JIMMA UNIVERSITY

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JIMMA INSTITUTE OF TECHNOLOGY/JIT/

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

HYDROLOGY AND HYDRAULIC ENGINEERING CHAIR

MASTERS OF SCIENCE IN HYDRAULIC ENGINEERING

HYDRAULIC PERFORMANCE EVALUATION OF EXISTING WATER SUPPLY DISTRIBUTION NETWORK; THE CASE OF DURAME TOWN, SNNPR, ETHIOPIA

A THESIS SUBMITTED TO THE CHAIR OF HYDROLOGY AND HYDRAULIC ENGINEERING, JIMMA INISTITUTE OF TECHNOLOGY, SCHOOL OF GRADUATE STUDIES, JIMMA UNIVERSITY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTERS OF SCIENCE IN HYDRAULIC ENGINEERING

By: - BEREKET KEBEDE

DECEMBER, 2019

JIMMA, ETHIOPIA

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DECEMBER, 2019

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APPROVAL PAGE

This thesis entitled with "Hydraulic performance evaluation of existing Water Supply Distribution Network in Durame town" has been approved by the following examiners for partial fulfilment of the requirement for the degree of Master of Science in Hydraulic Engineering.

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As members of the Board of Examiners of the MSc Thesis Open Defense Examination, we Certify that we have read, evaluated the Thesis prepared by Bereket Kebede and examined the candidate. We recommended that the Thesis be accepted as fulfilling the requirement for the degree of Master of Science in Hydraulic Engineering.

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DECLARATION

I, the undersigned, declare that this thesis entitled with "Hydraulic performance evaluation of Existing water supply distribution network": In case study in Durame town is my original work, and has not been presented and submitted for any degree in any other university. It is being submitted for the degree of Master of Science in Hydraulic Engineering, and all sources of material used for this thesis have been fully acknowledged.

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ABSTRUCT

The occurrence of ephemeral/transient flow in water distribution system creates large pressure forces and rapid fluid accelerations or deceleration that can cause failure of hydraulic equipment in a pipe network if adequate transient control measures are not in place. Safe and adequate delivery of water to a consumption node is an essential function of water distribution network. However, throughout the world especially in developing countries, the hydraulic performance of water distribution network is inadequate to transfer available water to a consumption node. Therefore, this study aims with the study of hydraulic parameters performance of Durame town water supply distribution network project as the case study. Durame town is the capital of Kembata Tembaro Administrative Zone of the SNNP Regional State. The water supply distribution network of Durame town was designed and constructed in 1983 by different spiritual missionaries. The methods used to carry out this research were desk study to evaluate previous designs, field data collection and data analysis. For conducting this study, both primary and secondary data were collected and hydraulic modeling software tool such as Bentley WaterGEMSv8i, ArcGIS version10.1, Geographic positioning system Garmin72 (GPS) and other relevant materials *were used. The study was carried out by selecting pipes having diameter greater or equal to 50 mm in diameter. The simulated result for both steady state and extended period simulation showed that the performance of distribution system related to pressure 36.47% for pressure value (< 15 mH_2O), 50.59% for pressure value (15 - 60) mH_2O and 12.94% for pressure value (> 60 mH_2O) pressure head and the velocity of pipe flow showed that 56.7% for velocity (< 0.6 m/s), 41.7% for velocity range (0.6 - 2 m/s) and 1.6% for velocity (> 2 m/s). Those problems are resulted from incorrect nodal placement and improper pipe connection during designing the system and when expanding the network to the newly established settlement area. The low pressure zone areas around Industrial College and below and above Durame general Hospital did not get water at these junctions during peak hour demand. The per capita domestic water consumption of study area was found to be 15.2 l/c/d in the year 2018. The minimum quantity of domestic water required in urban area of developing country in the radius 0.5 km taken as 20 l/c/day. Regarding to this value, the domestic water supply of Durame town only satisfies 76% of the standard value and the quantity of domestic water required in urban areas of Ethiopia is taken as 50 l/c/day. According to this value, the domestic water supply of Durame town only satisfies 30% of the standard value. The total loss of water in the town for the year was 700289 m^3 – 456580 m^3 which gives 243709 m^3 and approximately 34.8% of the total production. This figure is lower compared with the average for developing countries (35%). The average the amount of water, which actually reached the consumers, therefore accounts for only 65.2% of the total water produced. The potential of the projected water demand increment in Durame town is greater than the current supply potential of water sources. The current water demand is 6,196.75 m^3/day and the demand at end of design period of 2038 years would be around 22,467.82 m^3/day. Finally, the researcher recommends preventive measures of interruption water supply network using control valves and installing storage distribution balancing tank above the Durame industrial college for delivering enough and sufficient amount of water to the customers.*

Key words: *Demand, Hydraulic performance, Simulation, Water distribution Network, Water losses*

ACKNOWLEDGEMENTS

First and for most I am so grateful to the Almighty God for all I have received and for giving me health and strength to pass through all confronts during my life. I would like to express my profound gratitude and Many thanks to my research advisor Dr.-Ing. Fekad Fufa (PhD, Asso. Prof) and co-advisor Mr. Andualem Shigute (MSc) for rendering their immense and valuable supervision, support and guidance and valuable comments for the accomplishment of my research paper. Thank you!

I would like to extend my sincere gratitude to Jimma University to elaboration and giving this chance to me. I would also like to acknowledge Tropic Consultancy, which was my source of data to begin with my study, and my specific gratitude extends to Engr. Behayilu Tamiru, Mr. Muluneh Mathewos for helping me during field GPS survey and data collection for his technical support. Mr. Abenezer Tesfaye Zonal WMEO head, for his technical support and consultation when I begin my thesis.

I am indebted to my uncle Mr. Tekle Doboch for providing me the basic tools in life to recognize the importance of education, and the support necessary to help me realize my academic goals.

Finally, I would like to express my heartfelt thanks to all the staff off DTWSSO and AWWMEO, especially Mr. Temesgen Worku Head of AWWMEO, Getachewu Huro, Selamu Tadege, Markos Menta and Tarekegn Lodebo for their support in providing all the necessary information without a feeling of boredom whenever I went there.

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

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ACRONYMS

AWWA	American Water Works Association
AWWMEO	Angacha Woreda Water Mineral and Energy Office
BH	Borehole
CSA	Central Statistical Agency
DCI	Ductile Iron
DEM	Digital Elevation Model
DTWSSO	Durame Town Water Supply and Service Office
Dr.-Ing	Doctor Engineer
EPA	Environmental protection Agency
GIS	Geographic Information System
GPS	Global Position System
HC	House Connection
HDPE	High Density Polyethylene
HTU	House Tab User
ISO	Insurance Service Organization
KPa	Kilo Paskal
M.a.s.l	Mean above sea level
MDGR	Millennium Development Goal Report
MoH	Ministry of Health
MoWIE	Ministry of water Irrigation and Electricity
MoWR	Ministry of Water Resource
MSc	Master of Science
NGOS	Non-Governmental Organizations
NTU	Neighbour Tab Users
OWWDSE	Oromia Water Woks Design and Supervision Enterprise
Ph.D.	Philosophy of doctorate
PTU	Public Tab User
PVC	Polyvinyl Chloride
SSR	Summary and Statistical Report
TRex	Terrain Extractor

UFW	Unaccounted for Water
UNDP	United Nation Development Program
UNICEF	United Nations Children's Fund
WDN	Water Distribution Network
WDS	Water Distribution System
WHO	World Health Organization
WV	World Vision
YC	Yard Connection
YCS	Yard connection Share
YTU	Yard Tap Users

CHAPTER ONE

INTRODUCTION

1.1 Background

Water is the primary need to maintain life; every person in the World has the right to have access to potable water (GARG, 2010). Conditions of a safe, reliable, affordable, and easily accessible water supply is essential for good health and development (EPA, 2005b). To transport this precious substance also needed good water delivery system from single or multiple supply sources to consumers. Then water supply distribution systems are the most important public utility (Swammee, 2008).

The prerequisite of adequate and reliable water supply in developing countries is becoming a challenge for most water utilities especially public service providers (Khatri, 2007). Water demand has been increasing drastically in developing nation due to population growth as a result of rural to urban migration. As a consequence, in many countries public water service utilities have failed to provide consumers with adequate water supply and sanitation services. A partly from service coverage, there are other problems that affect public service providers such as high unaccounted for water (UfW) and financial problems due to a combination of low tariff, poor services, poor consumer records and inefficient billing practices (Kimey, 2008). The estimated water supply service level of Ethiopia in terms of coverage, quantity, quality and reliability is very low (Dighade *et al.*, 2014).

Water supply is an inevitable part of the urban infrastructure. The operational practices of large scale water supply networks still continue to be a major engineering challenge (Khatri, 2007). Water supply authorities are responsible to meet consistently the demand of different water consumer sectors including fire flows in the distribution system, to maintain reasonable flow velocities and service levels within the possible operational boundaries and to manage the available storage capacities for balancing the supply and demand in the pipe network (Atesu, 2015).

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

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

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

According to the Millennium Development Goals, of the Ethiopia water supply for all is emphasized. Government of Ethiopia aims to increase access to safe water supply to 98 present for rural areas and 100 present for urban areas and to provide all Ethiopians with access to basic sanitation. Proportion of population served living within a certain distance to an improved water point or scheme (1.5 km for rural and 0.5 km for urban areas) (WHO and UNICEF, 2012).

This study investigated the design of water distribution system of Durame town and evaluated the hydraulic performance of water distribution system under varying conditions of supply &

demand. It was made by reviewing the capacity of reservoir, water source, the total water demand and supply of the project which was Constructed by AG Consult with South Design and Supervision Works Enterprise in 2005 and revised by GTB Engineering Association in 2011. The evaluation of hydraulic performance of distribution system was made based on the design parameters, such as water pressure, flow velocity, terms of pressure using WaterGEMSv8i.

1.2 Statement of the Problem

Problems that affect the performance of the public water utilities, in the developing countries including Ethiopia, experience high (UfW) rate, which often average between 40 - 60%, meaning that about half of the potable water produced is lost somewhere in the supply process. Moreover, the public utilities often face financial challenges due to a combination of low tariffs, poor services, poor consumer records and inefficient billing and collection practice (Victor Kimey *et al.,* 2008).

The most important performance indicators in water supply distribution network is availability of optimum pressure at networks head and flow velocity in pipes. To deliver available water to every water consumer's optimum pressure and velocity in the system should be maintain to avoid water column separation and to ensure water supply demands at all time. Pressures in distribution system fail at maximum consumption hour and should not push water to the point of consumption node as well as during night time the consumption decreases and the pressure becomes high. The deficiency of hydraulic parameter (flow velocity and pressure) occurred due to random connection (placement) for nodes and pipe without any scientific method/mathematical calculation for flow and pressure (Misirdali, 2003).

Evaluating the performance of urban water supply system is important for identifying weakness and strengths of the system and to improve the water supply service level. A best performing system should provide safe, sufficient and realistic water supply service, with low water loss which full fills national and international standards. The major challenges of urban water supply systems in developing countries including the Ethiopia, low water supply service coverage, unavailability of sufficient water at all times, occurrence of very high amount of water loss, due to it does not meet national or international drinking water standards, (World Bank, 2006).

Durame is one of the medium level towns in the country with recent rapid urbanization and high population growth. The area has been experiencing frequent and regular disruption of water supplies for days to a week resulted from poor estimation of demand and supply relation, intermittent supply due to hydraulic problem, maintenance costs of existing water supply systems, local operation and management such as peak factors adjustment. Although the town water supply and service office trying to control the problems, delivering sufficient water without any interruption to the inhabitant persons remains dream. This study was undertaken using WaterGEMSv8i and the existing water distribution network was simulated for both steady sate and extended period simulation analysis to evaluate the performance of the system related to pressure and velocity.

1.3 Objectives

1.3.1 General Objective

The general objective of the study was evaluating the hydraulic performance of Durame Town Water Supply Distribution Network using WaterGEMSv8i.

1.3.2 Specific Objectives

- to model the existing water distribution network using WaterGEMSv8i
- to evaluate hydraulic parameters performance of the existing water distribution network
- to assess water losses in the network
- to evaluate the present water demand and forecast future demand

1.4. Research Questions

1. How can model the existing water distribution network of Durame town?
2. How can evaluate the Hydraulic parameters performance in the water supply distribution network?
3. How can assess the water losses in the network?
4. Is the present water supply satisfying the current and future water demand of the town?

1.5 Significance and rational of the study

Urban Water supply distribution network depends mainly on reservoirs/Wells, pump, Tanks, pipes, nodes and other relevant elements, based on this several government agencies shared the responsibility for the development of domestic water supplies in the state (Khatri, 2007).

According to Birerley (2006), Modeling and simulation are aimed at providing valuable insights in the problem structure instead of giving precise answers. With the advances of this technology, water utilities and engineers have been able to analyse the status and operations of the existing system as well as to investigate the impact of proposed changes. It helps to give insight for water sector, governmental organization, and NGOs for the type of problems existed in water supply distribution network to plan new water project and how to solve those problems for previous water project to meet the need for water to rapidly growing population. Despite of these efforts good water supply distribution system in Durame town is still characterises with inadequate distribution, insufficient coverage of services and water losses. This is due to increasing number of population and town expansion in different infra structures. Evaluating the town distribution network with WaterGEMSv8i software will be of great importance in solving problems related to hydraulic performance in water supply distribution network of the Durame town. Little has been done to review and analyze the performance of the urban water supply utilities.

In this study WaterGEMSv8i software is used. Its package is integrated with AutoCAD software technologies and its ability to interchange data between AutoCAD and WaterGEMSv8i software's was the reason to choose it for this study.

1.6 Scope of the Study

The primary objective of the study was to create the base line information and undergo simulation of existing water distribution network system by running the model for Steady State and extended period simulation analysis to identify system hydraulic performances related to pressure and velocity in the urban residences of Ethiopia particularly in the Durame town. This research was used hydraulic network analysis software Bentley WaterGEMSv8i. The performance of the system was observed under peak consumption and minimum time consumption and its performance was evaluated based on hydraulic conditions not including water quality.

1.7 Limitation of the Study

The main limitation of this study was un availability of documented data which describe the problems that related to poor performance of water distribution systems and the course of this study was associated with getting sufficient updated secondary data. For example, there was shortage of well-documented data sources and adequate report especially in the study area data not organized in the office Data like: Coordinates (x, y, z) of the nodes of the system, real existing design document of distribution system, the division of shape files for each Keble's and topography of the town.

Finally, during the time of this study, there were challenges to calibrate hydraulic networks and allocated nodal demand. All Pressure gages in distribution system were not working to measure pressure and there were no enough bulk meter along distribution system to measure, water loss at field surveying. This was difficult, to calibration of hydraulic network model to compare simulated and field survey results.

CHAPTER TWO

LITERATURE REVIEW

2.1 History of Water Supply Distribution Network

In history, human settlements have been founded following fresh water bodies is available, whether that is near springs, along riverside and lakes or in deltas and coastal areas (Boyd, 2005). These small settlements grew to develop to larger towns and cities. As industrialization progressed, increasing quantities (needs) of water was required for power generation and industrial processes. The influx of people from countryside to cities to work in the new industries meant that more water was needed for household purpose such as drinking, cooking and washing and also used as a means of human waste transport. Trifunovic'(2006) is elaborating general principles and practices in water transport and distribution in a practical and straight forward way.

Water distribution systems consist of a network of smaller to larger pipes with numerous connections that supply water directly to the users. The flow variations in such systems are much wider than in cases of water transport systems. In order to achieve optimal operation, different types of reservoirs, pumping stations, water towers, as well as various appurtenances (valves, hydrants, measuring equipment, etc.) can be installed in the system to control, measure & withdraw water flow in the pipe. Analysis of a pipe network is essential to understand or evaluate a physical system. In case of a single input system, the input discharge is equal to the sum of withdrawals. The known parameters in a system are the input pressure heads and the nodal withdrawals /discharge. In the case of a multi-input network system, the system has to be analysed to obtain input point discharges, out pipe discharges, and nodal pressure heads. Several researches have been made to study the behaviour of urban water distribution systems and to reach an optimal solutions and assumptions in order to improve the hydraulic performance, cost effectiveness, and to increase the efficiencies of the water supply distribution networks.

Shaher (2004) studied the hydraulic performance of water distribution systems under the action of cyclic pumping; the results show that the network under consideration is exposed to relatively high pressure values throughout. The velocity of the water through the network attained also high values. These high values of pressure and velocity have negative effects on the performance of the network. Water distribution Networks convey water drawn from the water source or treatment facility, to the point where it is delivered to the users. Unlike the transmission systems, these systems deal with water demand that varies considerably in the course of a day. Water consumption is highest during the hours that water is used for personal hygiene and cleaning, and when food preparation and clothes washing are done. Water use is lowest during the night. A typical WDNs consists of network of pipes, nodes linking the pipes, storage tanks, reservoirs, pumps, additional appurtenances like valves(Belay,2012).

Water distribution systems represent a major portion of the investment in urban infrastructure and a critical component of public works. The main goal is to design water distribution systems to deliver potable water over spatially extensive areas in required quantities and under satisfactory pressures. Therefore, hydraulic models for water distribution networks have become indispensable tools for understanding system behaviour by simulating pressures and flows at different locations and times in the networks (Nyende.*et al.*, 2012). The design of water distribution systems in general based on the assumption of continuous supply. However, in most of the developing countries, the water supply system is not continuous but intermittent (Khatri, 2007).

Design and Operation of Water distribution networks: Although almost all distribution systems are designed to meet peak hour demands, it will create low – flow conditions in some parts of the distribution system and may result in deterioration of microbial and chemical water quality of the system unless they are designed considering these water quality aspects. The purpose of water distribution network systems is to supply water at an adequate pressure and flow for the consumers. However, when designing piped water distribution system, excessive system capacity, low flow dead-end and loops, situations that may rise to negative pressure must be considered and avoided (AWWA, 2005).

2.2 Methods of Laying Distribution Pipes

2.2.1 Branched or Tree System

In this system, a main line is taken from the reservoir along the main road. The sub-mains are taken suitably from the main line. Cut-off values are provided at the entry of sub-mains. From the sub mains, the branch lines are taken from which service connections are given to consumer through the ferrule. The end of the sub-mains and branch lines are stopped by scour values which are known as dead – ends.

- Due to the dead – ends, there is no free circulation of water and the water remains stagnant within the pipe line.
- This system is suitable for regular developing town or city.

Advantages:

- Discharge and pressure at any point in the distribution system is calculated easily
- The valves required in this system of layout are comparatively less in number.
- The diameter of pipes used are smaller and hence the system is cheap and economical
- The laying of water pipes is used are simple.

Disadvantages:

- There is stagnant water at dead ends of pipes causing contamination.
- During repairs of pipes or valves at any point the entire downstream end is deprived of supply.
- Fire protection is at risk due to inability to isolate a break.

2.2.2 Grid Iron or Loop System

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

looped system Advantages are:

- Fluid velocities are lower, reducing head losses, resulting in greater capacity.
- Main breaks can be isolated to minimize loss of service to customers.
- Fire protection is greater due to greater capacity and ability to isolate breaks.
- Looped systems usually provide better residual chlorine content due to inline mixing and fewer dead ends.

looped system disadvantages

- Looped systems generally cost more because more length of pipes and number of valves are needed and hence there is increased cost of construction.
- Calculation of sizes of pipes and working out pressures at various points in the distribution system is laborious, complicated and difficult.

2.2.3 Circular or Ring System

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

2.2.4 Radial System

This is a zoned system. Water is pumped to the distribution reservoirs and from the reservoirs it flows by gravity to the tree system of pipes. The pressure calculations are easy in this system. Layout of roads needs to be radial to eliminate loss of head in bends. This is most economical system also if combined pumping and gravity flow is adopted.

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

2.3 Methods of Water Distribution

There are three methods of water distribution system delivered from the source to consumers' house (Zyoud, 2003). These are as follows:

2.3.1 Gravity Distribution

This is possible, when the source of water is elevated, so that sufficient pressure can be maintained in the systems. The main important of this method of water distribution system is saving power that needed for pumping.

2.3.2 Distribution by Pumping without Storage

In this method of distribution, water can have pumped directly into the main distribution lines without transfer water to service reservoir. The pumping rate should be sufficient to satisfy the demand. An advantage of direct pumping is that a large fire service pump may be used which can run up the pressure to any desired amount permitted by the construction of mains.

2.3.3 Distribution by Means of Pumps with Storage

This method used when there is an elevated reservoirs used to maintain the excess water pumped during periods of low consumption, and these stored quantities of water may use during the periods of high consumption.

2.4 Components of Water Distribution Network

Basically distribution system is divided into primary, secondary and tertiary mains which are defined as follows: Primary Main: that part of the system which conveys water from reservoirs to secondary distribution pipelines. The capacity of primary distribution main is determined by the peak hour demand. Generally, all pipelines of DN 200 mm and above which are not transmission mains will be considered as part of primary distribution main. Secondary Main: that part of the distribution systems which is fed by the primary pipe lines and conveys water to consumers, either directly or through a tertiary main, or that forms across-connection between two or more primary mains. The secondary distribution main is designed for peak hour demand. Generally, pipelines of DN 150 and 100 mm are considered part of the secondary distribution main. Tertiary Main: that part of the distribution system which is fed by main or secondary pipelines and conveys water to consumers through service connections. The tertiary distribution main is also designed to meet the peak hour demand consumers.

Distribution Reservoir

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

Storage Tank

Storage Tanks are artificial structures that store water and provide water to the system when needed. Equalizing and emergency storage are the two basic task of storage tanks. The variation in flow can be dealt with by operating pumps in parallel and /or building balancing storage in the system. Moreover, in low demand hours when the water consumptions of consumers are almost zero, amount of pumped water is higher than system demand and extra water coming from pumps are stored at storage tank and equilibrium of water distribution system is satisfied again. This equilibrium purpose of storage tank is called as equalizing storage. In addition, storage tanks help water utility to easily manage pressure distribution by prevention pressure fluctuations Emergency storage ability of storage tanks provide required water to perform fire-fighting operations or maintenance operations. For instance, if the pump of distribution network is turned off due to power cut, distribution network continues to serve to the customers by using water stored in the storage tank till the end of power outages (Al-Rayess, 2015).

Pipes

Pipes are the essential elements of a water distribution system. All the elements of water Distribution system, such as junction nodes, pumps, reservoirs, valves and tanks are linked to each other by pipes. Earlier, only limited sizes and types of water supply pipes were available, but nowadays with the help of developing technology, pipes are produced in different materials and sizes to be used in residential and commercial water supply network applications (Kay Chamber, 2004).

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

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

Pump

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

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

$$Q_O = Q_t \left(\frac{P_s - P_O}{P_s - P_t}\right)^{0.5} \qquad 2.1$$

Where

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

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

Valves

In a water supply system, valves are the major component to control the flow of water. By operating the valve, the flow can be controlled in different ways. Completely preventing water flow, adjusting the amount of water flow, directing flow to different paths and reducing flowing water pressure are some capabilities of valves in water supply system. Valves may be operated manually, either by a handle, lever or wheel. Valves may also be operated automatically by electronic devices and may be operated remotely (Newbold, 2009).

Fire Hydrants

A fire hydrant is an essential element of water distribution network to provide required water for fire-fighting. In fire-fighting operation, pressure and flow of water are important factors while extinguishing fire hydrants are designed to provide required high water pressure and flow.

Therefore, fire hydrants are connected to the distribution network with pipe having larger diameters to provide excessive water flow required for fire-fighting (Almasri, 2010).

Junctions

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

2.5 Water Distribution Network Hydraulic Modeling

2.5.1 Needs for Hydraulic Modeling

The small community's towns do not have very complex networks as compared to cities; however, they have poor data and records regarding their systems. In such cases, when one has to evaluate the hydraulics and the water quality of the distribution systems, it is advantageous to use computer models. Computer models making use of hydraulic simulation software are capable of mimicking the behave or of a real time system and have the capability of predicting the performance of the same system for future (Zhang, 2009).

Initially hydraulic models were simulating flows and pressures in a distribution system under steady state conditions assuming all demands and operations remained constant, but since demands and flows vary over the course of a day, Extended Period Simulation Models which can simulate distribution systems behaviour under time-varying conditions were developed (EPA, 2005a). Steady state simulations were advanced to EPS using the technique developed by Rao and Bree in the late 1970's (Laura Baumberger *et al*., 2007).

2.5.2 Modeling Concept

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

Conservation of Mass: it is equivalent to conservation of volume and with the assumption that water is an incompressible fluid, this principle requires that the sum of mass flows of all pipes entering a junction must be equal to the sum of mass flows of all pipes leaving that junction.

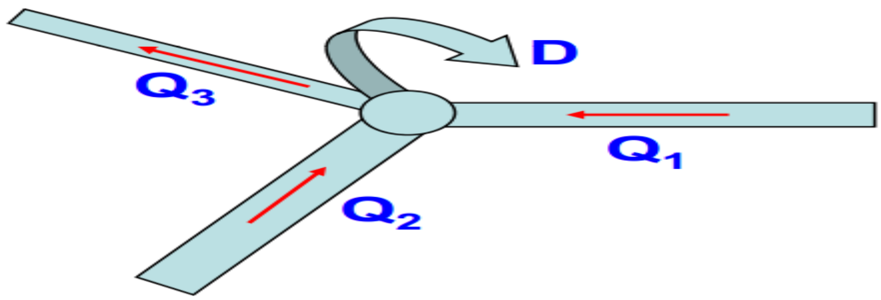

Figure 2. 1 Algebraic sum of flow rates entering and withdrawing from the node (Almasri, 2010).

$$Q_1 + Q_2 = Q_3 + D \qquad 2.2$$

$$D = Q_1 + Q_2 - Q_3 \qquad 2.3$$

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

D = external demand withdrawn from the node.

Conservation of Energy: According to Bernoulli's equation (EPA, 2005a), It consists of the pressure head, elevation head, and velocity head. There may be also energy added to the system (such as by a pump), and energy removed from the system due to friction.

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

$$\frac{p}{\gamma} + z_1 + \frac{v_1^2}{2g} = z_2 + \frac{v_2^2}{2g} + H_L \qquad 2.4$$

$Where, \quad \frac{p}{\gamma} = Pressurehead, \quad \frac{V^2}{2g} = Velocityhead, \quad Z = elevationhead,$

$H = totalhead$

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

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

2.5.3 Modeling Process

The task of assembling, testing, calibrating, validating and using the water distribution system model requires breaking the task in to its components and working through each step. Some tasks can be done in parallel and some tasks in series.

2.5.4 Gathering a Model

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

2.5.5 Data Requirements for Modeling of Water Distribution Network

Sources of Data: Electronic maps, recordings and CAD drawings are common target data sources. Non-graphical data such as tracking and inventory data base or text based models can also be used, but with great care; because simple topographic errors in a non-graphical network is very difficult to detect (Walski, 2003).

2.5.5.1 Basic Hydraulic Model Inputs

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

Water demand inputs: Data concerning existing demand from water billing systems, spatial allocation of data from billing and GIS, time varying factors, projected future demands and their allocations from the water utility and regional planning documents can be collected. Operational and model control inputs: Information on source nodes, pump stations, reservoirs, control valves and zonal valves can be collected from the operational staff. It is necessary to define a set of rules that tells how the water system operates in an EPS model. These operation rules maybe a set of 'logical controls' in which operations such as pump on/off, valve status, pump speeds, tank water levels, node pressures, demands etc. are controlled using 'what-if…then else' logical operators (Grayman and Rossman,1994).

2.5.6 Application of Water Distribution Models

WDN Simulations are used for long range master planning including new developments and rehabilitation, for fire protection studies, water quality investigations, energy management, system design, daily operational uses, operator training and emergency responses. ''Properly assembled model is an asset for the water utilities, much like a pipe or fire hydrant'' (Walski, 2003).

2.5.6.1 Role of Models in Operations

Operation personnel have to accept computer simulations as a tool to keep the WDN running smoothly. As a result, they will spend relatively few field observations to identify what is occurring in the distribution system. Models enable operational personnel to formulate solutions that will work correctly for the first time as an alternative of trial and error changes on the actual system for identifying the problems occurred in the system.

2.6 Urban Water Supply

Safe drinking water is the birthright of all humankind as much a birthright as clean air (Rao, 2002) while access to clean water can be considered as one of the basic needs and rights of a human being. Health of people and dignified life is based on access to clean water (Korkeakoski, 2006). Water is important in a number of ways; these include domestic and productive uses. Domestic water use takes the form of drinking, washing, cooking and sanitation, while productive water uses includes those for agriculture, Beer brewing, brick making, etc. Safe drinking water matched with improved sanitation contributes to the overall well-being of people; it has significant bearing on infant mortality rate, longevity and productivity. However, the majority of the world's population in both rural and urban settlements does not have access to safe drinking water (Alaci, 2009). According to WHO (2006), only 16 % of people in sub-Saharan Africa had access to drinking water through a household connection (an indoor tap or a tap in the yard). The primary goal of all water supply utilities is to provide customers with a private connection to the piped water supply network. For many public officials, policy makers and politicians a household or yard connection (here after referred to as a private connection) is considered the most satisfactory way to meet the following key objectives; Public health objectives: by ensuring better quality and access. Commercial objectives: by facilitating cost recovery and revenue generation. Social objectives: by improving access for the poorest and enhancing security and safety. Environmental objectives: by enabling better demand management and water conservation (AWUP, 2003).

2.7. Performance Evaluation of Urban Water Distribution System

Performance of a water distribution network can be defined as its ability to deliver a required quantity of water under sufficient pressure and an acceptable level of quality during different normal and abnormal operational situations (Tabesh and Dolatkhahi, 2006). Evaluating the performance of water supply systems is an important for water industry to deliver competent levels of service. A good distribution system should be a capable of supplying water at all intended place within the city with reasonably sufficient pressure head and the requisite amount of water for various types of demand (Garg, 2010). The performance of urban water supply scheme can be evaluating based on four performance measures: Hydraulic, Structural, Water quality and Customers perception.

2.7.1 Hydraulic Performance

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

2.7.2 Structural Performance

Water mains generally consist of a variety of pipe work and fittings, and which over time are subject to various episodes of augmentation, refurbishment, renewal, replacement, repair and extension. Physical performance of water supply system is the ability of the distribution system to act as a physical barrier that prevents external contamination from affecting the quality of the internal, drinking water supply (Tabeshe and A. Dolakhahi,2006).

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

2.7.3 Customer Perception

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

2.8. Complications of Water Distribution Network

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

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

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

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

2.8.1 Head Losses

There are different factors that cause the energy losses. The main reason of the energy loss is due to internal friction between fluid particles traveling at different velocities (Zyoud, 2003). There are two forms of resistance which causes energy loss in distribution system.

Surface Resistance: Head loss on the account of surface resistance; depend on pipe length, coefficient of surface resistance and friction factor. Surface resistance is characterized as major loss.

Form Resistance: The form resistance loss is due to bends, elbows, valves, enlargements, reducers, and so forth categorized as minor loss.

2.8.2 Head Loss Equations

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

Table 2. 1 Head loss equations and area of application Melaku (2015)

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

where: - V~ velocity, n ~ Manning's roughness coefficient, R ~ hydraulic radius, S~ slope,
C ~ Hazen-William roughness coefficient.

Friction Losses

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

Minor Losses

Minor losses are a result of localized areas of increased turbulence and are Frictional head losses, which cause energy losses within a pipe. A drop in the energy and hydraulic grades caused by valves, meters, and fittings, the value of these minor losses is often negligible.

Other Causes of Poor Water Pressure

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

2.9 Water Hammer

When the velocity of flow in a pipe changes suddenly, surge pressures are generated as some, or all, of the kinetic energy of the fluid is converted to potential energy and stored temporarily via elastic deformation of the system (Zyoud, 2003). As the system rebounds and the fluid returns to its original pressure, the stored potential energy is converted to kinetic energy and a surge pressure wave moves through the system. Ultimately, the excess energy associated with the wave is dissipated through frictional losses. This phenomenon, generally known as “water hammer”, occurs most commonly when valves are opened or closed suddenly, or when pumps are started or stopped. The excess pressures associated with water hammer can be significant under some circumstances in distribution network.

2.10 Water Transmission

The water needs to be transported from the source to the treatment plant, if there is one, and onward to the area of distribution. Depending on the topography and local conditions the water may be conveyed through free-flow conduits, closed conduits or a combination of both. The water conveyance will be either under gravity or by pumping. Free-flow conduits are generally laid at a uniform slope that closely follows the hydraulic grade line. Examples of such conduits are canals, aqueducts, tunnels or partially filled pipes. If a pipe or tunnel is completely full, the hydraulic gradient and not the slope of the conduit will govern the flow. The hydraulic laws of closed conduit flows, also commonly called pressurised flows, apply in this case. Pressurised pipelines can be laid up- and downhill as needed, as long as they remain at sufficient distance below the hydraulic grade line, i.e. a certain minimum pressure is maintained in the pipe. Free-flow conduits have a limited application in water supply practice in view of the danger that the water will get contaminated. They are never appropriate for the conveyance of treated water, but may well be used for transmission of raw water. For community water supply purposes, pressurised pipelines are the most common means of water transmission. Whether for free flow or under pressure, water transmission conduits generally require a considerable capital investment. A careful consideration of all technical options and their costs and discussion with the community groups that will support and manage the system are therefore necessary when selecting the best solution in a particular case. Trifunovic, N. (2006). the formula employed for the calculation of the diameter of the pipe is the Lea formula which states that:

$$D = K(Q)^{1/2} \qquad 2.5$$

Where: D is diameter, K is a constant between 0.97 and 1.22., Q is Maximum day demand

2.11 Hydraulic Design Parameters

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

2.11.1 Pressure

The pressure at nodes depends on the adopted minimum and maximum pressures within the network, topographic circumstances, and the size of the network. The minimum pressure should maintain to ensure that consumers' demand provided at all times. The maximum pressure also contains limitation of leakage and lead to water losses in distribution system.

2.11.2 Flow Velocity

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

$$V = \frac{4Q}{\pi D^2} \qquad 2.6$$

$$D = \sqrt{\frac{4Q}{\pi V}} \qquad 2.7$$

Where:-D: diameter of the pipe (m); Q: discharge (m3 /se); V: velocity (m/sec).

2.12 Water Distribution Network Simulation

Simulation is the process of imitating the behavior of one system through the functions of another (Amdework, 2012). Simulation can be used to predict system responses under a wide range of conditions without disturbing the actual system. In our case, the term simulation refers to the process of using a mathematical representation of real system, called a model. There are two most basic types of simulation that model may perform, depending on what the modeler is trying to observe or predict. These are steady- state simulation and extended period simulation (EPA, 2005a).

2.12.1 Steady-State Simulation

It is the simplest simulation type and solves the system of equations as if the system is in equilibrium. In other words, the dynamic variables such as pipe flows, junction demands, and tank elevations kept constant. Steady-state simulations commonly used to model peak demands or a short time period.

2.12.2 Extended-Period Simulations

Demand patterns; the amount of water that consumed in the morning when everyone is getting ready for work is different at midnight. The extended-period simulation will choose for this analysis because of its capability to model varying demands. The total simulation time was 24 hours with a one-hour time-step. The other capabilities of the WaterGEMSv8i software are as follows: Evaluate the hydraulics for different demands at a single node with varying time patterns, solve for different frictional head losses using Hazen-William, Darcy Weisbach or the Chezy-Manning equation, determine fire flow capacities for hydrants, Model tanks, including those, which are not circular, Model various valve operations, Perform energy cost calculations, Model fire sprinklers, irrigation systems, leakages and pressure dependent demands at any particular node(Bhadbhade, 2004).

2.13 Urban Water Demand

(Kimey,2008) the demand for public water is made up of authorized consumption by domestic, non-domestic consumers and water losses. The water distribution networks should meet demands for potable water. If designed correctly, the network of interconnected pipes, storage tanks, pumps, and regulating valves provides adequate pressures, adequate supply, and good water quality throughout the system. If incorrectly designed, some areas may have low pressures, poor fire protection, and even health risks. Physically lost from the system (leakage). It usually expressed as per capita demand. Per-capita water usage varies widely due to the differences in climatic conditions, standard of living, population growth, type of commercial and industrial activity and water pricing. Water demand increases with time due to mainly population growth. Therefore, new water resources ought to be developed in order to meet the increasing water demand at present and in future. Twort *et.al* (2000) distribution losses comprise leaks from mains, joints, valves, hydrants and washouts, and leaks from service pipes upstream of consumers' meters or boundary stopcocks. These components of leakage rise due to high pressure developed in the distribution system of pipe network at the demand nodes. The pressure at each node or junctions of distribution system network is determined by the WaterCAD model. This model determines the end users demand profile of the distribution system network. Data needed for this model includes: type of connection, the amount of demand, duration of supply, and topographic data of the area.

2.13.1 Urban Water Demand Forecasting

Water demand should be forecasted in time and space. Many water resource projects have relatively long useful life. Therefore, in studies of water demand forecasting the plan should have extended to about 50 years for long term. In medium scale development plans, a lead-time of 15 to 25 years may apply. The projection planning should have made for at least three levels, namely normal, minimum, and maximum condition (Karamouz. Met al., 2003).

2.13.2 Spatial Allocation of Demands

Consumption or water demand is that part of the water leaving the system at customers' faucet, leaky mains or open hydrants. This demand is the driving force behind the hydraulic dynamics in the distribution system (Amdewerk, 2012). It is possible to evenly distribute the overall demand data to each node starting from the bottom/from the customers' billing records/ or from the top/the treatment plant production data/.

Although water utilities make a large number of flow measurements, such as those at customer meters for billing and at treatment plants and wells for production monitoring, data are usually not compiled on the node-by-node basis needed for modeling. The modeler is thus faced with the task of spatially aggregating data in a useful way and assigning the appropriate usage to model nodes. The most common method of allocating baseline demands is a simple unit loading method. This method involves counting the number of customers [or acres (hectares) of a given land use, number of fixture units, or number of equivalent dwelling units] that contribute to the demand at a certain node, and then multiplying that number by the unit demand [for instance, number of gallons (liters) per capita per day] for the applicable load classification. Two basic approaches exist for filling in the data gaps between water production and computed customer usage: top-down and bottom-up. Both of these methods are based on general mass-balance concepts. Top-down demand determination involves starting from the water sources (at the "top") and working down to the nodal demands (Amdework, 2012).

2.13.3 Water Demand Variations

Water demand in a distribution system fluctuates over time. For example, residential water use on a typical weekday is higher than average in the morning before people go to work, and is usually highest in the evening when residents are preparing dinner, and washing clothes. This variation in demand over time can be modeled using demand patterns. Demand patterns are multipliers that vary with time and are applied to a given base demand, most typically the average daily demand (Vasava, 2007).

Seasonal Peak: According to MOWR urban water supply design criterion (2006), towns in Ethiopia are characterized by widely varying climatic conditions and so the variations in consumption during the year, reflected by a peak seasonal factor, will similarly vary. Some consultant has adopted seasonal peak factor of 1.1. The seasonal peak factor adopted for any particular scheme shall be selected according to the particular climatic conditions and existing consumption records (if reliable and unsuppressed). It is expected that seasonal peak factors will vary between 1.0 and 1.2, representing the relatively increase in the average daily demand during the dry and/or hot season months compared with the average annual demand.

Peak Day Factor: Many communities exhibit a demand cycle that is higher in one day of the week than in others. This situation shall be taken into account by the use of a peak day factor. Some consultants have used peak day demand factors of between 1.0 and 1.3. The value adopted for the design of each individual scheme shall be selected according to judicious observance of the habits of consumers and the knowledge of the community and system operators. It is expected that any value selected for the peak day factor would not fall outside the above range (MOWR, 2006). According to (Amdework, 2012), Peaking factors can be determined by dividing the maximum daily usage rate by the average daily usage rate as below

$$P_f = \frac{Q_{max}}{Q_{ava}} \qquad 2.8$$

Where P_f = peaking factor; Q_{max} = maximum daily demand; and Q_{ava}= average daily demand Firefighting flows are usually accounted for in maximum daily flows. There are several time related demands that should be considered in the model such as seasonal demands, weekly demands population growth and industrial demands. Seasonal demands such as hot dry summers cause increase low watering.

Fire demand: It is possible to meet fire demand either directly from the distribution network with minimum allowable pressure of 50 meters or by installing fire hydrants at lower elevation nodes so that the fire trucks can fill in by the available head (OWWDSE, 2010). The fire flow in the distribution system should be within the range of 1890 L/min. and 32400 L/min.

2.14 Water Losses in Distribution System

There are two types of water losses in distribution system (R.R. Dighade et al., 2014).

Real Losses: The real losses consist of water lost through burst pipes, leaking joints, fittings, service pipes, and connections. A high level of real loss reduces the amount of precious water reaching to customers and increases the operating costs of the utility.

Apparent Losses: Result from illegal connections, under- registration of customers meters, inaccurate meters, stopped meters, vandalized meters, by passed meters, billing errors, in adequate meter reading policy, bribery and corruption of meter readers.

CHAPTER THREE

MATERIALS AND METHODS

3.1. Descriptions of the study area

3.1.1 Location

The study conducted in Durame town, which is the administrative capital of the Kembata Tembaro Zone of the Southern Nations, Nationalities and Peoples Regional State (SNNPR) of Ethiopia. The town is located 7° 12’ 50" to 7° 16’ 30" N latitude and 37º 52’ 0" to 37º 54’ 50" E longitude. As such, the town can be accessed from the regional town (Hawassa) through a 125 km all-weather road. From Addis Ababa, Durame can be reached using two alternative roads; the Alemgene – Butajira – Hossaina – Durame road with a total length of 320 km and using the Addis Ababa – Mojo – Shashamene – Durame road having a total length of 352 km and Covering a total area of the Town is 53.2 s q km.

3.1.2 Topography and land form

The topography of Durame town is mixed consisting of gently sloping area at about the center following the main road which falls to the west and east and rises again. In general, the town has rolling topography bounded by Ambericho Mountain in the north and also consists of small hills and elevated areas. The elevation of the town ranges from 1990 - 2240 m.a.s.l showing a difference of 250 m within the boundary. The topography of the town has a higher evaluated area at about the geographic center of the town and then falls down and rises to the boundaries of the town in the northeast, southwest and west.

3.1.3 Climate

According to the traditional temperature zone classification of Ethiopia, (which is based on altitude) the town is found within the ‘Woina-Dega’ Agro-ecological zone. Consequently, it experiences mean annual temperature between 14 ^{0}c and 26 ^{0}c (NUPI, 1999, and AG. *et al.*, 2010). The highest temperature is experienced between January and March and the lowest temperature record is between July and September (National Meteorological Services Agency, 2006).

The air is usually humid as a result of abundant vegetation cover and enough rainfall. Durame gets rainfall almost throughout the year. The annual rainfall is 1080 - 1350 mm (SNNPRS, 2011) and reaches up to 2000 mm. The highest rainfall is recorded between July and September and the lowest rainfall occurs between November and February which are relatively dry months of the year.

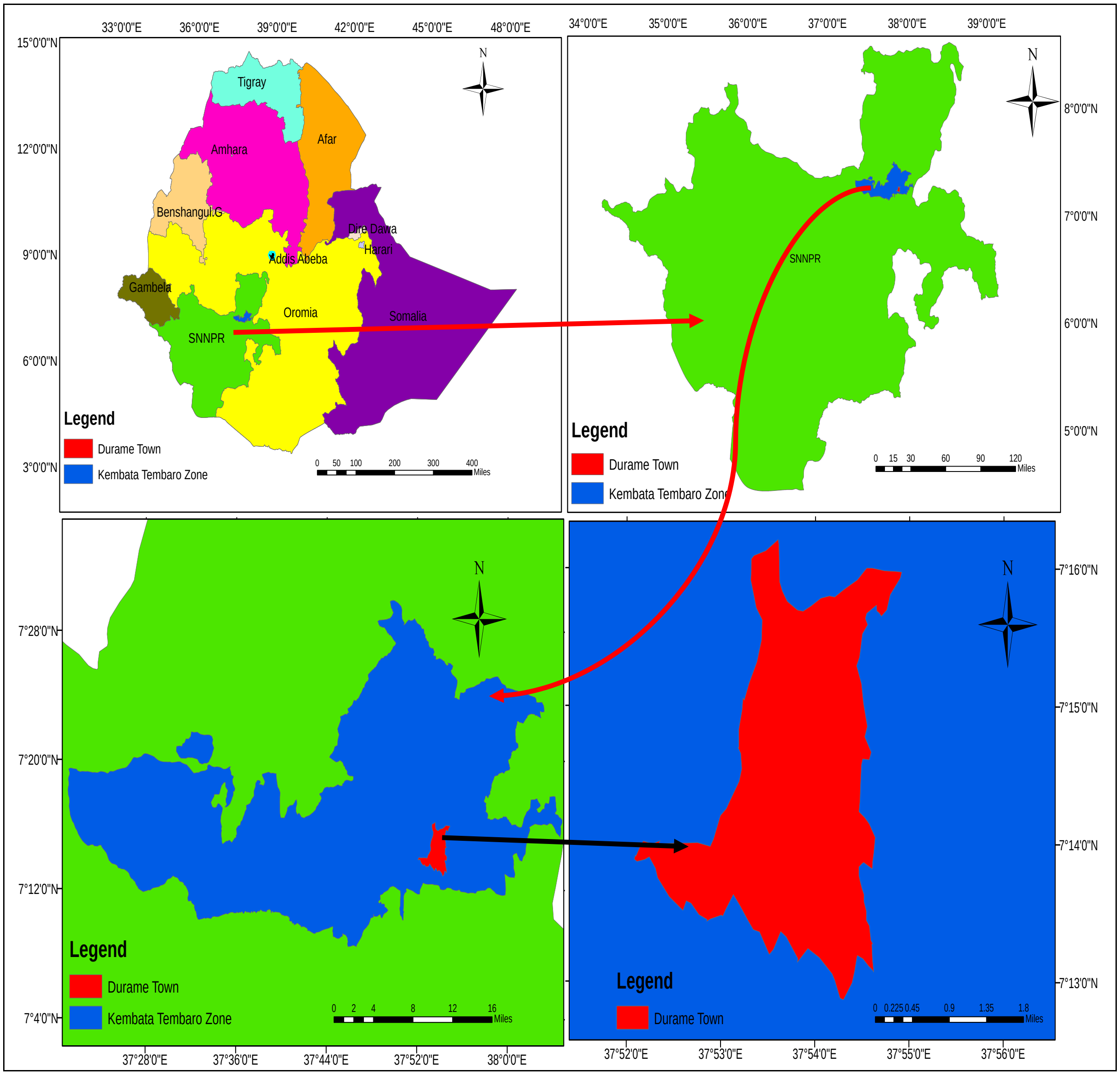

Figure 3. 1 Map of Durame town include biophysical features

3.1.4 Demographic characteristics

According to the National Population and Housing Census carried out in 2007, Durame town has the total number of population was 24,472, out of which 12,173 were males and 12,299 were females. Although, the population of the town become increasing from time to time in relation with the town development in investment, trade and due to expansion of town by including 5 rural kebeles from kedida Gamela distrikt to the town administration centre and town expansion was rural to urban migration as result of remittance send from South Africa in the town. The current population of the town is projected and approving to report of Administration of Town Finance and Economic Development, (2018), Durame has a total number of population is 82,300, out of which 41,220 (50.1 %) are males and 41,080 (49.9 %) are females.

3.2 Study Variables

3.2.1 Independent Variables

Independent variables were more related with specific objectives. Independent variables can change/affect the dependent variables. The variables which cause significant effect on the dependent variables in water distribution network were: Elevation, Family size, pipe:(diameter, length, joint condition, roughness coefficient). As the elevation decreases the pressure increases which cause high pressure zone. The diameter of a pipe can affect the model significantly. Using proper pipe diameter used to meet peak demand and fire protection while maintaining an adequate dynamic pressure in the system.

3.2.2 Dependent Variable

Dependent variables, which observed and measured to determine the effective of the independent variables, which was directly, related to the general objectives. The dependent variable includes; Hydraulic parameters (Pressure, Velocity and existing nodal demand).

3.3 Approaches and Techniques

3.3.1 Study Design

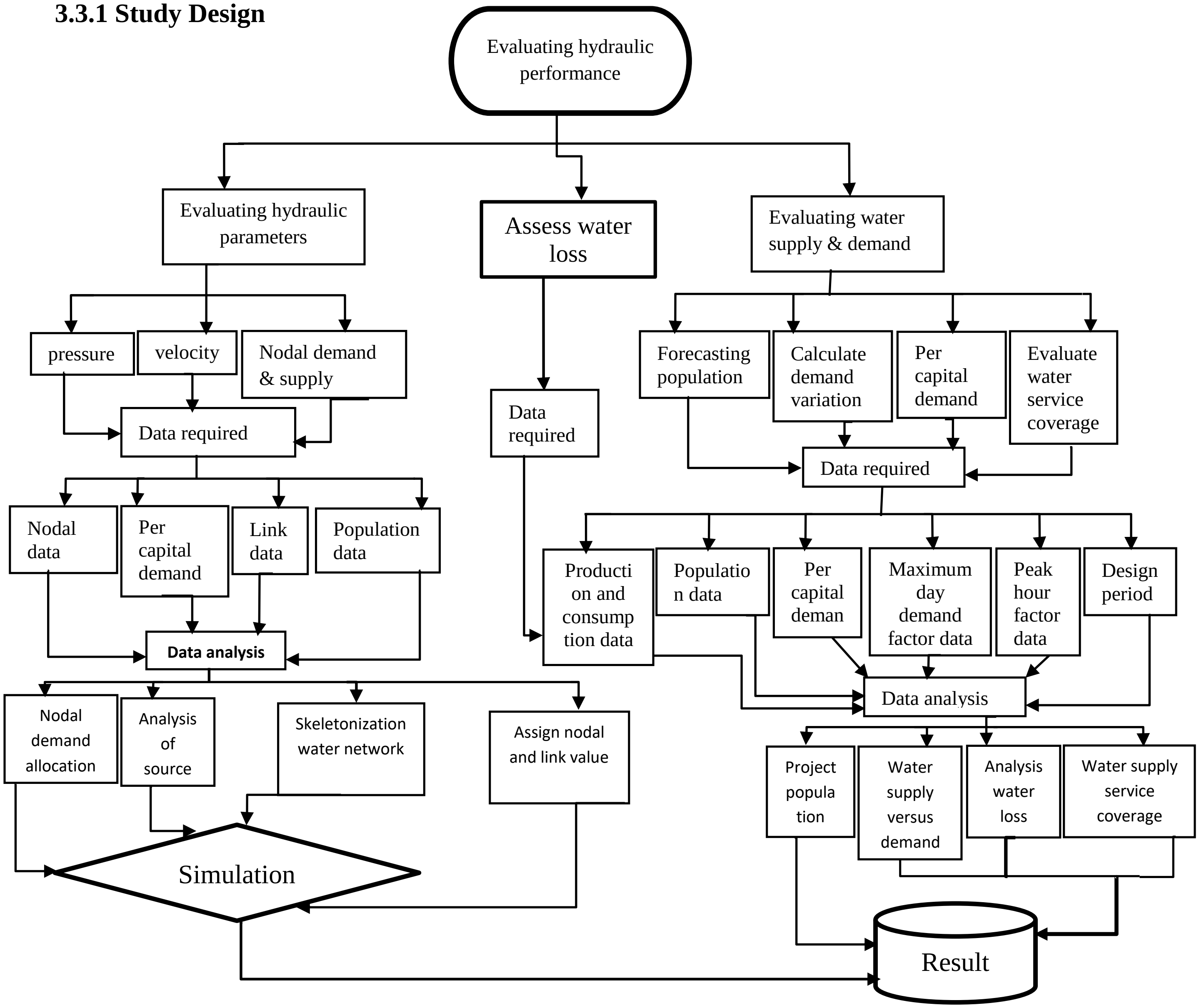

Figure 3. 2 Study design

3.3.2 Sample Size and Sampling Techniques

To get reliable data providing the necessary information required to answer the research questions of the study and for the achievement of the intended objectives of the study, both probability and purposive sampling techniques were employed in the study purposive sampling used to gather general information with the issues understudy to provide the general information required to realize the objectives of the study. And probability sampling technique was used to select sample households/respondents of the survey from the target population.

To this end to get the representative population and the necessary information accordingly, this research used Stratified random sampling techniques to select household respondents, officials and stakeholders. From the total residents in the town three kebele's such as Lalo, Zeraro anad Kasha respondents were selected using Systematic Sampling techniques and Furthermore, stratified random sampling techniques were employed to select the respondent from each three kebele's. Accordingly; 398 participants were selected using stratified random sampling techniques.

Table 3. 1 Total households and sampled respondents/households

No	Name of kebele	Total households (N)	Sampled households (n)
1	Lalo	5200	130
2	Zeraro	6500	140
3	Kasha	4800	128
Total		16500	398

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

3.3.3 Data Types and Methods of Data Collection

The most important step in any research study was data collection. In building the model of the distribution network, the data were first gathered regarding all the distribution system parameters. The collection process was performed using both primary and secondary data collection techniques to get the required information.

3.3.3.1 Sources of Primary Data Collection

Questionnaire and Household Survey: In this study, to generate first-hand and additional information from sampled households/respondents, the questionnaire with both open ended and close ended questions was distributed. The questionnaire was prepared in English language and to avoid language barriers it was translated to Amharic language. Then, the questionnaire was distributed to randomly sampled households/respondents to gather the relevant information required for the study to achieve its general and specific objectives, data concerning all relevant variables such as problems of urban water supply, parameters performance misuses/miss house hold connections and physical condition of water supply points under study, other problems with technical and institutional issues (Appendix - 4).

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

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

3.3.3.2 Secondary Data Collection

Existing available data describing the system have been gathered from different concerned organization. From Durame town water supply and service office, Municipality and South water, irrigation and energy bureau the following archived/recorded data has been collected.

- Water production and consumption,
- The existed elevation of the distribution system,
- Settlement map town and road route,

- Pipe data like material type, size and length
- Tank data,
- CSA data and other necessary data from Journals, internet and books. From list of population and housing census report (CSA, 2010), the growth rates and population for Durame town has been collected in order to forecast the future population.

The Water distribution network layout has been collected from tropical consultancy engineering.

3.3.3.3 Data Quality Control

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

3.3.4 Existing Water Supply System in Durame Town

3.3.4.1 Water Sources and Water Production with their Short Description

The water supply source of the entire town is ground water sources from three areas of ground water/deep bore-holes and spring developed source which is located in the distance of the town.

Table 3. 2 Durame Town water sources/reservoirs and their short description (DTWSS, 2018).

Item No	Name of the Sources/Local names	Direction from the town	Distance from the source (km)	Discharge (l/s)	Daily working hour	Daily water production (m^3/day)
1	Weta BH-1	North-East	4.1	8.5	19	582
2	Weta BH-2	North-East	3.9	7	19	479
3	Benara BH-1	North - West	3.1	7.5	19	513
4	Benara BH-2	North - West	3	6	19	410
5	WV BH	North-East	4	4.5	19	308
6	Gocho BH	North - West	2	4	19	274
7	Ambaricho Spring	North - West	8.7	1.5	24	130
8	Total Sum			39		2,669

As shown in the Table 3.2 the total daily water production of the wells was calculated by multiplying the discharging capacity of wells by pump operating hours (19) hours and Spring within 24 hours a day which indicates total daily production was 2,669 m^3/day.

3.3.4.2 Existing Distribution Network

The existing water distribution system of the town is both pump and gravity system. The water from the four sources is taken to relief/break tank by pump and the water from the relief tank as the remaining sources, taken to storage reservoir by booster pump station excluding spring source. Then, the stored water is distributed to the town by the gravity (uPVC, HDEP and DCI). The water distribution network of the town consists of about 59.93 km. The existing distribution system consists of a variety of pipe types: ductile iron (DCI), uPVC (u-polyvinyl chloride) and HDEP (high-density polyethylene). The town supplied by water intermittently by water Staff who is managing the system mainly by using controlling valves in order to supply all customers at least twice a week.

3.3.4.3 Distribution Network Pipes

Pipes are the essential elements of a water distribution system. All the elements of distribution system, such as junction (nodes), pumps, reservoirs, valves and tanks are linked to each other by pipes (Melaku, 2015).

Table 3. 3 Pipe diameter and corresponding length used as software inputs (DTWSSO, 2018).

Diameter (mm)	Pipe Length (m) and Material type			Total Length (m)
	DCI	HDPE	uPVC	
50	4145			4,145
63		11115		11,115
75	15905			15,905
90		5245		5,245
100	4270			4,270
110		3292		3,292
125	8325	3405	860	12,590
150	345	1150	1450	2,945
200		420		420
Total	32,990	24,627	2,310	59,927

To deals performances of hydraulic parameters for all pipe individual up to individual house hold is difficult and time consumed to represent by software. So, skeletonization was needed. Skeletonization is the process of selecting for inclusion in the model for enabling quicker calculation (Walski, Thomas M. 2007). Using skeletonization pipes having diameter greater or equal to 50 mm were selected for modeling the distribution system. As indicated in Table 3.3, the total length of pipes in the water distribution network was 59,927 m.

3.3.4.4 Storage Tank

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

Table 3. 4 Location and capacity of existing storage tanks (DTWSSO, 2018)

Tank label	Elevation (m)	X (m)	Y (m)	Capacity (m^3)
Tank-1	2,174.14	380146	803011	500
Tank-2	2,174.05	380239	802482	150
Tank-3	2,170.17	380605	803028	200
Tank-4	2,097.48	378054	798027	500
Tank-5	2,168.30	377873	800465	50
Total				1,400

Table 3. 5 Summary of water distribution network elements

System components	Number of element represented
Junctions	85
Pipes	127
Tanks	5
Pumps	8
Reservoirs/Sources	7
Flow control valves	3

The system elements are organized for the purpose of modeling the distribution network. As shown in Table 3.5 pipe networks connects the junctions, tanks, reservoirs, regulating valves and pumps.

3.4 Materials

The study was focussed on hydraulic performance evaluation of existing water supply in distribution network in Durame town. To achieved the goal of the study the materials that were used are computer, Bentley WaterGEMSv8i, Endnote, Arc GIS Version 10.1 and GPS Garmin72.

GPS Garmin 72 was used to take coordinates and elevation data for cross-checking with the coordinates of the boundary nodes obtained from the AutoCAD source and elevations generated by TRex. It was also used to take coordinates of some components of the distribution system which have no spatial data on the AutoCAD network. Also It has an ability to locate the latitude and longitude of the system on the ground by receiving satellite information.

Bentley WaterGEMSv8i was used to model the behavior of water distribution systems. Bentley WaterGEMSv8i has a capability of modeling water distribution behavior at steady-state and time varying situation. To design and evaluate hydraulic performance of the existing water distribution system, a model was developed using Bentley WaterGEMSv8i. Bentley WaterGEMSv8i is selected for this study because of the following reason: - Graphical user interferences and latest as camper to Epanet and WaterCAD 6.5 software, integration with external software, like Auto CAD and Microsoft excel and requires less effort and shorter time to build a model than others and It is aided with good quality of manual. WaterGEMSv8i is a powerful, easy-to-use program that helps hydraulic engineers design and analyzes water distribution systems. It provides spontaneous access to the tools you need to model complex hydraulic parameters performance. It can be used for many different kinds of applications in distribution system analysis. In this study, it was used to carry out the hydraulic parameters performances analysis of the distribution networks in the Durame town. The data sorting and color coding capability of WaterGEMSv8i software was used to identify very small and very large values and to focus on these values when testing and calibrating the hydraulic model. The other advantages of WaterGEMSv8i supported several methods of exchanging data with external applications, preventing duplication of effort and allowing us to save time by re using data already present in other locations.

Arc GIS version 10.1 was used to delineate the study area. Endnote program used to place citation and references in the document.

3.5 Data Analysis

The general steps of the data analysis were first developing distribution network based on collected data by selecting junctions, pipe links and pump links, second enter input data by using data entry dialog box, third run the model and finally compare the result in the real situation and already designed results of the consultant. The water distribution system layout was prepared using the collected data obtained from office and field survey data like: base demand, elevation, directions of Northing and Easting of Junction, water source, tank position and pipe data using the computer software application origin8 and excel was used to analyze for distribution system evaluated by using the engineering software called WaterGEMSv8i.The data analysis was simulated by developing water distribution network scenarios to evaluate hydraulic parameters performance of pressure, velocity and water flow rate of water supply distribution system of the town.

3.5.1 Network Simulation

WaterGEMSv8i is capable of performing two types of simulations, steady-state and extended period simulation. Analysis of the model of existing systems has been done by running the model at current year daily average at peaking and temporal variation of demand with different scenarios.

3.5.1.1 Steady-State Simulation

It is the simplest simulation type and solves the system of equations as if the system Junction demands and tank elevations kept constant.

3.5.1.2 Extended-Period Simulations

Demand Patterns: - the amount of water that consumed in the morning when everyone is getting ready for work is different at midnight. The extended-period simulation was choosing for this analysis because of its capability to model varying demands. The total simulation time was every two-hour time setup in the twenty- four-hours. Analysis at peak and minimum time consumption was simulated to identify the current problems of the system.

3.5.2 Hydraulic Parameters

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

3.5.2.1 Pressure

The adequacy of a system, the first parameter to check is the predicted pressure. There are generally three design pressures that are defined for each community: maximum pressure, minimum pressure during peak hour, and minimum pressure during a fire flow. The pressure at nodes depends on the adopted minimum and maximum pressures within the network, topographic circumstances, and the size of the network. The minimum pressure should have maintained to ensure that consumers' demand provided at all times. The maximum pressure also contains limitation of leakage and lead to water losses in distribution system. The operating pressure in the distribution network is given in Table 3.6.

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

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

3.5.2.2 Flow

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

$$v = \frac{4Q}{\pi D^2} \tag{3.1}$$

$$D = \sqrt{\frac{4Q}{V\pi}} \tag{3.2}$$

Where, D = diameter of the pipe (m); Q = discharge (m^3/s); and V = velocity (m/s), maximum velocities in distribution system 2 m/s and minimum velocity 0.6 m/s.

3.6. Model Building and Data Entry in the Existing Distribution Network

3.6.1 Importing the Network

To import the AutoCAD drawing in to the WaterGEMSv8i software, the AutoCAD drawing was first converted to .dxf file allowance after correcting all types of errors on the AutoCAD drawing. Then the dxf file network was imported in to Bentley WaterGEMSv8i software using the shape file link wizard. The modeling was performed using the following steps:

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

3.6.2 Data Entering and Data Proofing

The distinguishing and correcting data errors related to network data, demand data and operational data, which occur during data gathering process, data preparation and data analysis processes were undertaken. The input data should be entered into the software using different techniques these were model builder from dxf. File to software, use the properties editor for each element by individually opening the properties editor or used flex table for similar element data used by model builder so that the total input data for the analysis of distribution system included: - Nodes (Elevations and base demand), Pipes (Pipe diameters, lengths, material type), Tanks(Base , minimum and maximum elevation and diameter of the tank), Pumps (The most important parameter defining the pump operation is the pump curve, Other input needed is the elevation of the pump), Reservoir (Elevation) and Hazen -Williams pipe coefficient values, and other necessary values used by flex table.

Throughout the process, International System Unit (SI) has been used. To request the use of these units in WaterGEMSv8i, the user chooses SI flow unit under the hydraulics option. In this study, it was selected liters per second for flow in this model, which also defines all other units using the SI system. Hence, lengths, pressure, head, elevations are taken in meters, and diameters of pipes are defined as millimetres.

3.6.3 Model Representation

The town water supply distribution network performance was evaluated using application model of WaterGEMSv8i.The analysis of the data which were simulated by models were hydraulic design of pressure line from six borehole sources and one spring source to the service reservoir and reservoir outlet to pipe line network of the town. System distribution networks were drawn as a combination of various system components. The model was commonly, in water distribution system represented by system elements, such as reservoir, tank, pipe, node, pump and valves.

Reservoir: Reservoir is a type of storage node and that represent an infinite external source or sink of water to the network. A storage node is a special type of node where a free water surface exists, and the hydraulic head is the elevation of the water surface above sea level. The water surface elevation of a reservoir does not change as water flows into or out of it during an extended period simulation.

Storage Tank: Storage tank is a structure used to store water and provide water to the system when needed. Storage tank is crucial to continuously supply during a pump turned off and equalize water during peak demand hours.

Pipe: Every pipe is connected to two nodes at its ends. In a pipe network system, pipes are the channels used to convey water from one location to another. The physical characteristics of a pipe include the length, inside diameter, roughness coefficient, and miner loss coefficient. The pipe roughness coefficient was associated with the pipe material and age. The miner loss coefficient is due to the fitting along the pipe. Pipe length and diameters are then inputted as well as roughness. The Hazen-William formulae was selected and its roughness coefficient (C-value) of 130 was selected for DCI pipes,150 for the PVC pipes and 120 for steel and uPVC pipes. The formula was selected and used for this study Hazen-Williams formulas (OWWDSE, 2010).

$$H_f = \frac{10{\cdot}7\ L\ Q^{1{\cdot}852}}{C^{1{\cdot}852}\ D^{4{\cdot}87}} \qquad 3.3$$

Where, Hf = Head friction; Q = discharge (m3/s); L = Length of the pipe (m); D = Diameter (mm) and C = Roughness coefficient which varies for different pipe materials and age. The pipe roughness coefficient refers to a value that defines the roughness of the interior of a pipe. Two common roughness coefficients are the Hazen-Williams C-value and the Darcy Weisbach f-value. Although the Darcy-Weisbach term is generally considered more accurate and flexible by giving information about flow regime, it is also more complicated and difficult to determine. Therefore, the Hazen-Williams C-value is commonly used in network modelling as in this study due to highly usage materials type in the study area water supply distribution contains many DCI & HDPE pipe types.

Table 3. 7 Hazen-William roughness coefficients for pipe material (Chase *et al.*, 2003)

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

Nodes: Nodes are the locations where pipe connected and they should have their elevation specified above sea level. Nodes, besides representing the connection point between pipes, can represent the following components in a network:

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

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

3.6.4 Hydraulic Calculation

Bentley WaterGEMSv8i program solves for the distribution of flows and hydraulic grades using the energy equations. The quantities can be used to express the head loss or head gain between two locations. The conservation of energy principles states that the head loss through the system must balance at each point. Head loss between any two nodes must be sign consistent with the assumed flow direction. Any internationally recognized formula may be used in the hydraulic computations. The study was conducted using Hazen-Williams equation as it is commonly used in the design and analysis of pressurized pipe systems.

Nodal Demand Calculation

Bentley WaterGEMSv8i enables to allocate demand to the model of water distribution system. Load Builder greatly facilitates the tasks of demand allocation and projection. In the study the nodal demand was allocated using unit line demand allocation method. Demand for each node was calculated and analyzed based on the number of population point load data selection nearest node based on available load builder methods for each consumption node, and the period of supplying water to calculate the peak factor of demand for each node. This yields the demand of a certain area, which has to be converting into demand at a point (pipe junctions). Node usually had one of the two main functions; it receives a supply for the system or it delivers the demand required by consumers. Population around the node was identify, and the people served by the node were multiplied by per capita demand. Nodal demand is calculated using the following formulae:

$$N_d = \sum p_i \, d_j \qquad 3.4$$

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

3.6.5 Water Distribution System Network

A water distribution system is a pipe network that delivers water from single or multiple supply sources to consumers. There is a general belief arising out of carelessness on behalf of service providers, that water supply networks can be expanded indefinitely. Many water supply providers, in a drive to provide wide water supply coverage increase the number of customer connection through a massive network expansion. Because of rapid population growth and existence of unacceptable pressure and velocity in the distribution network water demand exceeds available production capacity. The flow and pressure distributions across a network are affected by the arrangement and sizes of the pipes and the distribution of the demand flows. Since a change of diameter in one pipe length will affect the flow and pressure distribution everywhere, network simulation is not an explicit process. Pipe network analysis involves the determination of the pipe flows and pressure heads that satisfy the continuity and energy conservation equations.

3.6.5.1 Pressure in Distribution System

According to Swamee and Sharma (2008), nodal pressure is stated as the minimum design pressure to discharge design flows on to the systems. It is based on the population served, types of dwellings in the area, and firefighting requirements. The pressure at node depends on the adopted minimum and maximum pressure within the network, topographic circumstances, and the size of the network. The static pressure in the distribution piping system is the pressure head with no water flowing in the network is equal to the height to which the column of liquid could be raise.

The general consideration is the water should reach up to the upper stories of low-rise buildings in sufficient quantity and pressure, considering firefighting requirements. In case of high- rise buildings, booster pumps are installed in the water supply system to water for pressure head requirements. The MoWR water supply design criteria (2006) recommended the pressure range in distribution system to be 15 - 60 m water head. However, there is no defined maximum and minimum pressure ranges set by the office, regarding to this literature based recommendation for optimum operating pressure was used to asses system hydraulic performances.

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

3.6.5.2 Pipe Velocity in Water Supply Network

Different design guide line has been developed by different researchers for the standard velocity in pipe flows. They recommended optimum velocities for pipe flow in transfer and distribution mains are presented in Table 3.8.

Table 3. 8 Pipe velocity range from various sources

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

3.6.5.3 Model Testing

Before going to the time consuming and tedious model calibration process, model testing was undertaken using the standard input data to check if the model perform correctly without any mathematical instabilities. Significant differences between the run results and expected system performance were investigated before going to further works. During the model testing process, there were many errors associated with the network topology and the allocated demands. Therefore, some further manual adjustments on physical and hydraulic parameters were made to make the model simulated parameter values reasonable. The demands of some nodes was transferred to another nodes and some slight adjustments on pump operating conditions were made to eliminate model error messages and to go for the next process. During the model testing, the model showed large values of pressure and flows in areas where there is no flow in the real situation and vice-versa. This condition suggested to undertake the calibration process.

3.7. Water supply Coverage Analysis of Durame Town

The coverage of water supply for the town has been evaluated based on the average per capita consumption and by mode of service. Water demand is the daily water requirement for use by human being for different domestic purposes. The annual total volume of water consumed for domestic purpose has been converted to average daily per capita consumption using the total number of population (OWWDSE, 2010). The annual consumption data has been converted to average daily per capita consumption using the number of population. The average daily per capita consumption (liter/person/day) was derived using the following terminologies (Desalegn, 2005).

$$Per\ capita\ consumption\ (l/p/d = \frac{Annual\ Consumption\ (m^3)\ x\ 1000L/m^3}{Population\ number\ of\ the\ Town\ x\ 365} \qquad 3.5$$

Data on individual domestic water consumptions, total water consumption (m^3) and total production (m^3) were collected from Durame water supply and service office bill documents for analyzing average per capita consumption.

3.7.1 Water Loss (unaccounted for Water) Analysis

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

$$Total\ water\ loss\ (\%\ ) = \frac{(Total\ water\ produced - Total\ water\ billed)(m^3)x\ 100}{Total\ water\ produced\ (m^3)} \qquad 3.6$$

3.8 Present and Future Water Demand Forecasting

3.8.1 Population Forecasting

Urbanization and population growth follow a very complex process and affected by a range of economic, political, social, cultural, and environmental factors. The design of the water supply project was done based on projected population and it was the main factors that affects the water supply project. Future population growth can be influenced by affecting birth, death, or migration rates due to social, economic, political, technological, and scientific developments. The geometric increase method is mostly applicable for developing countries, rapid growing towns and cities having vast scope of expansion. It is based on the assumption that the percentage increase remains constant (Lee and Tuljapurkar, 1994). The future population of Durame town was projected using geometric increase method.

According to Kharagpur web courses in geometric mean increase method the percentage increase in population from decade to decade is assumed to remain and is used to find out the future increment in population. Since this method gives higher values and hence should be applied for a new industrial town at the beginning of development progress for only few decades. The population at the end of n^{th} decade P_n can be estimated as:

$$p_n = p_o\left(1 + \frac{r}{100}\right)^n \qquad 3.7$$

Where P_n = population at n decades or year; P_o = Base year population; n = decade or year; r = percentage (geometric) increase.

The current population of the town is projected and approving to report of Administration of Town Finance and Economic Development, (2018), Durame has a total number of population is 82,300, which indicate base year population.

Table 3. 9 Population Projection Based on Growth rate (%) of Urban Population of (CSA, 2010)

Year	2019 - 2022	2023 – 2027	2028 – 2032	2033 – 2037	2038
Growth rate (%)	4.8	4.58	4.05	3.65	3.25

3.8.2 Water Demand Variations

Water demand in a distribution system fluctuates over time. For example, residential water use on a typical weekday is higher than average in the morning before people go to work, and is usually highest in the evening when residents are preparing dinner, and washing clothes. This variation in demand over time can be modeled using demand patterns. Demand patterns are multipliers that vary with time and are applied to a given base demand, most typically the average daily demand (Vasava, 2007).

Seasonal Peak

Towns in Ethiopia are characterized by widely varying climatic conditions and so the variations in consumption during the year, reflected by a peak seasonal factor, will similarly vary. Some consultants have adopted a seasonal peak factor of 1.1. The seasonal peak factor adopted for any particular scheme shall be selected according to the particular climatic conditions and existing consumption records (if reliable and unsuppressed). It is expected that seasonal peak factors will vary between 1.0 and 1.2, representing the relative increase in the average daily demand during the dry and/or hot season months compared with the average annual demand. Durame town seasonal peak factor taken as 1.1 by which average day demand was adjusted.

Peak Day Factor

Many communities exhibit a demand cycle that is higher in one day of the week than in others. This situation shall be taken into account by the use of a peak day factor. Some consultants have used peak day demand factors of between 1.0 and 1.3. The value adopted for the design of each individual scheme shall be selected according to judicious observance of the habits of consumers and the knowledge of the community and system operators. It is expected that any value selected for the peak day factor would not fall outside the above range. Durame town peak day factor taken as 1.2 and maximum day demand was adjusted.

Peak Hour Factor

Water demand varies greatly during the day. The distribution system must be designed to scope with the peak demand, which is taken into account by the use of a peak hour factor. This peak hour factor is expressed as a multiple of the annual average daily demand and applied additionally to the seasonal and peak day factors. The peak hour factor varies inversely with the size of the consumer base. The peak hour demand was adjusted by 1.8. with population range 50001 to 100000, based on the study area number of populations.

Table 3. 10 The peak hour range based on number of populations (MOWR, 2006).

Population Range	Peak hour factor
< 20000	2
20001 to 50000	1.9
50001 to 100000	1.8
X > 100000	1.6

3.8.3 Present and Future Water Demand Forecasting

For this study, water demand was classified in to two major categories as domestic and nondomestic water demand. Domestic water demand is water that is required for cooking, toilet flushing, bathing, drinking, and washing of face, clothes and utensils, etc. Non-domestic water demand includes Industrial demand, institutional demand, firefighting demand, water lost and waste, and public demand.

3.8.3.1 Domestic Water Demand Forecasting

The design of water supply project was necessary to estimate the amount of water that was required to satisfy and serve up to the end of the design period. It is the portion of that municipal water supply, which is used in home and largest portion of total demand for most water system (DOH, 2009). It includes toilet flash, cooking, drinking, washing, bathing, and other uses. There are four modes of services identified for domestic water consumers of Durame town. These are house connection was considered water supply from House connection (HC), yard own connection (YC), yard share connection (YCS), Public fountain (PF), Unserved (US) mode of service connections, (MOWR, 2006). The per capita water demand for various demand categories of the study area was adopted by taking into account the different development factors and standards used by the Ministry of Water Resources, Irrigation and Electricity (MoIE, 2015). In projecting the domestic water demand of the town, the following procedures were used.

Population Percentage Distribution by Mode of Service

Although the standard approach for formulating the percentage of population served by different modes would normally involve a detail analysis of past consumption trends based on office expert household survey, the base year (2018) percentage of population by mode of service was adopted from Durame town records and documents. The distribution of population for each mode of service was determined by considering socio-economic situation and living standard of the town. After establishing the population, distribution for the base year a forecast was made.

Establishment of per-capita Water Demand (l/c/d) For Each Mode of Service

Domestic per capita water demand was estimated taking into consideration analyzed actual and desired expressed demand during the survey; expected increase in per capita consumption and population of users with time and domestic water consumption variation based on the mode of service connections and based on liter per capita per day consumption of service connection taken from (MoWR, 2006). Domestic water consumption varies according to the mode of services, climatic conditions, socio-economic condition and other related factors. Subsequently reviewing previous design criteria in the country, arrived to the following per capita water consumption.

Table 3. 11 The Domestic daily demands of different consumption from (MoWR, 2006)

Mode of service	Per capita water demand (l/c/d)	
	Phase I	Phase II
House connection (HC)	50 l/c/day	70l/c/day
Yard connection own (YCO)	25l/c/day	40l/c/day
Yard connection shared (YCS)	30l/c/day	30l/c/day
Public tap users (PTU)	20l/c/day	25l/c/day

These values were given for the year of 2006; to convert into base year that is 2018, the annual rate of projection of 2% for public tape users, while for house connection and yard connection 3% was adopted by considering the living standard and socio-economic activities of the town.

Projection of Consumption by Mode of Service

Distributions of mode of service were established based on available data. The forecast envisages decrease in the public tap and neighbourhood users. The assumption was that more people would have yard connection. Besides, the number of house connection would increase in certain amount. Due to this, a significant increase of yard connections was estimated.

Adjustment for Climate and Socio-Economic Activity

In order to account change in climate, which affects water demand of a given area, the value of average per capita domestic demand was factored for climatic changes using the climatic factor. The demand adjustment factor due to climatic effect is given in Table 3.12.

Table 3. 12 Demand adjustment factor due to climatic effects (OWWDSE, 2010)

Group	Mean annual precipitation (mm)	Factor
A	900 or less	1.1
B	900 – 1200	1.0
C	1200 or more	0.9

According to the National Meteorological Service Agency, the mean annual rainfall of Durame town is 1080 mm, the correction factor was taken as 1.0. The domestic water demand also depends on the socio-economic situation of the area. Thus, per capita domestic water demand was modified using appropriate factor.

Table 3. 13 Demand adjustment factor for socio-economic situation (OWWDSE, 2010)

Group	Description	Factor
A	Town enjoying high living standard and high potential for development	1.1
B	Town with high potential for development but lower living standards at present	1.05
C	Town under normal Ethiopian conditions	1.0
D	Advanced rural towns	0.9

Due to the different socio-economic condition of Durame town can be categorized under group C. Consequently, a socio-economic factor of 1.0 was used.

Projection of Domestic Water Demand

Estimation of water demand per mode of service and estimation of population by mode of service was used to calculate the average per capita water demand. The average per capita domestic water demand for each year was computed by combining water demand by mode of service and population percentage distribution by mode of service for the year 2018 to 2038.

3.8.4 Non Domestic Water Demand

Non-domestic water demand was the quantity of water required for Government, and non - government office, Schools, Hospitals, TVTE College. To compute such water demands, the actual figure of public service, Institutions and commercial centers consumption data were interpolated from ministry of water, Irrigation and Electricity design manual. The recommended value of non-domestic demand by cost effective design guideline for urban water supply presented by MoWR (2006), was 20 to 40% of the domestic water demand. These water required for institutional, commercial and public purposes site are called non-domestic water demands.

Institutional and Commercial Demand

This refers to the water demand of facilities such as schools, hospitals, hotels, etc. and small commercial enterprises, and also public demand where appropriate. The review will assess the extent and development of the institutional and commercial base in each town and vary the likely daily demand, if necessary, based on the following consumptions.

Table 3. 14 The non-domestic day demands of different consumption from (MOWR, 2006).

Design Criteria	Unit
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 l/worshipper
Cinema house	4 l/seat
Abattoir	150 l/cow
Hospitals	50-75 l/bed
Hotels	25-50 l/bed
Public Bath	30 l/visitor
Railway & Bus station	5 l/user
Military Camps	60 l/person
Public Bath (with water facility connection)	20 liters/seat

Considering the non-domestic capacity of Durame town, the daily demands non domestic of different consumptions was take from (MoWR, 2006) design manual.

Non-Revenue (Unaccounted-for) Water (NRW)

The non-revenue is divided into physical, and non-physical water losses. Physical water losses are defined as “that amount of water which is lost without being used due to failures and deficiencies in the distribution facilities.”. Non-physical water loss is defined as “the amount of water which is not registered, due to incorrect reading of the measuring instruments installed (measurement errors) and/or absent or inaccurate estimates in the absence of measuring instruments (estimation errors) (Walski, and Savic, A., 2003).

In more Vocabulary, it is defined as the difference between the volume of water supplied and the volume of water billed/revenue expressed as a percentage of net water supplied. NRW represents water that has been produced and is ‘lost’ before it reaches the customer (either through leaks, theft or through legal usage for which no payment is made). This indicator captures not only physical losses but also commercial losses due to inefficient billing or illegal connections, system leakage, inaccuracies in metering, overflowing of reservoirs, and legitimate unmetered use such as firefighting, flushing, may indicate poor system management and poor commercial practices as well as inadequate network maintenance. NRW cannot be assessed easily without adequate and reliable metering, but sometimes take as losses 15 to 30% (MoWIE, 2015).

Fire Fighting

Water demand for firefighting purposes shall be assessed on a town-by-town basis, depending on the existence of equipment and the capacity of any firefighting service. According to (AWWA, 2005), minimum needed fire flow demand should not be less than 32 l/s and the maximum needed fire flow should not exceed 757 l/s.

Fire hydrants should be installed at public and municipality interest such as Schools, Shops, Hospitals, Fuel stations and at salient points of distribution network (MoWR, 2006), recommends enlarging total reservoir volume by 10% to the reserve water for firefighting. The amount of water required for firefighting was usually taken as small, the trend of the town frequency of the fire breakout was rare and from economical point of view, installing fire hydrant at lower elevated nodes of the distribution system, so that the fire brigade trucks could fill in by the available head was recommended.

3.9 History and Description about Durame Town Water Supply Distribution System

The current water supply source of the town consists of six Groundwater deep borehole and one developed spring locally named as Ambaricho spring with average estimated yield of 39 lit/sec. 4 concrete tanks with total volume of 1350 m^3 and one old masonry trapezoidal Tank with capacity of 50 m^3 which is used for storing and balancing water supply distribution system in Durame town. They located in different sites: The three reservoirs are located near KMG (Kembata Women Self Help), covers most part of the town. The oldest one reservoir is located near to the Hiddase high school and the last one is located in locally named as Danshe site. Also there are two booster reservoirs located on four borehole pressure line. The total number of customers in 2018 was described in the demand projection section. The existing sources which are Ambaricho spring developed and two existing boreholes source (Gocho BH and World Vision sources) are constructed in 1983 and the systems was updated new construction with additional four deep boreholes in 2011. north-west and north-east direction of the town.

Even though, Water demand in Durame town was high and shown severe shortage of water in the town. During the study time, Durame was used intermitted water supply system. Because of, in Durame town the quantity of water that wells produced not enough to meet the needs of consumers and system flushing, and other needs. The problem arises due to limited source capacity, high population growth in town, poor operation and maintenance, inequity of water in distribution due to the topography. This also high pressure on the existing infrastructure, which usually results in infrastructural decay, there by interrupted the efficient of water distribution system. Moreover, another problem of water supply in Durame town was associated with unpredictable/erratic power supply that humped continued operation of the water supply system. In Durame following rapidly development of the town construction field such as buildings, expansion of road including five rural kebele's and population number increase rapidly. This also highly challenge and make stressed on water supply system. In other hand Durame rapidly grown commercial town and has good prospect for development. To support the progress a potable, reliable and adequate water supply system must be established.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Model Results of WaterGEMSv8i

Model in the water supply distribution network consists, input data of the distribution system are elevations of node, base demand of the node, locations junctions/nodes, pipe length & its diameter and section and elevations of service reservoir. WaterGEMSv8i could show pressure, demand, and hydraulic grade in different nodes as well as flows, velocities, head-loss gradient and head-loss in different pipes throughout the distribution system. The results of model were generally displayed in tabular and graphic forms by the different scenarios.

4.1.1 Model Representation

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

Table 4. 1 Pipe diameter and corresponding lengths with their coverage (DTWSSO, 2018).

Diameter (mm)	Material type			Total Length (m)	Coverage (%)
	DCI	HDPE	uPVC		
50	4145			4,145	7
63		11115		11,115	18.5
75	15905			15,905	26.5
90		5245		5,245	8.7
100	4270			4,270	7.1
110		3292		3,292	5.5
125	8325	3405	860	12,590	21
150	345	1150	1450	2,945	5
200		420		420	0.7
Total	32,990	24,627	2,310	59,927	
Coverage (%)	55	41.1	3.9		

As described in Table 4.1 the total length of pipes represented in the model materials were DCI covers 55%, HDPE covers 41.1% and uPVC covers 3.9%. In the model different diameter pipes are represented to contribute their function to water distribution network. In the model 63 mm, 75 mm and 125 mm pipes are used in high percentage compared to other diameter represented in the model.

4.2 Simulation

4.2.1 Steady State Simulation

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

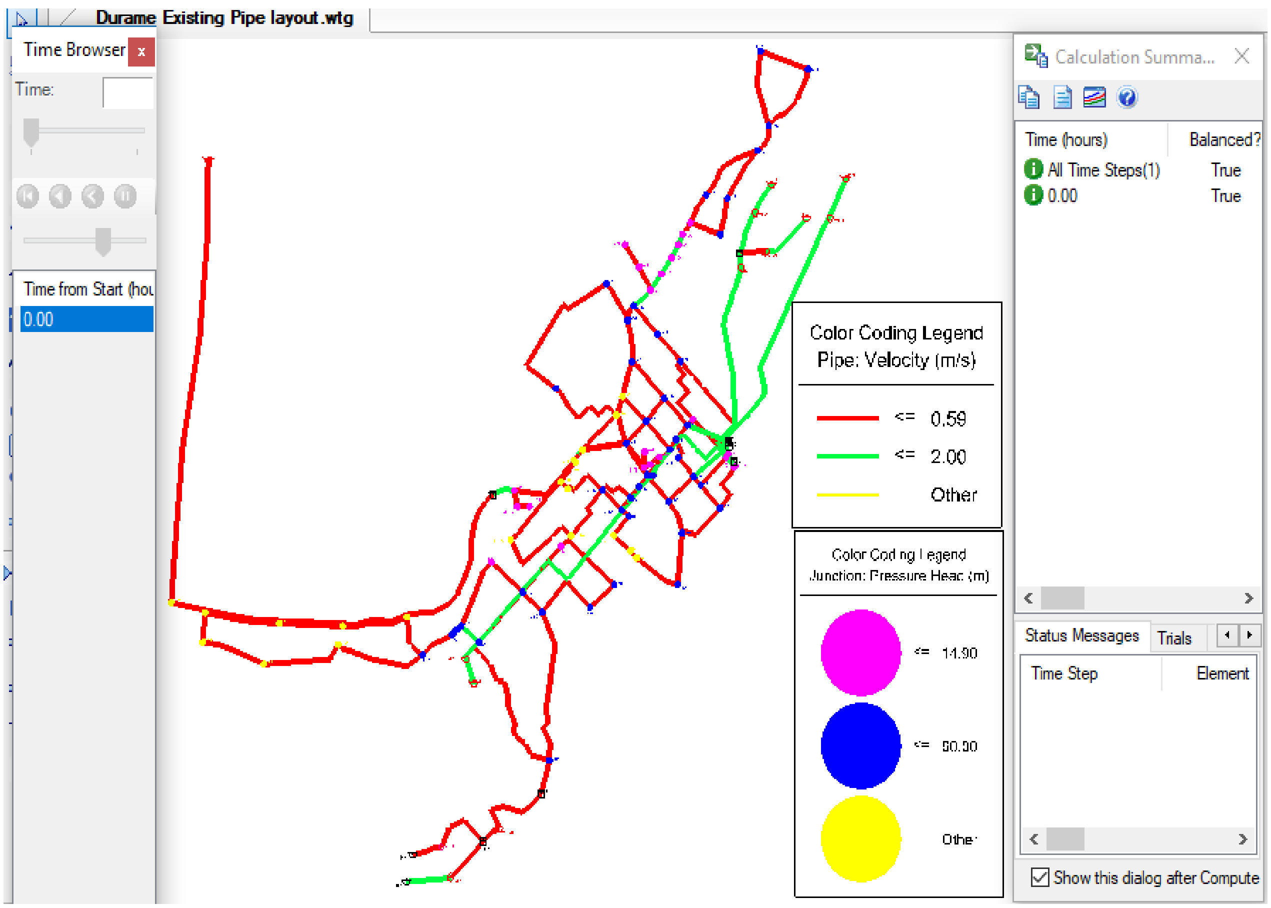

Figure 4. 1 Steady State Simulated pressures and velocities

4.2.2 Extended-Period Simulation

The system conditions have been computed over 24 hours with a specified time increment of two hours and starting model run at time 12:00 PM. The software simulates dynamic state hydraulic calculation based on mass and energy conservation principle. The model has been simulated for every two-hour time setup in the twenty- four-hour duration. However, for the analysis the peak and minimum hours' demand has been simulated to identify the current performance of the system related to system parameter like pressure and velocity. The model has been performed 12:00 AM to 3: 00 AM for minimum hour consumption and 6:00 AM to 8:00 AM for the peak hour consumption. It is noted that minimum hour model run has been made at 0:00 hour from starting time and peak hour model has been made at 7:00 AM from the starting to 8:00 AM. The dynamic simulated results have been presented in Appendix (2).

4.3 Hydraulic Parameters Performance in Water Supply Distribution Network

4.3.1 Pressure

Pressure in water distribution systems has to be maintained optimum; as to efficiently make water available to each demand category including at instances of firefighting (high withdrawal period) and as to reduce leakage as well as pipe breakage across the system. Swamee and Sharma (2008) described that the minimum design nodal pressure is the pressure assigned to discharge flow on to the system. At minimum peaks through night hours the pressure in the system becomes high and the leakage loses expected to increase whereas at high peaks the pressure becomes small and the leakage losses expected to decrease When the pressure exceeds the elevation of the storage tank, water can start to fill the storage tank. The higher the pressure is more water start to enter to the tank. In this study, the model run from the input of existing data a total node of 85 was reported from the project inventory dialog box. Based on Table 3.6 the results show that 42 nodes from a total 85 nodes have been observed out of the recommended serviceable pressure (15 mH_2O to 60 mH_2O).

The steady state analysis describes the behavior of the system at a specific point in time with flow rate and hydraulic grade remains constant overtime. Extended period simulation indicates the performance of the distribution system better than steady state simulation during high consumption or at stress condition. The simulated result for extended period simulation at 31(thirty-one) junctions showed that changes from positive pressure to negative pressure during high consumption period. The negative pressure indicates that the area supplied by nodes should not gate water at maximum consumption hours. The pressure at 11 nodes greater than the recommended pressure 60 mH_2O. High value of pressures affects adversely the hydraulic performance of the distribution network at night time during low consumption period, the pressure in the system become high and it causes pipe burst at the lower location which were presented in Table 4.3 below.

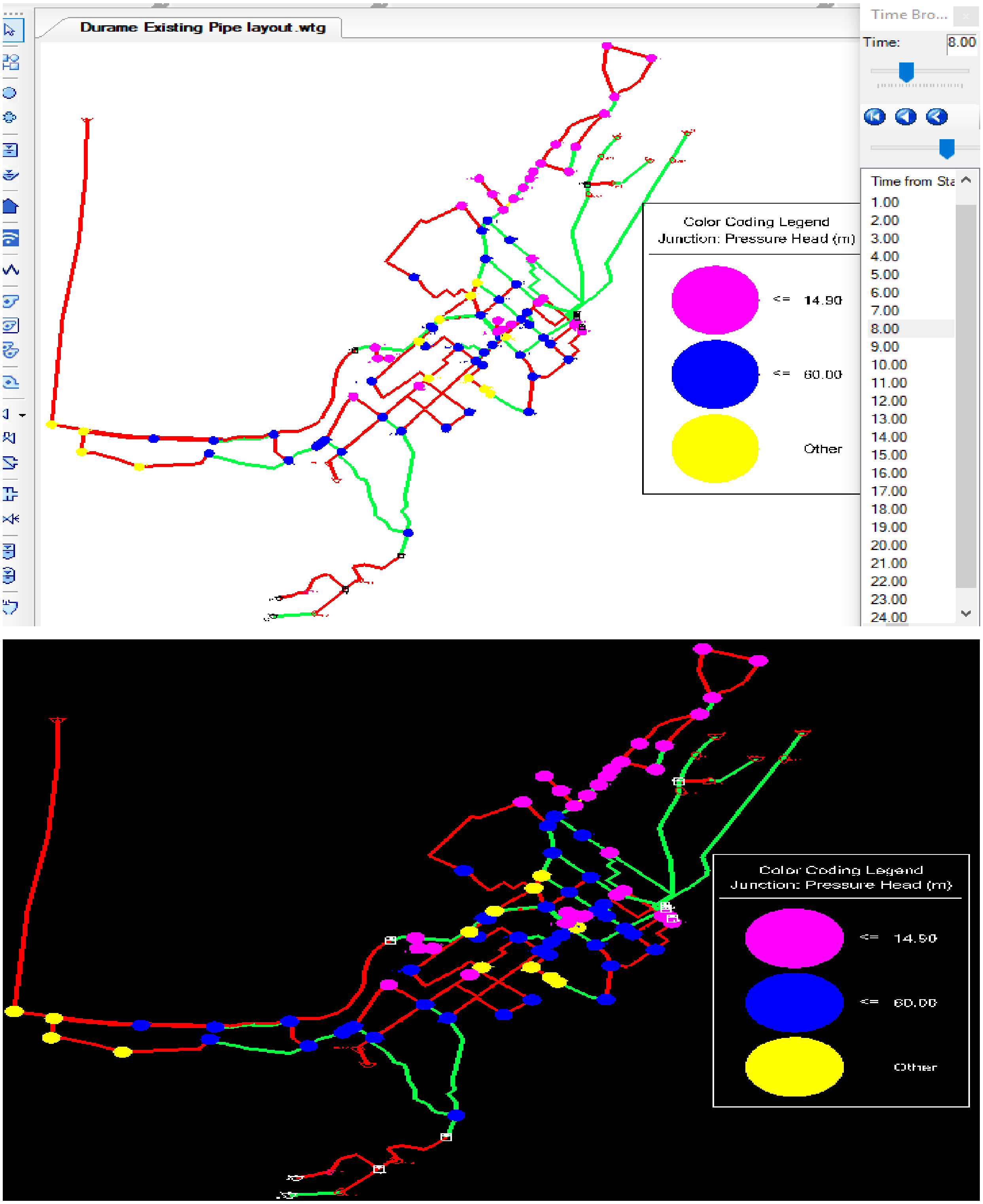

Figure 4. 2 Pressures at 7:00 AM to 8:00 AM in the peak-hour consumption

Table 4. 2 Simulated Pressure distribution in system at peak hour consumption (8:00)

Pressure range	< 15 mH_2O	(15 - 60 mH2o)	> 60 mH_2O
Number of Nodes	31	43	11
Percent (%)	36.47	50.59	12.94

As shown in the Table 4.2 from the total nodes 36.47% nodes had pressure below 15 mH_2O, 50.59% had permissible pressures between 15 mH_2O and 60 mH_2O and 12.94% of the node had above 60 mH_2O pressure.

Table 4. 3 Nodes/Junctions were selected in the network modeling process

Pressure ranges	Name of nodes
< 15 mH_2O	J-55, J-42, J-98, J-112, J-23, J-29, J-115, J-114, J-24, J-113, J-15, J-22, J-17, J-16, J-33, J-73, J-11, J-34, J-20, J-12, J-43, J-81, J-32, J-44, J-116, J-86, J-9, J-51, J-8, J-39, J-7.
15 mH2o - 60 mH_2O	J-49, J-65, J-13, J-31, J-36, J-83, J-48, J-57, J-14, J-69, J-2, J-59, J-38, J-35, J-1, J-18, J-64, J-58, J-54, J-53, J-5, J-127, J-74, J-66, J-85, J-128, J-129, J-26, J-6, J-50, J-37, J-46, J-19, J-27, J-41, J-25, J-67, J-125, J-4, J-40, J-3, J-52, J-30.
> 60 mH_2O	J-56, J-10, J-71, J-28, J-45, J-61, J-47, J-21, J-63, J-126, J-62.

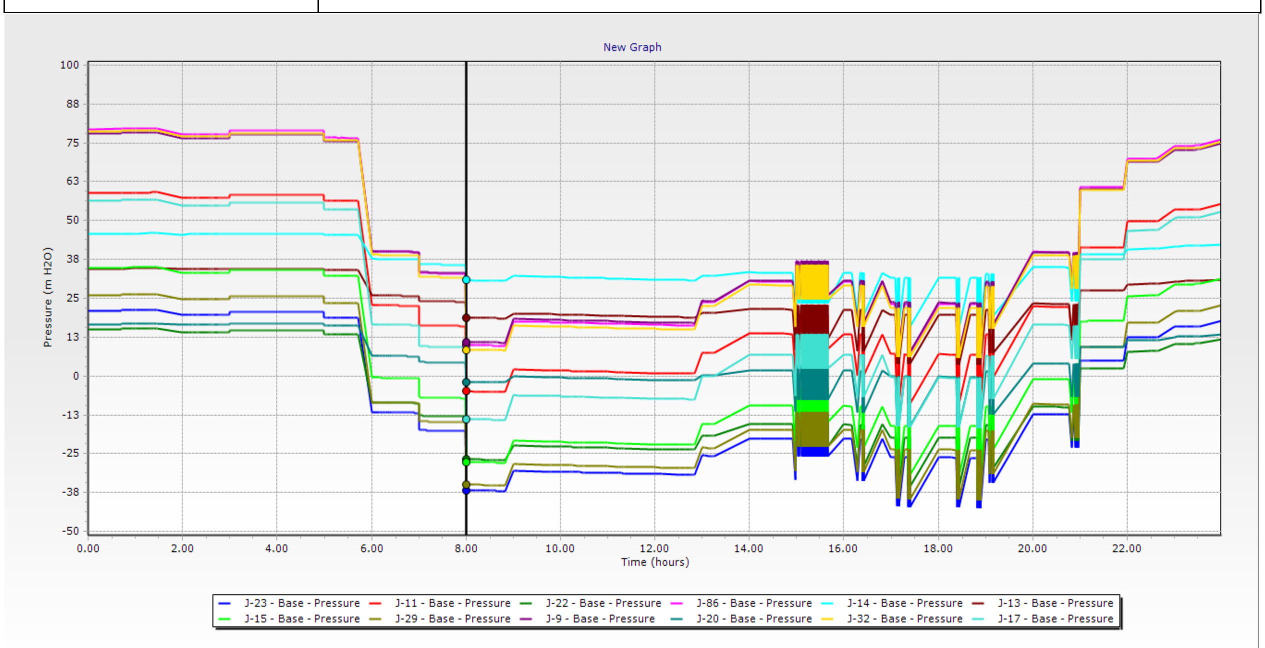

Figure 4. 3 Graph showing nodes with negative pressure

As shown in Figure 4.3 when the system operated at steady state the demand at every node did not changing, the pressure for all the twelve nodes was positive. However, as the demand changed to the peak demand the pressure decreased to negative (J – 23, J – 15, J – 11, J – 29, J – 22, J – 9, J – 86, J – 20, J – 114, J – 113, J – 32, J – 17) therefore, during this time water could not reach to consumers supplied by nodes as shown in the Table 4.3.

The results of the WaterGEMSv8i model of pressure in the water distribution network nodes shows minimum hydraulic pressure which less than a minimum of the design criteria of 15 mH_2O water column at the distribution system around Industrial College, Stadium areas and Hospital areas. This represents that the areas or village did not receive water from these junctions which shows negative pressure they produce low velocities which accelerate the deterioration and corrosion of the pipes in the distribution network. The pressure at nodes as shown Table 4.3 are greater than the recommended pressure 60 mH_2O. High value of pressures affects adversely the hydraulic performance of the distribution network at night time during low consumption period, the pressure in the system become high and it causes pipe burst at the lower location.

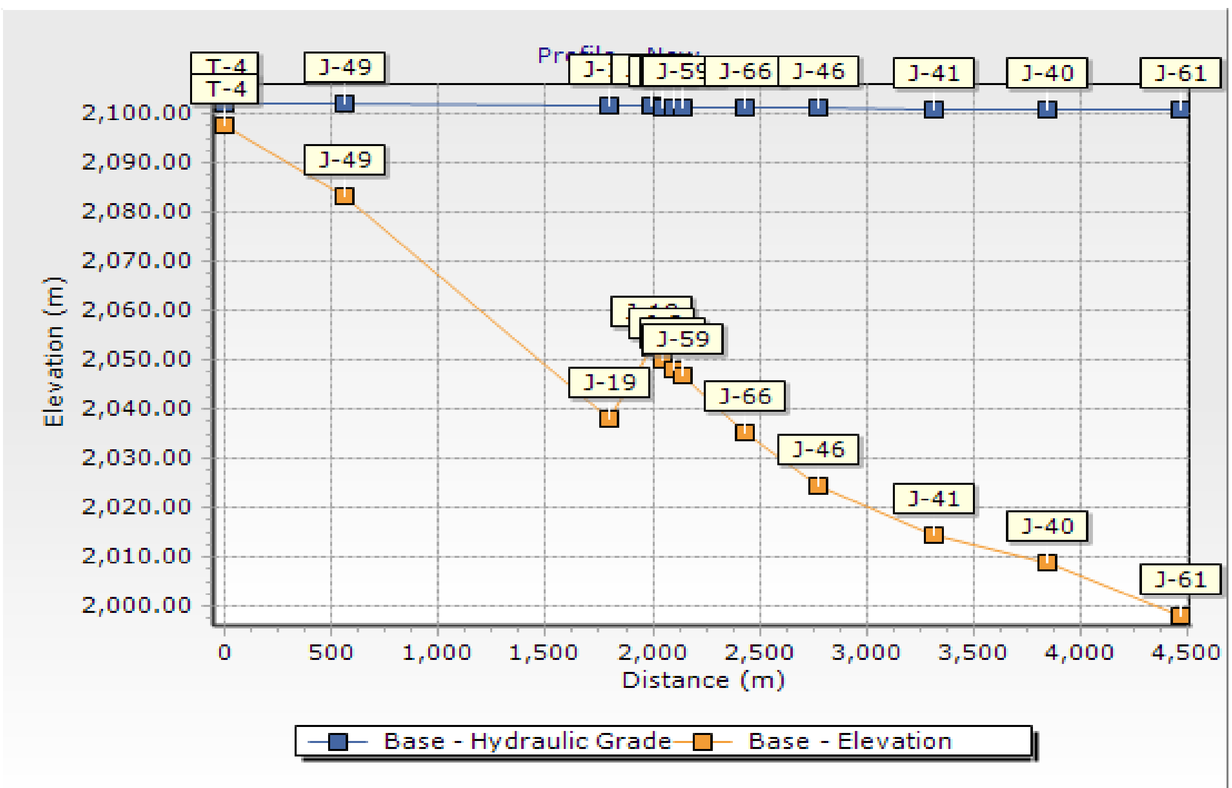

Figure 4. 4 Profiles of nodes showing distance from storage Tank - 4 (T- 4) with elevation

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

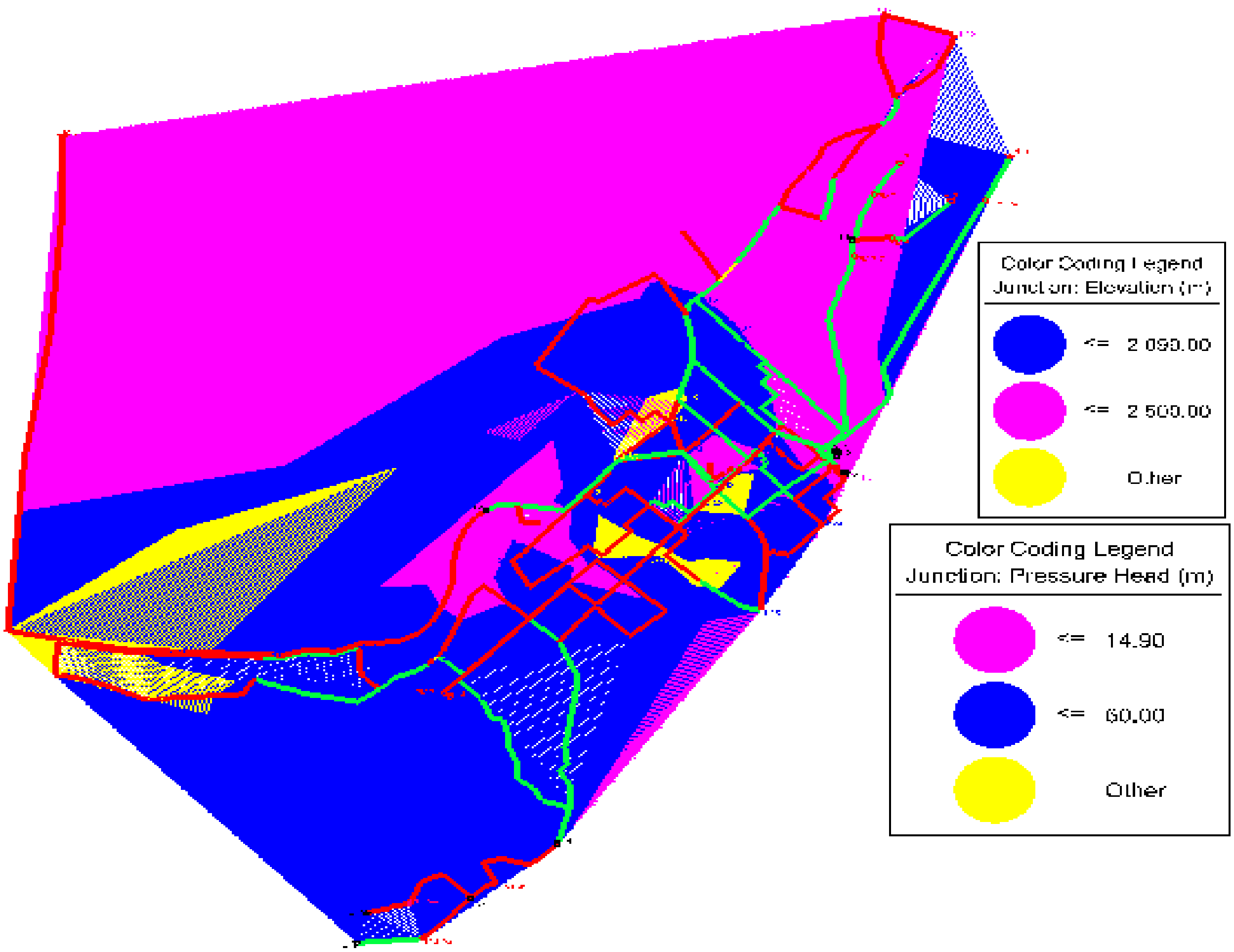

Figure 4. 5 Pressure contour at peak hour demand displayed with elevation

The pressures at nodes depend on the topography of the area and the performance of the input energy like pumps. The area highlighted by pink color in the Figure 4.5 is pressure deficit area below 15 mH_2O water head. It is based on the population served, types of dwellings in the area, and firefighting requirements. The pressure at node depends on the adopted minimum and maximum pressure within the network, topographic circumstances, and the size of the network. The contour map of pressure clearly shows the pressure difference in the whole systems in the study town.

4.3.2 Pipe Flow Velocity

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

Figure 4. 6 Velocity distribution at peak hour consumption

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

Velocity range (m/s)	Count		Effect
	Number	(%)	
< 0.6	72	56.7	Sedimentation problem
0.6 - 2	53	41.7	Normal
> 2	2	1.6	High head loss occurred

As indicated in Table 4.4, 56.7% of the pipes are below the permissible range of velocity; 41.7% at permissible range; and 1.6% above the range according to Table 3.8 criteria set by MoWR (2006). Low velocity in pipe flow affects the proper supply of water and undesirable for hygienic reason. When the diameter value of pipe increases the velocity decreases and long-time of retention causes sediment formation. The flow and pressure distributions across a network are affected by the arrangement andrsizes of the pipes and the distribution of the demand flows. Since a change of diameter in one pipe length will affect the flow and pressure distribution everywhere, network simulation is not an explicit process Pipe network analysis involves the determination of the pipe flows and pressure heads that satisfy the continuity and energy conservation equations (Rossman, 2000).

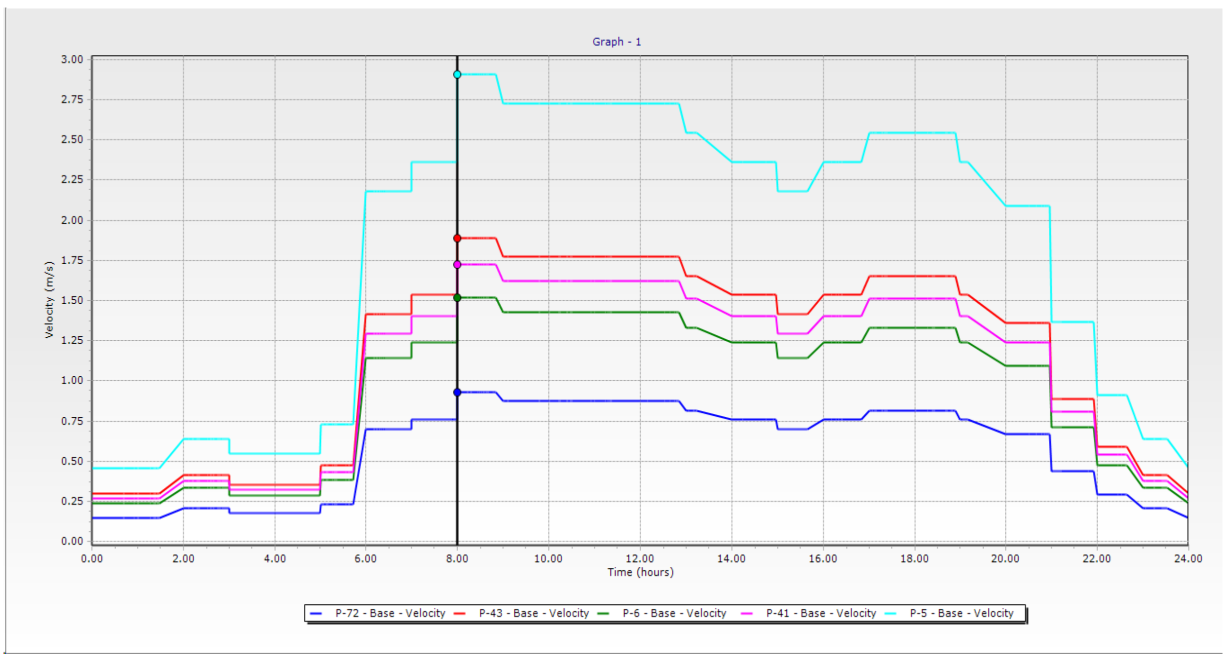

Figure 4. 7 Main transmission line showing velocity verses time graph (P-5, P-43, P-41, P-6, P-72)

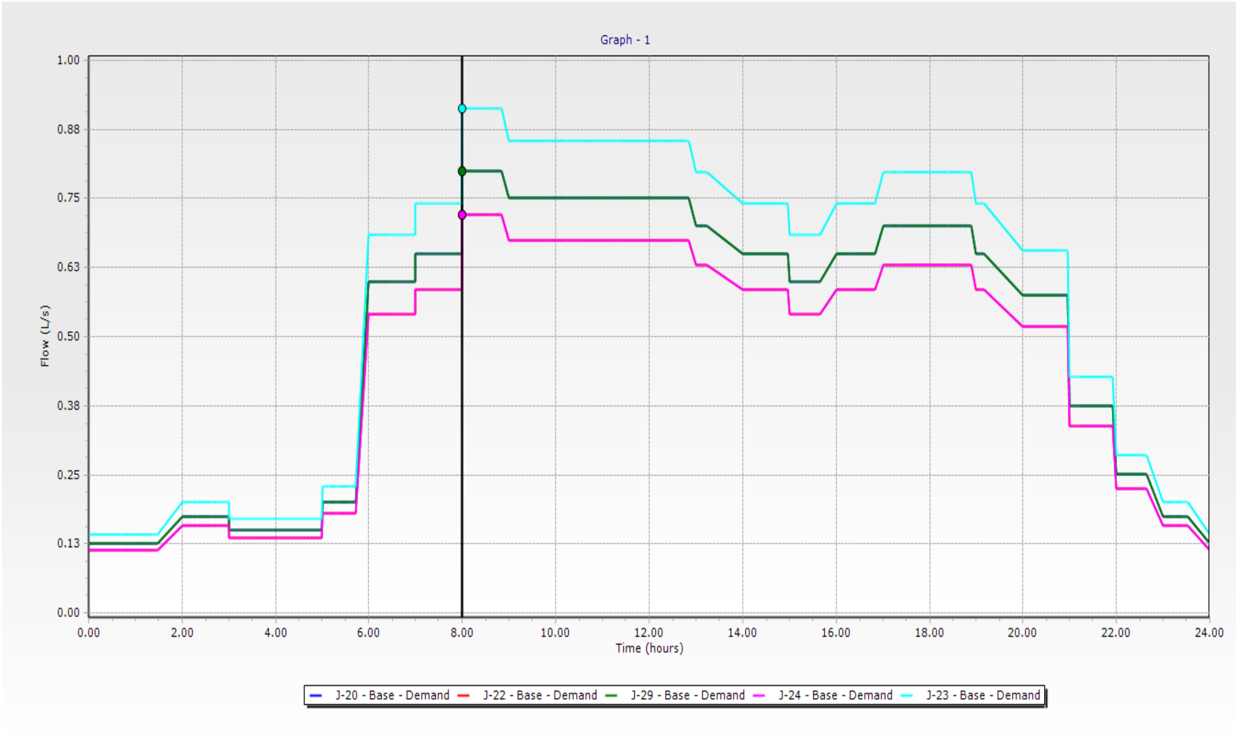

Figure 4. 8 Main transmission line showing flow verses time graph

The above Figures 4.7 and 4.8 shows velocity in distribution network is in high consumption time for selected pipes (J-20, J-22, J-24, J-23 and J-29) the velocity and flow increase linearly as the time increases from 2:00 hour up to 24:00 hour.

4.3.3 Demand Pattern

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

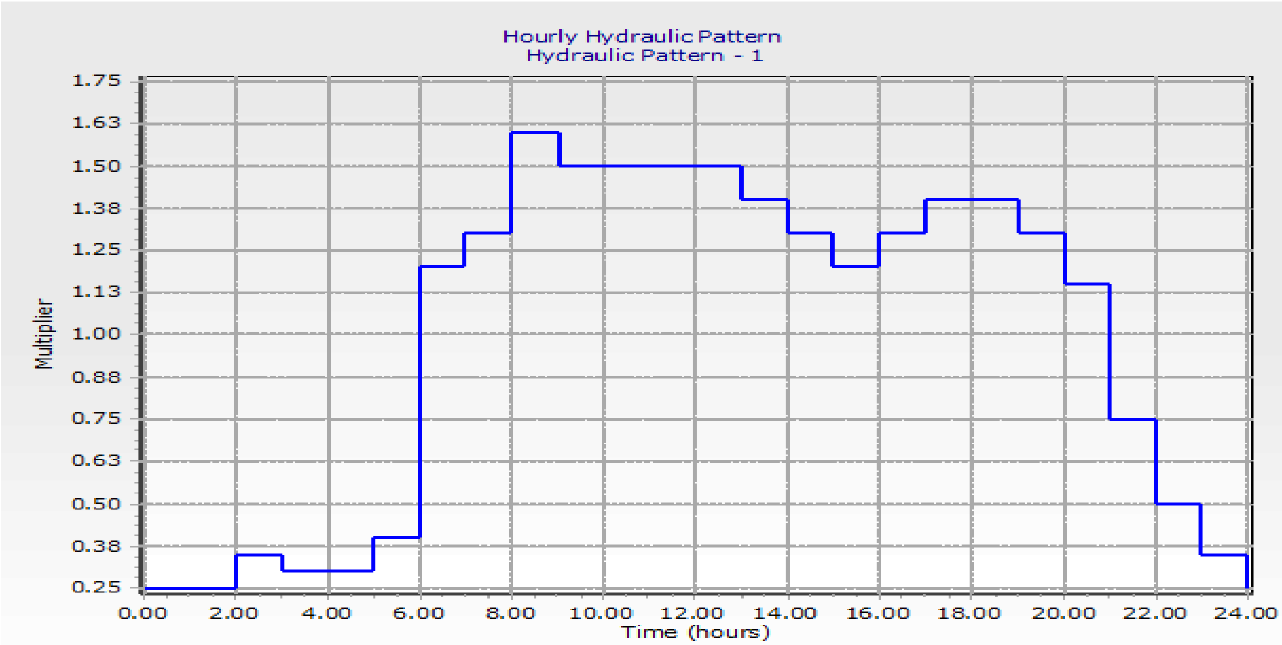

Figure 4. 9 Demand pattern in Water distribution of Durame Town within 24 hours

Figure 4. 10 The distribution network demand in Durame Town

4.4 Behavior of Storage tank at different consumption hours of a day

The service reservoir (storage tank) is provided to balance (constant) supply rate from the water source or treatment plant with the fluctuating water demand in distribution area. Dynamic (EPS) simulation result was used to show the fluctuating storage volume with time increments during high and low consumption.

Moreover, in low demand hours when the water consumption of consumer is almost zero, amount of pumped water is higher than system demand so that extra water coming from pumps are stored at storage tank and equilibrium of water distribution system satisfied again. The time varying simulation indicates that storage Tank- 4 starts to decrease its volume at 8:00 AM hour that means up to 8: hour AM the volume in the tank is full.

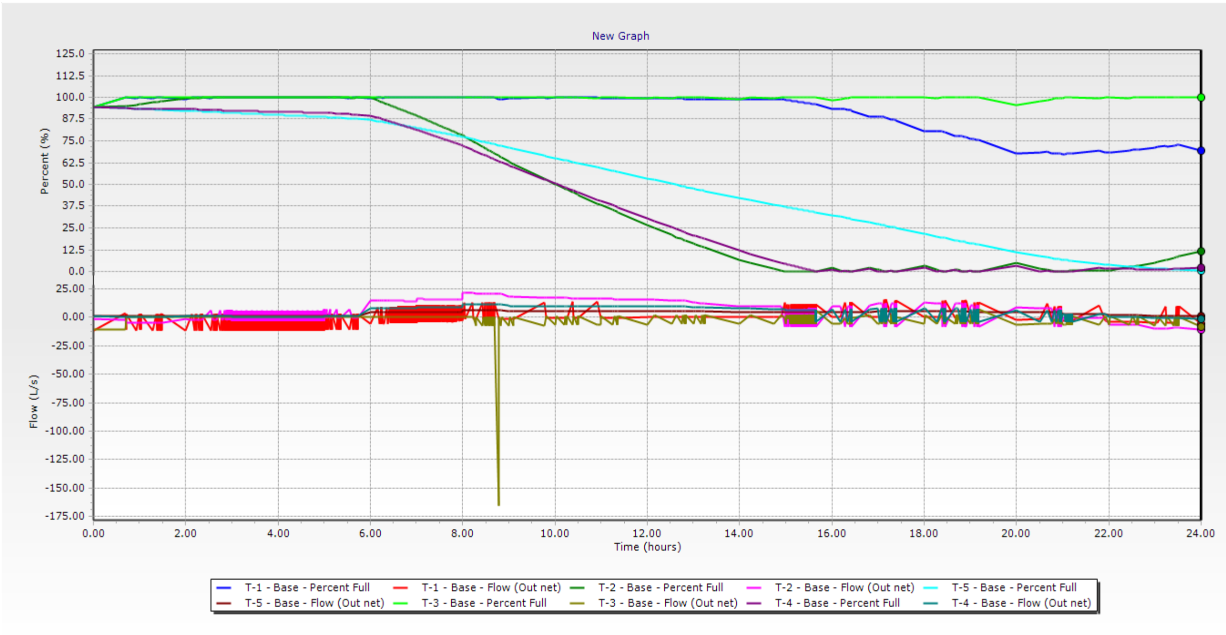

Figure 4. 11 Tank - 4 Water volume fluctuation over 24 hour periods

Figure 4.11 shows during the extended period simulation the storage level of the tank fluctuate for 24-hour period which shows the change in percent of full in different time interval. When the simulation run begins the tank was full and then the volume starts to decrease up to 16 hours, so that the pumps should have to operate to replenish the volume of the tank starting from 16 hours.

4.5 Pump

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

4.5.1 Pump capacity curve

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

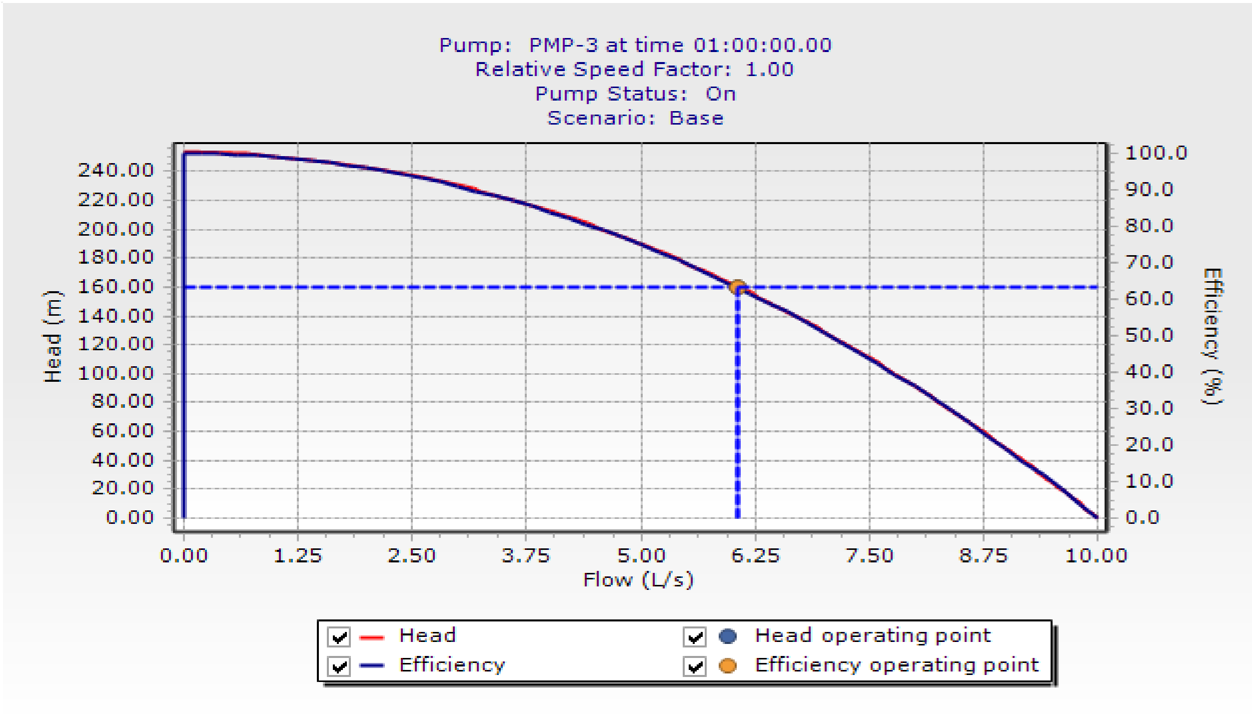

Figure 4. 12 Pump – head verses flow curve

Pump head is the head gain imparted to the water by the pump and plotted on the vertical of the curve in meter. Flow rate is plotted on the horizontal in litter per second. A valid pump curve must have decreasing head with increasing flow. An efficiency curve determines pump efficiency in vertical percent as a function of pump flow rate in horizontal flow. Figure 4.12 shows as the blue line the head increases the amount of discharge pushed by the pump decreases. When the head decreases the pump can push high amount of discharge to a lower elevation so that pump curve indicates decreasing head with increasing flow illustrate by red line both of them had been overlapped.

4.6 Evaluating the Current Water Supply of Durame Town

4.6.1 Water Sources and Production

The main source of water supply for the study town is from six ground water deep borehole and one spring developed source with a total discharge capacity of the town was 39 l/s. According to the information found during discussion and field observation with the experts of Durame town water supply and service office no other sources of water for potable drinking water except these sources.

As described in Table 3.2 the town water supply is an 86% dependent on electric power with working (19 hours), the wells produce a total volume of discharge 39 l/s which is equal to 31l/s in 24 hours. Therefore, the total volume of water entered to the storage tank within 24 hours is 2,669 m^3. According to the information obtained during discussion with the experts of water supply and service office, one problem of the water supply of town is the source of power for pump motor or shortage of power supply due to absence of power supply generators. That means, the supply of water to the town was 86% percent electric power dependent.

4.6.2 Coverage of Potable Water

Average per capita consumption was used to assess the domestic water supply coverage of the town. Data on individual domestic water consumptions, total water consumption (m^3) and total production (m^3) were collected from Durame town Water Supply and service office billed documents for analysing average per capita consumption. As we have seen in equation 3.5, the potable water supply coverage, the quantity of per capita water consumption was used. The average per capita water consumption was derived from the yearly consumption of the town that have been aggregated from the individual water meter and public tap. Thus, the annual water consumption data was converted to average daily per capita consumption using the population data of Durame town.

Table 4. 5 Annual water consumption of Durame Town

Year	Population	Annual billed Consumption (m^3)	Per capita Consumption (l/c/d)
2018	82300	456580	15.2

The average percapita domestic water consumption was derived from the yearly consumption of the town that have been aggregated from the individual water meter and public tap. Thus, the annual water consumption data was converted to average daily per capita consumption using the population data of Durame town using equation 3.5. As shown in Table 4.5, the per capita domestic water consumption of Durame town was found to be 15.2 l/c/d in the year 2018. According to WHO (2008), the minimum quantity of domestic water required in urban areas of developing country in the radius 0.5 km taken as 20 l/c/day. Regarding to this value, the domestic water supply of Durame town only satisfies 76% of the standard value and the quantity of domestic water required in urban areas of Ethiopia is taken as 50 l/c/day (MoWIE, 2015). According to this value, the domestic water supply of Durame town only satisfies 30% of the standard value. the per capita water consumption of town is viewing very low. The main reasons for reduction in the town's per capita water consumption as time goes is the increase in the population number of the town, pump failure and seasonal fluctuation of the source. The population number of the town is increasing from time to time with increasing demand on the existing water supply system of the town. As a result, the per capita domestic water consumption of the town gets lower and lower. Thus, it is advisable to develop the public preferred nearby source with supply and install new pump to improve the per capita water consumption of the town.

4.6.3 Water Losses

The monthly water produced and distributed to the distribution system and the water billed that was collected from the individual customer meter readings in the year 2018, used to quantify the total water loss for the entire town. The total water loss has been also evaluated based on percentage of system input volume, length of main and number of connections as explained under the performance indicators sub title later in this section.

The designed water production capacity of the town was 2,669 m^3/day. However, the actual production of water has been lower than the maximum designed capacity due to pump failure and seasonal fluctuation of the source. Production data computed from all sources in (DWSSO) shows that actual average production of water at present from the system was 1,918.6 m^3/day, which indicates 72% of its designed capacity (2,669 m^3/day).

The total production of water recorded by the office of water in the year 2018 was 700289 m^3 and the total billed consumption of the town was 456580 m^3. The amount of total consumed water is less than the amount of water supplied. Water loss from water distribution systems (WDSs) has long been a feature of the WDN operations management. According to Motie.etal (2007), total water loss or unaccounted for water (UFW) is the difference between the volume of water produced, and the volume that is billed or consumed. Therefore, the total loss of water in the town for the year was 700289 m^3 – 456580 m^3 which gives 243709 m^3 and approximately 34.8% of the total production. This figure is lower compared with the average for developing countries (35%) according to (Kingdom, 2006). The average the amount of water, which actually reached the consumers, therefore accounts for only 65.2% of the total water produced. According to Mckenzie *et al* (2006), the system efficiency is good (acceptable) if above 75% of water produced reaches the consumer. Therefore, Durame town water supply distribution network is not good.

The main reasons for this high loss of water are the present way of water. And also the average tariff for 1(one) m^3 of water in the town as 3.5 birr, the water loss is estimated to be 852,981.5 birr every year. However, the real loss is beyond this as the water tariffs like other developing countries are usually subsidized.

4.7 Population and Water Demand Projection

4.7.1 Population Projection

In order to forecast the population of the study area in 2018 based on last population census report the population and housing census report of 2007 which was prepared by Ethiopian Central Statistical Agency (CSA) have been used. As described Table 3.9 the growth rates within deferent years' percentages which was reported by CSA for Durame town was used for the current population projection.

Table 4. 6 Population projection of Durame Town based no growth rate % (2018 – 2038), (CSA, 2007).

Year	2018	2019	2023	2028	2033	2038
projected Population	82300	86250	102954	122411	140909	156027

Applying the geometric population projection method for Durame town the population has been projected up to year (2018 – 2038).

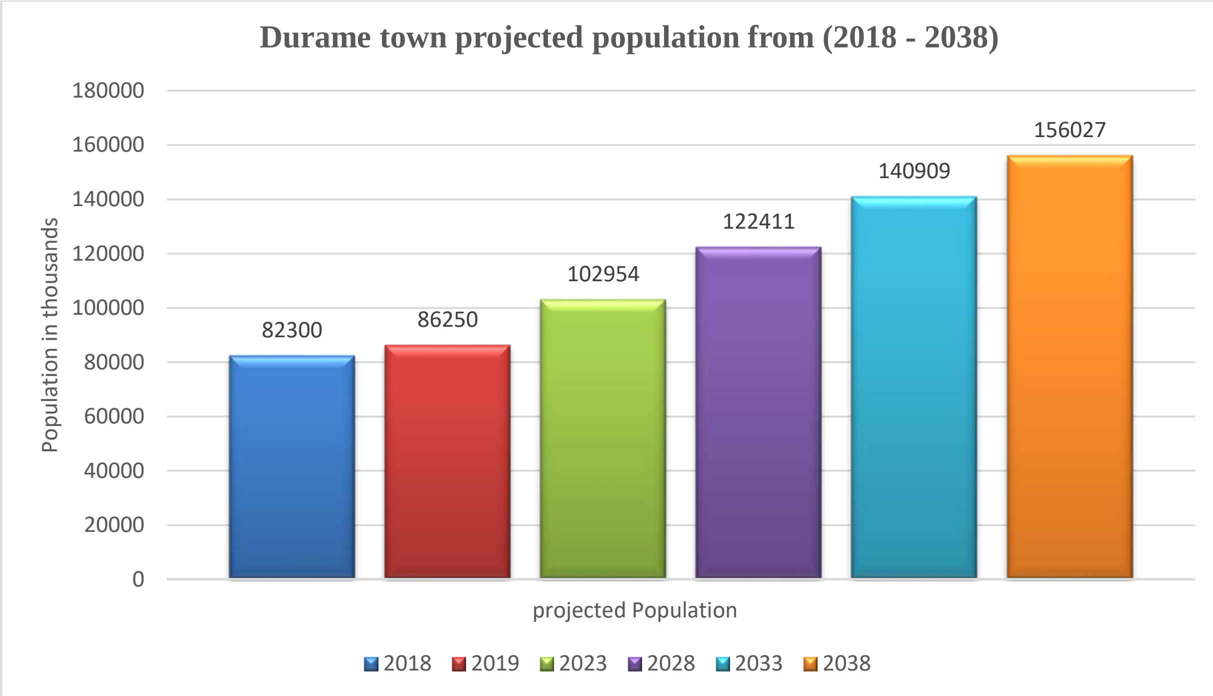

Figure 4. 13 Population projection of Durame Town up to design period (2018-2038)

4.7.2 Demand Projection by Mode of Services

Water engineers and managers forecast future water demand for a variety of purposes. These analyses can help managers understand spatial and temporal patterns of future water use to optimize system operations, plan for future water purchases or system expansion, or for future revenue and expenditures. There are several mathematical methods in use for estimating future demand. For this particular study, per capita use approach was employed due to the availability of data and the simplicity of the method. Thus, population was projected from 2018 to 2038 using geometric increases by using regional given growth rate in % and the corresponding water demand per mode of service was estimated till 2038.

According to MOWR (2006), a plan was prepared and entered in to action to improve the water supply schemes of Ethiopian Towns. As the schemes are changed to modern Technology, the quality of the water and system of supply also become better than the existing one. This change in quality and supply system makes the people to consume more water that makes the per capita water demand high in the coming years. Population growth influences water demand through increased demand by households, but also indirectly through uses in maintaining particular lifestyles (Pretoriou and Shutte, 1997). Using per capita per day water demand, the percentage of population served by house connection (HC), yard connection (YC), and public fountain (PF) has been estimated for each respective year (2018 - 2038). To estimate per capita water demand, the population for each mode of service should be first determined. The distribution of population for each mode of service was determined by considering socio-economic situation and living standard of the town. After establishing the population distribution for the base year a forecast was made.

Table 4. 7 Population Projection by Mode of Service.

Description	Phase I					Phase II	
	Unit	2018	2019	2023	2028	2033	2038
Population Growth Rate			4.80%	4.80%	4.05%	3.65%	3.25%
Projected population	No	82300	86250	102954	122411	140909	156027
Population Percentage Distribution by Mode of service							
HTU	%	1.82	2.97	7.57	13.32	19.07	24.82
YTU	%	42.53	43.86	49.18	55.83	62.48	69.13
NTU	%	0.61	0.70	1.06	1.51	1.96	2.41
PTU	%	55.04	52.47	42.19	29.34	16.49	3.64
Total	%	100.00	100.00	100.00	100.00	100.00	100.00
Population served by							
HTU	No	1500	2564	7796	16308	26875	38730
YTU	No	35000	37827	50630	68339	88036	107858
NTU	No	500	602	1089	1845	2758	3756
PTU	No	45300	45258	43439	35919	23239	5683
Per Capita Demand by Mode of Services							
HTU	l/c/d	96	98	106	115	125	134
YTU	l/c/d	48	48	50	53	55	58
NTU	l/c/d	58	59	62	67	72	77
PTU	l/c/d	38	39	41	43	46	48
Domestic Water Demand by Mode of Services							
HTU	m3/d	144.00	251.06	823.29	1,878.72	3,354.01	5,205.32
YTU	m3/d	1,680.00	1,833.86	2,551.75	3,608.29	4,859.61	6,212.60
NTU	m3/d	28.80	35.24	68.04	124.36	199.40	289.94
PTU	m3/d	1,739.52	1,760.49	1,776.48	1,558.58	1,066.41	274.98
Total Water Demands	m3/d	6,196.75	6,723.22	9,199.48	12,905.91	17,418.44	22,467.82

The socio- economic and climatic adjustment factor was combined 1*1= 1 used to calculate the adjusted water demand by multiplying the total water demand for respective years. Using 10 %

for institutional and commercial demand, 10 % for industrial water demand, 5 % for firefighting demand from the adjusted water demand the average daily water demand was calculated. Taking 25 % for unaccounted water demand from average daily water demand the projected water demand was calculated. The maximum daily water demand and peak hour demand was projected using the maximum day factor of 1.2 for maximum day factor and 1.8 for peak hour factor which is recommended for towns having population between 50001 to 100,000 (MOWR, 2006). The detailed result of the projected population and water demand is presented in appendix (3).

4.7.3 Average Per Capita Daily Water Demand

Water demand was a summation of all consumptions given in the preceding sections and will determine the capacity needed from the sources. Under constrained resources, water is allocated to consumers and the actual water supply may not be able to meet the demand. The total annual recorded consumption of the town has been converted to average daily per capita consumption using the number of population in the year 2018. Daily per capita consumption (l/c/day) from billed consumption was presented in Table 4.5.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Based on the model results, field survey and data analysis using models, urban water supply distribution network the best performance indicators for water distribution modeling results are: pressure head at network nodes (15 mH_2O - 60 mH_2O head) and flow velocity in pipes (0.6 – 2 m/s) (MOWR, 2006). On the water distribution network hydraulic parameters performance has been evaluated using the existing water distribution layout, surveyed data of base demand, elevation and directions of Northing and Easting of Junction, Source, Tanker and pipe data by using WaterGEMSEv8i software.

The results of the WaterGEMSv8i model of pressure in the water distribution network nodes shows minimum hydraulic pressure which less than a minimum of the design criteria of 15 mH_2O water column at the distribution system around Industrial College, Stadium areas and Hospital areas. Which represents that the areas or village did not receive water from these junctions which shows negative pressure they produce low velocities which accelerate the deterioration and corrosion of the pipes in the distribution network. Result shows during peak hour consumption, parts of the distribution system receive water with low pressure and under some conditions risk of obtaining no water is observed because of the pressure in the distribution system is below permissible minimum requirement. Which shows the performance of the distribution system has 49.41 % of consumption nodes have out of the indicated range of performance. 50.59 % nodes have acceptable pressure limits between (15 mH_2O - 60 mH_2O)

The simulated result for the distribution system was resulted 36.47 % nodes has been found below the acceptable limits of pressure value (< 15 mH_2O), in pressure zone one due to high elevation difference and long distance from the source. The result for dynamic simulation for 24-hour period and during high consumption period 31(thirty-one) nodes have negative pressure.
The nodes are furthest from the source of supply point storage tank (T- 4 and T-1). As the distance increases the water pressure diminishes(reduces) in distribution system. The negative value of pressure indicates that the area supplied by those nodes could not get water in peak demand hours. Therefore, areas which need zoning or installing another new tank to enable the system to

continuously supply the residents at all demanding time is known. Low velocity in pipe flow affects the proper supply of water and undesirable for hygienic reason. When the diameter value of pipe increases the velocity decreases and long-time of retention causes sediment formation. The flow and pressure distributions across a network are affected by the arrangement andsizes of the pipes and the distribution of the demand flows. The cause of sedimentations has been identified from 56.7 % of water distribution pipes with velocity below the acceptable limits (< 0.6 m/s). When the flow is moving very slow in pipes the very tinny materials remains on the inner wall of the pipes and causes water quality problems. The locations of pipes with high head loss are known from 1.6 % for velocity values (> 2 m/s).

The total average per capita consumption of the Durame town in the year 2018 was 15.2 l/p/d which shows lower performance as compared to 20 l/p/d which is set by WHO (2008) within a radius of 0.5 km. For the study year which only satisfies 30 % of the minimum urban water consumption value set by (MoWIE, 2015) and 76 % value set by (WHO, 2008).

The future population and water demand of the town has been projected up to the year 2038 using year 2018 as base year. The potential of the projected water demand increment in Durame town is greater than the current supply potential of water sources. Which shows current water demand is 6,196.75 m^3/day and the demand at end of design period of 2038 years would be around 22,467.82 m^3/day.

5.2 Recommendations

In order to improve in terms of hydraulic parameters performance in the distribution and water supply coverage of the town the following activity should be performed:
As this study, was specifically limited to evaluate hydraulic performance related to pressure and pipe flow velocity and to forecast the future population and demand a study should be undertaken related to spatial allocation of demand and leakage detection of the system. So that the administrative or other concerned body can use the findings and layout of this research to overcome the distribution network problems of the study town.

Durame town water supply and service office should gather x, y, coordinates of all of the components of water supply distribution network, its customers water meter and prepare population layer by shape file in order to model the distribution system using WaterGEMSv8i software for future improvement of distribution network and expansion works with Technology technics to address the problems and to satisfy the customer needs. The Ambaricho spring developed was the one source of potable water for the town, around the upper spring developed watershed area have high gully erosion, the soil and water conservation practice should be conducted to increase the recharge for sustainable uses.

To increase the reliability of water supply coverage, the ground water sources has to be augmented by surface water and water losses control measures should be taken.
The simulated result has shown low pressure at a nodes supplied by storage tank one; therefore, the water supply and service office should have to construct additional reservoir or have to increase zoning around the above the Industrial College area of the town to get high pressure head.

Necessary pressure supplementing valves should be installed to upraise pressure during peak demand time. More of the pipes are found under low flow velocity so that, the diameter of the pipes should be minimized to upgrade the quality and sufficient amount of water in the system. To reduce such problems in the town distribution system should check every gate valves (flow control valves) as per their control status due to the shortage of the sources.

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APPENDIXES

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

Flex Table: Pipe Table
Current Time: 0.00 hours

Label	Diameter (mm)	Length (m)	Material	Flow (L/s)	Velocity (m/s)	Head loss Gradient (m/km)
P-48	75	3,450	DCI	0.00	0.00	0.000
P-14	125	220	DCI	0.00	0.00	0.000
P-144	120	85	DCI	0.00	0.00	0.000
P-20	75	3,310	DCI	0.00	0.00	0.000
P-74	75	335	DCI	0.00	0.00	0.000
P-9	63	420	HDPE	0.00	0.00	0.000
P-60	90	475	HDPE	0.01	0.00	0.000
P-85	50	80	HDPE	0.00	0.00	0.002
P-50	63	230	HDPE	0.02	0.01	0.001
P-131	63	610	HDPE	-0.03	0.01	0.005
P-31	110	160	DCI	0.13	0.01	0.004
P-46	63	205	HDPE	-0.08	0.02	0.022
P-59	50	195	HDPE	0.05	0.02	0.029
P-93	90	750	HDPE	0.18	0.03	0.019
P-42	75	130	DCI	0.13	0.03	0.023
P-30	75	120	DCI	0.13	0.03	0.022
P-47	63	195	HDPE	-0.09	0.03	0.029
P-92	63	535	HDPE	-0.09	0.03	0.032
P-87	90	50	HDPE	0.21	0.03	0.027
P-82	75	390	DCI	-0.15	0.03	0.034
P-11	125	260	DCI	0.44	0.04	0.019
P-10	63	590	HDPE	0.11	0.04	0.044
P-156	63	210	DCI	-0.11	0.04	0.044
P-146	63	220	HDPE	0.11	0.04	0.045
P-3	63	570	HDPE	-0.11	0.04	0.045
P-98	63	595	HDPE	-0.12	0.04	0.050
P-95	90	265	HPE	0.25	0.04	0.033
P-145	75	80	DCI	0.17	0.04	0.043
P-49	63	625	HDPE	0.13	0.04	0.045
P-107	110	530	HDPE	0.42	0.04	0.034
P-24	150	240	DCI	-0.82	0.05	0.026
P-132	63	750	HDPE	-0.15	0.05	0.072
P-94	110	400	HDPE	0.46	0.05	0.039
P-103	75	200	DCI	0.23	0.05	0.069
P-102	90	345	HDPE	0.33	0.05	0.059
p-65	90	595	HDPE	0.36	0.06	0.067
P-45	75	70	DCI	-0.25	0.06	0.083
P-69	75	405	DCI	-0.25	0.06	0.086
P-133	75	480	HDPE	-0.26	0.06	0.089
P-148	63	230	HDPE	-0.20	0.06	0.126
P-51	50	1,285	HDPE	0.12	0.06	0.165
P-89	63	55	HDPE	-0.20	0.06	0.133

P-54	63	685	HDPE	-0.21	0.07	0.138
P-149	75	370	DCI	-0.30	0.07	0.115
P-8	63	450	HDPE	0.21	0.07	0.143
P-2	63	555	HDPE	-0.22	0.07	0.155
P-96	90	345	HDPE	0.47	0.07	0.110
P-101	50	450	HDPE	0.15	0.07	0.221
P-53	63	525	HDPE	-0.24	0.08	0.142
P-7	63	580	HDPE	-0.25	0.08	0.197
P-13	125	50	DCI	-1.03	0.08	0.095
P-29	75	230	DCI	0.37	0.08	0.177
P-71	75	250	HDPE	-0.38	0.09	0.178
P-76	100	185	DCI	0.68	0.09	0.130
P-83	75	160	DCI	0.38	0.09	0.183
P-44	110	240	HDPE	-0.84	0.09	0.122
P-147	75	240	DCI	-0.39	0.09	0.191
P-36	125	846	DCI	1.09	0.09	0.106
P-100	75	320	DCI	0.42	0.09	0.214
P-137	50	275	HDPE	0.19	0.10	0.380
P-91	125	145	HDPE	1.22	0.10	0.131
P-86	100	405	DCI	-0.79	0.10	0.173
P-28	75	1,334	DCI	0.45	0.10	0.248
P-52	63	760	HDPE	-0.32	0.10	0.307
P-84	75	130	HDPE	-0.46	0.10	0.256
P-12	63	285	HDPE	0.32	0.10	0.315
P-63	75	280	DCI	-0.46	0.11	0.263
P-139	110	260	HDPE	1.01	0.11	0.173
P-4	63	230	HDPE	0.34	0.11	0.340
P-104	125	1,200	HDPE	1.33	0.11	0.154
P-105	90	1,240	HDPE	-0.69	0.11	0.227
P-35	125	195	HDPE	1.34	0.11	0.155
P-55	63	400	HDPE	0.34	0.11	0.352
P-56	100	310	DCI	-0.87	0.11	0.207
P-134	75	295	DCI	0.50	0.11	0.303
P-97	63	545	HDPE	-0.36	0.11	0.375
P-21	200	40	uPVC	-3.66	0.12	0.100
P-70	125	295	HDPE	-1.43	0.12	0.177
P-62	50	395	HDPE	0.23	0.12	0.524
P-66	150	560	uPVC	2.14	0.12	0.152
P-80	100	115	DCI	0.95	0.12	0.245
P-61	75	280	DCI	0.54	0.12	0.349
P-57	100	980	DCI	0.98	0.12	0.256
P-58	125	285	HDPE	1.55	0.13	0.203
P-78	100	265	DCI	1.00	0.13	0.268
P-106	75	320	DCI	-0.57	0.13	0.386
P-73	75	240	DCI	-0.57	0.13	0.388
P-79	100	120	DCI	1.03	0.13	0.282
P-144	63	120	HDPE	0.41	0.13	0.490
P-64	90	230	HDPE	-0.86	0.14	0.340
P-90	100	114	DCI	1.09	0.14	0.313
P-77	125	55	HDPE	1.70	0.14	0.241
P-68	90	290	HDPE	0.90	0.14	0.368

P-72	90	120	HDPE	0.93	0.15	0.387
P-135	75	605	DCI	0.68	0.15	0.537
P-99	90	195	HDPE	1.00	0.16	0.452
P-140	125	285	HDPE	-1.95	0.16	0.311
P-23	150	300	uPVC	-3.00	0.17	0.284
P-75	110	65	HDPE	-1.62	0.17	0.410
P-81	50	110	HDPE	-0.34	0.17	1.055
P-67	110	195	HDPE	-1.77	0.19	0.483
P-25	150	90	uPVC	-3.65	0.21	0.408
P-27	75	183	DCI	0.95	0.22	0.989
P-22	200	380	uPVC	6.98	0.22	0.334
P-6	75	90	DCI	1.05	0.24	1.192
P-19	125	360	uPVC	2.96	0.24	0.673
P-41	75	175	DCI	1.19	0.27	1.507
P-43	75	150	DCI	1.30	0.30	1.781
P-1	63	40	HDPE	-0.95	0.30	2.311
P-18	63	50	HDPE	-1.10	0.35	3.033
P-38	50	535	HDPE	-0.71	0.36	4.203
P-5	63	160	HDPE	-1.42	0.45	4.854
P-26	125	255	DCI	6.07	0.49	2.549
P-15	125	655	DCI	7.13	0.58	3.433
P-16	125	2,300	DCI	-8.27	0.67	4.522
P-17	125	100	DCI	8.27	0.67	4.523
P-143	100	40	DCI	6.07	0.77	0.000
P-33	150	115	DCI	14.23	0.81	5.082
P-34	150	1,635	DCI	14.23	0.81	5.082
P-145	110	55	DCI	7.77	0.82	0.000
P-32	110	522	DCI	7.77	0.82	7.514
P-141	100	65	DCI	6.73	0.86	0.000
P-88	100	345	DCI	6.73	0.86	9.145
P-142	100	34	DCI	7.13	0.91	0.000
P-40	100	1,200	DCI	7.77	0.99	11.954
P-39	63	60	DCI	-3.28	1.05	22.905
P-37	50	50	HDPE	2.56	1.30	44.773

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

Flex Table: Junction Table
Current Time: 0.00 hours

Label	X (m)	Y (m)	Elevation (m)	Demand (L/s)	Pressure (m H2O)	Pressure Head (m)
J-55	379,060.89	800,883.08	2,143.30	0.13	-55	-55.59
J-42	378,984.50	800,824.89	2,140.79	0.13	-53	-53.08
J-112	378,934.85	800,800.98	2,139.21	0.08	-51	-51.49
J-98	378,940.17	800,923.94	2,139.12	0.13	-51	-51.41
J-115	377,988.53	800,502.78	2,115.00	0.13	-27	-26.99

J-114	377,886.48	800,504.14	2,115.00	0.25	-27	-26.99
J-113	377,862.66	800,625.25	2,111.49	0.13	-23	-23.44
J-34	377,674.50	800,075.69	2,093.04	0.11	9	8.57
J-43	379,632.24	800,906.15	2,169.43	0.13	9	8.86
J-33	378,778.72	802,513.60	2,168.31	0.11	9	9.51
J-116	379,682.16	800,804.92	2,165.86	0.00	10	10.19
J-44	379,613.91	800,881.41	2,166.38	0.13	12	11.92
J-73	378,893.98	802,342.25	2,165.82	0.11	12	12.01
J-22	379,079.58	802,289.46	2,162.10	0.11	15	14.96
J-8	379,338.90	801,174.78	2,162.39	0.14	16	15.98
J-24	379,162.54	802,412.63	2,160.79	0.11	16	16.01
J-20	378,988.41	802,166.52	2,161.24	0.13	17	16.60
J-7	379,294.43	801,127.78	2,160.79	0.13	18	17.61
J-81	378,249.41	800,195.07	2,083.11	0.11	18	18.53
J-49	378,153.56	798,549.69	2,083.34	0.11	19	18.55
J-23	379,219.71	802,513.40	2,155.55	0.14	21	20.99
J-31	379,197.41	801,017.21	2,153.38	0.20	25	24.99
J-36	379,563.12	800,488.63	2,152.72	0.11	26	25.57
J-29	379,254.17	802,592.78	2,150.43	0.13	26	26.00
J-39	379,238.70	801,618.16	2,151.71	0.11	26	26.33
J-51	378,623.09	802,212.32	2,151.04	0.13	27	26.93
J-65	378,689.72	799,899.65	2,072.66	0.13	29	29.00
J-57	379,218.25	800,879.06	2,149.32	0.11	29	29.08
J-48	379,147.70	800,940.99	2,148.70	0.20	30	29.66
J-13	378,850.57	802,044.52	2,143.51	0.13	34	34.43
J-15	379,319.54	802,682.64	2,141.64	0.11	35	34.74
J-83	379,046.64	801,824.78	2,141.14	0.11	37	36.84
J-69	379,102.48	801,333.40	2,140.24	0.11	38	38.06
J-54	379,344.42	800,733.07	2,140.25	0.05	38	38.17
J-38	378,960.33	800,745.04	2,139.11	0.11	39	39.18
J-53	379,404.13	800,665.26	2,137.63	0.08	41	40.76
J-58	378,895.92	800,654.92	2,135.72	0.08	42	42.53
J-127	378,953.11	801,158.81	2,133.43	0.11	45	44.84
J-64	378,489.59	799,726.94	2,056.68	0.11	45	44.99
J-35	377,929.53	799,845.56	2,056.52	0.11	45	45.11
J-14	378,800.25	801,932.26	2,132.26	0.11	46	45.71
J-128	379,138.18	800,540.96	2,132.52	0.11	46	45.81
J-5	378,822.67	800,569.91	2,132.33	0.13	46	45.90
J-85	379,214.26	799,903.22	2,132.31	0.11	46	46.00
J-74	378,788.99	800,987.46	2,131.82	0.13	46	46.40
J-18	377,423.56	799,586.65	2,052.20	0.13	49	49.33
J-103	379,008.13	800,743.06	2,139.35	0.00	50	49.67
J-2	377,392.03	799,559.93	2,049.95	0.08	51	51.43
J-16	379,564.62	802,591.82	2,124.70	0.11	52	51.68
J-129	378,750.25	800,476.18	2,126.41	0.13	52	51.76
J-26	378,593.38	800,627.67	2,125.66	0.13	52	52.50
J-6	378,810.26	800,427.68	2,125.04	0.13	53	53.12
J-1	377,365.16	799,536.61	2,047.99	0.08	53	53.38
J-59	377,347.93	799,516.12	2,046.56	0.05	55	54.72
J-17	379,898.25	804,001.94	2,119.66	0.11	56	56.46
J-50	378,093.99	799,689.26	2,044.80	0.11	57	56.91

J-37	378,204.92	801,409.66	2,119.70	0.05	58	58.48
J-11	379,449.88	802,893.59	2,117.44	0.13	59	58.89
J-25	379,249.64	800,299.14	2,118.71	0.11	59	59.62
J-19	377,569.05	799,459.39	2,038.08	0.11	63	63.54
J-27	378,833.56	801,616.10	2,113.92	0.13	64	64.17
J-66	377,104.66	799,362.29	2,034.99	0.11	66	66.18
J-67	378,305.17	800,637.78	2,109.58	0.14	68	68.47
J-125	377,832.77	800,247.85	2,109.53	0.11	68	68.53
J-4	378,369.77	800,841.61	2,109.23	0.13	69	68.84
J-56	378,425.64	800,938.11	2,107.94	0.11	70	70.25
J-12	379,624.82	802,868.40	2,105.94	0.11	70	70.35
J-10	378,704.81	801,204.49	2,107.70	0.13	70	70.50
J-3	378,354.33	800,859.47	2,107.08	0.13	71	70.99
J-30	378,247.48	800,692.85	2,105.02	0.13	73	73.04
J-71	378,684.76	800,281.47	2,104.29	0.13	74	73.87
J-28	378,757.88	801,349.77	2,103.74	0.11	74	74.43
J-45	378,824.19	800,161.54	2,101.82	0.13	76	76.35
J-46	376,972.97	799,651.65	2,024.43	0.11	77	76.71
J-21	378,874.30	800,107.13	2,100.23	0.14	78	77.94
J-9	379,877.02	803,239.79	2,097.96	0.13	78	78.26
J-32	380,292.37	803,862.89	2,097.24	0.11	79	78.88
J-86	379,966.57	803,430.01	2,096.55	0.11	79	79.59
J-62	375,023.15	799,763.85	2,003.98	0.00	84	84.25
J-41	376,443.03	799,582.79	2,014.13	0.11	87	86.80
J-126	378,333.53	800,278.58	2,089.41	0.11	89	88.70
J-52	376,406.56	799,439.68	2,010.01	0.11	91	90.92
J-40	375,919.51	799,604.82	2,008.54	0.11	92	92.32
J-61	375,303.69	799,685.64	1,997.83	0.11	103	103.00
J-47	375,790.55	799,291.34	1,997.68	0.11	103	103.16
J-63	375,284.34	799,456.21	1,993.35	0.11	107	107.48

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

Flex Table: Pipe Table
Current Time: 8.00 hours

Label	Diameter (mm)	Length (m)	Material	Flow (L/s)	Velocity (m/s)	Head loss Gradient (m/km)
P-40	100	1,200	DCI	0.00	0.00	0.00
P-48	75	3,450	DCI	0.00	0.00	0.00
P-145	110	55	DCI	0.00	0.00	0.00
P-39	63	60	DCI	0.00	0.00	0.00
P-14	125	220	DCI	0.00	0.00	0.00
P-32	110	522	DCI	0.00	0.00	0.00
P-144	120	85	DCI	0.00	0.00	0.00
P-20	75	3,310	DCI	0.00	0.00	0.00
P-9	63	420	HDPE	-0.01	0.00	0.00

P-60	90	475	HDPE	0.05	0.01	0.00
P-85	50	80	HDPE	0.02	0.01	0.00
P-74	75	335	DCI	0.05	0.01	0.00
P-86	100	405	DCI	0.29	0.04	0.01
P-50	63	230	HDPE	0.12	0.04	0.01
P-131	63	610	HDPE	-0.22	0.07	0.09
P-31	110	160	DCI	0.80	0.08	0.02
P-38	50	535	HDPE	0.19	0.10	0.19
P-37	50	50	HDPE	0.19	0.10	0.02
P-59	50	195	HDPE	0.25	0.13	0.12
P-24	150	240	DCI	-2.33	0.13	0.04
P-93	90	750	HDPE	1.10	0.17	0.40
P-46	63	205	HDPE	-0.55	0.18	0.17
P-47	63	195	HDPE	-0.56	0.18	0.17
P-42	75	130	DCI	0.80	0.18	0.09
P-30	75	120	DCI	0.80	0.18	0.09
P-92	63	535	HDPE	-0.60	0.19	0.54
P-36	125	846	DCI	2.50	0.20	0.42
P-87	90	50	HDPE	1.36	0.21	0.04
P-82	75	390	DCI	-1.00	0.23	0.43
P-11	125	260	DCI	2.79	0.23	0.16
P-10	63	590	HDPE	0.71	0.23	0.80
P-156	63	210	DCI	-0.72	0.23	0.29
P-3	63	570	HDPE	-0.73	0.23	0.80
P-146	63	220	HDPE	0.74	0.24	0.32
P-145	75	80	DCI	1.05	0.24	0.10
P-98	63	595	HDPE	-0.77	0.25	0.93
P-95	90	265	HPE	1.57	0.25	0.27
P-21	200	40	uPVC	-7.76	0.25	0.02
P-49	63	625	HDPE	0.84	0.27	0.87
P-107	110	530	HDPE	2.72	0.29	0.57
P-132	63	750	HDPE	-0.94	0.30	1.69
P-69	75	405	DCI	1.36	0.31	0.78
P-94	110	400	HDPE	2.98	0.31	0.51
P-90	100	114	DCI	2.50	0.32	0.17
P-103	75	200	DCI	1.44	0.33	0.43
P-102	90	345	HDPE	2.13	0.34	0.63
p-65	90	595	HDPE	2.29	0.36	1.23
P-45	75	70	DCI	-1.60	0.36	0.18
P-133	75	480	HDPE	-1.66	0.37	1.33
P-148	63	230	HDPE	-1.17	0.38	0.78
P-51	50	1,285	HDPE	0.75	0.38	5.92
P-54	63	685	HDPE	-1.32	0.42	2.93
P-149	75	370	DCI	-1.88	0.43	1.30
P-73	75	240	DCI	-1.90	0.43	0.85
P-89	63	55	HDPE	-1.35	0.43	0.24
P-8	63	450	HDPE	1.35	0.43	2.00
P-2	63	555	HDPE	-1.41	0.45	2.68
P-96	90	345	HDPE	3.00	0.47	1.18
P-101	50	450	HDPE	0.93	0.48	3.11
P-26	125	255	DCI	6.04	0.49	0.64

P-53	63	525	HDPE	-1.56	0.50	2.32
P-75	110	65	HDPE	-4.75	0.50	0.20
P-23	150	300	uPVC	-8.85	0.50	0.63
P-76	100	185	DCI	3.98	0.51	0.64
P-7	63	580	HDPE	-1.61	0.52	3.56
P-147	75	240	DCI	-2.33	0.53	1.25
P-83	75	160	DCI	2.35	0.53	0.85
P-13	125	50	DCI	-6.56	0.53	0.15
P-29	75	230	DCI	2.40	0.54	1.27
P-71	75	250	HDPE	-2.41	0.55	1.39
P-44	110	240	HDPE	-5.25	0.55	0.87
P-15	125	655	DCI	7.11	0.58	2.24
P-100	75	320	DCI	2.65	0.60	2.11
P-84	75	130	HDPE	-2.66	0.60	0.86
P-91	125	145	HDPE	7.39	0.60	0.53
P-137	50	275	HDPE	1.19	0.61	2.97
P-25	150	90	uPVC	-11.22	0.64	0.29
P-63	75	280	DCI	-2.82	0.64	2.07
P-28	75	1,334	DCI	2.88	0.65	10.30
P-52	63	760	HDPE	-2.04	0.66	7.27
P-35	125	195	HDPE	8.11	0.66	0.85
P-12	63	285	HDPE	2.07	0.66	2.79
P-16	125	2,300	DCI	-8.28	0.68	10.43
P-17	125	100	DCI	8.28	0.68	0.45
P-4	63	230	HDPE	2.16	0.69	2.44
P-104	125	1,200	HDPE	8.53	0.69	5.74
P-134	75	295	DCI	3.07	0.70	2.56
P-105	90	1,240	HDPE	-4.43	0.70	8.75
P-56	100	310	DCI	-5.60	0.71	2.02
P-62	50	395	HDPE	1.41	0.72	5.83
P-55	63	400	HDPE	2.26	0.72	4.60
P-97	63	545	HDPE	-2.28	0.73	6.35
P-78	100	265	DCI	5.73	0.73	1.80
P-61	75	280	DCI	3.30	0.75	2.78
P-139	110	260	HDPE	7.11	0.75	1.66
P-143	100	40	DCI	6.04	0.77	0.00
P-80	100	115	DCI	6.04	0.77	0.86
P-70	125	295	HDPE	-9.46	0.77	1.71
P-66	150	560	uPVC	13.68	0.77	2.65
P-106	75	320	DCI	-3.42	0.78	3.40
P-33	150	115	DCI	14.31	0.81	0.59
P-34	150	1,635	DCI	14.31	0.81	8.40
P-77	125	55	HDPE	10.09	0.82	0.36
P-140	125	285	HDPE	-10.16	0.83	1.89
P-58	125	285	HDPE	10.18	0.83	1.89
P-79	100	120	DCI	6.52	0.83	1.04
P-144	63	120	HDPE	2.60	0.83	1.79
P-141	100	65	DCI	6.69	0.85	0.00
P-88	100	345	DCI	6.69	0.85	3.12
P-57	100	980	DCI	6.77	0.86	9.07
P-64	90	230	HDPE	-5.52	0.87	2.44

P-142	100	34	DCI	7.11	0.91	0.00
P-68	90	290	HDPE	5.76	0.91	3.32
P-72	90	120	HDPE	5.92	0.93	1.45
P-135	75	605	DCI	4.12	0.93	9.08
P-22	200	380	uPVC	29.81	0.95	1.87
P-99	90	195	HDPE	6.43	1.01	2.74
P-19	125	360	uPVC	12.83	1.05	3.67
P-81	50	110	HDPE	-2.16	1.10	3.60
P-67	110	195	HDPE	-11.31	1.19	2.93
P-27	75	183	DCI	6.08	1.38	5.62
P-6	75	90	DCI	6.72	1.52	3.34
P-41	75	175	DCI	7.63	1.73	8.21
P-43	75	150	DCI	8.35	1.89	8.31
P-1	63	40	HDPE	-6.08	1.95	2.88
P-18	63	50	HDPE	-7.04	2.26	4.72
P-5	63	160	HDPE	-9.07	2.91	24.17

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Appendix 2.2 Extended period simulation for Junctions (Nodes) at maximum consumption hour

Flex Table: Junction Table

Current Time: 8.00 hours

Label	X (m)	Y (m)	Elevation (m)	Demand (L/s)	Pressure (m H2O)	Pressure Head (m)
J-55	379,060.89	800,883.08	2,143.30	0.80	-72	-71.95
J-42	378,984.50	800,824.89	2,140.79	0.80	-69	-69.42
J-98	378,940.17	800,923.94	2,139.12	0.80	-68	-67.66
J-112	378,934.85	800,800.98	2,139.21	0.48	-68	-67.66
J-23	379,219.71	802,513.40	2,155.55	0.91	-37	-36.83
J-29	379,254.17	802,592.78	2,150.43	0.80	-35	-35.04
J-115	377,988.53	800,502.78	2,115.00	0.80	-34	-34.50
J-114	377,886.48	800,504.14	2,115.00	1.60	-34	-34.42
J-24	379,162.54	802,412.63	2,160.79	0.72	-34	-33.86
J-113	377,862.66	800,625.25	2,111.49	0.80	-10	-29.64
J-15	379,319.54	802,682.64	2,141.64	0.72	-8	-27.71
J-22	379,079.58	802,289.46	2,162.10	0.72	1	-26.86
J-17	379,898.25	804,001.94	2,119.66	0.72	1	-13.91
J-16	379,564.62	802,591.82	2,124.70	0.72	2	-10.92
J-33	378,778.72	802,513.60	2,168.31	0.72	3	-9.62
J-73	378,893.98	802,342.25	2,165.82	0.72	3	-6.84
J-11	379,449.88	802,893.59	2,117.44	0.80	4	-4.89
J-34	377,674.50	800,075.69	2,093.04	0.72	4	-3.49
J-20	378,988.41	802,166.52	2,161.24	0.80	5	-1.84
J-12	379,624.82	802,868.40	2,105.94	0.72	5	5.05
J-43	379,632.24	800,906.15	2,169.43	0.80	6	5.58
J-81	378,249.41	800,195.07	2,083.11	0.72	7	7.07
J-32	380,292.37	803,862.89	2,097.24	0.72	8	8.51
J-44	379,613.91	800,881.41	2,166.38	0.80	9	8.65

J-116	379,682.16	800,804.92	2,165.86	0.00	9	9.14
J-86	379,966.57	803,430.01	2,096.55	0.72	10	10.00
J-9	379,877.02	803,239.79	2,097.96	0.80	11	11.03
J-51	378,623.09	802,212.32	2,151.04	0.80	12	12.17
J-8	379,338.90	801,174.78	2,162.39	0.91	13	12.63
J-39	379,238.70	801,618.16	2,151.71	0.72	14	14.23
J-7	379,294.43	801,127.78	2,160.79	0.80	14	14.43
J-49	378,153.56	798,549.69	2,083.34	0.72	15	15.01
J-65	378,689.72	799,899.65	2,072.66	0.80	18	18.45
J-13	378,850.57	802,044.52	2,143.51	0.80	19	18.83
J-31	379,197.41	801,017.21	2,153.38	1.28	21	21.19
J-36	379,563.12	800,488.63	2,152.72	0.72	22	22.09
J-83	379,046.64	801,824.78	2,141.14	0.72	23	22.91
J-48	379,147.70	800,940.99	2,148.70	1.28	25	25.52
J-57	379,218.25	800,879.06	2,149.32	0.72	26	25.79
J-14	378,800.25	801,932.26	2,132.26	0.72	31	30.95
J-69	379,102.48	801,333.40	2,140.24	0.72	32	32.20
J-2	377,392.03	799,559.93	2,049.95	0.48	32	32.20
J-59	377,347.93	799,516.12	2,046.56	0.32	32	32.57
J-38	378,960.33	800,745.04	2,139.11	0.72	33	33.31
J-35	377,929.53	799,845.56	2,056.52	0.72	34	33.66
J-1	377,365.16	799,536.61	2,047.99	0.48	34	34.01
J-18	377,423.56	799,586.65	2,052.20	0.80	35	34.68
J-64	378,489.59	799,726.94	2,056.68	0.72	35	34.70
J-58	378,895.92	800,654.92	2,135.72	0.48	36	35.66
J-54	379,344.42	800,733.07	2,140.25	0.32	36	35.71
J-53	379,404.13	800,665.26	2,137.63	0.48	38	38.04
J-5	378,822.67	800,569.91	2,132.33	0.80	38	38.19
J-127	378,953.11	801,158.81	2,133.43	0.72	38	38.23
J-74	378,788.99	800,987.46	2,131.82	0.80	39	38.58
J-66	377,104.66	799,362.29	2,034.99	0.72	41	40.81
J-85	379,214.26	799,903.22	2,132.31	0.72	41	41.21
J-128	379,138.18	800,540.96	2,132.52	0.72	41	41.55
J-129	378,750.25	800,476.18	2,126.41	0.80	42	42.32
J-26	378,593.38	800,627.67	2,125.66	0.80	43	42.74
J-6	378,810.26	800,427.68	2,125.04	0.80	44	43.59
J-50	378,093.99	799,689.26	2,044.80	0.72	48	47.82
J-37	378,204.92	801,409.66	2,119.70	0.35	49	49.42
J-46	376,972.97	799,651.65	2,024.43	0.72	50	50.20
J-19	377,569.05	799,459.39	2,038.08	0.72	51	51.53
J-27	378,833.56	801,616.10	2,113.92	0.80	53	52.69
J-41	376,443.03	799,582.79	2,014.13	0.72	54	54.14
J-25	379,249.64	800,299.14	2,118.71	0.72	55	55.32
J-67	378,305.17	800,637.78	2,109.58	0.91	56	55.71
J-125	377,832.77	800,247.85	2,109.53	0.72	56	55.85
J-4	378,369.77	800,841.61	2,109.23	0.80	56	56.24
J-40	375,919.51	799,604.82	2,008.54	0.72	57	57.41
J-3	378,354.33	800,859.47	2,107.08	0.80	58	58.43
J-52	376,406.56	799,439.68	2,010.01	0.72	58	58.52
J-30	378,247.48	800,692.85	2,105.02	0.80	60	60.27
J-56	378,425.64	800,938.11	2,107.94	0.72	61	61.17

J-10	378,704.81	801,204.49	2,107.70	0.80	62	61.83
J-71	378,684.76	800,281.47	2,104.29	0.80	64	64.22
J-28	378,757.88	801,349.77	2,103.74	0.72	65	64.95
J-45	378,824.19	800,161.54	2,101.82	0.80	67	66.87
J-61	375,303.69	799,685.64	1,997.83	0.72	67	67.24
J-47	375,790.55	799,291.34	1,997.68	0.72	68	67.92
J-21	378,874.30	800,107.13	2,100.23	0.91	69	68.69
J-63	375,284.34	799,456.21	1,993.35	0.72	72	71.71
J-126	378,333.53	800,278.58	2,089.41	0.72	78	77.66
J-62	375,023.15	799,763.85	2,003.98	0.00	83	83.49

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Bentley WaterGEMS V8i (SELECTseries 6)
[08.11.06.58]
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Appendix 2.3 Extended period simulation Table for pipes (Links) at minimum consumption hour

Flex Table: Pipe Table
Current Time: 1.00 hours

Label	Diameter (mm)	Length (m)	Material	Flow (L/s)	Velocity (m/s)	Head loss Gradient (m/km)
P-40	100	1,200	DCI	0.00	0.00	0.000
P-48	75	3,450	DCI	0.00	0.00	0.000
P-145	110	55	DCI	0.00	0.00	0.000
P-39	63	60	DCI	0.00	0.00	0.000
P-14	125	220	DCI	0.00	0.00	0.000
P-32	110	522	DCI	0.00	0.00	0.000
P-144	120	85	DCI	0.00	0.00	0.000
P-20	75	3,310	DCI	0.00	0.00	0.000
P-9	63	420	HDPE	0.00	0.00	0.000
P-85	50	80	HDPE	0.00	0.00	0.000
P-60	90	475	HDPE	0.01	0.00	0.000
P-74	75	335	DCI	0.01	0.00	0.000
P-50	63	230	HDPE	0.02	0.01	0.001
P-131	63	610	HDPE	-0.03	0.01	0.004
P-31	110	160	DCI	0.13	0.01	0.004
P-24	150	240	DCI	-0.36	0.02	0.006
P-59	50	195	HDPE	0.05	0.02	0.027
P-38	50	535	HDPE	0.05	0.02	0.028
P-37	50	50	HDPE	0.05	0.02	0.027
P-46	63	205	HDPE	-0.08	0.03	0.023
P-93	90	750	HDPE	0.17	0.03	0.017
P-42	75	130	DCI	0.13	0.03	0.023
P-30	75	120	DCI	0.13	0.03	0.022
P-47	63	195	HDPE	-0.09	0.03	0.031
P-92	63	535	HDPE	-0.09	0.03	0.032
P-87	90	50	HDPE	0.22	0.03	0.027
P-11	125	260	DCI	0.44	0.04	0.019
P-10	63	590	HDPE	0.11	0.04	0.044

P-146	63	220	HDPE	0.11	0.04	0.045
P-156	63	210	DCI	-0.11	0.04	0.044
P-82	75	390	DCI	-0.16	0.04	0.037
P-3	63	570	HDPE	-0.11	0.04	0.045
P-21	200	40	uPVC	-1.21	0.04	0.015
P-98	63	595	HDPE	-0.12	0.04	0.050
P-95	90	265	HPE	0.25	0.04	0.033
P-145	75	80	DCI	0.17	0.04	0.041
P-49	63	625	HDPE	0.13	0.04	0.045
P-107	110	530	HDPE	0.42	0.04	0.034
P-132	63	750	HDPE	-0.14	0.05	0.070
P-23	150	300	uPVC	-0.83	0.05	0.026
P-94	110	400	HDPE	0.46	0.05	0.040
P-69	75	405	DCI	0.21	0.05	0.063
P-103	75	200	DCI	0.23	0.05	0.069
P-102	90	345	HDPE	0.33	0.05	0.059
p-65	90	595	HDPE	0.36	0.06	0.067
P-45	75	70	DCI	-0.25	0.06	0.083
P-133	75	480	HDPE	-0.26	0.06	0.087
P-51	50	1,285	HDPE	0.12	0.06	0.144
P-91	125	145	HDPE	0.74	0.06	0.051
P-73	75	240	DCI	-0.28	0.06	0.102
P-86	100	405	DCI	-0.51	0.06	0.077
P-89	63	55	HDPE	-0.20	0.07	0.135
P-54	63	685	HDPE	-0.21	0.07	0.138
P-149	75	370	DCI	-0.29	0.07	0.112
P-25	150	90	uPVC	-1.18	0.07	0.051
P-8	63	450	HDPE	0.21	0.07	0.143
P-35	125	195	HDPE	0.85	0.07	0.067
P-2	63	555	HDPE	-0.22	0.07	0.155
P-101	50	450	HDPE	0.14	0.07	0.216
P-96	90	345	HDPE	0.47	0.07	0.110
P-148	63	230	HDPE	-0.23	0.07	0.170
P-53	63	525	HDPE	-0.24	0.08	0.142
P-83	75	160	DCI	0.35	0.08	0.158
P-7	63	580	HDPE	-0.25	0.08	0.197
P-13	125	50	DCI	-1.03	0.08	0.095
P-29	75	230	DCI	0.38	0.08	0.177
P-44	110	240	HDPE	-0.81	0.09	0.113
P-71	75	250	HDPE	-0.38	0.09	0.178
P-147	75	240	DCI	-0.39	0.09	0.195
P-84	75	130	HDPE	-0.41	0.09	0.211
P-100	75	320	DCI	0.41	0.09	0.212
P-137	50	275	HDPE	0.19	0.10	0.361
P-63	75	280	DCI	-0.43	0.10	0.227
P-28	75	1,334	DCI	0.45	0.10	0.248
P-52	63	760	HDPE	-0.32	0.10	0.307
P-12	63	285	HDPE	0.32	0.10	0.315
P-134	75	295	DCI	0.47	0.11	0.264
P-56	100	310	DCI	-0.83	0.11	0.191
P-139	110	260	HDPE	1.01	0.11	0.172

P-4	63	230	HDPE	0.34	0.11	0.340
P-104	125	1,200	HDPE	1.33	0.11	0.154
P-105	90	1,240	HDPE	-0.69	0.11	0.227
P-55	63	400	HDPE	0.35	0.11	0.358
P-22	200	380	uPVC	3.56	0.11	0.096
P-97	63	545	HDPE	-0.36	0.11	0.375
P-62	50	395	HDPE	0.23	0.12	0.511
P-80	100	115	DCI	0.95	0.12	0.242
P-70	125	295	HDPE	-1.48	0.12	0.187
P-140	125	285	HDPE	-1.48	0.12	0.187
P-106	75	320	DCI	-0.53	0.12	0.340
P-66	150	560	uPVC	2.14	0.12	0.152
P-78	100	265	DCI	0.96	0.12	0.247
P-61	75	280	DCI	0.57	0.13	0.388
P-58	125	285	HDPE	1.59	0.13	0.214
P-79	100	120	DCI	1.02	0.13	0.278
P-144	63	120	HDPE	0.41	0.13	0.484
P-57	100	980	DCI	1.04	0.13	0.286
P-77	125	55	HDPE	1.62	0.13	0.222
P-64	90	230	HDPE	-0.86	0.14	0.340
P-76	100	185	DCI	1.08	0.14	0.311
P-75	110	65	HDPE	-1.32	0.14	0.282
P-68	90	290	HDPE	0.90	0.14	0.368
P-72	90	120	HDPE	0.93	0.15	0.387
P-135	75	605	DCI	0.67	0.15	0.516
P-99	90	195	HDPE	1.00	0.16	0.452
P-36	125	846	DCI	1.94	0.16	0.309
P-81	50	110	HDPE	-0.34	0.17	1.076
P-67	110	195	HDPE	-1.77	0.19	0.483
P-27	75	183	DCI	0.95	0.22	0.989
P-6	75	90	DCI	1.05	0.24	1.191
P-90	100	114	DCI	1.94	0.25	0.917
P-19	125	360	uPVC	3.10	0.25	0.735
P-41	75	175	DCI	1.19	0.27	1.508
P-43	75	150	DCI	1.30	0.30	1.781
P-1	63	40	HDPE	-0.95	0.30	2.311
P-18	63	50	HDPE	-1.10	0.35	3.033
P-5	63	160	HDPE	-1.42	0.45	4.854
P-26	125	255	DCI	6.06	0.49	2.540
P-15	125	655	DCI	7.11	0.58	3.415
P-16	125	2,300	DCI	-8.27	0.67	4.521
P-17	125	100	DCI	8.27	0.67	4.521
P-143	100	40	DCI	6.06	0.77	0.000
P-33	150	115	DCI	14.24	0.81	5.089
P-34	150	1,635	DCI	14.24	0.81	5.089
P-141	100	65	DCI	6.71	0.85	0.000
P-88	100	345	DCI	6.71	0.85	9.107
P-142	100	34	DCI	7.11	0.91	0.000

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Bentley WaterGEMS V8i (SELECTseries 6) [08.11.06.58]

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

Flex Table: Junction Table

Current Time: 1.00 hours

Label	X (m)	Y (m)	Elevation (m)	Demand (L/s)	Pressure (m H2O)	Pressure Head (m)
J-55	379,060.89	800,883.08	2,143.30	0.13	-56	-55.64
J-42	378,984.50	800,824.89	2,140.79	0.13	-53	-53.12
J-112	378,934.85	800,800.98	2,139.21	0.08	-51	-51.54
J-98	378,940.17	800,923.94	2,139.12	0.13	-51	-51.45
J-115	377,988.53	800,502.78	2,115.00	0.13	-27	-27.04
J-114	377,886.48	800,504.14	2,115.00	0.25	-27	-27.04
J-113	377,862.66	800,625.25	2,111.49	0.13	-23	-23.48
J-34	377,674.50	800,075.69	2,093.04	0.11	9	8.53
J-43	379,632.24	800,906.15	2,169.43	0.13	9	9.14
J-33	378,778.72	802,513.60	2,168.31	0.11	10	9.75
J-44	379,613.91	800,881.41	2,166.38	0.13	12	12.19
J-73	378,893.98	802,342.25	2,165.82	0.11	12	12.25
J-116	379,682.16	800,804.92	2,165.86	0.00	13	12.72
J-22	379,079.58	802,289.46	2,162.10	0.11	15	15.20
J-8	379,338.90	801,174.78	2,162.39	0.14	16	16.22
J-24	379,162.54	802,412.63	2,160.79	0.11	16	16.25
J-20	378,988.41	802,166.52	2,161.24	0.13	17	16.84
J-7	379,294.43	801,127.78	2,160.79	0.13	18	17.83
J-81	378,249.41	800,195.07	2,083.11	0.11	18	18.49
J-49	378,153.56	798,549.69	2,083.34	0.11	18	18.51
J-23	379,219.71	802,513.40	2,155.55	0.14	21	21.23
J-31	379,197.41	801,017.21	2,153.38	0.20	25	25.18
J-36	379,563.12	800,488.63	2,152.72	0.11	26	25.84
J-29	379,254.17	802,592.78	2,150.43	0.13	26	26.24
J-39	379,238.70	801,618.16	2,151.71	0.11	27	26.58
J-51	378,623.09	802,212.32	2,151.04	0.13	27	27.17
J-65	378,689.72	799,899.65	2,072.66	0.13	29	28.96
J-57	379,218.25	800,879.06	2,149.32	0.11	29	29.26
J-48	379,147.70	800,940.99	2,148.70	0.20	30	29.86
J-13	378,850.57	802,044.52	2,143.51	0.13	35	34.67
J-15	379,319.54	802,682.64	2,141.64	0.11	35	34.98
J-83	379,046.64	801,824.78	2,141.14	0.11	37	37.09
J-69	379,102.48	801,333.40	2,140.24	0.11	38	38.28
J-54	379,344.42	800,733.07	2,140.25	0.05	38	38.33
J-38	378,960.33	800,745.04	2,139.11	0.11	39	39.38
J-53	379,404.13	800,665.26	2,137.63	0.08	41	40.96
J-58	378,895.92	800,654.92	2,135.72	0.08	43	42.73
J-64	378,489.59	799,726.94	2,056.68	0.11	45	44.96
J-127	378,953.11	801,158.81	2,133.43	0.11	45	45.05
J-35	377,929.53	799,845.56	2,056.52	0.11	45	45.07
J-14	378,800.25	801,932.26	2,132.26	0.11	46	45.95
J-128	379,138.18	800,540.96	2,132.52	0.11	46	46.01

J-5	378,822.67	800,569.91	2,132.33	0.13	46	46.10
J-85	379,214.26	799,903.22	2,132.31	0.11	46	46.21
J-74	378,788.99	800,987.46	2,131.82	0.13	47	46.61
J-18	377,423.56	799,586.65	2,052.20	0.13	49	49.29
J-2	377,392.03	799,559.93	2,049.95	0.08	51	51.39
J-16	379,564.62	802,591.82	2,124.70	0.11	52	51.92
J-129	378,750.25	800,476.18	2,126.41	0.13	52	51.96
J-26	378,593.38	800,627.67	2,125.66	0.13	53	52.70
J-6	378,810.26	800,427.68	2,125.04	0.13	53	53.33
J-1	377,365.16	799,536.61	2,047.99	0.08	53	53.34
J-59	377,347.93	799,516.12	2,046.56	0.05	55	54.68
J-17	379,898.25	804,001.94	2,119.66	0.11	57	56.70
J-50	378,093.99	799,689.26	2,044.80	0.11	57	56.87
J-37	378,204.92	801,409.66	2,119.70	0.05	59	58.69
J-11	379,449.88	802,893.59	2,117.44	0.13	59	59.13
J-25	379,249.64	800,299.14	2,118.71	0.11	60	59.82
J-19	377,569.05	799,459.39	2,038.08	0.11	63	63.50
J-27	378,833.56	801,616.10	2,113.92	0.13	64	64.39
J-66	377,104.66	799,362.29	2,034.99	0.11	66	66.14
J-67	378,305.17	800,637.78	2,109.58	0.14	69	68.68
J-125	377,832.77	800,247.85	2,109.53	0.11	69	68.73
J-4	378,369.77	800,841.61	2,109.23	0.13	69	69.04
J-56	378,425.64	800,938.11	2,107.94	0.11	70	70.45
J-12	379,624.82	802,868.40	2,105.94	0.11	70	70.59
J-10	378,704.81	801,204.49	2,107.70	0.13	71	70.71
J-3	378,354.33	800,859.47	2,107.08	0.13	71	71.19
J-30	378,247.48	800,692.85	2,105.02	0.13	73	73.24
J-71	378,684.76	800,281.47	2,104.29	0.13	74	74.07
J-28	378,757.88	801,349.77	2,103.74	0.11	74	74.64
J-45	378,824.19	800,161.54	2,101.82	0.13	76	76.55
J-46	376,972.97	799,651.65	2,024.43	0.11	77	76.67
J-21	378,874.30	800,107.13	2,100.23	0.14	78	78.14
J-9	379,877.02	803,239.79	2,097.96	0.13	78	78.50
J-32	380,292.37	803,862.89	2,097.24	0.11	79	79.12
J-86	379,966.57	803,430.01	2,096.55	0.11	80	79.83
J-62	375,023.15	799,763.85	2,003.98	0.00	84	84.20
J-41	376,443.03	799,582.79	2,014.13	0.11	87	86.76
J-126	378,333.53	800,278.58	2,089.41	0.11	89	88.91
J-52	376,406.56	799,439.68	2,010.01	0.11	91	90.89
J-40	375,919.51	799,604.82	2,008.54	0.11	92	92.28
J-61	375,303.69	799,685.64	1,997.83	0.11	103	102.96
J-47	375,790.55	799,291.34	1,997.68	0.11	103	103.12
J-63	375,284.34	799,456.21	1,993.35	0.11	107	107.44
J-103	379,008.13	800,743.06	2,139.35	0.00	311	311.45

Appendix 3

		Phase I				Year phase II	
Description	Unit	2018	2019	2023	2028	2033	2038
Population Growth Rate			4.80%	4.80%	4.05%	3.65%	3.25%
Projected/Forecasted population	No	82300	86250	102954	122411	140909	156027
Population Percentage Distribution by Mode of service							
HTU	%	1.82	2.97	7.57	13.32	19.07	24.82
YTU	%	42.53	43.86	49.18	55.83	62.48	69.13
NTU	%	0.61	0.70	1.06	1.51	1.96	2.41
PTU	%	55.04	52.47	42.19	29.34	16.49	3.64
Total	%	100.00	100.00	100.00	100.00	100.00	100.00
Population served by							
HTU	No	1500	2564	7796	16308	26875	38730
YTU	No	35000	37827	50630	68339	88036	107858
NTU	No	500	602	1089	1845	2758	3756
PTU	No	45300	45258	43439	35919	23239	5683
Per Capita Demand by Mode of Services							
HTU	l/c/d	96	98	106	115	125	134
YTU	l/c/d	48	48	50	53	55	58
NTU	l/c/d	58	59	62	67	72	77
PTU	l/c/d	38	39	41	43	46	48
Domestic Water Demand by Mode of Services							
HTU	m3/d	144.00	251.06	823.29	1,878.72	3,354.01	5,205.32
YTU	m3/d	1,680.00	1,833.86	2,551.75	3,608.29	4,859.61	6,212.60
NTU	m3/d	28.80	35.24	68.04	124.36	199.40	289.94
PTU	m3/d	1,739.52	1,760.49	1,776.48	1,558.58	1,066.41	274.98
Total Domestic Demand	m3/d	3,592.32	3,880.65	5,219.56	7,169.95	9,479.42	11,982.84
	l/s	41.58	44.91	60.41	82.99	109.72	138.69
Socio- Economic Factor		1	1	1	1	1	1
Climatic Factor		1	1	1	1	1	1
Adjusted Domestic Water Demand (ADD)	m3/d	3,592.32	3,880.65	5,219.56	7,169.95	9,479.42	11,982.84
	l/s	41.58	44.91	60.41	82.99	109.72	138.69
Non Domestic Water Demand							
Small Scale Industrial Water Demand (small industries 5% of ADD)	m3/d	-	-	-	-		-
	l/s	-	-	-	-		-
Commercial & Institutional Water Demand with allowance of small scale industry (25% of ADD)	m3/d	898.08	970.16	1,304.89	1,792.49	2,369.86	2,995.71
	l/s	10.39	11.23	15.10	20.75	27.43	20.80
Live stock Water Demand (0% of ADD)	m3/d	-	-	-	-	-	-
	l/s	-	-	-	-	-	-
Total Demands	m3/d	4,490.40	4,850.81	6,524.45	8,962.44	11,849.28	14,978.55
	l/s	51.97	56.14	75.51	103.73	137.14	159.49
Unaccounted for Water (UFW) "non-revenue-water" UFW (15-25% of TAD)	m3/d	673.56	751.88	1,141.78	1,792.49	2,666.09	3,744.64
	l/s	7.80	8.70	13.22	20.75	30.86	39.87
UFW (15-25%)		15.00	15.50	17.50	20.00	22.50	25.00
Average Day Water Demand	m3/d	5,163.96	5,602.69	7,666.23	10,754.93	14,515.36	18,723.19
	l/s	59.77	64.85	88.73	124.48	168.00	199.37
Max Day Factor		1.2	1.2	1.2	1.2	1.2	1.2
Max Day Demand	m3/d	6,196.75	6,723.22	9,199.48	12,905.91	17,418.44	22,467.82
	l/s	71.72	77.82	106.48	149.37	201.60	239.24
	m3/hr	258.20	280.13	383.31	537.75	725.77	936.16
Peak Hour Factor		1.80	1.80	1.80	1.80	1.80	1.80
Peak Hour Demand	m3/d	11154.15	12101.80	14719.16	20649.46	27869.50	35948.52
	l/s	129.10	140.07	170.36	239.00	322.56	382.79

Appendix 4

Questionnaires for Domestic Household Survey in Durame Town

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

Questionnaire No: ________Name of Interviewer: ______________________________Date of interview: __________

I. Personal Information of Respondents

1. Name of Kebeke/mender:______________________

2. House No: ______________

3. Sex: Male______ , Female_______

4. Age: Under 14 years_____, 15-39 years______, 40-64years______, above 65 years_______

5. Educational background: None____, Read-Write_____, Elementary school______, Secondary school______, High School______, College______, Graduated______, Higher education_____, Others ________,

6. Occupation: Government Sector_____, Private Sector______, Retired_____,

other (specify)_______________________

7. How many persons live in your household__________.

8. How long have you been living in this town? _________years.

Water Sources and Uses

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

A. Piped with Household taps D. River/pond

B. Private dug well E. Rainwater collection

C. Private water seller F. Other sources (specify)_______________________.

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

A. In own house

B. In own yard/plot

C. In neighbor's house

D. Other, specify__________________________________.

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

Yes No

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

A. Low quantity of the water

B. Low reliability of the scheme

C. Long distance to the scheme

D. Delay for maintenance

E. Scheme is non-functional

F. Other, specify__________________________.

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

A. High (quick) E. Low (slow)

B. Generally high F. Every time low

C. Sometime high G. Very low

D. Sometime low H. Every time no water

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

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

A. Water source problem C. Reservoir fails

B. Pipe break D. Pump failure

E. Other/specify__________________________________.

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

A. During road construction C. During electric line installation

B. During telephone line installation D. During building construction

E. When it gets old F. Other (specify)____.

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

A. Once a day E. 4 to 5 days a week

B. Twice a day F. 1 to 2 weeks a Month

C. 1 to 2 days a week G. Other, specify__________________________.

D. 2 to 3 days a week

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

A. Neighboring house C. Water vendors

B. Pond/river D. Well/borehole

E. Other, specify _______________________________.

11. Is there water loss in distribution system in Durame town?

Yes NO

a) Do you see Pipe breakage in distribution system?

Yes NO

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