * 1000\ \text{l/m3}}{\text{total number of connection} * 365\text{day}}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..3.8$$

3.4.3.3.Water loss as per pipe length

The other good indicator of water loss in the distribution network is determining loss as per pipe length (liters per kilometer of pipe line per day, l/km/d). Therefore using this, the estimated amount of water loss; as per kilometer of the pipe length of the town was calculated as;

$$\text{Loss per pipe length} = \left(\frac{\text{Unbilled volume} * 1000\ \text{l/m3}}{\text{total number of pipe length} * 365\ \text{day}}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..3.9$$

3.4.3.4.Calculating Infrastructure leakage index (ILI)

$$ILI = \frac{CARL}{UARL} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..3.10$$

Where,

CARL- current annual real losses

UARL- Unavoidable real losses

UARL or the minimum achievable annual physical losses (unavoidable annual real loss) in the system is computed according to (Farley 2008).

$$UARL\left(\frac{L}{day}\right) = (18 \text{ x } Lm + 0.80 \text{ x } Nc + 25 \text{ x } Lp) \text{x } P \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.11$$

Where, L_m = mains length (km); N_c = number of service connections; L_p = total length of private pipe, property boundary to customer meter (km); and P = average pressure (m)

3.5.Hydraulic Performance Assessment Criteria

3.5.1. Pressure and Velocity

In designing or improving a water distribution system there are sets of design criterion to be considered. Among them, pressure and velocity are important one. The design criteria used in the design of water supply distribution system components, nodal pressure during the period of peak demand, and optimum velocities of the transfer and distribution mains are as follows (MoWIE 2006).

- ✓ Minimum static head from the distribution system is 15 m.
- ✓ Maximum static head within a pressure zone was limited to 70 m.
- ✓ Maximum velocities of major transfer mains <2.5 m/s.
- ✓ Maximum velocities of distribution mains <2 m/s.
- ✓ The minimum velocity of flow in a pipeline is 0.6m/s, but for looped systems there will be pipe lines with sections of zero velocity.

3.5.2. Reservoir Capacity

The capacities of reservoirs in the water supply system were determined using different methods. The most appropriate and economical approach of determining storage volume of reservoir is the 24 hours supply demand simulation mass curves. In order to develop such type of curves, it requires reliable recorded historical data of hourly water demand figures of the town. But, in the absence of such type of data, to determine the size of reservoirs, it was adopted the commonly practiced in many water supply systems and based on the urban water supply design criteria of the ministry of water resources; it was used for sizing the reservoir volume as one third of the maximum daily demand.

3.5.3. Pump Capacity

One of the main components of water distribution systems is the pump stations. Pumps were deliver energy to the hydraulic system in order to overcome elevation difference and head losses due to pipe friction and fittings.

For this study, both maximum capacity of the pump and pump efficiency were conducted in order to determine the pumps capacity in the town water distribution system.

Maximum capacity of pump = Maximum Pump design capacity * Effective pump operation time

$$\text{Pump Efficiency} = \frac{\text{Water Power}_{\text{out,maximum}}}{\text{Pump Power}_{\text{in}}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.12$$

Pump power refers to the brake horsepower on the pump shaft, and it is difficult to measure in the field.

Water power is computed from the following relationship:-

$W_P = C_f Q h_P \Upsilon$

Where W_P= water power (hp, Watts)

Q= flow rate (l/s)

h_P= head added at pump (m)

Υ= specific weight of water (N/m3)

C_f = unit conversion factor

According to the pump characteristic comply with (ISO 9906:2012); most pumps which present and perform in a good condition have an efficiency of 60-80%.

3.6.Method of Collecting Sample Pressures

Sample pressure and flow were taken from locations that are representative of the distribution network. These points should include the samples representative of the conditions at the most unfavorable places in the supply system, particularly points of possible high & low-pressure zones ends and points of high consumption and less consumption. Sampling time pressures, tank level and flow data were taken at both peak hourly demand and minimum hourly demand (midnight). The major factors considered during the selection of sample areas are the zone of high problem found, topography, data availability, suitability to identify the problem. By taking the above criteria in to consideration the main pipe line in the system was selected to identify the bursting problem and for the pressure measurement location to perform calibration and validation. Selected location point for measuring pressure sample is shown in the table (3-9).

Table 3-9: Selected location of pressure sampling point

Sampling point	Location			Measured date	Measured time during peak demand time	Measured time during low demand time
	X	Y	Z			
S1	813180	1013946	1945	14/05/2019	7.00 AM	8.00 PM
S2	812557	1014615	1939	14/05/2019	8.00 AM	8.30 PM
S3	811784	1014459	1991	13/05/2019	6:00 AM	8:00 PM
S4	811363	1014,832	1981	14/11/2019	8:00 AM	10:00 PM
S5	811615	1015522	1921	13/05/2019	6:30 AM	9:00 PM
S6	811,157	1015992	1920	14/11/2019	8:30 AM	9:30 PM
S7	811116	1016669	1949	13/05/2019	8:00 AM	9:30 PM
S8	810,163	1017,247	1881	14/11/2019	1:00 PM	9:00 PM
S9	809529	1016850	1837	13/05/2019	7:00 PM	10:00 PM
S10	808,329	1016911	1908	14/11/2019	1:30 PM	10:30 PM
S11	808988	1016,344	1894	15/11/2019	7.00 AM	8:00 PM
S12	811879	1014072	2074	14/05/2019	8.00AM	9.00 PM
S13	812,633	1013,912	2037	15/11/2019	8.00 AM	8:30 PM
S14	812521	1013529	2033	14/05/2019	6.00PM	10.00 PM
S15	811,856	1013269	2103	15/11/2019	6.30 PM	9:30 PM

In this study pressure samples are taken from public tap and customer tab. Accordingly,30 pressure samples are taken from 15 different point in the distribution collected from places of high consumption and low consumption as well as from high pressure area and low pressure area during the day time and night using pressure gauge.

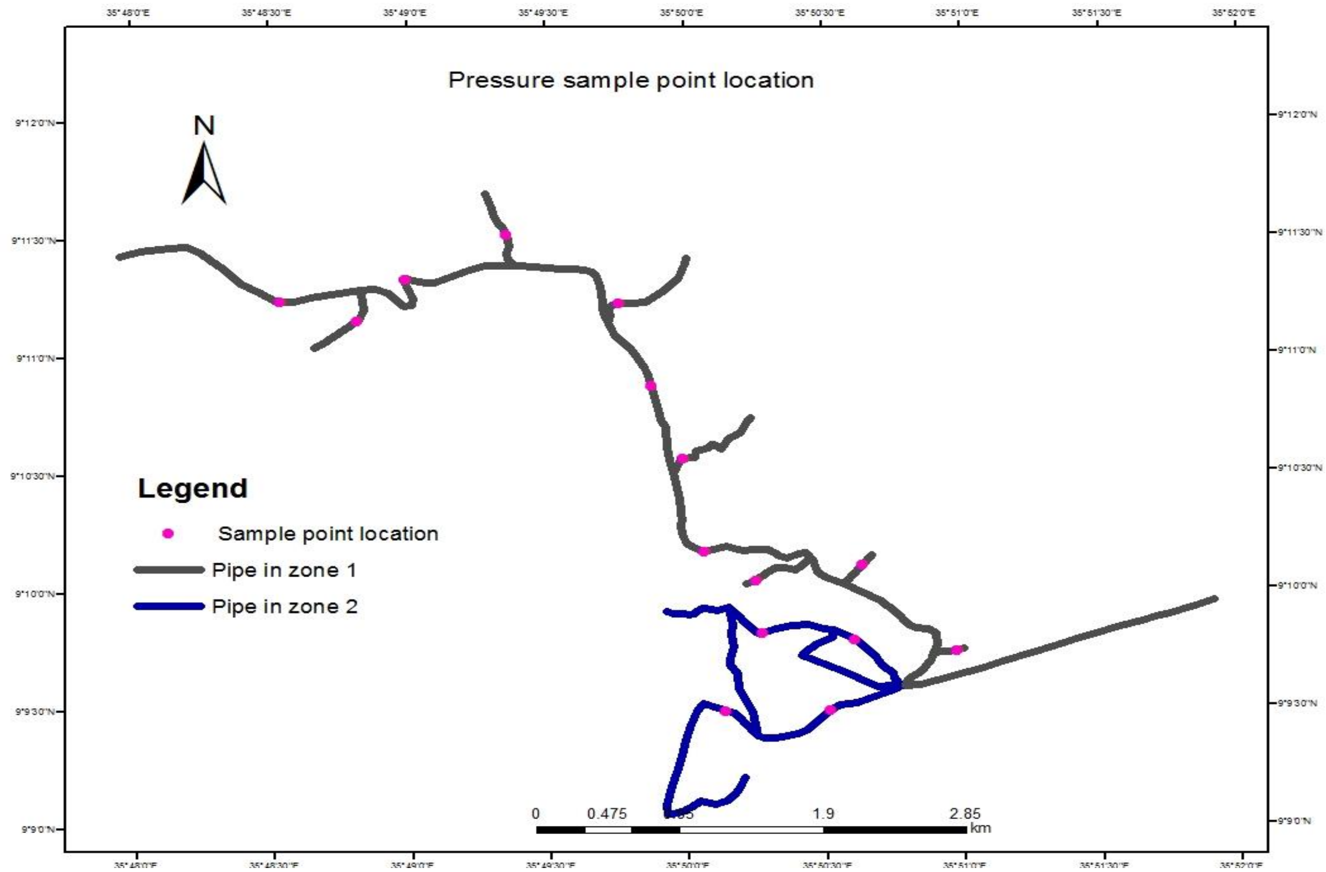


Figure 3-7: Pressure sample point location in the distribution system

Figure 3-8: Pressure gauge and measuring pressure in the distribution system

(Date of measurement: In the May 2019 and November 2019)

3.7. Assessment of Leakage Management Practice

The water distribution leakage management practices of the town water service office were assessed based on the management, technical and financial; plan, policy and strategies. Hence, field visits were made to identify the leakage in the system and its managing processes. During field observation, discussions were conducted with town water supply service personnel to obtain information on the common failure of system, financing mechanisms, and the maintenance culture and cost drivers of maintenance. While, cost related data was collected by reviewing the annual reports and financial statements of the utility. Finally, the collected data were analyzed and presented in the next chapter.

4. RESULT AND DISCUSSION

4.1. Model Skeletonization

The general skeletonized pipe length and pipe volume included in modeling is described in the table below.

Table 4-1: Skeletonized pipe length and pipe volume included in modeling

Material	Diameter (mm)	diameter (m)	Length(m)	Area(m2)	Volume
Ductile Iron	500	0.5	36	0.19625	7.065
	450	0.45	2962	0.158963	470.8469
	400	0.4	2156	0.1256	270.7936
PVC	350	0.35	1748	0.096163	168.0921
	300	0.3	466	0.07065	32.9229
	250	0.25	1684	0.049063	82.62125
	200	0.2	5535	0.0314	173.799
	150	0.15	4206	0.017663	74.29
Total			18793		1280.43

Generally, the Percentage of total length of pipe included in modeling after skelotenization is 64% and Percentage of total volume of pipe included in modeling after skelotenization is 91%.

4.2. Model Building Process

4.2.1. Population Projection

According to CSA census result of 2007 reveals, the total population of Gimbi Town is 30,981 (15,716 male and 15,265 female) and 7,422 households (CSA 2008). And the projected population by (CSA 2013) at the year 2015 is 45572(22921 male and 22651 female). This value has great difference with the data obtained from Gimbi Town Municipal Office, which is population size at the 2015 is estimated at 85,478 (45983male and 39495 female). Therefore, for this study; because of availability of CSA data and uncertainty of the municipality data, the population data by the CSA is used as base population, and the population of the town at the year 2019 was estimated.

Table 4-2: Population growth rate, Oromia regional state

Year G.C	2010	2015	2020	2025	2030	2035
Growth rate (%)	4.33	4.15	3.93	3.68	3.53	3.27

Source: CSA 2007 report

Using the above growth rate of CSA (2007) and projected population at the year 2015 by (CSA 2013) which is 45572 is used as a base and population at the (2019) is estimated using exponential method.

$$p_n = p_{0*}e^{rn} \text{ ……………………………………………………………..4.1}$$

Pn - estimated population at n year

Po - base population

r - Growth rate

n – Number of year

Table 4-3: Projected population from CSA (2015)

Population	Unit	2015	2019
Growth rate	%		4.15
Census data	No	43,414	51,253

Therefore, the current (2019) population of the town is estimated to 51253.

4.2.2. Determining Per Capital Water Consumption

The per-capita water consumption for various demand categories varies depending on the size of the town and the level of development. In Gimbi town, because of the growth of the socio-economic activity in both governmental and private sectors, there was the high water demand in the town. Using the annual water consumption and population figure in (2019), the average per capital consumption of the town was identified as below.

$$per\ capiatal\ consuption\left(\frac{\frac{l}{c}}{d}\right) = \frac{(\text{Annual consumption }(m^3)\ x\ 1000\ l/m^3)}{total\ population\ x\ 365} \text{ ……………… 4.2}$$

The annual cumulative domestic consumption amount in 2019 is 262040 m^3 and the projected total population of the town at the year 2019 was estimated as 51253. Therefore, by using the above expression the average daily per capita consumption is:

$$per\ capiatal\ consuption\left(\frac{\frac{l}{c}}{d}\right) = \frac{(262040\ (m^3)\ x\ 1000\ l/m^3)}{51253\ x365} = 14$$

But, according to existing town water supply design report; the average per capital water demand of the town at the year 2019 was estimated and adopted as 60 l/c/d. and According to (MoWIE 2015) on Second Growth and Transformation National Plan for the Water Supply and Sanitation Sub-sector (GTP-2) urban water supply access with GTP-2 minimum service level of 100 l/c/day for category-1 towns/cities (towns/cities with a population more than 1 million) , 80 l/c/day for category-2 towns/cities (towns/cities with a population in the range of 100,000-1million), 60 l/c/day for category-3 towns/cities (towns/cities with a population in the range of 50,000 - 100,000), 50 l/c/day for category-4 towns/cities (towns/cities with a population in the range of 20,000-50,000), up to the premises and 40 l/c/day for category-5 towns/cities (towns/cities with a population less than 20,000) within a distance of 250m with piped system for 75% of the urban population. As projected population Gimbi is the town found in category-3. Therefore, when we compare of this figure the above estimated per capital consumption value (14 l/c/d) was unrealistic and unacceptable. Therefore, further reviewing work is necessary to fix the recent per capital water consumption of the town.

4.2.3. Calculating Average Water Flow

Average water flow is total water flow in the distribution system and it is the sum of water used for domestic purpose, industrial purpose, commercial purpose, public demand and it also include non- revenue water. But, in Gimbi water distribution system there is no industrial water demand for the current year.

Therefore, in this study average water flow in the distribution is estimated by multiplying per-capital water consumption with the projected population of the year 2019 plus total non-revenue water in the system gives the average water flow in Gimbi town water distribution system. Which is calculated using the formula below.

Total average water flow in the distribution is calculated by:

$$Qavg = (percapital\ consumption\ \ x\ total\ population\ of\ the\ town) + NRW \ldots\ldots\ldots\ldots .4.3$$

Qavg = (14 l/capita/day X 51523) + 854.7 m^3/d =1576 m^3/d

But, according to the Gimbi town water supply design report the average water demand at the year 2019 is 5,477m3/day. When we compare the current average water flow 1576 m^3/d with the design report it is very minimum and have to be improved.

4.3. Hydraulic Model Calibration and Validation

4.3.1. Model Calibration

In the modern time, water utilities have able to analyze the status of their existing water supply system using hydraulic models. But, for assuring the entered water distribution model inputs data accuracy; the computed model results have been compared with the actual observed field conditions of study area and adjustment of the computed model result is required. During model calibration Hazen William C-factor and nodal demand was adjusted in the study.

Table 4-4: Time series representation of pressure value for peak demand time

Sampling point	Location (m)			Measured date	Measured time	Computed pressure (m)	Observed pressure (m)	Error
	X	Y	Z					
S1	813180	1013946	1945	14/05/2019	7.00 AM	60	57.2	2.8
S2	812557	1014615	1939	14/05/2019	8.00 AM	66	63.3	2.7
S3	811784	1014459	1991	13/05/2019	6:00 AM	14	12.9	1.1
S4	811363	1014,832	1981	14/11/2019	8:00 AM	19	17.1	1.9
S5	811615	1015522	1921	13/05/2019	6:30 AM	84	85	-1
S6	811,157	1015992	1920	14/11/2019	8:30 AM	86	84.6	1.4
S7	811116	1016669	1949	13/05/2019	8:00 AM	57	55.2	1.8
S8	810,163	1017,247	1881	14/11/2019	1:00 PM	114	112.8	1.2
S9	809529	1016850	1837	13/05/2019	7:00 PM	168	166.5	1.5
S10	808,329	1016911	1908	14/11/2019	1:30 PM	14	12.2	1.8
S11	808988	1016,344	1894	15/11/2019	7.00 AM	29	27.7	1.3
S12	811879	1014072	2074	14/05/2019	12.00PM	53	51.6	1.4
S13	812,633	1013,912	2037	15/11/2019	8.00 AM	89	87.4	1.6
S14	812521	1013529	2033	14/05/2019	12.30PM	93	94.5	-1.5
S15	811,856	1013269	2103	15/11/2019	6.30 PM	8	6.4	1.6

Table 4-5: Time series representation of pressure value; during low demand time

Sampling point	Location (m)			Measured date	Measured time	Computed pressure (m)	Observed pressure (m)	Error
	X	Y	Z					
S1	813180	1013946	1945	14/05/2019	8.00 PM	60	65.8	-5.8
S2	812557	1014615	1939	14/05/2019	8.30 PM	66	68.7	-2.7
S3	811784	1014459	1991	13/05/2019	8:00 PM	14	16.5	-2.5
S4	811363	1014,832	1981	14/11/2019	10:00 PM	19	21.2	-2.2
S5	811615	1015522	1921	13/05/2019	9:00 PM	84	86.3	2.3
S6	811,157	1015992	1920	14/11/2019	9:30 PM	86	88.9	2.9
S7	811116	1016669	1949	13/05/2019	9:30 PM	57	58.8	-1.8
S8	810,163	1017,247	1881	14/11/2019	9:00 PM	114	116.6	-2.6
S9	809529	1016850	1837	13/05/2019	10:00 PM	168	169.5	-1.5
S10	808,329	1016911	1908	14/11/2019	10:30 PM	14	15.6	-1.6
S11	808988	1016,344	1894	15/11/2019	8:00 PM	29	31.4	-2.4
S12	811879	1014072	2074	14/05/2019	9.00 PM	53	55.6	-2.6
S13	812,633	1013,912	2037	15/11/2019	8:30 PM	90	92.4	-2.4
S14	812521	1013529	2033	14/05/2019	10.00 PM	93	95.8	-2.8
S15	811,856	1013269	2103	15/11/2019	9:30 PM	8	11	-3

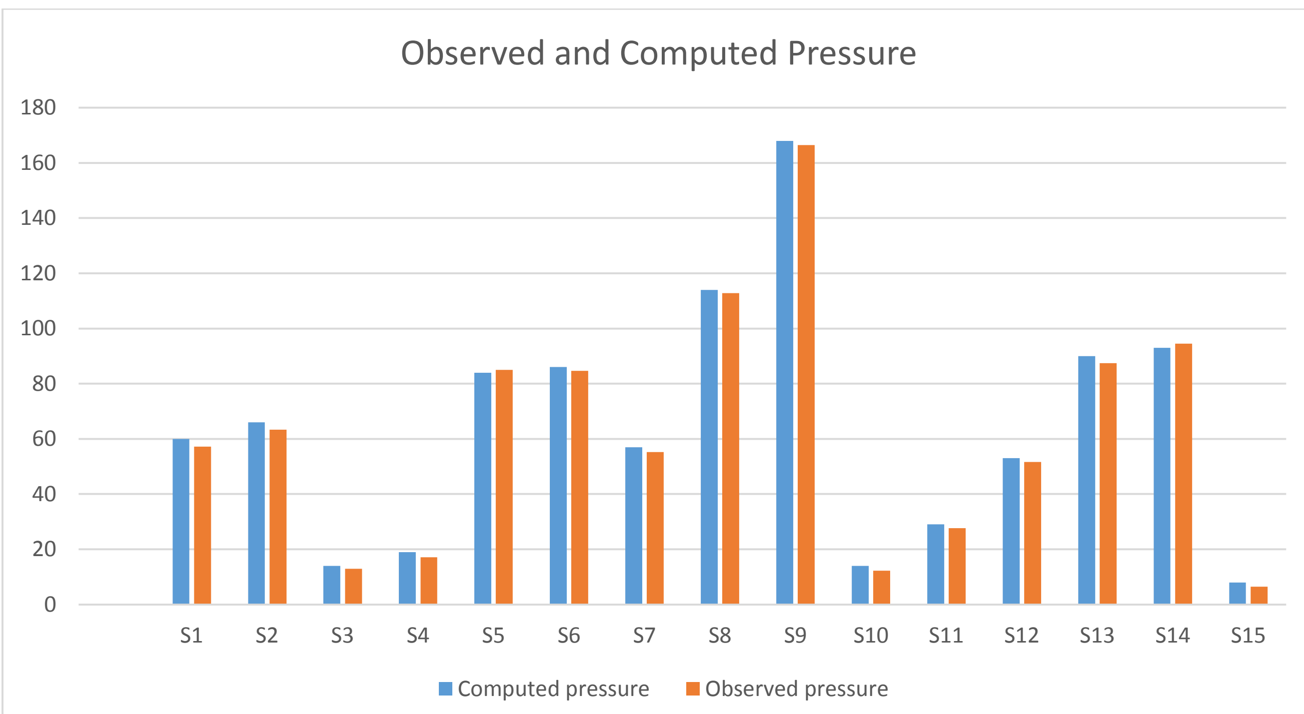


Figure 4-1: Graphical representation of the computed and observed pressure value during peak demand time.

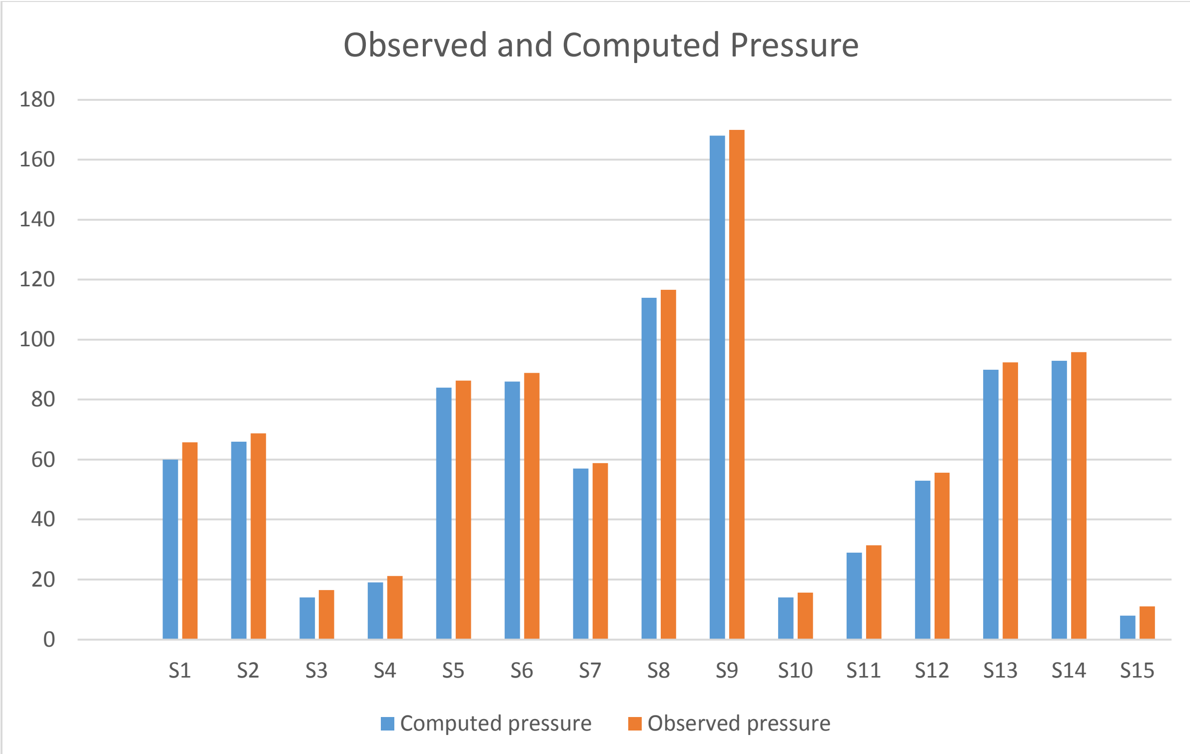


Figure 4-2: Graphical representation of the computed and observed pressure value during night flow (low demand time)

4.3.2. Model Validation

As shown in the figure (4-3) it explain the results of correlation value (R^2) is 96.9%. Thereby, the calibrated pressure value was validated within the recommended standard.

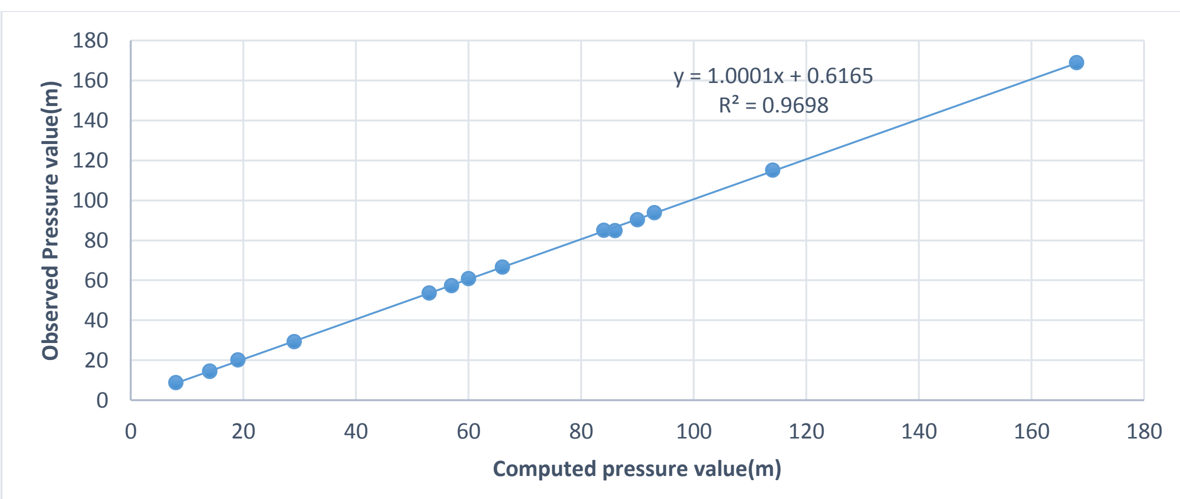


Figure 4-3: Correlated plot during pressure validation.

4.4. Hydraulic Analysis of Water Distribution Network

In the current Gimbi water supply system, clear water shall be delivered to the service reservoirs directly through the transmission main and which is completely isolated from the distribution system. This section describes the results of water distribution and hydraulic analysis in the system.

4.4.1. Existing Reservoirs Capacity

Therefore, as per the design criteria of the (MoWIE 2006) the maximum day factor usually varies between 1.0 and 1.3. Hence, a maximum day factor of 1.2 was adopted for assessing the maximum day water demand and reservoirs capacity for Gimbi town and applied it corresponding to the total average day demand of a particular year (2019).

$$\text{Maximum Day Demand} = \text{MDF X Average Day Demand} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots .4.4$$

But, the minimum service level required for the town to achieve the Second Growth and Transformation National Plan for the Water Supply and Sanitation Sub-sector (GTP-2) is 60l/c/d. Therefore, the average day demand required for the year 2019 is:-

Domestic demand = 60l/c/d x 51253 = 3075 m^3/d

Adding 10% for commercial, 5% for public demand, 20% for UFW and 10% for firefighting the required average day demand and maximum day demand of the Gimbi town is:

ADD = 4459 m^3/d

MDD = 1.2 x 4482 m^3/d = 5351 m^3/d

Accordingly, the current (2019) required reservoirs volume capacity for water demand of Gimbi town was estimated as;

$$\text{Reservoir capacity} = \text{Maximum day demand x}\frac{1}{3}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots .4.5$$

Reservoir capacity = Maximum day demand *1/3

= 5351 m^3/d *1/3 = 1784 m^3/d.

Hence, from the above finding to satisfy the current water demands of Gimbi town; the total clear water reservoir required is 1784 m^3 volume capacity of standard reservoir. But, in the existing water supply system of Gimbi, three storage tanks which serve as clear water tank and service reservoir had a capacity of 2900 m^3. This indicate, the existing reservoirs capacities is enough for the current population of 2019.

4.4.2. Pump Capacity

One of the main components of water distribution systems is the pump stations. Pumps were deliver energy to the hydraulic system in order to overcome elevation difference and head losses due to pipe friction and fittings. Pump head curve is one of the necessary input parameters for water distribution modeling and according to (Thomas M. W alski, Donald V . Chase et al. 2003) is an energy equation which used for solving pipe network problems. Hence, the developed pump head curve during model simulation work were presented as figure 4-4 below

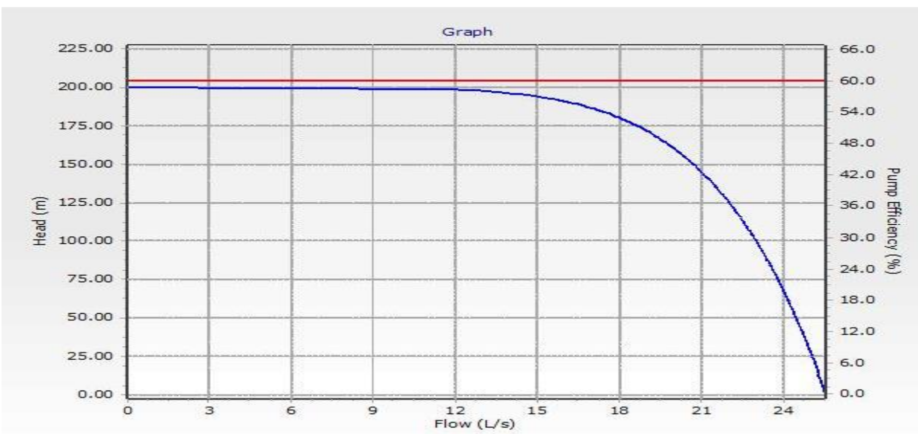


Figure 4-4: Developed pump head curve during model simulation

For this study, pump efficiency were conducted in order to determine the pumps capacity in the town water distribution system. According to field observed data and model simulated result which presented in (appendix- B &C); the pump brake horse power and maximum water power were collected as 22.5 kW and 14 kW, respectively. Therefore, using these finding the efficiency were assessed manually and discussed as below.

$$\text{Pump Efficiency} = \frac{\text{Water Power}}{\text{Pump Power}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..4.6$$

Pump power refers to the brake horsepower on the pump shaft, and it is difficult to measure in the field.

Water power is computed from the following relationship:-

$W_P = C_f \, Q \, H_P \Upsilon$ ……………………………………………………………………………… 4.7

Where W_P= water power (hp, Watts)

Q= flow rate (9, l/s)

H_P= head added at pump (154 m)

ϒ= specific weight of water (9810 N/m3)

C_f = unit conversion factor (4.058 x 10^{-6} English, 0.001 SI)

Maximum Pump Efficiency = 14kw/22.5kw =60%

According to the pump characteristic comply with (ISO 9906:2012); most pumps which present and perform in a good condition have an efficiency of 60-80%. Therefore the pump is in a good condition. But the main problem is that due to fear of the water power they doesn't connect the pump recommended in the design which can lift the current water demand but they are using two pumps with capacity of 10l/s. as per information from water utility when they used the designed pump capacity there is high leakage of water in the distribution. For this reason they may choice to operate with low capacity of the pump. But this doesn't give the solution. Therefore the water utility have to check the pipe laid in distribution whether they are connected properly or not.

As per the computed WaterGEMS model outputs (Appendix-B&C) and information obtained from Gimbi town water service office; those pumps performing in the system were operating an average of effective 24 hours in a day. With this the pumps maximum capacity of delivering water to the distribution system was discussed as the fallowing below.

$$\text{Maximum capacity of pump} = \text{Maximum Pump design capacity X Effective pump operation time} \ldots\ldots\ldots\ldots 4.8$$

=20 l/s*24 hr/day = 1728m^3/d

Therefore, the pump can only deliver maximum water of 1728 m^3/d to the system. But from the above finding the current maximum water demand of the town is 5351 m^3/d (62 l/s) and this indicates that the pumps capacity were not meet the current water demands of Gimbi town and have to be improved.

4.4.3. Transmission Main Line

It is discussed that the transmission main was isolated from the distribution network and it gives water from treatment plant to main reservoir, from main reservoir to booster station reservoir , from booster station to old tank by gravity means and from booster station to radio station tank by the aid of pump. Because of transmission main was isolated from the distribution network and there are no users from transmission main the hydraulic grade line is not decreased which give sufficient pressure to the system. In the transmission main there are low velocity and all the velocity where below 0.6m/s which is less than recommended velocity in distribution system as (MoWIE 2006). Hydraulic grade line in distribution system is expressed in Appendix E and F.

4.4.4. Distribution Line

Regards the topography of Gimbi town, the locations of nodes in the water distribution line is in close proximity to each other. The maximum and minimum water pressure in the distribution system was 180m and -19m head around downstream of the town and service reservoir respectively. According to the design criteria of (MoWIE 2006) , the maximum and minimum water pressure in the distribution system is 80m and 15m respectively. Beside these comparisons; the current Gimbi town existing water distribution network was operating out of the recommended limitation. This is because topography of the town lies on plateau landscape within this landscape there are also gullies and river course were water distribution network may go up and down toward the road. In the Distribution main there are low velocity and all the velocity where below 0.6m/s which is less than recommended velocity in distribution system as (MoWIE 2006).

4.4.5. Pressure Variation in the Distribution System

Variation of water pressure in the distribution system is mainly because of hourly fluctuation of water demand. As shown in figure 4-5 below; the water pressures in Gimbi water distribution system were a function of topography factor that is Variation of elevation difference in most part of the town has also an impact for the rising and reduction of water pressure in the network.

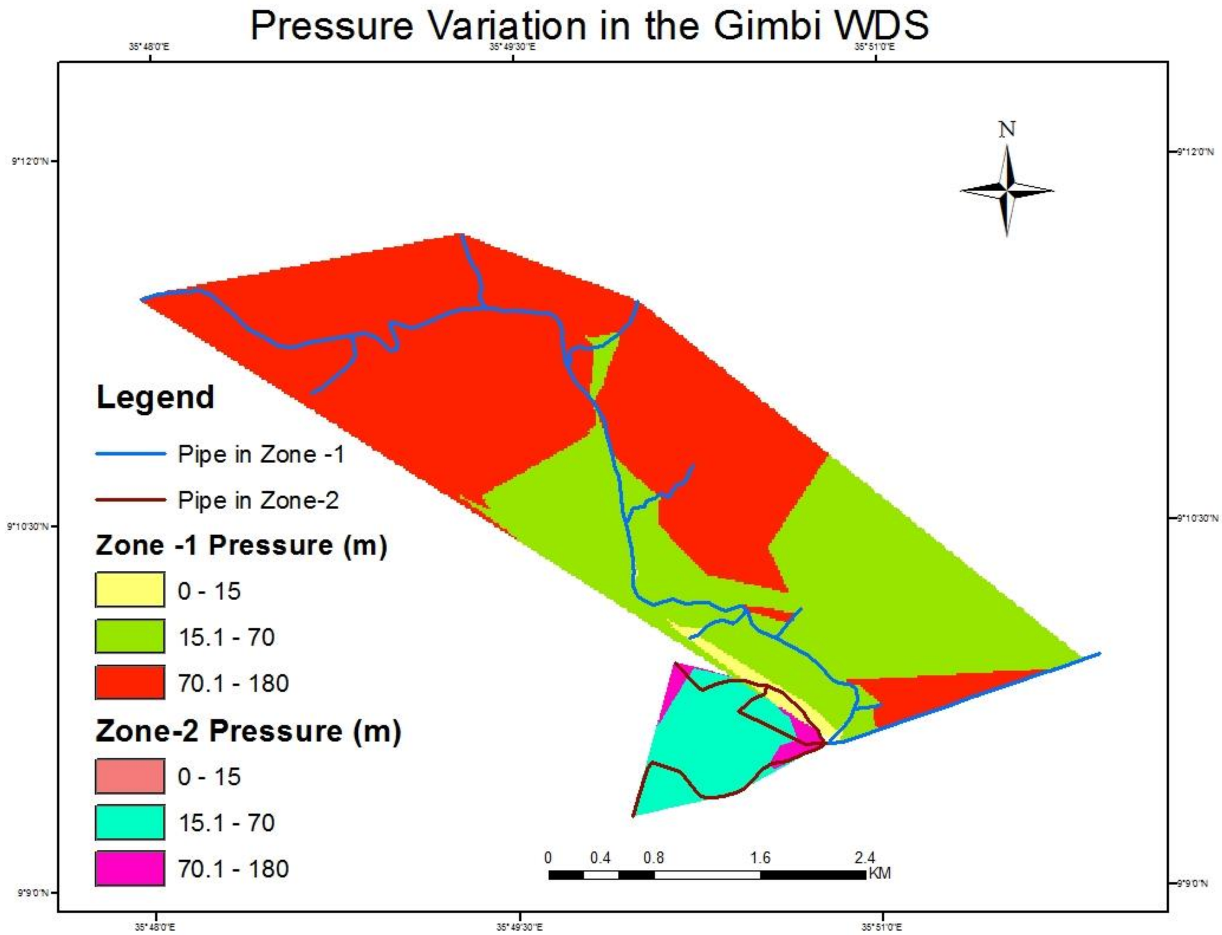


Figure 4-5: Pressure variation in the distribution system at average day demand.

4.5. Identified Problems and Improvement Mechanisms

4.5.1. Identified Problems

a) Negative pressure

Accordingly, the minimum water pressure recorded in the system was -19m head. From these results it was discussed that; the minimum value of water pressure may cause water shortage for the peoples found at higher elevation above the reservoir location.

b) High pressures in the distribution system

As per model analysis, maximum water pressure in the transmission main was found at the downstream of the town with the value 180m head during low demand time and 164m head during high demand time. Among the node with high pressure Node 109,110, 112 are one of them. And the pressure value at these node are 168,172 and 180 respectively.

Pressure in water distribution system 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. The former one is frequently achieved in setting minimum pressure to be maintained at each junction. The later one is achieved differently in setting allowable pressure to be maintained in the system.

Minimum pressures at peak hour demand sufficient to serve the highest supply point in the network is not less than 10m to 15m would be required to serve highest supply point in the distribution system. And Maximum static pressure during low demand periods typically at night should be as low as practicable to minimize leakage. The maximum pressure in mains is considered not to exceed 70m to limit leakage and stresses on pipes (MoWIE 2006).

With regard to current simulation of actual water distribution condition at average day demands as shown below in both tabular and figure and the results for pressure is summarized in the table (4-6) and detailed in Appendix E.

In Gimbi WDS the minimum pressures are observed mainly at the nodes situated near to tanks and at the node found in remotest area of the distribution system are susceptible to low pressure. Whereas majority of nodes located in middle of the network receive optimum pressure which does not violet minimum or maximum allowable pressure range. On the other hand, maximum pressure in the distribution system is found at the downstream of the country where the elevation in the area is low. The actual node pressure simulation in the distribution is shown in the table (4-6).

Table 4-6: Distribution of actual node pressure

Pressure (m)	Nodes (number)	Percentage (%
>70	93	48
70-15	102	51
<15	2	1
Total	197	100

As portrayed in Table (4-6), Figure (4-5) and detailed in appendix E, 48% of nodes are liable to extremely high pressure. This figure is relatively high. Minimum pressure less than 15m pressure head is about 1% only, and 51% of nodes were received water of optimum pressure at low consumption hour. The actual node pressure contour map is showed in the (Figure 4-5) above.

c) Low Velocity in the Distribution system

Velocity of water flow in a pipe is also one of the important parameters in hydraulic modeling performance evaluation of the efficiency of water supply distribution and transmission line. Velocity distribution is also varying with demand pattern changes. The water supply system network velocity is summarized in the table (4-7).

Table 4-7: Velocity distribution in the system

velocity (m/s)	pipe (number)	Percentage (%
>2.5	0	0
2.5-1.5	0	0
1.5-0.6	0	0
<0.6	203	100
Total	203	100

As stated in table (4-7) none of pipes shows the maximum velocity above the permissible velocity that is (>2.5m/s) and all of the pipes are below the minimum velocity in the distribution that is less than 0.6 m/s. While, none of the pipes shows the permissible velocity ranges. Improving or upgrading the existing water supply distribution is also considering and checking the velocity minimum and maximum limit based criteria.

In Gimbi WDS minimum velocity may occur due to the imbalance between the peak hour water demand adopted in design report and the current supply. The current peak hour demand supply is less than that of adopted in design report because of that the water utility may supply partially due to fear of the pipe burst. Velocity distribution in Gimbi WDS is presented in the figure (4-6).

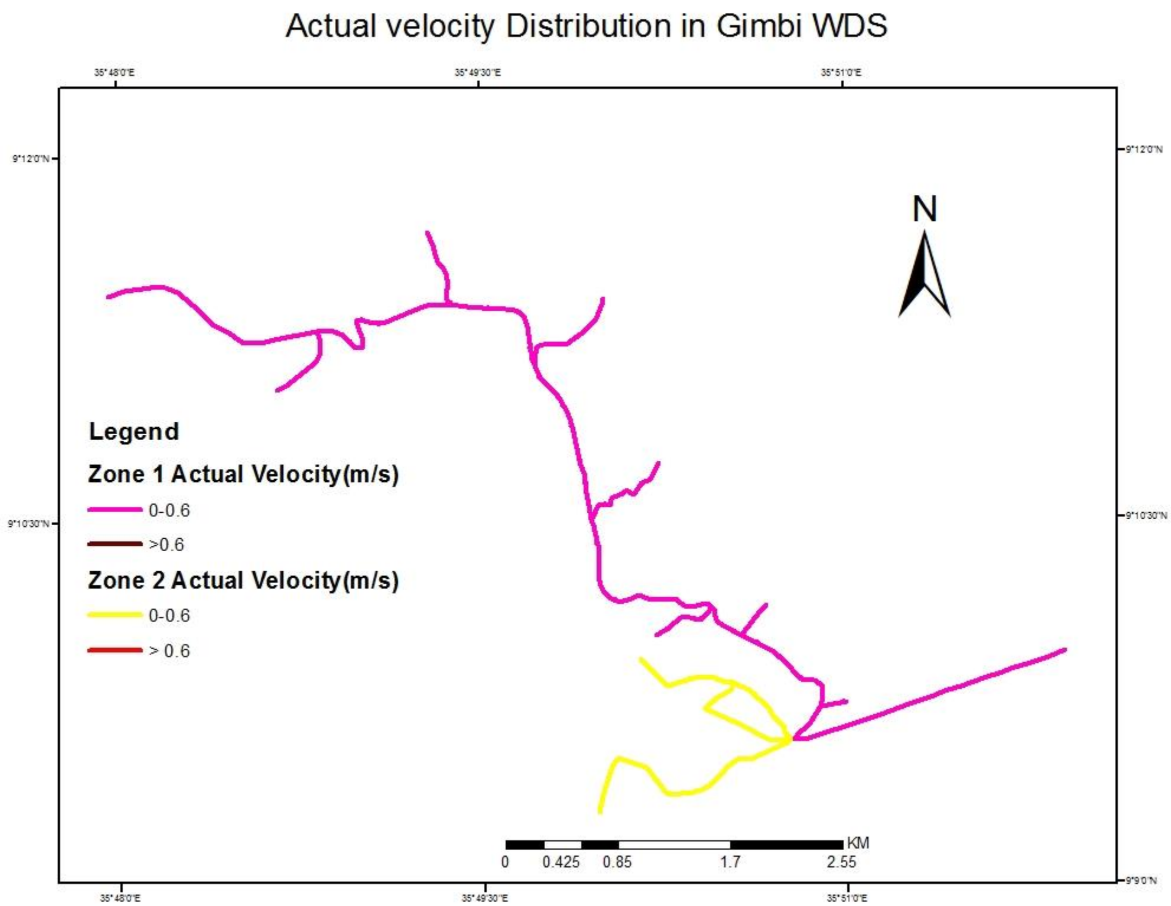


Figure 4-6: Actual velocity Distribution in Gimbi WDS

High pressures are usually caused by serving customers at low elevation in the WDS. Usually, high pressures are easiest to evaluate with model runs at low demands. This range corresponds to minimum night time demands for a typical system. If the engineer feels that pressures are too high, the usual solution is to establish a new pressure zone for the lower elevation using Pressure Resisting valves (PRV).

4.5.2. Improvement Mechanisms

In designing or improving a water distribution system there are sets of design criterion to be considered. Among them, pressure and velocity are important one. The design criteria used in the design of water supply distribution system components, nodal pressure during the period of peak demand, and optimum velocities of the transfer and distribution mains are as follows (MoWIE 2006).

- ✓ Minimum static head from the distribution system is 15 m.
- ✓ Maximum static head within a pressure zone was limited to 70 m.
- ✓ Maximum velocities of major transfer mains <2.5 m/s.
- ✓ Maximum velocities of distribution mains <2 m/s.
- ✓ The minimum velocity of flow in a pipeline is 0.6m/s, but for looped systems there will be pipe lines with sections of zero velocity.

Depending up on the above criterion most of the pressure value in the Gimbi WDS is above allowable pressure value and needs reduction of excessive pressure to the desired allowable value. Therefore, this excessive pressure is reduced by installing pressure reduce valves (PRV) at links which has maximum pressure. The location of PRV to be installed in the distribution system to decrease the excess pressure in the system is as shown in the table (4-8) and in the Appendix-G.

Table 4-8: Pressure reduce valves in the improved system

Zone	Label	X	Y	Elevation	Diameter(mm)
Zone 1	PRV-1	811633.35	1015549.98	1,914.50	200
	PRV-2	810898.09	1016427.13	1,917.79	350
	PRV-3	810265.98	1016953.14	1,883.01	300
Zone 2	PRV-4	812360.95	1014065.22	2052.32	250

The results of the model for the improved systems are illustrated in the table (4-9), figure (4-7) and in the appendix- F.

Table 4-9: Improved system pressure with PRV.

Pressure (m)	Nodes (number)	Percentage (%
>70	16	8
70-15	156	79
<15	25	13
Total	197	100

As the results of the improved systems are summarized in the above table (4-9) only 8% of nodes have maximum pressure above allowable (70m) in the system. This value is improved more than 80% compared with the actual node pressure in the distribution system before it is improved.

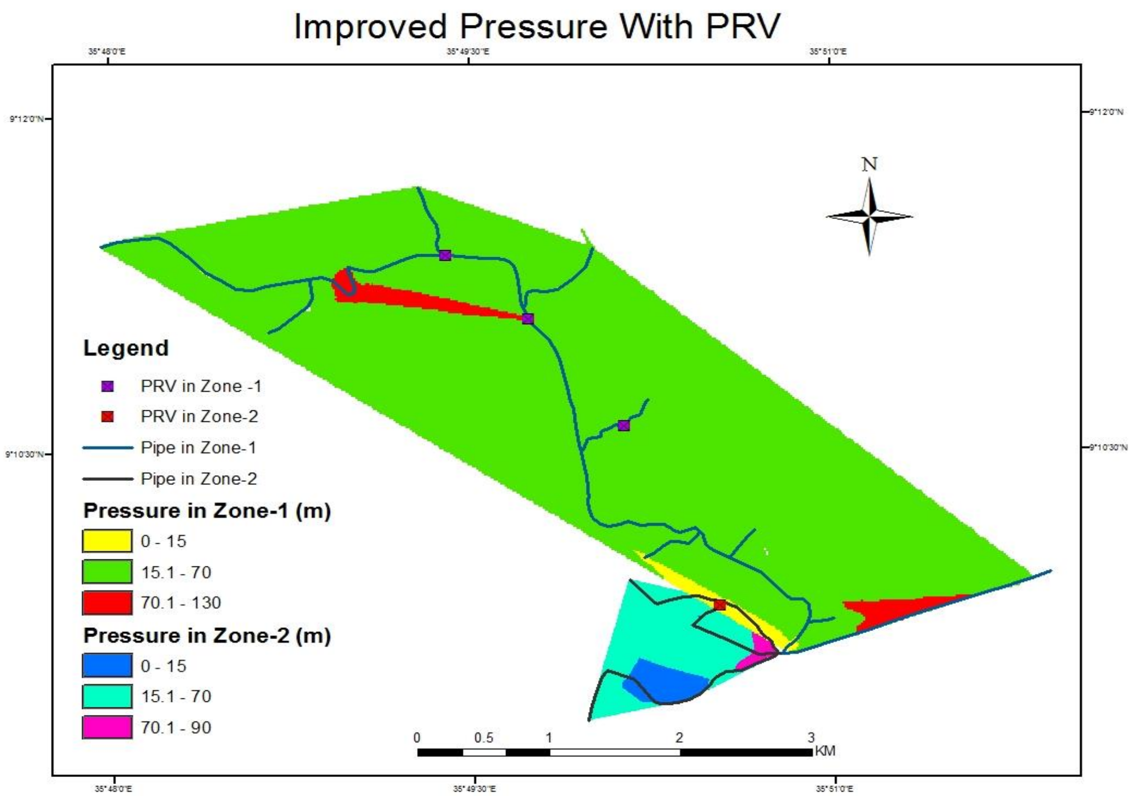


Figure 4-7: Improved pressure contour map With PRV.

Many of the scholars states that improving velocity in water distribution system is done by resizing the pipe diameter in the system. As we see from the table 4-7 actual velocity in the distribution system is less than 0.6 m/s this shows that the velocity in the system is low. And the reason for low velocity in the system is due to large diameter of pipe and less water flow in the system. This problem is occurred because of the water production in the system is decreased due to the fear of water leakage when the water is fully supplied. It is observed that the low velocity in the distribution system is improved when the water is fully supplied. Therefore, in this study resizing the pipe diameter in order to improve the velocity is not recommended. Because, this pipe material is an asset of the utility. Therefore, instead of resizing the pipe diameter, fully supplying of the water to satisfy peak hourly demand as per the design is recommended.

When the water is fully supplied for the current required water demand of the town, velocity distribution in both peak day demand and minimum day demand becomes under allowable velocity ranges (0.6m/s-2.5m/s). Improved velocity distribution in the model system is illustrated in the figure (4-8) and in the Appendix-G.

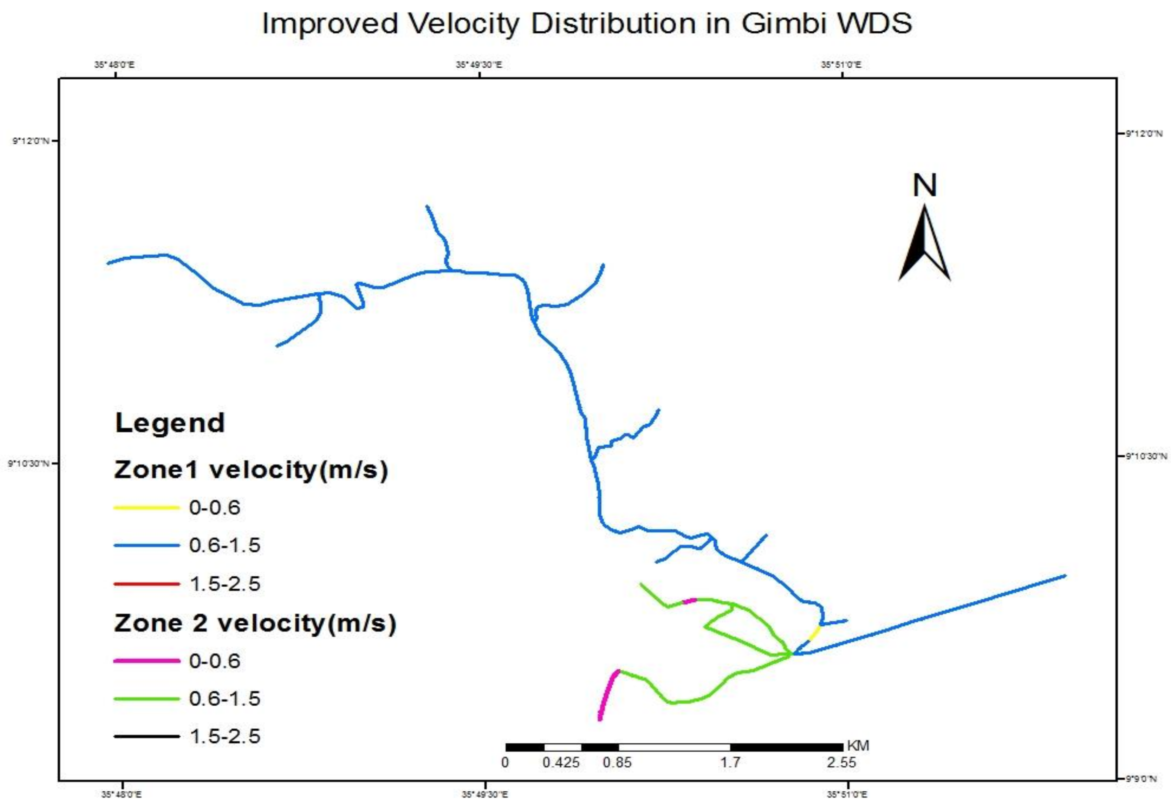


Figure 4-8: Improved velocity in the distribution system

4.6. Water Loss Analysis

Analyzing water loss and leakage needs a detail data due to its complex nature. The data are usually scarce in developing countries that the case of Gimbi town is also similar.

One of the major challenges of water utilities is high volume of water loss in their distribution networks. If a large quantity of supplied water is lost; it is difficult to meet the required quantity to demands, and correspondingly made challenges to keep the water tariffs in the system at a reasonable level. Whereby, water loss for Gimbi town was assessed and discussed as below;

4.6.1. Percentage of Water Loss

Non-Revenue Water includes water losses in the water distribution system, illegal connections, and improper metering and recording. The amount is expressed as percentage of the total produced water from the water supply system. According to the town water service office; the annual Non-Revenue Water in Gimbi town during the year 2019 was estimated using the formula below and the result is presented in the Table (4-10).

$$\textbf{Total percentage of water loss } (\%) = \left(\frac{\textbf{Unbilled volume}}{\textbf{System production}}\right) * \mathbf{100} \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots..4.9$$

Table 4-10: Percentage of Non-Revenue Water

Year	production(m^3/month)	Billed consumption(m^3/month)	Non-Revenue	
			m^3	%
October, 2018	43500	22185	21315	49
November, 2018	43950	21536	22414	51
December, 2018	44752	21481	23271	52
January, 2019	45380	20875	24505	53
February, 2019	46440	20898	25542	55
March, 2019	47025	21631	25394	54
April, 2019	48582	21862	26720	55
May, 2019	50054	22024	28030	56
June, 2019	51050	22519	28531	56
July, 2019	51100	22090	29010	57
August, 2019	51120	22570	28450	56
September, 2019	51151	22369	28782	56
Total	574004	262040	311964	55

As we have seen from the table (4-10), the amount of total water loss or NRW is increasing monthly with respect to water production and the average annual NRW in the system is 55%. But, according to (MoWIE 2006) for normal water distribution system 15% of water loss is generally regarded as good, and Uneconomical to try and reduce and for new water supply system Unaccounted for water may reach up to 40% of water production. Based on this, percentages of water loss in Gimbi Water distribution system are high and require strategic scheme management.

4.6.2. Water Loss as Per Number of Service Connection

One of the appropriate indicators of water loss in the distribution system is describing it as per number of service connection (liters per service connection per day, l/c/d), and it gives more precise figure than NRW as a percentage of inputs volume. Based on the obtained data; the total number of service connections in Gimbi town was 5734 based on this, the volume of water loss as per number of connection were analyzed from the total unbilled volume.

$$\text{Loss per connection} = \left(\frac{\text{water losses} * 1000\ \text{l/m3}}{\text{total number of connection} * 365\text{day}}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..4.10$$

Loss per connection = (311964 m^3 *1000 l/m3)/ (5734*365 day) = 150 liters/con/day.

According to the (José Sardinha 2017) guide line for water loss level in distribution system is Good condition of system if it is less than 250 Liter/connection /day, Average condition 250 -450 Liter/connection/day and Bad condition of system if it is greater than 450 Liter/connection/day. Based on the above guide line performance indicator of physical loss target matrix of Gimbi town water loss per connection were found in Good condition which is 150 l/connection /day < 250 l/connection/day.

4.6.3. Water Loss as Per Pipe Length

The other good indicator of water loss in the distribution network is determining loss as per pipe length (liters per kilometer of pipe line per day, l/km/d). According to the town water utility, the total length of water distribution line including both main and privet pipe line (from property boundary to customer's meter) were estimated about 144.4 km. Therefore using this, the estimated amount of water loss; as per kilometer of the pipe length of the town was calculated as;

$$\text{Loss per pipe length} = \left(\frac{\text{Unbilled volume} * 1000\ \text{l/m3}}{\text{total number of pipe length} * 365\ \text{day}}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..4.11$$

Loss per pipe length = (311964 m^3 *1000 l/m3)/ (144.4*365 day)

= 5920 liters/km/day

As per revised literature of (José Sardinha 2017) the general rule for water loss level guide line for water loss level in distribution system is Good condition of system, if loss per length of pipe is less than 10,000 Liter/km main/day. The average condition is, if water loss as per pipe length is 10,000 -18,000 liter/km main/day. And bad condition if water loss as per pipe length is greater than 18,000 liter/km main/day. Therefore, this figure shown that Gimbi water distribution system is in good condition in terms of water loss per pipe length.

Category of water loss

Non-revenue or total loss is the sum of real loss and apparent loss. However, for this study unable to compute the actual total real loss and apparent loss due to limitation of data like average leakage flow, number of pipe burst, number of water loss due to customer meter inaccuracy, illegal connection and etc. Therefore, in this study physical losses and commercial losses are estimated according to sectorial estimates of worldwide NRW volumes by (H.E.Mutikanga, Sharma et al. 2010) which is expressed in the table (4-16).

Table 4-11: Sectoral estimates of worldwide NRW volumes

	Supplied population (in millions ,2002)	system input (L/cap/d)	Level of NRW (% of system input volume)	Ratio	
				physical losses	commercial losses
Developed countries	744.8	300	15	80	20
Eurasia countries (CIS)	178	500	30	70	30
Developing countries	837.2	250	35	60	40

Source:(H.E.Mutikanga, Sharma et al. 2010)

Real losses (physical losses) and apparent losses (commercial losses)

Commercial losses are at times known as apparent Losses (AL). Such losses represent those losses that are a result of faulty meter reading, meter inaccuracies, unauthorized water usage, and data miss handling (AWWA, 2009).

From the table (4-15), the total non-revenue water was recorded as 311964 m^3. These amount of water loss were categorized as physical/real and apparent loss. Multiplying ratio of physical losses and commercial losses from sectorial estimates of worldwide NRW volumes according to

(H.E.Mutikanga, Sharma et al. 2010) for developing countries I have got the physical and commercial losses and the result is presented in the table (4-12).

Table 4-12: Estimated physical losses and commercial losses in the system

Total NRW	Physical Losses (60% of NRW)	Commercial Losses (40% of NRW
311964	187179 m^3	124785 m^3

4.6.4. Calculating Infrastructure leakage index (ILI)

$$ILI = \frac{CARL}{UARL} \quad \ldots 4.12$$

Where,

CARL- current annual real losses.

UARL- Unavoidable real losses.

Unavoidable real lose (UARL)

According to (Lambert and Lalonde 2005) the general equation for the UARL Calculation is

$$\mathrm{UARL}\left({}^{\mathrm{liters}}/_{\mathrm{day}}\right) = [18\,\mathrm{Lm} + 0.8\,\mathrm{Nc} + 25 * \mathrm{Lp}] * \mathrm{P} \quad \ldots 4.13$$

Where, L_m = mains length (km); N_c = number of service connections; L_p = total length of private pipe, property boundary to customer meter (km); and P = average operating pressure.

According to (José Sardinha 2017) the UARL can be applied to networks with an average operating pressure between 20 and 100 m, a service connection density of between 10 and 120 connections per km of network and meters for clients on service connections with medium length of up to 30 m. Jose also states that in networks with a low density of service connections, usually rural areas with less than 20 service connections per km of mains, it is more appropriate to express the indicators of losses in terms of length of mains, rather than relating them with the number of service connections or consumption locations. In the case of Gimbi town WDS the average operating pressure in the system is 77 which has fitting the criteria, service connection density is 40connections per km of network and as discussion with water utilities length of meters for clients on service connections is estimated to more than 40m. And this is disagree with the above criterion.

(Farley 2008) describe UARL as the most reliable predictor of water loss for the system with more than 5000 service connection, connection density number of connection (Nc)/length of main(Lm) more than 20 per km and average pressure more than 25meters. In the case of Gimbi town WDS it satisfy this criteria.

According to (GAWP 2016) there are several basic methods for calculating average operating pressure. Those are:-

- For water system with distribution model, an average pressure can be easily calculated by averaging the pressure at each node in the model.
- For water system with a single pressure zone, representative sample of static pressure readings across the zone should be taken and averaged.
- For water systems with multiple pressure zones, representatives samples of static pressure readings across each zone should be taken, and then the averages for all zones should be combined in to a total weighted average, based on length of per zone.

Gimbi WDS has two zones. Therefore, to compute average operating pressure the method by (GAWP 2016) is used. Applying this method the average operating pressure for both zone is calculated in the table (4-13).

Table 4-13: Multiple pressure zone calculation

Zone	Average zone pressure(m)	Length of main (km)	Weighted % of Total length of mains.
1	82	14	(14/18.4)*100 = 76%
2	61	4.4	(4.4/18.4)*100 =24%

Average operating pressure = (82m x 76%) + (61m x 24%) = 77

Therefore, the value of UARL (liters/day) is:

UARL (liters/day) = (18*29.4+0.8*5734+25*0.03)*77= 394020 liters/day = 394m^3/day

Therefore, Infrastructure leakage index (ILI) is computed as;

$$ILI = \frac{CARL}{UARL} = \frac{512819\ liters/day}{394020\ \text{liters/day}} = 1.3$$

ILIs of the system is related to banding of World Bank Institute (WBI) to judge the Performance Indicators of ILI of the system. Therefore, the performance indicators of ILIs of Gimbi water supply system was 1.3 which is band A of developing countries. According to World Bank Institute (WBI) this shows that further loss reduction may be uneconomic unless there are shortages; careful analysis is needed to identify cost effective improvement.

Table 4-14. World Bank Institute (WBI) Banding System to Interpret ILIs

World Bank Institute (WBI) Banding System to Interpret ILIs			
Developing countries	**Developed countries**	**BAND**	General description of real loss performance management categories
<4	<2	A	Further loss reduction may be uneconomic unless there are shortages; careful analysis is needed to identify cost effective improvement
4 to 8	2 to 4	B	Potential for marked improvements; consider pressure management, better active leakage control practices, and better network maintenance
8 to 16	4 to 8	C	Poor leakage record; tolerable only if water is plenty and cheap; even then analyze level and nature of leakage and intensify leakage reduction efforts
>16	>8	D	Very inefficient use of resources; leakage Reduction programs imperative & high priority

Source:(H.E.Mutikanga, Sharma et al. 2010).

Finding on water loss analysis

The current Gimbi town water supply system contain more than 5734 private connection and more than 144 km of total pipe length. But, the system is supplying water that are less in ammount compared with the designed one. Therefore, the system losses analysis in terms of number of connection, main pipe length and ILI values are insensitive to indicate the system lose. Because, minimum water production results minimum water loss volume. When dividing this minimum water loss volume for number of service connection or for main pipe length the obtained value is also minimum. In order to compute UARL the general criterion to be fulfilled is missed especially average length of customer meter from service connection. As we have seen from the above results the water loss in the system in terms of service connection, main pipe length and ILI value shows water loss in Gimbi town WDN is in good condition which is unexpected without taking any management action and far from the reality. Therefore, in this study percentage of water loss are good indicator of water loss in the system.

4.7. Major Factors Contributing to Water Loss in Gimbi Town

There are several reasons for the high level of water loss in the water distribution networks. Beside to these, the major sources of water loss experienced throughout Gimbi water distribution system were as a result of:-

4.7.1. Improper Installation

According to the information obtained from discussion with the staff of Gimbi water supply service office, due to the political pressure from the administration office the contractor was forced to lay the pipe earlier before basic treatment work and reservoir work is finished. Also for installing of the pipe the mass of peoples are called by the administrators. Therefore, the pipe was installed by the participants of the people without knowledge and experience of laying the pipe. According to staff of Gimbi water supply service office the pipe laid was carried out without being checked the proper diameter of the pipe in the system. Therefore, the improper installation of the pipe in the distribution system may cause high water losses especially in the transmission line of the town water distribution network.

4.7.2. Age and Size of Pipe

As per information from Gimbi water supply service office except the main line and secondary line most of the distribution and service pipes that deliver water directly to the consumers is unchanged and the new system directly connected to the previous line that may contain a lot of aged pipe. Whereby, these pipe materials suffered its quality due to long service time, water carrying capacity and environmental conditions and previously used small diameter pipe doesn't fit for the current demand in most area which may require modifications. Therefore, age and pipe size are the main factors for frequent pipe bursting and real losses in the town water distribution network. According to the water utility water lost from the system mostly occur in areas where the new pipe is not replaced and water loss may also occur in the transmission main and distribution line served around center of the town in the areas having large consumer.

Small size and aged pipes laid in Gimbi town water distribution network without any replacement is shown as in the table (4-20).

Table 4-15: Small size and aged pipes in Gimbi town water distribution network

Description	Average year	Diameter (mm)	Length (km)
DCI	10 Years And More	150-200	4.8
	10 years and less	150 -200	5.2
GI	10 Years And More	80-150	15
	10 years and less	80-150	12
PVC	10 Year and less	50-150	13

4.7.3. Metering Inaccuracy

The type and brand of water meter has an effect in the accuracy of customer water metering. From the total of 5,734 a single-jet type (class B, dry dial) customer water meters; 3617 DN15mm, 2068 DN 20mm, 106 DN 25mm, 24 DN 32mm and 12 DN 90 mm were installed in Gimbi town water service system. As per the town water service office; 70% of these water meter were manufactured in China, while remain 30% was manufactured in Poland. In this study due to the lack of data the volume of water lost due to metering inaccuracy is not defined. But as the discussion with water utility staff members even though they don't know its volume, there was water lost due to metering inaccuracy.

4.7.4. Data Handling Errors

Data handling error in the meter reading and billing process were contributed for apparent losses. Customer meter reading practice; especially unbilled metered trends were the common problem of Gimbi town water service office. Whereby, recording of under/overestimated figure lead the water utility to improper collection of revenue, and at the end of the month the authority were lost money. Accordingly, as per the water authority due to poor recording of data some peoples may make conflict when they come to pay for water they used monthly and also they states as they found the problem is data handling error. As per water utility due to silence of the customer who pay less for water, the utility faced problem to know the total volume of water loss in this cases.

4.7.5. Illegal Connections

As a developing town there are a significant number of illegal users of water within Gimbi town water distribution network, and were contribute to the reduction in service level to authorized consumers. The town water utility were not know the actual figure of residences that do not pay water tariffs but received water from the distribution system. But as per the feedback from the water utility; construction sectors, different enterprises and hotels in the town are mainly contributing in large number. Due to limitation of data, water losses as result of illegal connections were not analyzed in this study. But the water utility have to be properly manage illegally connected users contributing large volume of water loss which may also decrease financial capacity of the system.

4.8. Leakage Management Practice of the Town

Many water utilities have less attention for water loss as a result of their poor maintenance capacities. In Gimbi water service it was observed that; there are no enough budget, proper instrument, accessories, and strong policies for suitable leakage management. As per discussion with water utility they only repair the system after they received the pipe burst report and they doesn’t make any management practice before the burst was occur. And these have considerable impact for physical losses in the town water distribution system.

A strong management and organizational setup is the main reason for good leakage management practice in the water supply services and to the satisfaction of the customer’s. Good leakage management practice is one of the ways of reducing water loss in the distribution system. The leakage management trends of Gimbi town water supply service office were assessed and described as below.

According to Gimbi town water service office, the total numbers of staff members including supporting sections are 43 in number. Of these six are professionals from technique section (one electrician, one electromechanical, two plumbers and two operator). Based on the policy and guidelines of the regional administration, the town water supply service manager can organize, directs and administers the overall activities of the water service unit. While, regard to the leakage management view, the operation and maintenance section of the utility is the most important section in the water supply service and the head of this section shall be accountable to the manager of the water service office. The main activities of this section are direct, coordinate and control the water supply operations and maintenances work including production, distribution, leakage and laboratory units.

4.8.1. Management Practice

As per the collected information and field observation, the operation and maintenance culture of Gimbi town water distribution system were presented as below:

Operation system:

Operation system carried out in the utility includes the operation of the intake, treatment units, pumping stations and the associated transmission mains. There is no well-educated permanent operators who control the above assets in the treatment plant site. The person work at this site is only for inspection and it has no skill in controlling the system. Mostly the operation system was controlled by the utility guards (non-skilled man power) who protects the intake and treatment plant site**.** There is the practice of recording treated water production and consumption figures by the utility experts.

Maintenance culture:

The overall maintenances of system components were not checked by schedule and it was maintained during failure or damage is occurring. Valves and all accessories installed in the main were not properly used and maintained regularly (greasing and cleaning). No functional controlling check valves within the network. Accordingly, when failure is happened; the utility were maintaining the system by switching off the pumps operating at pumping station. Whereby, the technique group forced to stay until the water pressure was reduced in the system. Maintaining, removing and replacing of both customers service line and bulk flow meters were done when the utility was reported or requested by the customers during failure is appear. The other collected information was due to limitation of budget; when failure is occur at customer’s service line, the customers forced to supply all the necessary accessories. Accordingly, until the required accessories is purchased and replaced a considerable quantity of water is lost.

4.8.2. Water Utility Organizational Structure

The total numbers of personnel of the WSSE including the supporting staffs are 43. Out of the staffs 6 are working in the technical sector, 17 in the administration and 11 in the finance section and the remaining 9 are general service workers. According to categories given to town water supply services by the Oromia Water, Mineral and Energy Bureau, Gimbi town Water Supply and Sewerage Enterprise is in the 2nd class.

The existing organization chart is shown in the figure (4-10).

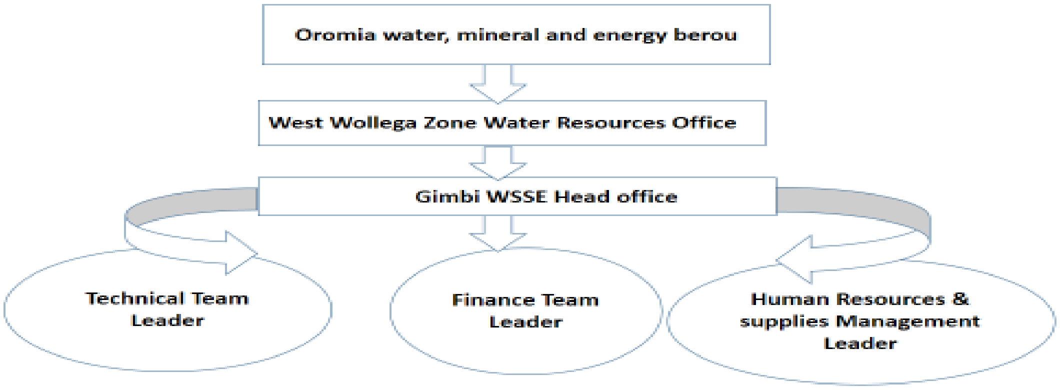


Figure 4-9: Organizational Structure of Gimbi Town WSS

The technical team leader to maintain the existing water supply system were only 6 person which is difficult in monitoring and managing the water supply system because the total pipe length found in Gimbi water distribution system is 145 km including service pipe. This implies that the duty of one person is around 24km which is very difficult in managing and monitoring effectively and efficiently.

4.8.3. Financial Analysis

The main financial source of Gimbi town water service office is the government budget and regional contribution. Accordingly, the financial plan and polices in the town water service system was assessed as below;

Water tariff policies

One of the leakage management strategies in the water utilities is the water tariff carried out in the system. The major objective of water tariff is to make financial sustainability and cost recovery, with the consideration of low-income groups. Accordingly, the water tariff structure for Gimbi town water system was reviewed and applied based on the regional water, mineral and energy bureau recommended value, and the expected capital benefit of the water utility.

As per the town water service office, the tariff structure is adopted as flat and graded rate. Table4.7 below shown that; public fountain users are charged flat rate i.e. the same rate for all consumption. While, house and yard connection users are charged progressive rate polices i.e. the tariff rate increases with the consumption volume of water.

Table 4-16: Water tariff policies

Block	Consumption range (m3/month)	Tariff (ETB)/m3
1	less than 5	4.6
2	6-10	5.5
3	11-30	6.6
4	More than 30	7.9
PF	All consumption	4

Source: Gimbi town water supply service office

From the above table (4-21) the average unit price of 5.72 birr and financially, the water utility were collect 1,586,585 birr from authorized revenue water. But, the authority was lost an estimated of 1,381,561.5 birr as result of total water loss. Therefore, from this figure it was discussed that; the general leakage management trends of Gimbi town water service system were in poor status, and the utility was given less attention for water loss.

5. CONCLUSION

Assessment of hydraulic performance of water supply distribution system to address water distribution bottlenecks within an urban water supply system is important. This can be achieved through investigating the status of the existing distribution system of the network. The main objectives of this study is to assess sustainability and to identify the challenges of Gimbi town WDS.

Currently, Gimbi town is getting clean water from the second water supply project of the town and this project start commissioning since October, 2018. But, this project has stopped its function more than two times due to frequent bursting of pipe with high sound in the distribution system. The basic hydraulic problem found in the distribution system is frequent pipe burst with high sound, high water leakage in most of the area and the communities living at high elevation do not get water.

From the analysis it is identified that, the current (2019) annual consumption of water in the area is only 262040 m^3 which gives the per capita consumption of 14 l/c/d and the size of existing pump capacity which is (20 l/s) is minimum when compared with the current required maximum water demand that is 5351 m^3/d (62 l/s).

Generally, the major constraints of distribution system identified are low capacity of pump, low velocity in which all of the pipes have the velocity less than 0.6 m/s, high water pressure which has maximum value of 180 m and also minimum water pressure observed is (-19 m). These indicate pressure in the system doesn't performing within the allowable pressure value (15-70) according to the urban water supply design criteria set by (MoWIE 2006). Therefore, the water distribution network were faced a frequent pipe bursts and failures during low demand time and exposed to large volume of water loss especially in high pressure zone areas, while during high demand time mostly residences found in dense population and higher level of the town were not received and/served continuous water from the system.

Water loss analysis of the system using annual water balance of production and consumption figure of the town indicate that 55% of treated water was lost as a total Non-Revenue Water which indicate that the leakage management trends of Gimbi town water service system were in poor status, and the utility was given less attention for water loss reduction.

In general, high water pressure, improper installation of the pipe, aging and size of pipe material, metering inaccuracies, illegal connections, systematic data handling errors and poor maintenance practices were found as the major sources of water losses in Gimbi water distribution networks.

6. RECOMENDATION

Based on the analyzed findings the following recommendations were mentioned for existing water supply system of Gimbi town:-

- As the current water demand in the town is much greater of the daily water production of the system, so it is necessary improving the pump capacity and replacing the new pumps with the required hydraulic performance that can deliver for the current year (2019) maximum day demand of the water (62 l/s) solves water scarcity in the town.
- Pipe of old system which has average of 30 year should be replaced with new pipe due to most of the leakage occurred around the old pipe system.
- In downstream of the town where pressure is high it is necessary to provide pressure reducing valve or pressure break tank in the distribution system that may reduce pressure of system.
- To minimize the pressure in the distribution system to an acceptable level it is recommended to install at least 4 PRV in the WDN at different location. The installation of 4 PRV in the model reduces the system pressure to an acceptable range of pressure value (15-70).
- Check the installation of pipe which is carried out before 6 years that is laid by political pressure from administrator on the contractor and done without checking of size, materials and its order according to design document.
- It is also recommended that to install stand by generator in the pump station that is used during the failure of the electric power to minimize interruption of water supply occurring due to electric power related issues.
- It is also advised to improve and use old water supply system as alternative that was stopped its function currently due to high treatment cost.
- During water loss analysis, it was observed that large amount of water were lost. So for commercial loss management, it is much advised that; the town water utility should be regularly examine and taste all customer metering accuracies in accordance with the manufacturer’s recommendations, the water authority should also be provides customer awareness programs and should be encouraged to report illegal connections, and regulations should be in place to penalize the water thieves. While, town water utility

should improve their data handling system supporting with computerized recording technology. And for real losses management, the town water utility recommended to manage the water pressure, actively control leakage, increase their speed and quality of leakage repair, and improve system maintenance, rehabilitation and replacement

- It is also recommended to hire new qualified persons by the professions in order to manage the water supply scheme and to minimize water loss. Because, most of the staff members are working by experience without knowledge of the sciences.
- It was also advised to conducting a complete customer survey within each district meter areas, whereby utility representatives visit every property in the district meter area whether or not customers are recorded in the billing system. In addition to this, the water authority should be pay enough attention and put forward dimensions on water loss management strategies through the processes which involving engineering approaches and international experience.

7. REFRENCE

AWWA, M. (2014). "Water Audits and Loss Control Programs."

Bentley (2008). Bentley WaterGEMS V8iUser's Guide.

Benyam (2016). Assessment of the water distribution network of metu Town water supply system, Ethiopia, Addis Ababa.

Bridges, G. and M. MacDonald (1994). Affordable Water Supply And Sanitation. leakage control - the neglected solution. Colombo, Sri Lanka.

CSA (2008). Summary and statistical report of the 2007 population and housing census. F. d. r. o. e. p. c. commision. Addis Ababa.

CSA (2013). Population Projection of Ethiopia for All Regions at Wereda Level from 2014 – 2017 F. D. R. o. Ethiopia and C. S. Agency. Addis Ababa.

E.Hickey, H. (2008). water supply system and evaluation methods. USA, U.S.Fire Adminstration. I.

Farley (2008). Leakage management and control. WHO. Geneva.

GAWP (2016). Georgia water system audits and water loss controll manual version 2.0. N. resources. Georgia. **.**

H.E.Mutikanga, S. Sharma, et al. (2010). "assesement of apparent losses in urban water system." Water and Environment.

Izinyon, O. C. and B. U. Anyata (2011). "Water Distribution Network Modelling of Asmall Community Using Watercad Simulator." Global Journal Of Engineering Research 10(1&2, 2011): 35-47.

J.G. Saldarriaga, S. O., D. Rodríguez, and J. Arbeláez (April 2009). Water Distribution Network Skeletonization Using the Resilience Concept.

José Sardinha, F. S., Andrew Donnelly, Vera Marmelo, Pedro Saraiva, Nuno Dias, Ricardo Guimarães, Daniel Morais, Vitor Rocha (2017). Active Water Loss Control. 2.

Lambert, A. (2003). "Assessing non-revenue water and its components." IWA Task Force Water 21.

Lambert, A. and A. Lalonde (2005). "Using practical predictions of economic intervention frequency to calculate short-run economic leakage level with or without pressure management."

Lina Perelman, Morris L. Maslia, et al. (2015). "Using aggregation/skeletonization network models for water quality simulations in epidemiologic studies " American Water Works Association.

Mahmoud A. Elsheikh, Hazem I. Saleh, et al. (2013). "Hydraulic modelling of water supply distribution for improving its quantity and quality." Sustainable Environment Research.

Mamade, A., D. Loureiro, et al. (2018). "Top-Down and Bottom-Up Approaches for Water-Energy Balance in Portuguese Supply Systems." water.

MoWIE (2006). Urban Water Supply Design Criteria. W. R. Administration and U. W. S. a. Sanitation.

MoWIE (2015). Second Growth and Transformation National Plan for the Water Supply and Sanitation Sub-sector(2015/16-2019/20) Addis Ababa.

Nemanja (2006). introduction to urban water distribution, Taylor & Francis/Balkema.

Nitin P. Sonaje and M. G. Joshi (2015). "A Review Of Modeling And Application of Water Distribution Networks Softwares." International Journal of Technical Research and Applications 3(5).

Rakesh Bahadur, Jeffrey Johnson, et al. (2006). Impact of Model Skeletonization on Water Distribution Model Parameters as Related to Water Quality and Contaminant Consequence Assessment W. I. P. D. U. S. EPA/NHSRC. USA, ResearchGate.

Shinde, P., P. P. , et al. (2018). "Design and Analysis of Water Distribution Network Using Water GEMS." International Journal Of Advance Research In Science And Technology 0**7**(03).

Thomas M. W alski, Donald V . Chase, et al. (2003). Advanced Water Distribution Modeling and Management. C. a. E. E. a. E. M. Faculty, University of Dayton eCommons. 18.

USEPA (2005). Water Distribution System Analysis: field studies, modeling and management Areferance guide for utilities. U. E. P. Agency. Washington DC.

Vermersch, M., F. Carteado, et al. (2016). Guidance Notes On Apparent Losses And Water Loss Reduction Planning.

8. APPENDIX

APPENDIX – A: Assigned base flow

Junction	No.House	No. People	Nodal consumption (m^3/day)	Average base flow (m^3/day)
N92	32	160	2.24	4.91
N316	28	140	1.96	4.29
N41	10	50	0.7	1.53
N320	22	110	1.54	3.37
N71	10	50	0.7	1.53
J-01	12	60	0.84	1.84
J-43	0	0	0	0.00
N88	32	160	2.24	4.91
J-48	50	250	3.5	7.67
N104	26	130	1.82	3.99
N68	20	100	1.4	3.07
N70	12	60	0.84	1.84
J-50	0	0	0	0.00
N46	20	100	1.4	3.07
N91	10	50	0.7	1.53
N250	20	100	1.4	3.07
N114	16	80	1.12	2.45
N214	0	0	0	0.00
N83	32	160	2.24	4.91
N89	30	150	2.1	4.60
J-26	14	70	0.98	2.15
N339	10	50	0.7	1.53
N248	6	30	0.42	0.92
N62	38	190	2.66	5.83
N66	10	50	0.7	1.53
N82	14	70	0.98	2.15
N210	8	40	0.56	1.23
N220	66	330	4.62	10.12
N93	22	110	1.54	3.37
N221	26	130	1.82	3.99
N340	18	90	1.26	2.76
N57	14	70	0.98	2.15
N117	110	550	7.7	16.87
N119	74	370	5.18	11.35
N209	20	100	1.4	3.07
N101	36	180	2.52	5.52
N122	20	100	1.4	3.07

Junction	No.House	No. People	Nodal consumption (m^3/day)	Average base flow (m^3/day)
N21	74	370	5.18	11.35
N113	134	670	9.38	20.55
N217	140	700	9.8	21.47
J-20	90	450	6.3	13.80
N365	106	530	7.42	16.25
N110	60	300	4.2	9.20
N249	30	150	2.1	4.60
N115	20	100	1.4	3.07
N112	16	80	1.12	2.45
N22	24	120	1.68	3.68
J-15	12	60	0.84	1.84
J-51	20	100	1.4	3.07
N54	24	120	1.68	3.68
N86	50	250	3.5	7.67
N314	62	310	4.34	9.51
N253	86	430	6.02	13.19
J-02	42	210	2.94	6.44
N67	36	180	2.52	5.52
N44	30	150	2.1	4.60
N219	20	100	1.4	3.07
N17	0	0	0	0.00
N313	0	0	0	0.00
N244	16	80	1.12	2.45
N23	24	120	1.68	3.68
N202	18	90	1.26	2.76
N102	80	400	5.6	12.27
N98	230	1150	16.1	35.27
N317	110	550	7.7	16.87
N36	30	150	2.1	4.60
N38	42	210	2.94	6.44
N16	12	60	0.84	1.84
J-33	24	120	1.68	3.68
J-49	38	190	2.66	5.83
N18	18	90	1.26	2.76
N15	26	130	1.82	3.99
N252	20	100	1.4	3.07
N100	8	40	0.56	1.23
N338	16	80	1.12	2.45
N40	36	180	2.52	5.52

Junction	No.House	No. People	Nodal consumption (m^3/day)	Average base flow (m^3/day)
N344	24	120	1.68	3.68
N76	20	100	1.4	3.07
N49	22	110	1.54	3.37
J-34	12	60	0.84	1.84
N106	8	40	0.56	1.23
N87	10	50	0.7	1.53
N366	18	90	1.26	2.76
N247	14	70	0.98	2.15
J-42	6	30	0.42	0.92
N242	10	50	0.7	1.53
J-13	20	100	1.4	3.07
N343	14	70	0.98	2.15
N47	16	80	1.12	2.45
J-40	10	50	0.7	1.53
J-30	22	110	1.54	3.37
N203	64	320	4.48	9.81
N55	64	320	4.48	9.81
N80	40	200	2.8	6.13
N37	30	150	2.1	4.60
N75	36	180	2.52	5.52
J-10	48	240	3.36	7.36
N99	42	210	2.94	6.44
J-46	24	120	1.68	3.68
N94	28	140	1.96	4.29
N367	80	400	5.6	12.27
N56	30	150	2.1	4.60
J-19	42	210	2.94	6.44
N105	56	280	3.92	8.59
N312	38	190	2.66	5.83
N315	34	170	2.38	5.21
N241	62	310	4.34	9.51
N64	42	210	2.94	6.44
J-32	30	150	2.1	4.60
N120	26	130	1.82	3.99
N124	42	210	2.94	6.44
N319	40	200	2.8	6.13
N51	28	140	1.96	4.29
J-29	24	120	1.68	3.68
N42	20	100	1.4	3.07

Junction	No.House	No. People	Nodal consumption (m^3/day)	Average base flow (m^3/day)
N246	16	80	1.12	2.45
N107	36	180	2.52	5.52
N34	20	100	1.4	3.07
N311	30	150	2.1	4.60
J-47	26	130	1.82	3.99
N111	16	80	1.12	2.45
N346	12	60	0.84	1.84
N126	8	40	0.56	1.23
N243	16	80	1.12	2.45
J-21	6	30	0.42	0.92
N218	10	50	0.7	1.53
N85	8	40	0.56	1.23
N59	10	50	0.7	1.53
J-35	6	30	0.42	0.92
J-38	0	0	0	0.00
N60	8	40	0.56	1.23
N108	6	30	0.42	0.92
N215	10	50	0.7	1.53
N118	12	60	0.84	1.84
N24	8	40	0.56	1.23
N245	14	70	0.98	2.15
N341	12	60	0.84	1.84
N95	6	30	0.42	0.92
N20	10	50	0.7	1.53
N81	20	100	1.4	3.07
N52	40	200	2.8	6.13
N342	40	200	2.8	6.13
N364	70	350	4.9	10.73
N251	30	150	2.1	4.60
N19	22	110	1.54	3.37
N50	30	150	2.1	4.60
N39	6	30	0.42	0.92
N45	50	250	3.5	7.67
N53	24	120	1.68	3.68
N125	210	1050	14.7	32.20
N109	50	250	3.5	7.67
N216	46	230	3.22	7.05
N61	60	300	4.2	9.20
J-04	20	100	1.4	3.07

Junction	No.House	No. People	Nodal consumption (m^3/day)	Average base flow (m^3/day)
J-54	10	50	0.7	1.53
N92	60	300	4.2	9.20
N316	28	140	1.96	4.29
N41	20	100	1.4	3.07
N320	30	150	2.1	4.60
N71	6	30	0.42	0.92
J-01	16	80	1.12	2.45
J-43	20	100	1.4	3.07
N88	60	300	4.2	9.20
J-48	28	140	1.96	4.29
N104	60	300	4.2	9.20
N68	40	200	2.8	6.13
N70	0	0	0	0.00
J-50	120	600	8.4	18.40
N46	10	50	0.7	1.53
N91	20	100	1.4	3.07
N250	0	0	0	0.00
N114	32	160	2.24	4.91
N214	30	150	2.1	4.60
N83	18	90	1.26	2.76
N89	34	170	2.38	5.21
J-26	28	140	1.96	4.29
N339	10	50	0.7	1.53
N248	34	170	2.38	5.21
N62	30	150	2.1	4.60
N66	20	100	1.4	3.07
N82	40	200	2.8	6.13
N210	16	80	1.12	2.45
N220	22	110	1.54	3.37
N93	8	40	0.56	1.23
N221	20	100	1.4	3.07
N340	32	160	2.24	4.91
N57	20	100	1.4	3.07
N117	50	250	3.5	7.67
N119	12	60	0.84	1.84
N209	10	50	0.7	1.53
N101	30	150	2.1	4.60
N122	0	0	0	0.00
N21	14	70	0.98	2.15

Junction	No.House	No. People	Nodal consumption (m^3/day)	Average base flow (m^3/day)
N113	16	80	1.12	2.45
N217	10	50	0.7	1.53
J-20	0	0	0	0.00
N365	0	0	0	0.00
N110	6	30	0.42	0.92
N249	0	0	0	0.00
N115	10	50	0.7	1.53
N112	30	150	2.1	4.60
N22	0	0	0	0.00
J-15	10	50	0.7	1.53
J-51	310	1550	21.7	47.53
N54	34	170	2.38	5.21
N86	16	80	1.12	2.45
N314	110	550	7.7	16.87
N253	56	280	3.92	8.59
J-02	106	530	7.42	16.25
N67	16	80	1.12	2.45
N44	34	170	2.38	5.21
N219	18	90	1.26	2.76
N17	18	90	1.26	2.76
N313	30	150	2.1	4.60
N244	46	230	3.22	7.05
N23	10	50	0.7	1.53
N202	6	30	0.42	0.92
N102	16	80	1.12	2.45
N98	12	60	0.84	1.84
N317	36	180	2.52	5.52
N36	30	150	2.1	4.60
N38	20	100	1.4	3.07
N16	10	50	0.7	1.53
J-33	18	90	1.26	2.76
J-49	80	400	5.6	12.27
N169	10	50	0.7	1.53
N187	230	1150	16.1	35.27
N177	20	100	1.4	3.07
N176	16	80	1.12	2.45
N193	18	90	1.26	2.76
N163	50	250	3.5	7.67
N184	16	80	1.12	2.45

Junction	No.House	No. People	Nodal consumption (m^3/day)	Average base flow (m^3/day)
N194	30	150	2.1	4.60
N168	70	350	4.9	10.73
N166	12	60	0.84	1.84
N167	10	50	0.7	1.53
N181	18	90	1.26	2.76
N130	70	350	4.9	10.73
N173	26	130	1.82	3.99
N165	12	60	0.84	1.84
N138	30	150	2.1	4.60
N189	22	110	1.54	3.37
N162	26	130	1.82	3.99
N159	10	50	0.7	1.53
N179	8	40	0.56	1.23
N170	16	80	1.12	2.45
N183	12	60	0.84	1.84
N185	26	130	1.82	3.99
N171	30	150	2.1	4.60
N137	22	110	1.54	3.37
N175	14	70	0.98	2.15
N192	12	60	0.84	1.84
N178	26	130	1.82	3.99
N180	38	190	2.66	5.83
N172	14	70	0.98	2.15
N161	30	150	2.1	4.60
N28	28	140	1.96	4.29
N182	14	70	0.98	2.15
N160	12	60	0.84	1.84
N174	10	50	0.7	1.53
N191	30	150	2.1	4.60
N186	22	110	1.54	3.37
N164	10	50	0.7	1.53
N135	26	130	1.82	3.99
N188	14	70	0.98	2.15
N165	100	500	7	15.33
N190	12	60	0.84	1.84
N169	14	70	0.98	2.15
N187	28	140	1.96	4.29
N177	20	100	1.4	3.07
N176	18	90	1.26	2.76

Junction	No.House	No. People	Nodal consumption (m^3/day)	Average base flow (m^3/day)
N193	16	80	1.12	2.45
N163	30	150	2.1	4.60
N184	36	180	2.52	5.52
N194	34	170	2.38	5.21
N168	16	80	1.12	2.45
N166	90	450	6.3	13.80
N167	66	330	4.62	10.12
N181	54	270	3.78	8.28
N130	32	160	2.24	4.91
N173	18	90	1.26	2.76
N165	40	200	2.8	6.13
N138	24	120	1.68	3.68
N189	20	100	1.4	3.07
N162	26	130	1.82	3.99
N159	30	150	2.1	4.60
N179	70	350	4.9	10.73
N170	74	370	5.18	11.35
N183	56	280	3.92	8.59
N185	38	190	2.66	5.83
N171	28	140	1.96	4.29
N137	40	200	2.8	6.13
N175	34	170	2.38	5.21
N192	50	250	3.5	7.67
N178	46	230	3.22	7.05
N180	36	180	2.52	5.52
N172	30	150	2.1	4.60
N161	50	250	3.5	7.67
N28	16	80	1.12	2.45
N182	20	100	1.4	3.07
N160	30	150	2.1	4.60
N174	80	400	5.6	12.27
N191	42	210	2.94	6.44
N186	10	50	0.7	1.53
N164	8	40	0.56	1.23
N135	0	0	0	0.00
N188	0	0	0	0.00
N165	10	50	0.7	1.53
N190	6	30	0.42	0.92
N169	16	80	1.12	2.45

Junction	No.House	No. People	Nodal consumption (m^3/day)	Average base flow (m^3/day)
N187	12	60	0.84	1.84
N177	20	100	1.4	3.07
N176	20	100	1.4	3.07
N193	18	90	1.26	2.76
N163	20	100	1.4	3.07
N184	30	150	2.1	4.60
N194	6	30	0.42	0.92
N168	10	50	0.7	1.53
N166	14	70	0.98	2.15
N167	16	80	1.12	2.45
N181	22	110	1.54	3.37
N130	50	250	3.5	7.67
N173	166	830	11.62	25.45
N165	26	130	1.82	3.99
N138	20	100	1.4	3.07
N189	22	110	1.54	3.37
N162	20	100	1.4	3.07
N159	20	100	1.4	3.07
N179	30	150	2.1	4.60
Total	10250	51250	717.5	1571.70

APPENDIX –B

Operating Pump Power

Time	OPPERATING PUMP POWER(KW)
1:00	37
2:00	37
3:00	37
4:00	37
5:00	37
6:00	37
7:00	37
8:00	37
9:00	37
10:00	37
11:00	37
12:00	37
13:00	37
14:00	37
15:00	37
16:00	37
17:00	37
18:00	37
19:00	37
20:00	37
21:00	37
22:00	37
23:00	37
24:00	37

APPENDIX –C

Actual Nodal Pressure Result

Zone	Label	Elevation (m)	Hydraulic Grade(m)	Pressure(m)
1	N92	1902.69	2005.52	103
1	N316	1933.73	2005.51	72
1	N41	1956.88	2005.87	49
1	N320	1896.04	2005.51	109
1	N71	1967.99	2005.65	38
1	J-01	2078.28	2094.74	16
1	J-43	1908.83	2005.51	96
1	N88	1936.17	2005.54	69
1	J-48	1882.24	2005.45	123
1	N104	1865.92	2005.45	139
1	N68	1984.4	2005.66	21
1	N70	1971.42	2005.65	34
1	J-50	1880.42	2005.37	125
1	N46	1961.07	2005.77	45
1	N91	1889.16	2005.52	116
1	N250	1902	2005.58	103
1	N114	1834.61	2005.39	170
1	N214	1956.16	2005.76	49
1	N83	1920.21	2005.57	85
1	N89	1923.49	2005.53	82
1	J-26	1986.11	2005.66	20
1	N339	1864.93	2005.46	140
1	N248	1928.31	2005.59	77
1	N62	1980.99	2005.68	25
1	N66	1989.87	2005.67	16
1	N82	1941.33	2005.59	64
1	N210	1951.31	2005.77	54
1	N220	1986.84	2005.74	19
1	N93	1915.74	2005.51	90
1	N221	1990.18	2005.74	16
1	N340	1871.97	2005.46	133
1	N57	1974.78	2005.71	31
1	N117	1876.91	2005.36	128
1	N119	1890.07	2005.35	115
1	N209	1939.61	2005.78	66
1	N101	1898.87	2005.48	106

Zone	Label	Elevation (m)	Hydraulic Grade(m)	Pressure(m)
1	N122	1908.97	2005.32	96
1	N21	1927.38	2042.27	115
1	N113	1825.89	2005.4	179
1	N217	1984.94	2005.75	21
1	J-20	1968.46	2005.7	37
1	N365	1861.94	2005.38	143
1	N110	1833.51	2005.41	172
1	N249	1921.42	2005.59	84
1	N115	1850.65	2005.38	154
1	N112	1826.56	2005.4	178
1	N22	1942.1	2040.97	99
1	J-15	1954.64	2005.8	51
1	J-51	1886.77	2005.29	118
1	N54	1959.14	2005.73	46
1	N86	1938.05	2005.55	67
1	N314	1902.9	2005.52	102
1	N253	1870.79	2005.58	135
1	J-02	2032.78	2036.98	4
1	N67	1987.87	2005.66	18
1	N44	1964.29	2005.78	41
1	N219	1986.43	2005.74	19
1	N17	1946.9	2045.63	99
1	N313	1895.08	2005.52	110
1	N244	1963.77	2005.6	42
1	N23	1959.94	2040.19	80
1	N202	1945.91	2005.92	60
1	N102	1887.48	2005.47	118
1	N98	1893.19	2005.5	112
1	N317	1949.23	2005.51	56
1	N36	1954.72	2005.95	51
1	N38	1952.86	2005.92	53
1	N16	1977.4	2046.48	69
1	J-33	1967.35	2005.64	38
1	J-49	1910.71	2005.33	94
1	N18	1920.87	2044.62	124
1	N15	2032.19	2093.76	61
1	N252	1886.01	2005.58	119
1	N100	1897.41	2005.49	108
1	N338	1868.1	2005.47	137
1	N40	1953.21	2005.89	53

Zone	Label	Elevation (m)	Hydraulic Grade(m)	Pressure(m)
1	N344	1868.63	2005.45	137
1	N76	1947.7	2005.62	58
1	N49	1940.43	2005.75	65
1	J-34	1964.76	2005.63	41
1	N106	1856.72	2005.43	148
1	N87	1938.47	2005.55	67
1	N366	1872.23	2005.37	133
1	N247	1934.35	2005.59	71
1	J-42	1897.75	2005.5	108
1	N242	1967.1	2005.62	38
1	J-13	1946.17	2005.76	59
1	N343	1875.28	2005.45	130
1	N47	1951.56	2005.76	54
1	J-40	1939.4	2005.51	66
1	J-30	1892.94	2005.52	112
1	N203	1946.55	2005.92	59
1	N55	1968.83	2005.72	37
1	N80	1955.41	2005.6	50
1	N37	1951.56	2005.93	54
1	N75	1958.12	2005.63	47
1	J-10	1991.86	2005.74	14
1	N99	1892.31	2005.49	113
1	J-46	1880.04	2005.47	125
1	N94	1923.76	2005.51	82
1	N367	1894.02	2005.37	111
1	N56	1976.06	2005.71	30
1	J-19	1978.89	2005.68	27
1	N105	1865.5	2005.44	140
1	N312	1899.65	2005.52	106
1	N315	1910.54	2005.52	95
1	N241	1969.09	2005.63	36
1	N64	1981.03	2005.67	25
1	J-32	1926.59	2005.56	79
1	N120	1900.14	2005.34	105
1	N124	1879.14	2005.28	126
1	N319	1904.16	2005.51	101
1	N51	1972.65	2005.74	33
1	J-29	1966.68	2005.64	39
1	N42	1966.98	2005.85	39
1	N246	1941.75	2005.59	64

Zone	Label	Elevation (m)	Hydraulic Grade(m)	Pressure(m)
1	N107	1855.24	2005.42	150
1	N34	1971.64	2005.97	34
1	N311	1900.24	2005.52	105
1	J-47	1864.36	2005.38	141
1	N111	1830.39	2005.4	175
1	N346	1900.87	2005.44	104
1	N126	1859.83	2005.25	145
1	N243	1964.86	2005.61	41
1	J-21	1984.42	2005.67	21
1	N218	1985.21	2005.75	20
1	N85	1930.89	2005.56	75
1	N59	1967.26	2005.7	38
1	J-35	1865.03	2005.58	140
1	J-38	1904.08	2005.51	101
1	N60	1979.95	2005.69	26
1	N108	1843.6	2005.42	161
1	N215	1969.01	2005.75	37
1	N118	1882.4	2005.35	123
1	N24	2015.1	2037.74	23
1	N245	1950.03	2005.6	55
1	N341	1881.11	2005.46	124
1	N95	1918.85	2005.51	86
1	N20	1907.74	2043.38	135
1	N81	1951.87	2005.6	54
1	N52	1968.31	2005.74	37
1	N342	1881.46	2005.45	124
1	N364	1849.37	2005.38	156
1	N251	1898.59	2005.58	107
1	N19	1909.4	2043.87	134
1	N50	1948.06	2005.75	58
1	N39	1953.85	2005.9	52
1	N45	1968.04	2005.77	38
1	N53	1962.52	2005.73	43
1	N125	1867.3	2005.27	138
1	N109	1837.36	2005.41	168
1	N216	1975.2	2005.75	30
1	N61	1981.46	2005.68	24
1	J-04	1960.55	2005.96	45
1	J-54	1857.08	2005.24	148
2	N169	2,039.48	2,127.55	88

Zone	Label	Elevation (m)	Hydraulic Grade(m)	Pressure(m)
2	N187	2,059.13	2,127.87	69
2	N177	2,103.31	2,126.92	24
2	N176	2,107.96	2,126.99	19
2	N193	2,043.39	2,127.52	84
2	N163	2,058.31	2,127.91	69
2	N184	2,066.10	2,126.54	60
2	N194	2,044.18	2,127.53	83
2	N168	2,039.74	2,127.57	88
2	N166	2,039.17	2,127.60	88
2	N167	2,041.80	2,127.58	86
2	N181	2,103.56	2,126.73	23
2	N130	2,115.34	2,127.97	13
2	N173	2,080.73	2,127.13	46
2	N165	2,037.01	2,127.69	90
2	N138	2,040.35	2,127.62	87
2	N189	2,083.42	2,127.83	44
2	N162	2,101.49	2,127.95	26
2	N159	2,075.21	2,126.53	51
2	N179	2,085.97	2,126.80	41
2	N170	2,033.62	2,127.43	94
2	N183	2,063.20	2,126.55	63
2	N185	2,072.04	2,126.51	54
2	N171	2,033.68	2,127.32	93
2	N137	2,039.90	2,127.65	88
2	N175	2,102.94	2,127.05	24
2	N192	2,067.76	2,127.66	60
2	N178	2,092.63	2,126.85	34
2	N180	2,099.37	2,126.76	27
2	N172	2,053.12	2,127.22	74
2	N161	2,053.32	2,131.62	78
2	N28	2,088.70	2,129.84	41
2	N182	2,108.79	2,126.63	18
2	N160	2,046.70	2,132.27	85
2	N174	2,097.05	2,127.09	30
2	N191	2,074.14	2,127.69	53
2	N186	2,065.68	2,126.51	61
2	N164	2,042.23	2,127.89	85
2	N135	2,032.43	2,127.75	95
2	N188	2,067.70	2,127.85	60
2	N165	2,025.02	2,127.80	103

APPENDIX - D

Actual Pipe Velocity

Zone	Label	Diameter (mm)	Material	Hazen-Williams C -factor	Flow (L/s)	Velocity (m/s)	Length (m)
1	P-14	400	Ductile Iron	120	19	0.15	175
1	P-16	400	Ductile Iron	120	19	0.15	150
1	P-17	400	Ductile Iron	120	19	0.15	180
1	P-18	400	Ductile Iron	120	19	0.15	133
1	P-19	400	Ductile Iron	120	19	0.15	86
1	P-20	400	Ductile Iron	120	19	0.15	197
1	P-21	400	Ductile Iron	120	19	0.15	232
1	P-22	400	Ductile Iron	120	19	0.15	137
1	P-23	400	Ductile Iron	120	19	0.15	434
1	P-24	400	Ductile Iron	120	19	0.15	107
1	P-31	450	Ductile Iron	120	17.10	0.11	115
1	P-32	450	Ductile Iron	120	17.04	0.11	55
1	P-33	450	Ductile Iron	120	15.24	0.1	64
1	P-34	450	Ductile Iron	120	15.17	0.1	94
1	P-35	450	Ductile Iron	120	15.09	0.09	63
1	P-36	450	Ductile Iron	120	15.066	0.09	83
1	P-37	450	Ductile Iron	120	15.02	0.09	55
1	P-38	450	Ductile Iron	120	15.01	0.09	127
1	P-39	450	Ductile Iron	120	14.93	0.09	146
1	P-40	450	Ductile Iron	120	14.89	0.09	262
1	P-41	450	Ductile Iron	120	13.70	0.09	133
1	P-42	450	Ductile Iron	120	13.66	0.09	72
1	P-43	450	Ductile Iron	120	13.60	0.09	49
1	P-44	450	Ductile Iron	120	13.52	0.09	41
1	P-45	450	Ductile Iron	120	13.44	0.08	36
1	P-46	450	Ductile Iron	120	11.81	0.07	44
1	P-47	450	Ductile Iron	120	11.75	0.07	47
1	P-48	450	Ductile Iron	120	11.68	0.07	78
1	P-49	450	Ductile Iron	120	11.66	0.07	55
1	P-50	450	Ductile Iron	120	11.63	0.07	37
1	P-51	450	Ductile Iron	120	11.59	0.07	52
1	P-52	450	Ductile Iron	120	11.59	0.07	90
1	P-53	450	Ductile Iron	120	11.51	0.07	73
1	P-54	450	Ductile Iron	120	11.51	0.07	43
1	P-55	450	Ductile Iron	120	11.51	0.07	74
1	P-56	450	Ductile Iron	120	11.42	0.07	64
1	P-57	450	Ductile Iron	120	11.40	0.07	91
1	P-58	450	Ductile Iron	120	11.40	0.07	63
1	P-59	450	Ductile Iron	120	11.32	0.07	50
1	P-60	450	Ductile Iron	120	11.32	0.07	38
1	P-61	450	Ductile Iron	120	11.31	0.07	43
1	P-62	450	Ductile Iron	120	11.3	0.07	36

Zone	Label	Diameter (mm)	Material	Hazen-Williams C -factor	Flow (L/s)	Velocity (m/s)	Length (m)
1	P-63	450	Ductile Iron	120	11.17	0.07	46
1	P-64	450	Ductile Iron	120	11.17	0.07	57
1	P-65	450	Ductile Iron	120	11.17	0.07	29
1	P-66	450	Ductile Iron	120	11.17	0.07	26
1	P-67	450	Ductile Iron	120	11.15	0.07	111
1	P-68	450	Ductile Iron	120	11.13	0.07	30
1	P-69	450	Ductile Iron	120	11.1	0.07	129
1	P-70	450	Ductile Iron	120	11.1	0.07	41
1	P-71	350	PVC	130	9.2	0.1	60
1	P-72	350	PVC	130	9.2	0.1	50
1	P-73	350	PVC	130	9.2	0.1	97
1	P-76	350	PVC	130	9.1	0.09	63
1	P-77	350	PVC	130	8.66	0.09	86
1	P-78	350	PVC	130	8.66	0.09	169
1	P-79	350	PVC	130	8.66	0.09	109
1	P-80	350	PVC	130	8.56	0.09	61
1	P-81	350	PVC	130	8.51	0.09	101
1	P-82	350	PVC	130	8.51	0.09	59
1	P-83	350	PVC	130	8.46	0.09	66
1	P-84	350	PVC	130	8.46	0.09	91
1	P-86	350	PVC	130	7.12	0.07	58
1	P-87	350	PVC	130	7.11	0.07	65
1	P-88	350	PVC	130	7.11	0.07	67
1	P-89	350	PVC	130	7.07	0.07	60
1	P-90	350	PVC	130	7.07	0.07	80
1	P-91	350	PVC	130	7.07	0.07	38
1	P-92	350	PVC	130	6.07	0.06	55
1	P-93	300	PVC	130	5.983	0.08	65
1	P-94	300	PVC	130	5.941	0.08	94
1	P-95	300	PVC	130	5.891	0.08	83
1	P-96	300	PVC	130	5.821	0.08	112
1	P-97	300	PVC	130	5.781	0.08	62
1	P-99	250	PVC	130	4.219	0.09	194
1	P-100	250	PVC	130	4.219	0.09	104
1	P-101	250	PVC	130	4.119	0.08	258
1	P-102	250	PVC	130	4.054	0.08	65
1	P-103	250	PVC	130	4.054	0.08	79
1	P-104	250	PVC	130	4.054	0.08	49
1	P-105	250	PVC	130	4.054	0.08	22
1	P-106	250	PVC	130	4.034	0.08	165
1	P-107	250	PVC	130	3.994	0.08	50
1	P-108	250	PVC	130	3.971	0.08	48
1	P-109	250	PVC	130	3.911	0.08	145
1	P-110	250	PVC	130	3.811	0.08	102
1	P-111	250	PVC	130	3.811	0.08	96
1	P-112	200	PVC	130	2.373	0.08	329

Zone	Label	Diameter (mm)	Material	Hazen-Williams C -factor	Flow (L/s)	Velocity (m/s)	Length (m)
1	P-113	200	PVC	130	2.313	0.07	115
1	P-114	200	PVC	130	2.283	0.07	123
1	P-115	200	PVC	130	2.233	0.07	138
1	P-116	200	PVC	130	1.133	0.04	142
1	P-117	150	PVC	130	1.09	0.06	139
1	P-118	150	PVC	130	1.09	0.06	221
1	P-119	150	PVC	130	1.08	0.06	91
1	P-120	150	PVC	130	1	0.06	123
1	P-121	150	PVC	130	1	0.06	192
1	P-195	200	PVC	130	1.215	0.04	32
1	P-196	200	PVC	130	1.215	0.04	32
1	P-197	200	PVC	130	1.215	0.04	40
1	P-198	200	PVC	130	1.215	0.04	48
1	P-199	200	PVC	130	1.215	0.04	47
1	P-200	200	PVC	130	1.215	0.04	103
1	P-201	200	PVC	130	1.215	0.04	87
1	P-202	200	PVC	130	1.155	0.04	167
1	P-203	200	PVC	130	1.145	0.04	123
1	P-204	200	PVC	130	1.055	0.03	115
1	P-205	200	PVC	130	1.01	0.03	44
1	P-212	200	PVC	130	1.519	0.05	63
1	P-213	200	PVC	130	1.519	0.05	48
1	P-214	200	PVC	130	1.519	0.05	58
1	P-215	200	PVC	130	1.519	0.05	95
1	P-216	200	PVC	130	1.419	0.05	57
1	P-217	200	PVC	130	1.389	0.04	52
1	P-218	200	PVC	130	1.389	0.04	54
1	P-219	200	PVC	130	1.389	0.04	70
1	P-220	150	PVC	130	1.3	0.07	112
1	P-230	200	PVC	130	1.348	0.04	80
1	P-231	200	PVC	130	1.348	0.04	79
1	P-232	200	PVC	130	1.338	0.04	80
1	P-233	200	PVC	130	1.258	0.04	287
1	P-234	200	PVC	130	1.2	0.04	81
1	P-122	150	PVC	130	1	0.06	138
1	P-245	150	PVC	130	1.7	0.1	156
1	P-246	150	PVC	130	1.2	0.07	49
1	P-250	150	PVC	130	1.14	0.06	198
1	P-251	150	PVC	130	1.1	0.06	95
1	P-253	200	PVC	130	1.572	0.05	48
1	P-254	200	PVC	130	1.512	0.05	68
1	P-255	200	PVC	130	1.472	0.05	24
1	P-256	200	PVC	130	1.442	0.05	73
1	P-257	200	PVC	130	1.432	0.05	64
1	P-258	200	PVC	130	1.392	0.04	59
1	P-259	200	PVC	130	1.372	0.04	52

Zone	Label	Diameter (mm)	Material	Hazen-Williams C -factor	Flow (L/s)	Velocity (m/s)	Length (m)
1	P-260	200	PVC	130	1.35	0.04	51
1	P-261	200	PVC	130	1.3	0.04	78
1	P-282	200	PVC	130	1.79	0.06	34
1	P-283	200	PVC	130	1.79	0.06	90
1	P-284	200	PVC	130	1.78	0.06	97
1	P-285	200	PVC	130	1.72	0.05	43
1	P-286	200	PVC	130	1.72	0.05	68
1	P-287	200	PVC	130	1.72	0.05	24
1	P-288	200	PVC	130	1.72	0.05	24
1	P-289	200	PVC	130	1.72	0.05	21
1	P-290	200	PVC	130	1.68	0.05	57
1	P-292	200	PVC	130	1.15	0.04	21
1	P-293	200	PVC	130	1.15	0.04	69
1	P-294	200	PVC	130	1.11	0.04	90
1	P-295	200	PVC	130	1.1	0.04	48
1	P-75	350	PVC	130	-9.1	0.09	154
1	P-344	400	Ductile Iron	120	19	0.15	97
1	P-345	400	Ductile Iron	120	19	0.15	94
1	P-13	400	Ductile Iron	120	-19	0.15	134
1	P-343	450	Ductile Iron	120	19	0.12	58
1	P-347	450	Ductile Iron	120	19	0.12	58
1	P-348	500	Ductile Iron	120	-19.26	0.1	36
1	P-1	200	PVC	130	1.18	0.04	92
1	P-2	200	PVC	130	1.18	0.04	92
1	P-3	350	PVC	130	8.41	0.09	160
1	P-4	350	PVC	130	8.41	0.09	160
1	P-5	300	PVC	130	5.75	0.08	50
1	P-6	300	PVC	130	5.75	0.08	50
2	P-123	250	PVC	130	4	0.07	85
2	P-124	250	PVC	130	4	0.07	55
2	P-26	200	PVC	130	6	0.18	102
2	P-27	200	PVC	130	6	0.18	278
2	P-28	200	PVC	130	6	0.18	286
2	P-125	250	PVC	130	4	0.07	111
2	P-127	200	PVC	130	2	0.08	145
2	P-128	200	PVC	130	2	0.08	96
2	P-129	200	PVC	130	2	0.07	90
2	P-130	200	PVC	130	2	0.07	79
2	P-131	200	PVC	130	2	0.06	73
2	P-132	200	PVC	130	2	0.06	41
2	P-133	200	PVC	130	2	0.06	44
2	P-134	200	PVC	130	2	0.06	43
2	P-135	200	PVC	130	2	0.06	41
2	P-136	200	PVC	130	2	0.05	77
2	P-137	150	PVC	130	1	0.07	109
2	P-138	150	PVC	130	1	0.07	127

Zone	Label	Diameter (mm)	Material	Hazen-Williams C -factor	Flow (L/s)	Velocity (m/s)	Length (m)
2	P-139	150	PVC	130	1	0.06	118
2	P-140	150	PVC	130	1	0.05	104
2	P-141	150	PVC	130	1	0.05	46
2	P-142	150	PVC	130	1	0.05	58
2	P-143	150	PVC	130	1	0.04	76
2	P-144	150	PVC	130	1	0.04	81
2	P-145	150	PVC	130	1	0.03	98
2	P-146	150	PVC	130	1	0.03	61
2	P-147	150	PVC	130	0	0.02	69
2	P-148	150	PVC	130	0	0.02	49
2	P-149	150	PVC	130	0	0.02	233
2	P-150	150	PVC	130	0	0.02	227
2	P-151	150	PVC	130	0	0.02	71
2	P-152	150	PVC	130	0	0.01	117
2	P-153	150	PVC	130	0	0.01	123
2	P-154	150	PVC	130	0	0.01	129
2	P-170	150	PVC	130	1	0.06	72
2	P-171	150	PVC	130	1	0.06	103
2	P-172	150	PVC	130	1	0.05	113
2	P-173	150	PVC	130	1	0.05	106
2	P-174	100	PVC	130	1	0.09	108
2	P-175	100	PVC	130	1	0.09	38
2	P-176	100	PVC	130	1	0.09	255
2	P-338	200	PVC	130	6	0.18	25
2	P-339	200	PVC	130	6	0.18	52
2	P-1	250	PVC	130	4	0.07	56
2	P-2	250	PVC	130	4	0.07	56

APPENDIX- E: Introduced PRV Table

Zone	Label	Elevation (m)	Diameter (Valve) (mm)	Hydraulic Grade Setting (Initial) (m)	Pressure Setting (Initial) (m H2O)	Flow (L/s)	Hydraulic Grade (From) (m)	Hydraulic Grade (To) (m)	Head loss (m)
1	PRV-1	1,914.50	200	1,934.53	20	1.18	2,005.92	1,934.54	71.38
1	PRV-2	1,917.79	350	1,962.87	45	8.41	2,005.90	1,962.90	43
1	PRV-3	1,883.01	300	1,923.07	40	5.75	1,962.88	1,923.10	39.78
2	PRV-4	2,052.32	250	2,112.42	60	4	2,127.99	2,112.46	15.53

APPENDIX- F: Improved System Pressure with PRV

Zone	Label	Elevation (m)	Hydraulic Grade(m	Pressure (m)
1	N92	1,902.69	2006	60
1	N316	1,933.73	2006	29
1	N41	1,956.88	2006	47
1	N320	1,896.04	2006	66
1	N71	1,967.99	2006	32
1	J-01	2,078.28	2094	16
1	J-43	1,908.83	2006	53
1	N88	1,936.17	2006	62
1	J-48	1,882.24	2006	41
1	N104	1,865.92	2006	56
1	N68	1,984.40	2006	16
1	N70	1,971.42	2006	28
1	J-50	1,880.42	2006	33
1	N46	1,961.07	2006	42
1	N91	1,889.16	2006	73
1	N250	1,902.00	2006	32
1	N114	1,834.61	2006	82
1	N214	1,956.16	2006	46
1	N83	1,920.21	2006	78
1	N89	1,923.49	2006	74
1	J-26	1,986.11	2006	14
1	N339	1,864.93	2006	58
1	N248	1,928.31	2006	67
1	N62	1,980.99	2006	20
1	N66	1,989.87	2006	10
1	N82	1,941.33	2006	57
1	N210	1,951.31	2006	52
1	N220	1,986.84	2006	16
1	N93	1,915.74	2006	47
1	N221	1,990.18	2006	12
1	N340	1,871.97	2006	51
1	N57	1,974.78	2006	27
1	N117	1,876.91	2006	38
1	N119	1,890.07	2006	25
1	N209	1,939.61	2006	64
1	N101	1,898.87	2006	62
1	N122	1,908.97	2006	5
1	N21	1,927.38	2042	75
1	N113	1,825.89	2006	72
1	N217	1,984.94	2006	18

Zone	Label	Elevation (m)	Hydraulic Grade(m	Pressure (m)
1	J-20	1,968.46	2006	33
1	N365	1,861.94	2006	53
1	N110	1,833.51	2006	85
1	N249	1,921.42	2006	73
1	N115	1,850.65	2006	66
1	N112	1,826.56	2006	81
1	N22	1,942.10	2040.	79
1	J-15	1,954.64	2006	49
1	J-51	1,886.77	2006	27
1	N54	1,959.14	2006	43
1	N86	1,938.05	2006	60
1	N314	1,902.90	2006	60
1	N253	1,870.79	2006	61
1	J-02	2,032.78	2036	14
1	N67	1,987.87	2006	12
1	N44	1,964.29	2006	39
1	N219	1,986.43	2006	16
1	N17	1,946.90	2045	69
1	N313	1,895.08	2006	67
1	N244	1,963.77	2006	33
1	N23	1,959.94	2040	80
1	N202	1,945.91	2006	60
1	N102	1,887.48	2006	73
1	N98	1,893.19	2006	69
1	N317	1,949.23	2006	13
1	N36	1,954.72	2006	51
1	N38	1,952.86	2006	52
1	N16	1,977.40	2046	69
1	J-33	1,967.35	2006	32
1	J-49	1,910.71	2006	4
1	N18	1,920.87	2044	74
1	N15	2,032.19	2093	61
1	N252	1,886.01	2006	47
1	N100	1,897.41	2006	64
1	N338	1,868.10	2006	55
1	N40	1,953.21	2006	51
1	N344	1,868.63	2006	54
1	N76	1,947.70	2006	51
1	N49	1,940.43	2006	62
1	J-34	1,964.76	2006	35
1	N106	1,856.72	2006	63
1	N87	1,938.47	2006	59

Zone	Label	Elevation (m)	Hydraulic Grade(m	Pressure (m)
1	N366	1,872.23	2006	42
1	N247	1,934.35	2006	61
1	J-42	1,897.75	2006	64
1	N242	1,967.10	2006	31
1	J-13	1,946.17	2006	56
1	N343	1,875.28	2006	48
1	N47	1,951.56	2006	51
1	J-40	1,939.40	2006	23
1	J-30	1,892.94	2006	70
1	N203	1,946.55	2006	59
1	N55	1,968.83	2006	33
1	N80	1,955.41	2006	43
1	N37	1,951.56	2006	54
1	N75	1,958.12	2006	41
1	J-10	1,991.86	2006	11
1	N99	1,892.31	2006	69
1	J-46	1,880.04	2006	43
1	N94	1,923.76	2006	39
1	N367	1,894.02	2006	19
1	N56	1,976.06	2006	25
1	J-19	1,978.89	2006	22
1	N105	1,865.50	2006	56
1	N312	1,899.65	2006	63
1	N315	1,910.54	2006	52
1	N241	1,969.09	2006	30
1	N64	1,981.03	2006	19
1	J-32	1,926.59	2006	72
1	N120	1,900.14	2006	15
1	N124	1,879.14	2006	34
1	N319	1,904.16	2006	58
1	N51	1,972.65	2006	30
1	J-29	1,966.68	2006	33
1	N42	1,966.98	2006	37
1	N246	1,941.75	2006	54
1	N107	1,855.24	2006	64
1	N34	1,971.64	2006	34
1	N311	1,900.24	2006	62
1	J-47	1,864.36	2006	51
1	N111	1,830.39	2006	88
1	N346	1,900.87	2006	22
1	N126	1,859.83	2006	52
1	N243	1,964.86	2006	32

Zone	Label	Elevation (m)	Hydraulic Grade(m	Pressure (m)
1	J-21	1,984.42	2006	16
1	N218	1,985.21	2006	17
1	N85	1,930.89	2006	67
1	N59	1,967.26	2006	34
1	J-35	1,865.03	2006	67
1	J-38	1,904.08	2006	58
1	N60	1,979.95	2006	21
1	N108	1,843.60	2006	76
1	N215	1,969.01	2006	34
1	N118	1,882.40	2006	33
1	N24	2,015.10	2037	23
1	N245	1,950.03	2006	46
1	N341	1,881.11	2006	42
1	N95	1,918.85	2006	43
1	N20	1,907.74	2043	65
1	N81	1,951.87	2006	47
1	N52	1,968.31	2006	34
1	N342	1,881.46	2006	41
1	N364	1,849.37	2006	66
1	N81	1,951.87	2006	47
1	N52	1,968.31	2043	34
1	N342	1,881.46	2006	41
1	N364	1,849.37	2006	66
1	N251	1,898.59	2006	35
1	N19	1,909.40	2006	74
1	N50	1,948.06	2006	54
1	N39	1,953.85	2006	51
1	N45	1,968.04	2006	35
1	N53	1,962.52	2006	39
1	N125	1,867.30	2006	45
1	N109	1,837.36	2006	82
2	N142	2039.48	2109	70
2	N179	2059.13	2112	53
2	N151	2103.31	2108	16
2	N150	2107.96	2109	11
2	JA-03	2043.39	2112	69
2	N132	2058.31	2127	69
2	N158	2066.1	2108	42
2	JA-10	2044.18	2109	65
2	N141	2039.74	2109	70
2	N139	2039.17	2110	71
2	N140	2041.8	2110	68

2	JA-08	2103.56	2108	15
Zone	Label	Elevation (m)	Hydraulic Grade(m	Pressure (m)
2	N130	2115.34	2127.84	12
2	N147	2080.73	2109.15	28
2	N136	2037.01	2111.03	74
2	N138	2040.35	2110.4	70
2	N181	2083.42	2112.3	29
2	N131	2101.49	2127.74	26
2	N159	2075.21	2108.68	33
2	N153	2085.97	2108.87	23
2	N144	2033.62	2109.38	76
2	JA-09	2063.2	2108.68	45
2	N160	2072.04	2108.68	37
2	N145	2033.68	2109.3	75
2	N137	2039.9	2110.71	71
2	N149	2102.94	2109.08	16
2	N184	2067.76	2112.27	44
2	N152	2092.63	2108.91	16
2	N154	2099.37	2108.82	19
2	N146	2053.12	2109.22	56
2	N27	2053.32	2131.62	78
2	N28	2088.7	2129.84	41
2	N156	2108.79	2108.73	10
2	N26	2046.7	2132.27	85
2	N148	2097.05	2109.12	12
2	N183	2074.14	2112.27	38
2	N161	2065.68	2108.68	43
2	JA-01	2042.23	2112.4	70
2	N135	2032.43	2111.4	79
2	N180	2067.7	2112.34	45
2	N134	2025.02	2111.8	87
2	JA-02	2081.72	2112.27	30

APPENDIX-G

Improved pipe velocity in the model

Zone	Label	Length (m)	Diameter(mm)	Material	Hazen-Williams C	Flow(L/s)	Velocity (m/s)
1	P-14	175	400.0	Ductile Iron	120.0	62.00000	0.49
1	P-16	150	400.0	Ductile Iron	120.0	62.00000	0.49
1	P-17	180	400.0	Ductile Iron	120.0	62.00000	0.49
1	P-18	133	400.0	Ductile Iron	120.0	62.00000	0.49
1	P-19	86	400.0	Ductile Iron	120.0	62.00000	0.49
1	P-20	197	400.0	Ductile Iron	120.0	62.00000	0.49
1	P-21	232	400.0	Ductile Iron	120.0	62.00000	0.49
1	P-22	137	400.0	Ductile Iron	120.0	62.00000	0.49
1	P-23	434	400.0	Ductile Iron	120.0	62.00000	0.49
1	P-24	107	400.0	Ductile Iron	120.0	62.00000	0.49
1	P-31	115	450.0	Ductile Iron	120.0	61.68998	0.39
1	P-32	56	450.0	Ductile Iron	120.0	61.48998	0.39
1	P-33	64	450.0	Ductile Iron	120.0	59.68998	0.38
1	P-34	94	450.0	Ductile Iron	120.0	58.68998	0.37
1	P-35	63	450.0	Ductile Iron	120.0	57.78998	0.36
1	P-36	83	450.0	Ductile Iron	120.0	56.91998	0.36
1	P-37	55	450.0	Ductile Iron	120.0	56.21998	0.35
1	P-38	128	450.0	Ductile Iron	120.0	55.61998	0.35
1	P-39	146	450.0	Ductile Iron	120.0	55.01998	0.35
1	P-40	262	450.0	Ductile Iron	120.0	54.61998	0.34
1	P-41	133	450.0	Ductile Iron	120.0	53.31998	0.34
1	P-42	72	450.0	Ductile Iron	120.0	53.11998	0.33
1	P-43	49	450.0	Ductile Iron	120.0	52.91998	0.33
1	P-44	42	450.0	Ductile Iron	120.0	52.61998	0.33
1	P-45	36	450.0	Ductile Iron	120.0	52.51998	0.33
1	P-46	44	450.0	Ductile Iron	120.0	49.79998	0.31
1	P-47	47	450.0	Ductile Iron	120.0	49.59998	0.31
1	P-48	78	450.0	Ductile Iron	120.0	49.19998	0.31
1	P-49	55	450.0	Ductile Iron	120.0	49.19998	0.31
1	P-50	37	450.0	Ductile Iron	120.0	49.19998	0.31
1	P-51	52	450.0	Ductile Iron	120.0	48.49998	0.30
1	P-52	90	450.0	Ductile Iron	120.0	47.89998	0.30
1	P-53	74	450.0	Ductile Iron	120.0	47.39998	0.30
1	P-54	43	450.0	Ductile Iron	120.0	47.19998	0.30
1	P-55	74	450.0	Ductile Iron	120.0	46.99998	0.30
1	P-56	64	450.0	Ductile Iron	120.0	46.79998	0.29
1	P-57	91	450.0	Ductile Iron	120.0	46.49998	0.29
1	P-58	63	450.0	Ductile Iron	120.0	46.29998	0.29
1	P-59	50	450.0	Ductile Iron	120.0	45.99998	0.29
1	P-60	38	450.0	Ductile Iron	120.0	45.79998	0.29
1	P-61	43	450.0	Ductile Iron	120.0	45.39998	0.29
1	P-62	36	450.0	Ductile Iron	120.0	45.19998	0.28

Zone	Label	Length (m)	Diameter(mm)	Material	Hazen-Williams C	Flow(L/s)	Velocity (m/s)
1	P-63	46	450.0	Ductile Iron	120.0	44.69998	0.28
1	P-64	57	450.0	Ductile Iron	120.0	44.59998	0.28
1	P-65	29	450.0	Ductile Iron	120.0	44.09998	0.28
1	P-66	26	450.0	Ductile Iron	120.0	44.09998	0.28
1	P-67	111	450.0	Ductile Iron	120.0	43.77998	0.28
1	P-68	30	450.0	Ductile Iron	120.0	43.57998	0.27
1	P-69	129	450.0	Ductile Iron	120.0	43.37998	0.27
1	P-70	41	450.0	Ductile Iron	120.0	43.17998	0.27
1	P-71	60	350.0	PVC	130.0	39.28998	0.41
1	P-72	50	350.0	PVC	130.0	38.88998	0.40
1	P-73	97	350.0	PVC	130.0	37.68998	0.39
1	P-76	63	350.0	PVC	130.0	36.18998	0.38
1	P-77	86	350.0	PVC	130.0	35.78998	0.37
1	P-78	169	350.0	PVC	130.0	34.98998	0.36
1	P-79	109	350.0	PVC	130.0	33.98998	0.35
1	P-80	61	350.0	PVC	130.0	33.88998	0.35
1	P-81	100	350.0	PVC	130.0	33.48998	0.35
1	P-82	59	350.0	PVC	130.0	33.48998	0.35
1	P-83	67	350.0	PVC	130.0	32.98998	0.34
1	P-84	91	350.0	PVC	130.0	32.18998	0.33
1	P-86	58	350.0	PVC	130.0	27.63000	0.29
1	P-87	65	350.0	PVC	130.0	26.53000	0.28
1	P-88	67	350.0	PVC	130.0	25.63000	0.27
1	P-89	60	350.0	PVC	130.0	25.43000	0.26
1	P-90	79	350.0	PVC	130.0	25.03000	0.26
1	P-91	38	350.0	PVC	130.0	24.63000	0.26
1	P-92	55	350.0	PVC	130.0	24.43000	0.25
1	P-93	65	300.0	PVC	130.0	24.13000	0.34
1	P-94	94	300.0	PVC	130.0	23.83000	0.34
1	P-95	82	300.0	PVC	130.0	23.63000	0.33
1	P-96	112	300.0	PVC	130.0	23.33000	0.33
1	P-97	63	300.0	PVC	130.0	23.33000	0.33
1	P-99	194	250.0	PVC	130.0	19.20000	0.39
1	P-100	104	250.0	PVC	130.0	18.80000	0.38
1	P-101	258	250.0	PVC	130.0	17.60000	0.36
1	P-102	65	250.0	PVC	130.0	17.10000	0.35
1	P-103	79	250.0	PVC	130.0	16.20000	0.33
1	P-104	49	250.0	PVC	130.0	15.90000	0.32
1	P-105	22	250.0	PVC	130.0	15.60000	0.32
1	P-106	165	250.0	PVC	130.0	15.40000	0.31
1	P-107	50	250.0	PVC	130.0	15.00000	0.31
1	P-108	48	250.0	PVC	130.0	14.80000	0.30
1	P-109	145	250.0	PVC	130.0	14.20000	0.29
1	P-110	102	250.0	PVC	130.0	13.20000	0.27
1	P-111	96	250.0	PVC	130.0	12.30000	0.25

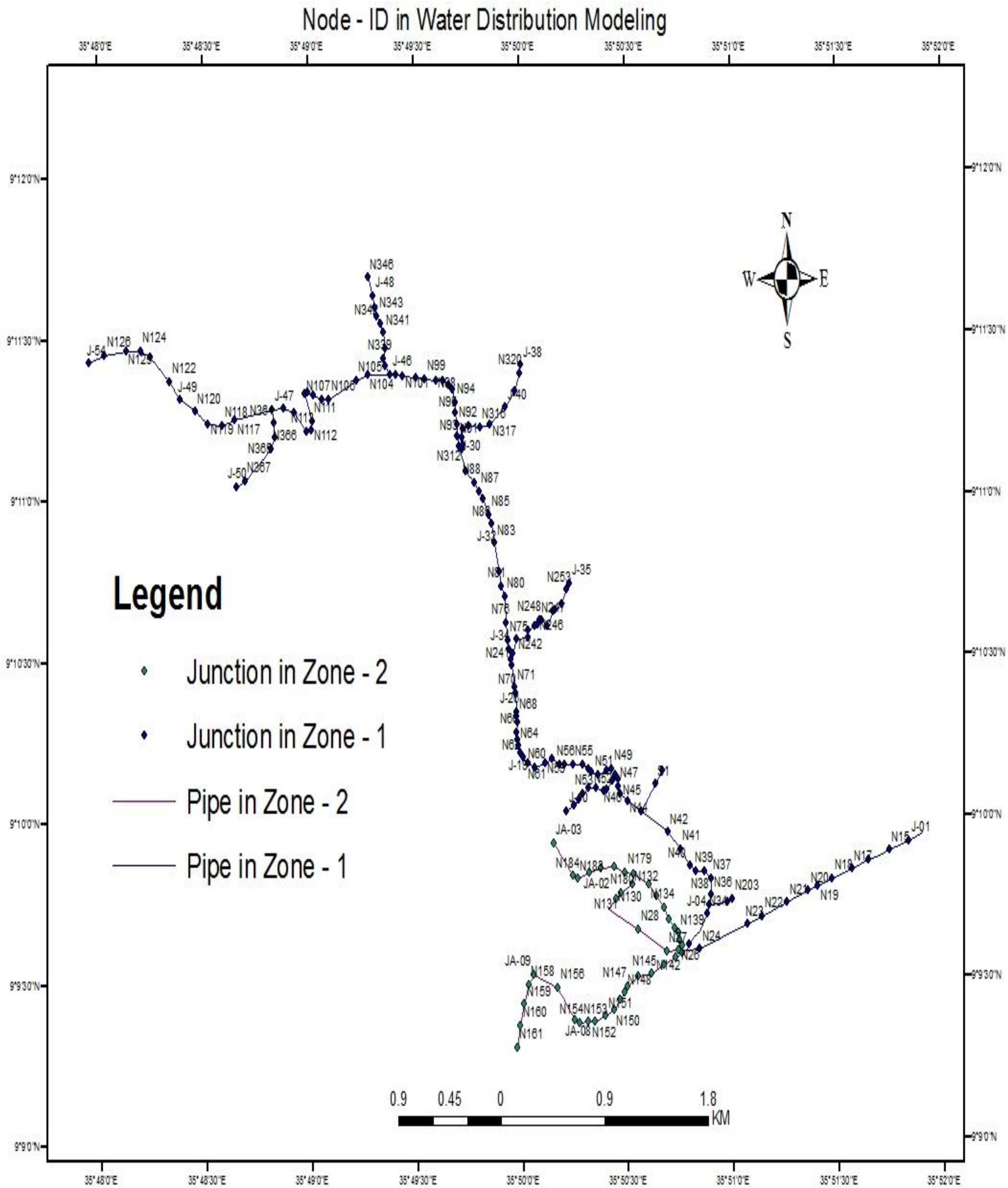
Zone	Label	Length (m)	Diameter(mm)	Material	Hazen-Williams C	Flow(L/s)	Velocity (m/s)
1	P-112	329	200.0	PVC	130.0	16.60000	0.21
1	P-113	115	200.0	PVC	130.0	15.90000	0.19
1	P-114	123	200.0	PVC	130.0	14.80000	0.15
1	P-115	138	200.0	PVC	130.0	13.90000	0.12
1	P-116	142	200.0	PVC	130.0	12.80000	0.09
1	P-117	139	150.0	PVC	130.0	12.30000	0.13
1	P-118	221	150.0	PVC	130.0	12.30000	0.13
1	P-119	91	150.0	PVC	130.0	12.10000	0.12
1	P-120	124	150.0	PVC	130.0	11.80000	0.10
1	P-121	192	150.0	PVC	130.0	11.40000	0.08
1	P-195	32	200.0	PVC	130.0	13.26000	0.10
1	P-196	32	200.0	PVC	130.0	13.26000	0.10
1	P-197	40	200.0	PVC	130.0	12.66000	0.08
1	P-198	48	200.0	PVC	130.0	12.46000	0.08
1	P-199	47	200.0	PVC	130.0	12.26000	0.07
1	P-200	103	200.0	PVC	130.0	11.96000	0.6
1	P-201	86	200.0	PVC	130.0	11.76000	0.6
1	P-202	167	200.0	PVC	130.0	11.70000	0.5
1	P-203	123	200.0	PVC	130.0	11.50000	0.5
1	P-204	115	200.0	PVC	130.0	10.90000	0.3
1	P-205	44	200.0	PVC	130.0	10.40000	0.3
1	P-212	63	200.0	PVC	130.0	13.53000	0.51
1	P-213	48	200.0	PVC	130.0	13.23000	0.50
1	P-214	58	200.0	PVC	130.0	13.00000	0.50
1	P-215	95	200.0	PVC	130.0	13.00000	0.50
1	P-216	57	200.0	PVC	130.0	13.00000	0.50
1	P-217	52	200.0	PVC	130.0	12.60000	0.4
1	P-218	54	200.0	PVC	130.0	11.40000	0.4
1	P-219	70	200.0	PVC	130.0	11.40000	0.4
1	P-220	112	150.0	PVC	130.0	10.80000	0.5
1	P-230	80	200.0	PVC	130.0	15.30000	0.17
1	P-231	79	200.0	PVC	130.0	14.90000	1.6
1	P-232	80	200.0	PVC	130.0	13.60000	1.1
1	P-233	287	200.0	PVC	130.0	11.80000	0.6
1	P-234	81	200.0	PVC	130.0	10.50000	0.2
1	P-122	138	150.0	PVC	130.0	11.00000	0.6
1	P-245	156	150.0	PVC	130.0	11.40000	0.8
1	P-246	49	150.0	PVC	130.0	10.90000	0.5
1	P-250	198	150.0	PVC	130.0	11.10000	0.6
1	P-251	95	150.0	PVC	130.0	10.80000	0.5
1	P-253	48	200.0	PVC	130.0	12.72000	0.9
1	P-254	68	200.0	PVC	130.0	12.72000	0.9
1	P-255	24	200.0	PVC	130.0	11.62000	0.5
1	P-256	73	200.0	PVC	130.0	11.62000	0.5
1	P-257	64	200.0	PVC	130.0	11.22000	0.4

Zone	Label	Length (m)	Diameter(mm)	Material	Hazen-Williams C	Flow(L/s)	Velocity (m/s)
1	P-258	59	200.0	PVC	130.0	11.22000	0.4
1	P-259	52	200.0	PVC	130.0	11.02000	0.33
1	P-260	51	200.0	PVC	130.0	10.90000	0.3
1	P-261	73	200.0	PVC	130.0	10.70000	0.3
1	P-282	34	200.0	PVC	130.0	13.59000	1.1
1	P-283	90	200.0	PVC	130.0	13.49000	1.1
1	P-284	97	200.0	PVC	130.0	12.99000	0.9
1	P-285	43	200.0	PVC	130.0	12.39000	0.8
1	P-286	68	200.0	PVC	130.0	12.39000	0.8
1	P-287	24	200.0	PVC	130.0	12.04000	0.6
1	P-288	24	200.0	PVC	130.0	12.04000	0.6
1	P-289	21	200.0	PVC	130.0	11.64000	0.5
1	P-290	57	200.0	PVC	130.0	11.60000	0.5
1	P-292	21	200.0	PVC	130.0	11.30000	0.4
1	P-293	70	200.0	PVC	130.0	11.10000	0.4
1	P-294	90	200.0	PVC	130.0	10.90000	0.3
1	P-295	48	200.0	PVC	130.0	10.50000	0.2
1	P-75	154	350.0	PVC	130.0	36.88998	0.38
1	P-344	97	400.0	Ductile Iron	120.0	62.00000	0.49
1	P-345	94	400.0	Ductile Iron	120.0	62.00000	0.49
1	P-13	134	400.0	Ductile Iron	120.0	62.00000	0.49
1	P-343	58	450.0	Ductile Iron	120.0	40.00000	0.25
1	P-347	58	450.0	Ductile Iron	120.0	40.00000	0.25
1	P-348	36	500.0	Ductile Iron	120.0	40.26000	0.21
1	P-1	33	200.0	PVC	130.0	41.60000	0.5
1	P-2	59	200.0	PVC	130.0	41.60000	0.5
1	P-3	30	350.0	PVC	130.0	31.58998	0.33
1	P-4	131	350.0	PVC	130.0	31.59000	0.33
1	P-5	30	300.0	PVC	130.0	22.73000	0.32
1	P-6	20	300.0	PVC	130.0	22.73000	0.32
2	P-123	85	250.0	PVC	130.0	19	0.40
2	P-124	55	250.0	PVC	130.0	19	0.40
2	P-26	102	200.0	PVC	130.0	36	1.14
2	P-27	278	200.0	PVC	130.0	36	1.14
2	P-28	286	200.0	PVC	130.0	36	1.14
2	P-125	111	250.0	PVC	130.0	19	0.40
2	P-127	145	200.0	PVC	130.0	14	0.45
2	P-128	96	200.0	PVC	130.0	14	0.44
2	P-129	90	200.0	PVC	130.0	13	0.42
2	P-130	79	200.0	PVC	130.0	13	0.40
2	P-131	73	200.0	PVC	130.0	12	0.39
2	P-132	41	200.0	PVC	130.0	12	0.38
2	P-133	44	200.0	PVC	130.0	12	0.37
2	P-134	43	200.0	PVC	130.0	11	0.36
2	P-135	42	200.0	PVC	130.0	11	0.34

Zone	Label	Length (m)	Diameter(mm)	Material	Hazen-Williams C	Flow(L/s)	Velocity (m/s)
2	P-136	60	200.0	PVC	130.0	10	0.33
2	P-137	108	150.0	PVC	130.0	9	0.52
2	P-138	127	150.0	PVC	130.0	9	0.49
2	P-139	118	150.0	PVC	130.0	8	0.46
2	P-140	104	150.0	PVC	130.0	8	0.44
2	P-141	46	150.0	PVC	130.0	7	0.41
2	P-142	58	150.0	PVC	130.0	7	0.40
2	P-143	76	150.0	PVC	130.0	7	0.38
2	P-144	81	150.0	PVC	130.0	6	0.35
2	P-145	98	150.0	PVC	130.0	6	0.31
2	P-146	61	150.0	PVC	130.0	5	0.28
2	P-147	69	150.0	PVC	130.0	5	0.27
2	P-148	49	150.0	PVC	130.0	4	0.25
2	P-149	233	150.0	PVC	130.0	4	0.23
2	P-150	227	150.0	PVC	130.0	3	0.19
2	P-151	71	150.0	PVC	130.0	3	0.16
2	P-152	117	150.0	PVC	130.0	3	0.14
2	P-153	123	150.0	PVC	130.0	2	0.11
2	P-154	129	150.0	PVC	130.0	2	0.11
2	P-170	72	150.0	PVC	130.0	5	0.28
2	P-171	103	150.0	PVC	130.0	4	0.22
2	P-172	113	150.0	PVC	130.0	7	0.65
2	P-173	106	150.0	PVC	130.0	8	0.7
2	P-174	108	100.0	PVC	130.0	8	0.7
2	P-175	38	100.0	PVC	130.0	8	0.7
2	P-176	255	100.0	PVC	130.0	10	0.8
2	P-338	25	200.0	PVC	130.0	19	1.14
2	P-339	52	200.0	PVC	130.0	19	1.14
2	P-1	21	250.0	PVC	130.0	19	0.40
2	P-2	35	250.0	PVC	130.0	19	0.40

Appendix –H

Node - ID in Modeling Water Distribution Network



Appendix-I

Pipe - ID in Water Distribution Modeling