ASSESSMENT OF THE WATER DISTRIBUTION NETWORK PERFORMANCE OF DURAME TOWN WATER SUPPLY SYSTEM

MASTER OF SCIENCE THESIS

TESFAHUN MATHEWOS

HAWASSA UNIVERSITY, HAWASSA, ETHIOPIA

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This is to certify that the thesis entitled "Assessment of the Water Distribution Network Performance of Durame Town Water Supply System" submitted in partial fulfillment of the requirements for the degree of Master's with specialization in Water Resource Engineering and Management, the Graduate Program of the Faculty of Bio-Systems and has been carried out by Tesfahun Mathewos Id. No WREM/009/10 School of under our supervision. Therefore we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

Kannan Narayanan (PhD)	___________________	__________________
Name of Major Advisor	Signature	Date
Mihret Dananto (PhD)	___________________	___________________
Name of Co-Advisor	Signature	Date

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DECLARTION

I hereby declare that this Thesis is my original work and has not been presented for a degree in any other university, and all sources of material used for this thesis have been duly acknowledged.

Name: Tesfahun Mathewos

Signature: ______________ Date: _________________

This Thesis has been submitted for examination with my approval as Thesis advisors.

Kannan Narayanan (PhD)	__________________	_________________
Name of Major Advisor	Signature	Date
Mihret Dananto (PhD)	__________________	__________________
Name of Co-Advisor	Signature	Date

Place and Date of Submission: Hawassa University,

LISTS OF ABBREVIATIONS

a.m.s.l	above mean sea level
ADD	Average Day Demand
AWWA	American Water Works Association
BAC	Billed Authorized Consumption
BH	Bore Hole
CSA	Central Statistics Agency
CWSS	Community Water Supply Systems
CAPL	Current Annual Volume of Physical Losses
DN	Diameter Nominal
DP	Distribution Point
DCI	Ductile Cast Iron
DTWSS	Durame Town Water Supply Service
EWT	Elevated Water Tank
EFC	Environmental Finance Center
EPA	Environmental Protection Agency
EELP	Ethiopian Electric Power Corporation
FDRE	Federal Democratic Republic of Ethiopia
GRP	Glass Reinforced Plastic
GPS	Global Position System
GLR	Ground Level Type
HU	Hawassa University
HDPE	High Density Polyethylene
HC	House Connections
IMF	International Monetary Fund
IWA	International Water Association
ITCZ	Inter Tropical Convergence Zone
KMG	Kembati Ment Gezima
MDD	Maximum Day Demand
MAAPL	Minimum Achievable Annual Physical Losses

MoWIE	Ministry of Water, Irrigation and Electric
MWR	Ministry of Water Resources
NRC	National Research Council
NPSH	Net Positive Suction Head
NRW	Non-Revenue Water
O&M	Operation & Maintenance
PHD	Peak Hour Demand
PRV	Pressure Release Valves
PYT	Private Yard Tap
PF	Public Fountain
PSP	Public Stand Pipes
RWPS	Raw Water Pumping Station
RC	Reinforced Concrete
SEI	Significant Environmental Impact
SNNPRS	Southern Nations Nationalities and Peoples Regional State
SYT	Shared Yard Tap
RSWW	Standard for Water Works
UARL	Unavoidable Average Real Losses
UPVC	Universal Polyvinyl Chloride
USAID	United States Agency for International Development
WDNS	Water Distribution Networks System
WME	Water Mine and Energy
WSP	Water Supply and Sanitation
WUC	Water User Committee
WUB	Water Users Board
WUAM	Water Utility Asset Management
WHO	World Health Organization
WLCC	Water Loss Control Committee
YTU	Yard Tap Users
ZWMED	Zonal Water Mine and Energy Department

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ABSTRACT

The research *work was focused on the assessment of water supply distribution network performance of Durame Town, which is the Zonal Town of Kembata Tembaro Zone in Southern Nation Nationality People Regional State of Ethiopia. The main objective of this thesis was to* assess *Durame Town water supply system by analyzing water demand and supply, water distribution network, water losses and leakage management of the pipe system in the service. To examine the hydraulic performance of the water distribution network system, water GEMS modeling was adopted. The model was used to identify the zone of high pressure and low pressure in junctions and the level of velocity through pipe system. The model run was performed for average velocity, peak demand and low demand to analyze the system model. Furthermore the model analysis result shows different problems of aged pipes, oversized and undersized pipes, high pressure during night flow and low pressure during day flow. As per the analyzed results; the current maximum water demand in Durame town is estimated at 6392.304m^3/day, while small reservoirs capacity and low pump efficiency were observed in the town water distribution networks and the maximum pressure occurred at night demand is 85m which is above the recommended value. Further, the analyzed water losses result in Durame Town indicates that about 32.58% of production is Non-Revenue Water. Thereby, apparent losses cover 1.76% of total loss and 30.81% were physical losses. Accordingly, for the case of Durame Town apparent losses are less significant, however physical losses contributed unconsidered volume of lost. In general, high demand of water, small* capacities of *existing infrastructures and large volume of water loss lead to intermittent water distribution in Durame Town. Therefore, it is significant to rehabilitate and improve the , water demand and supply system and providing more attention for the causes of water losses, and leakage management of the Town.*

Keywords: *Water GEMS Model, water demand and supply system, water loss and leakage management, Durame Town, Ethiopia.*

1. INTRODUCTION

1.1 Background

Water is an essential and life sustaining natural resource and is critical for the survival of all living organisms, food production and economic development. One of the principal roles of public work is providing water in sufficient quantity to users. The water uses are increasing with enormous rapidity; whereas the supply is static. Problems in providing satisfactory water supply to the rapidly growing population especially that of the developing countries is increasing from time to time. The sustainable provision of adequate and safe drinking water is the most important of all public services (Dessalew et al., 2017).

Water is the basic and most fundamental resource for humans, animals and plants, all are dependent on it. The water uses are increasing with enormous rapidity; whereas the supply is static (Hockensmith, 2006). ‘All peoples, whatever their stage of development and social and economic condition, have the right to have access to drinking-water in quantities and of a quality equal to their basic needs’. Water is the precious gift of nature, the source of prosperity, most crucial for sustaining life, basic to most economic activities and its role in human survival and health is well known (WHO, 2014).

Water supply and distribution is a complex system and that exists to satisfy the various needs of peoples. Whereby, it consists of various components of physical assets including reservoirs, pipes, pumps, and different hydraulic controlling accessories that make up the water distribution system. It is generally desired that water should be supply continuously in the required quantities with adequate pressure and flow from sources to all customers. However, occasional disruptions due to failures of their system components and variation of demands may occur over the service life (Jalal et al., 2008).

Problems with access to sufficient water are most happen in the developing world, and more than one billion people were suffer without access to water for their basic needs. Thereby, the United Nations Millennium Declaration and the plan of implementation of the world; was set reducing the proportion of people having without adequate access to water by one-half for the year 2019. Hence, adequate water distribution is one of the international goals for sustainable development (Renwick et al., 2013).

Most water distribution system across the world was built decades ago, and many are reaching their expected life spans within 30 years (NRC et al., 2006). Accordingly, in developing countries; one of the commonly cited constraints to effective water provisioning is the “aging infrastructure” problem. And these were presents many technical limitations for effective and continues water distribution system to customers (Citilin A. Gady, 2014).

Almost all over the developing world Intermittent piped water networks were found widely. From that it is estimated one third of urban water supplies in Africa were operated intermittently. As result of; fast population growth rate, scarcity of water sources, treatment plant size, reservoirs and storage tank capacity, power outages to run water pumps, high leakage problems, or some combination of these conditions were the primary causes for intermittent water distribution in the water system (Renwick et al., 2013).

According to statistical surveys Non-Revenue Water in developing countries is around 45 to 50% i.e. half of the total system input volume. This is largely because most of the water utilities do not have enough attention and monitoring systems within water losses and its management systems. Further; metering error, water theft, and lack of effective data recording and handling systems is the other problems of the water utility in developing countries. Accordingly, the water distribution system in such countries does not meet the need of water for various demands since high levels of water losses in their distribution networks (Dighade et al.,2014).

Large quantity of water is also losses through leaking pipes, joints, valves and fittings of the distribution systems. Age of the installations, bad quality of materials used, and/or poor workmanship are the main sources of these water losses. Therefore, Constraints to water access also include limitations directly related to the aging, operation and maintenance capacity of the water services (Grady et al.,2014).

Water utilities in many developing countries are struggling to ensure that customers to be receive a reasonable supply of adequate drinking water. But, problems related with less engineering aspects, low level of technology and costs associated with water provisioning were lead to poor management and controlling of the water system including non-revenue water. Thus, the water tariff systems and revenue collection policies were not reflect the true value of water supplied, which limits the utility's cost recovery and encourages customers to undervalue the service (Malcom Farley, 2008).

In general, water problem is a growing global concern and that has an impact on countries' economic prospects. Rising water stress, large supply variability, and lack of access to safe and adequate drinking water are a frequent problems in many parts of the world. Especially, developing countries face greater challenges of adequate water distribution because of their institutional capacity, poor infrastructure, lower income levels, less developed policy and larger population growth (Fund, 2015).

According to Durame Town water supply service office, one of the common problems in the town water supply system was related with intermittent supply due to the current performance of the water distribution system. Thereby, there were inadequate amount of water supply and low coverage problems in the town. Therefore, this research work was prepared to assess Durame Town water supply and demand system, water distribution network in terms of the hydraulic performance, water loss and leakage management practice.

1.2 Statement of the Problem

Most of the water infrastructure in developing world including Ethiopia is in service for decades and can be a significant source of water loss through leaks. That means, water can be lost through improper management, unauthorized consumption (water theft), data handling errors, metering inaccuracies/failure and administrative error practices. Sufficient amount of water is a precondition for health and development and a basic human right; yet it is still denied to hundreds of millions of people throughout the developing world.

Almost, in all developing countries water supply system the problem of intermittent water distribution is a growing concern. One of the major indicators of intermittent water distribution is the capacities and configuration of system components. Reservoirs capacity, pump size, pipeline and pipeline network schematization has a negative impact on quantity of water distribution in a system. One of the major challenges on reduction of the performance of towns' water supply system is the demand on water increases due to the growth of population and urbanization of the town. In urban areas around Durame Town majority of householders consume their total water needs from the town's water supply system either directly through private connections or public taps.

According to Durame Town water supply service office, existing water supply system has served beyond its design period and currently there is intermittent water distribution in the town. The level of water loss in towns' water distribution system depends not only on aging of the infrastructure, but also the quality of material used, workmanship, and customers' awareness and attitude towards water.

In addition to water supply and demand in the Town, again water leakage management practice is a great problem. Therefore; this study paper was prepared to address the current performance of Durame town existing water distribution network.

1.3 Objective

1.3.1 General objective

The general objective of this work was to assess the hydraulic performance of Durame Town water distribution network systems and to improve the knowledge on the management and performance evaluation of the supply system.

1.3.2 Specific objective

The specific objective includes:

The demand and supply system assess by modeling

- To assess the demand and supply by modeling
- To assess the major factors of water loss in the distribution network system, and
- To assess the town water utility leakage management practice

1.4 Research Questions

- What is the demand and supply coverage balance of the town and major factors for intermittent water supply experienced in Durame Town water distribution network?
- How much water is lost in the system comparing with system production?
- How are the town water supply service office; plan, policy and strategy for leakage management practice?

1.5 Scope of the Study

This study specifically focuses on presenting the fundamental concept of hydraulics applied to Durame Town water supply network, in order to support municipal officials of the town for to a better evaluation and decision making of water distribution and delivery systems.Therefore, the research work was limited to assess the water distribution network (from clear water well to distribution end point) of Durame Town water supply system in South region of Ethiopia.

It mainly focus and was assess to identify the hydraulic performance and the factors for intermittent water supply, water loss, leakage management strategy and water supply service of the town water distribution system. This was achieved with hydraulic modeling, water loss analysis, and by made of discussion with the town water utility personnel to gather relevant information in the subject area. But, due to lack of enough budgets, occurrence of covid-19/corona virus, chemical reagents, resources and distance of the study area from laboratory these research exclude the water quality analysis in the distribution system of the study area in the town.

1.6 Significance of the Study

The existing source of Durame Town water supply system is from Hambericho spring and from different Bore Holes surrounding in the Town. It is significant to provide the status of the water distribution network systems and providing more attention to water losses, leakage reduction policies and strategies are vital for remedial measures. Better management decision should be taken to change the current water losses and to minimize the weakness problem in high leakage management of the Durame Town water distribution network system.

1.7 Outline of the Studies

The studies were divided in to five chapters. Chapter one provides the brief introduction of the study, statement of the problem and objective of the research. Chapter two deals with brief explanation of literature review. Detail description of the study area and data set and data sources for software model were included in chapter three and four respectively. The fourth chapter explains result and discussion part. Conclusions and recommendations are included in chapter five.

2. LITERATURE REVIEW

2.1 Introduction

In water distribution system, the reliability of water with a constant flow rate should be available to customers throughout the design time. If water is not available in sufficient quantities it should be pumped for a short period of time and at high flow rate, to meet the various demand of customers. Accordingly, service reservoir/storage tanks usually provided in order to store water when the pumping rate is higher than the demand at low/night times. But, this can be also used in the case that the pumping rate is below the needed demand, since to equalize the pressure in the network (Hussni & Zyoud, 2003).

In developing countries; many water authorities are facing the challenges of uniform distribution system by providing adequate water supply to the rapidly growing populations'. Thereby, most of the existing water supply systems are unable to meet the various demands of water. Beside to this; infrastructural aging problem, poor management of the existing system components/assets and utilities capacity shortages were increases the level of water losses in the distribution system (Welday, 2005; Jalal, 2008).

According to (Tomas, et al., 2003), the water distribution network can be classified as Looped and Branched systems.

2.1.1 Branched water distribution system

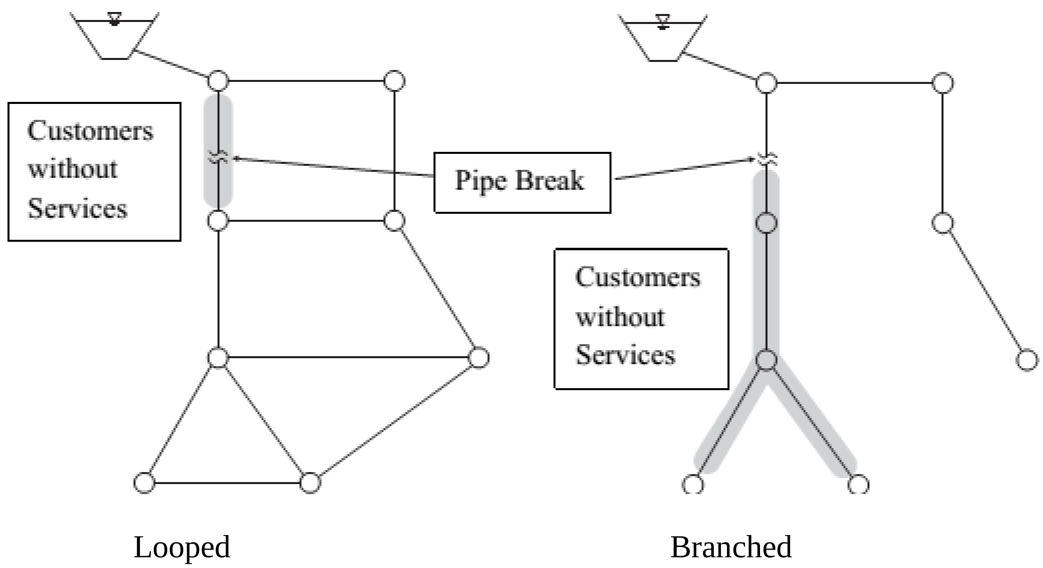
Branched water distribution system is also called a tree system. The water has only one possible path from the source to a customer. Thereby, these are applicable for small capacity water suppliers, and are common in most developing countries. The advantage of branched systems is the most economical because of its low cost, but it has some disadvantages as presented below;

- ❖ Low reliability, affects all users especially located downstream of any breakdown in the system. So that, their water services were interrupted until the repairs are finished.
- ❖ Fluctuating in water demand, producing rather large pressure variations in the system.

- ❖ When there is a need for developing the network, new branches follow that development and new dead ends will be constructed. It also danger of contamination during the network without water.

2.1.2 Looped water distribution system

Looped water distribution system serves different paths that water can follow to get from the source to a particular customer. The systems are generally more desirable than branched systems because it coupled with sufficient valves and accessories, and can provide reliability in the water distribution. In these systems because of more than one path for water, the system capacity is greater and it improves the hydraulics of the distribution system. For example, consider a main break occurring near the reservoir in each system depicted in Figure 2.1 below. In the looped system, that break can be isolated and repaired with little impact on customers outside of that immediate area. While, the effects of water service interruption is more significant to branched system.



(Source: Advanced water distribution modeling and Management: Haestad Methods)

Figure. 2. 1: Looped and branched networks after network failure

2.2 Factors Causing Loss of Hydraulic Integrity in Water Distribution Network

'In most of the developing regions, the design of water distribution systems is based on the assumption of direct supply, although most of these systems are intermittent systems which result in severe supply, insufficient pressure in the distribution system (pressure losses in several areas in the network), inequitable distribution of the available water and very short duration of supply. However, the purpose of hydraulic integrity in the water distribution system is to supply water at adequate or acceptable pressure and flow (Zyoud, 2003).

2.2.1 Low pressure

The water distribution system is one of the most important and vital systems that cannot be dispensed with. The problems of low pressure in the water supply system is common problem faced from long time and affect the quality of water distribution system in most countries. The main cause of this problem is improper design for the network, the design consider that the water continuous pumping for 24 hours while the fact said adverse that (Yahia et al., 2018).

The pressure loss by the action of friction at the pipe wall and its magnitude also dependent on the water demand, properties of the fluid that is passing through the pipe, the speed at which it is moving, the internal roughness of the pipe, pipe length, gradient and diameter of the pipe. Such situations may occur where there are: properties on high ground, remote properties at the end of long lengths of pipe, demands that are greater than the design demand, pipes of inadequate capacity or too small diameter, rough pipes. For example corroding iron pipes or pipes with a build-up of sediment and equipment failures such as pumps and valves. Finally, the poor pressures tend to be caused by inadequate capacity in a pipe or pump, high elevations, or some combination of the two' (Chambers et al., 2004).

Therefore, one of the most hydraulic integrity is maintaining adequate water pressure inside the pipe. Hence, the water utilities should be achieving a high degree of hydraulic integrity through a combination of proper system design, good monitoring, operation and immediate maintenance (Zyoud et al., 2003).

2.2.2 High pressure during low demand conditions

High pressure during low demand conditions can cause pipe bursting, leakage and large amount of water losses through the distribution networks. Therefore, when dealing with high pressures, PRVs should be used to reduce and regulate pressure in the system (M. D. Tomas et al., 2003).

Accordingly, pipes and pumps must be sized to overcome these problem and to provide acceptable pressure in the system. Although, sizing of control valves based on the desired flow conditions and pressure differential is vital (NRC et al., 2006).

2.2.3 Pump capacity

A pump is device in which mechanical energy is applied and transferred to the water as total head, and these head is a function of flow rate through the pump (Donald, 2003). While, 'the familiarity, location, size and capacity of pumps in water distribution are the major impacts for low flow or negative pressure raised in the system, and this can lead to intermittent water supply in the distribution system' (Chambers et al.,2004).

There are many reasons and factors why a pump is not performing well in a certain situation of water distribution system. But, as per Marta & Rudolf, 1987; the important and possible reasons to less performing of pumps were identified as below;

- When the pump is of poor design and quality.
- If it is not suitable for the given situation and does not work in its optimal range.
- If the pump is not being used properly and maintained regularly (cleaning, greasing, etc.)
- If the pump is excessively exposed to sun, rain, dust, etc.
- If it is overused and was not repaired properly after a break-down and
- If supply of spare parts is difficult.

2.2.4 Demand increase

In developing countries rising water demand as a result of population growth and urbanization has an effect on the reliability and availability of existing water distribution system. Therefore, 'water demands need to be assessed on the basis of considering the year and date supplying water through the distribution system. The primary objective is to make sure that the community is being serviced adequately. If there are deficiencies in meeting current or future goals because of population growth, this needs to be identified for the areas of the community where there may be inadequate flows to meet customers consumption during peak hour water demand of the day' (Hickey, 2008).

2.2.5 Poor infrastructures

Currently in most of the developing world it has been observed that pipe network is very old and which is laid many years ago. With aging problem there is considerable reduction in carrying capacity of the pipelines. Although, most of the distribution pipeline were get corroded and leakage were occur, since resulting in loss of water and pressure reduction. Hence, ‘All these materials suffer from degradation over time and result in leakage in the network.

It is therefore, preventive maintenance of distribution system assures and providing conditions for adequate flow through the pipelines. Incidentally, this will prolong the effective life of the pipeline and restore its carrying capacity. Some of the main functions in the management of preventive maintenance of pipelines are assessment, detection and prevention of loss of water from pipelines through leaks, maintaining the capacity of pipelines, cleaning of pipelines and relining’ (Dighade, 2014).

2.2.6 Operation and maintenance activities

‘Water distribution systems are occasionally subject to emergencies or planned maintenance activities in which certain components become not workable and the system can no longer provide the minimum level of service to customers. Planned maintenance activities include supplies going off line (e.g., reservoir shutdown for inspection, cleaning, or repairs; installation of new pipe connections; pipe rehabilitation or break repairs; and transmission main valve repairs). All these activities can result in a reduction in system capacity and supply pressure, and changes to the flow paths of water within the distribution system’ (Council, 2006).

Therefore, lack of attention to the important aspect of operation and maintenance of water supply schemes were leads to deterioration of the useful life of the distribution systems. Further, as Dighade, et al., 2014; some of the key issues contributing to the poor operation & maintenance have been identified as follows;

- Lack of funds, operation manuals and real time field information
- Inappropriate system design and poor workmanship,
- Overlapping responsibilities and inadequate training of personnel,

- Inadequate emphasis on preventive maintenance,

Whereby, there is a need for clear policies, legal framework and decision of responsibilities and mandates within the water supply authority.

2.3 Basic Principles of Hydraulic Modeling

In line with Jalal, 2008; the main reason for modeling a system is to assist designers, managers and planners to explore the governing laws of such systems and to accurately analyze their behavior. Hence, models are employed to resolve problems in system's design and operation. 'Model-based simulation is a method for mathematically approximating the behavior of real water distribution systems.

To effectively utilize the capabilities of distribution system simulation software and interpret the results produced, the modeler must understand the mathematical principles involved' (Tomas, et al., 2003). 'In networks of interconnected hydraulic elements, every element is influenced by each of its neighbors; the entire system is interrelated in such a way that the condition of one element must be consistent with the condition of all other elements. These conditions are mainly controlled by two laws' (Tomas, et al., 2003).

2.3.1 Law of conservation of mass

Several formulations of conservation of mass and energy can be written for a water distribution system under steady conditions (Boulos Lansey, 2004). Here the pipe flow equation formulation is summarized. For a junction that connects two or more pipes, conservation of mass is written as:

$$\sum_{pipes} Qi - U = 0 \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (2.1)$$

Where Q = inflow to node in I in pipe (L_3/T)
U = water used at node (M^3/T)
During extended-period simulations; a term to the accumulation of water at certain nodes are considered, because water can be stored and withdrawn from storage tanks (Tomas, et al., 2003).

$$\sum_{pipes} Q - U - \frac{s}{dt} = 0 \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (2.2)$$

Where $\frac{ds}{dt}$ = change in storage (M^3/T)

Therefore, the concept to conservation of mass is applied to all junction nodes and tanks in a water distribution networks.

2.3.2 Law of conservation of energy

The Energy equation is known as Bernoulli's equation. 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 changes in energy are referred to as head gains and head losses. In hydraulics, energy is converted to energy per unit weight of water, "head".

The equation for conservation of energy is written in terms of head as follows: Within a hydraulic analysis, the equation is written in terms as follows:

$$Z1 + \frac{P1}{\gamma} + \frac{vi2}{2g} + \sum hp \; = Z2 + \frac{\boldsymbol{P2}}{\boldsymbol{\gamma}} + \frac{v}{2g} + \sum h\,\mathrm{L} + \sum h\,\mathrm{m} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots .2.3)$$

Where Z= Elevation (M)

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

γ = Fluid Specific weight ($M/L/T^2$)

V = Velocity (L/T)

g = gravitational acceleration Constant (M/T^2)

hp = head loss in pipes (M)

hp = head loss due to minor losses

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

2.3.3 Water distribution network simulation

'The term simulation generally refers to the process of imitating the behavior of one system through the functions of another. It can be used to predict system responses to events under a wide range of conditions without disrupting the actual system. Using simulations, problems can be anticipated in proposed or existing systems, and can be evaluated before time, money, and materials are invested in a real-world project' (Tomas et al., 2003).

As per Tomas, et al., 2003; in water distribution networks the most basic type of model simulations are either steady-state or extended-period simulation.

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

B. Extended-period simulations: are determined the dynamic behavior of a system over a period of time, and it analyze the system on assumption that the hydraulic demands and boundary conditions were change with respect to time. Hence, 'extended period analysis used to evaluate system performance over time and allows the user to model pressures and flow rates changing, tanks filling and draining, and regulating valves opening and closing throughout the system in response to varying demand conditions and automatic control strategies formulated by the modeler. Therefore, regardless of project size, model-based simulation can provide valuable information to assist an engineer in making well-informed decisions' (Tomas et al., 2003).

2.4 Types of Water Distribution System Models

- Epanet Software
- Water CAD Software model and;
- Water GEMS Software model

2.5 Water GEMS: Modeling Capabilities

Bentley Water GEMS is a comprehensive and easy to use water distribution modeling application. Its objective is; to study the Water GEMS software, to enhance our technical skills and tounderstand practical implementation of Pipe network analysis in broad area. It's used for:

- ❖ Building a water-distribution network and performing steady state as well as extended period simulations
- ❖ Water Quality Analysis
- ❖ Network Design
- ❖ Model Calibration
- ❖ Network Design Optimization

Water GEMS provides and allowing modeling practically for any distribution system aspect. Therefore, working with Bentley Water GEMS used as for decision-support tool for water infrastructures and were help to assess and/or operate (Bentley Water GEMS V8i (Select Series 5) software: user manual, 2008).

The hydraulic analysis at a steady-state or an extended-period simulation

- ❖ Pressure, flow and demands in the system and to see how behaves over time,
- ❖ The size of pipes, pump and computer system head curves,
- ❖ Tank, pump and valve behavior in the system,
- ❖ Leakage and water loss from the network

Calibration the model either manually or use the Darwin Calibrator methods and, generate fully customizable in graphs, charts and reports form.

2.5.1 Input data for assembling the model

In practice, pipe networks consist not only of pipes, but composed of vary fittings, services, storage tanks and reservoirs, meters, regulating valves, pumps, and electronic and mechanic al controls. For modeling purposes, these system elements were organized into the following categories (Bentley Water GEMS V8i (Select Series 5) software: user manual, 2008).

Table 2.1: Input parameters and primary purposes of water GEMS tools

Element	Type	Primary Modeling Valves	Input Data
Reservoir	Node	Provides water to the system	Hydraulic grade line (water surface elevation)
Tank	Node	Stores excess water within the system and releases that water at times of high usage	Base Elevation, Max. Elevation, Min. Elevation, and Diameter
Junction	Node	Discharge the demand require d or recharge the inflow water from/to the system	Elevation
pipe	Link	transport water from one node to another	Elevation, Diameter, Material and Roughness coefficient
pumps	Node/ Link	Provide energy to the system a nd raise the water pressure to o vercome elevation differences and friction losses	Elevation, Pump definition (Characteristics of max. operati on and design discharge and he ad efficiency)
Valves	Node/ Link	Controls flow or pressure through a pipe and results in a loss of energy in the system	Elevation, Diameter, Valve type,

(Source; Bentley Water GEMS: user manual, 2008).

2.6 Water Demand Modeling

The first question in the design and operation of WDN is: How much water is needed? The answer to this question is difficult because the required water is a function of various factors. While, some of the factors are completely independent and time varying. Therefore, water demand modeling is one of the most important challenges in the design of WDN, since it reflects the changes in population, climate, land use, the number of service connections and customer life style (Jalal et al., 2008).

2.6.1 Demand modeling approaches

In the water distribution system, there are two main approaches for water demand modeling (Jalal et al., 2008).

A. Deterministic water demand estimation: In this approach, the actual water demand for all users is estimated based on predicted water consumption over the service time. One simple approach for deterministic water demand is estimating individual needs based on type of customers and their activities and finally adding these lead to get total water demand. For example, the water demand can be estimated on the basis of per capital demand in small urban areas (Jalal et al., 2008).

B. Stochastic demand forecasting: this method mostly considers and adopts the uncertain fluctuations on demand over time and location spans. Risks and sensitivity of forecasts such as the consequence of total loss of supply and the effect of variations in rates income should be considered and included. Hence, demand estimation based on historical consumption per user category (domestic, industrial/commercial) and expected changes (increasing or decreasing) in user category over the forecasting period is good example of stochastic demand forecasting (Jalal et al.,2008).

2.6.2 Variations in water demand

The per capital demand of a particular town is the average consumption of water for a year. In practice it has been seen that this demand does not remain uniform throughout the year, but it various from season to season, even hour to hour (Venkateswara et al., 2005).

A. Seasonal Variation: Water demand varies from season to season. In dry season the water demand is maximums, because the people will use more water for bathing, cooling, lawn watering and street sprinkling. While, demand will becomes minimum in rainy/wet season because less water is used in bathing and there is no lawn watering. Therefore, 'maximum day water demand is considered to meet water consumption changes with seasons and it used to size source, treatment plant and rising mains. Hence, maximum day demands can be obtained by multiplying the average-day demands to the peaking factor applied to the node' (Venkateswara, et al., 2005).

$$Qmax = PF * Qavg \quad (2.4)$$

Where Qmax = Max day demand (cfs, m^3/s)

PF =Peaking factor between maximum day and average day demand

Q avg = Average day demand (cfs, m^3/s)

B. Daily Variation: This variation mainly depends on the general behavior of people, climatic conditions and character of city as industrial, commercial or residential. More water demand is on Saturday and holidays due to more comfortable bathing, washing etc. as compared to other working days. Accordingly, 'Average daily water demand is the sum of the domestic, non-domestic and NRW which is used to estimate the maximum day& the peak hour demand' (Venkatswara et al.,2005).

It expressed as economic calculations over the projects lifetime.

$$Q_{avg} = \text{Per Capital water consumption} * \text{Total population of the town} \ldots \quad (2.5)$$

Where, Qavg = Average day demand (cfs, m^3/s)

C. Hourly Variation: In most developing countries the maximum hour water demand is happen during morning and evening time over 24 hour, because in these time most people use water for bathing, washing and cooking purpose. Therefore, 'peak hour demand is the highest demand of any one hour over the maximum day. And it represents the hourly variations in water demand resulting from the behavioral patterns of the local population' (Venkateswara et al., 2005).

$$Q_{hr} = PF * Qavg \ldots \quad (2.6)$$

Where, Q hour = Peak hour demand (cfs, m^3/s)

PF = Peaking factors between maximum hour and average day demand

Qavg = Average day demand (cfs, m^3/s)

2.6.3 Baseline demands

'The most common method of allocating baseline demands is a simple unit loading method. This method involves counting the number of customers (hectares of a given land use, number of fixture units, or number of equivalent dwelling units) that contribute to the demand at a certain node, and then multiplying that number by the unit demand (for instance, number of gallons/liters per capital per day) for the applicable load classification' (Tomas et al., 2003).

Therefore, average day demands were used to estimate the baseline demand and other demand in the water distribution system including unaccounted-for water. Hence, most modelers determine the water demand analysis of a given town by applying baseline demand to a variety of peaking factors and demand multipliers (Bhadhade et al., 2009).

2.6.4 Demand diurnal pattern and multipliers factors

‘The variations in water usage for water supply systems typically follow a 24-hour cycle. However, in reality, water demand varies over time and for extended period simulation to reflect the dynamics of the real system, these demand fluctuations must be incorporated into the model and it requires both baseline demand data and information on how demands vary over time. These demands can be determined by applying a multiplication factors or a peaking factor. Multiplication/ Peaking factors from average day to maximum day tend to range from 1.2 to 3.0, and factors from average day to peak hour are typically between 3.0 and 6.0. Of course, these values are system-specific, so it must be determined based on the demand characteristics of the system at hand’ (Tomas et al., 2003).

Therefore, When more than one demand type is served by a particular junction, the total demand for a junction at any given time is equal to the sum of each baseline demand times with its respective pattern multiplier, and it is used in most software packages to assign a different pattern to the different components of the composite demand as per below (Tomas et al., 2003).

$$Q\ i,t\ = \sum_k Bi,j\ Pi,j,t \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.7)$$

Bi,j = Baseline demand for demand type j at junction I (cfs, m^3/s)

P i,j,t = Pattern multiplier for demand type j at junction i at time t.

2.7 Model Calibration and Validation

‘Model calibration is the process of fine-tuning a model until it simulates field conditions for a specified time horizon to an established degree of accuracy’. Fine-tuning includes making minor adjustments to the input data to achieve the desired output data’ (Gregory, 2002). Therefore, model will not be hundred percent correct and to be calibrating it must be accurately simulate the observed data. So that, calibration is a major portion of modeling process and proper calibration were achieved through accurate field data. Further, according to Tomas, et al., 2003; hydraulic model calibration is the necessary process of modeling and it is calibrated in order to have better confidence, understanding and identifying errors made during the model-building process.

2.7.1 Pressure calibration

Collecting pressures data throughout the water distribution system used to indicate the level of service. Pressure readings are done using pressure gauge commonly taken at pump stations, storage tanks, reservoirs, fire hydrants, home faucets, air release and other types of valves. However, different factors can contribute to deviation between model simulation and actual field data.

Therefore, 'calibration can be accomplished by adjusting only internal pipe roughness values or estimates of nodal demands until an agreement between observed and computed pressures and flows is obtained. The basis for this claim is that unlike pipe lengths, diameters, and tank levels, which are directly measured, pipe roughness values and nodal demands are typically estimated, and thus have room for adjustment' (Tomas et al., 2003).

2.7.2 Acceptable levels of calibration

According to (Toamas et al., 2003), 'regardless of which approach to calibration is adopted a realistic model should achieve some level of performance criteria. Accordingly, outlines the criteria for pressure through extended-period simulations has been established'.

2.7.2.1 Pressure criteria

A. 85% of field test measurements should be within ± 0.5 m or ± 5 % of the maximum head loss across the system, whichever is greater.

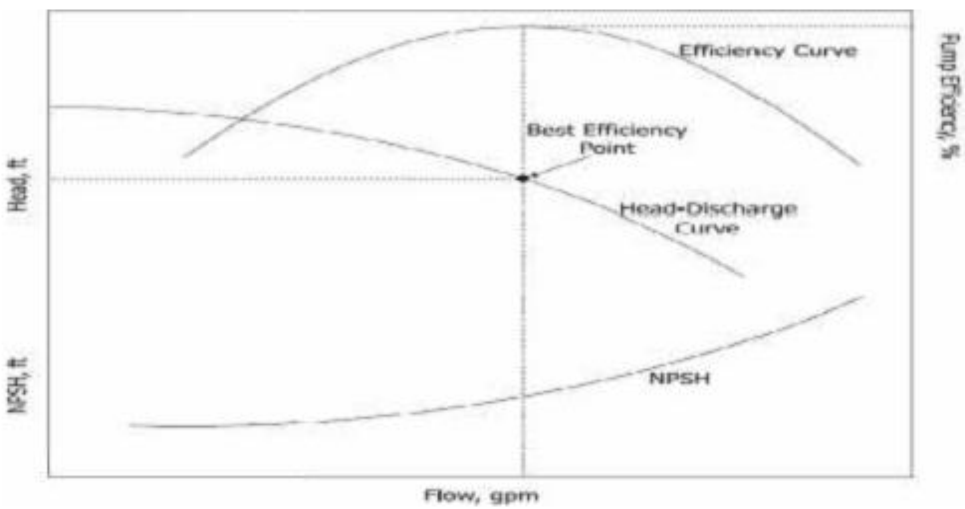
B. 95% of field test measurements should be within ± 0.75 m or ±7.5 % of the maximum head loss across the system, whichever is greater.

C. 100% of field test measurements should be within ± 2 m or ± 15 % of the maximum head loss across the system, whichever is greater.

2.8 Pump Capacity Test

Pump is a device that adds energy to the system in the form of increasing hydraulic grade to water. In water distribution systems, the most frequently type of pump is the centrifugal pump. Accordingly, the performance of these centrifugal pumps is a function of flow rate, and is described by the following four parameters listed as below (Tomas et al., 2003).

Head: Total dynamic head added by pump in units of the length.
Efficiency: Overall pump efficiency (wire-to-water efficiency) in units of the percent.
Brake horsepower: Power needed to turn pump (in power units) and
Net positive suction head (NPSH): Head above vacuum (in units of the length) required to prevent cavitation.



(Source: Advanced water distribution modeling and Management: Haestad Methods)

Figure 2.2: Pump efficiency curve.

'Typically, only the head characteristic curve is needed for modeling; however, some models determine energy usage at pump stations as well as flow and head. To determine energy usage, the model must convert the water power produced by the pump into electric power used by the pump. This conversion is done using the efficiency relationships summarized below' (Tomas et al., 2003).

$e_p = (Water\ Power\ out|Pump\ Water\ in)$ … … … … … … … … … … … … … … … …. (2.8)

Where, WP = Water power, watts

Q = flow rate, l/s

HP = head at pump, m

C = Specific weight of water, 9810 N/m^3 and

Cf = Units conversion factor, 0.001 for S

2.9 Water Loss In Distribution Network

Water losses occur in all water distribution networks, even new one and it is only the volume that varies. Thereby, the volume of these losses reflects the capacity of water authorities to manage their distribution networks (Dighade et al., 2014).

In general, ‘water losses consist of real and apparent losses. And to most water utilities , the level of Non-revenue Water (NRW) is a key performance indicator of efficiency. Utility managers should use the water balance to calculate each component and determine where water losses are occurring. By quantifying NRW from the water balance concept, volumes of lost water into system can be calculate and they will then priorities and implement the required policy changes and operational practices which lead to the proper understood and take the required actions’ (Bentley Water GEMS: user manual, 2008).

Therefore, the water balance can guides water loss estimation in the distribution system while also indicating the level of accuracy of the Non- Revenue Water calculation.

Table 2.2: Water balance showing NRW components; IWA water loss task force

<table>
<tr><td rowspan="8">System input volume</td><td rowspan="4">Authorized Consumption</td><td rowspan="2">Billed Authorized Consumption</td><td>Billed metered consumption</td><td rowspan="2">Revenue water billed unmetered</td></tr>
<tr><td>Billed unmetered consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorized Consumption</td><td>Unbilled metered consumption</td><td rowspan="3">Nonrevenue Water</td></tr>
<tr><td>Unbilled unmetered consumption</td></tr>
<tr><td>Water losses</td><td>Apparent losses</td><td>Unauthorized consumption
Customer meter inaccuracies and data handling error</td></tr>
<tr><td rowspan="3"></td><td rowspan="3">Physical/Real loss</td><td>Leakage on mains (Transmission and Distribution)</td><td rowspan="3"></td></tr>
<tr><td>Leakage and overflow from utility storage tank</td></tr>
<tr><td>Leakage on service connection up to the customer meter</td></tr>
</table>

(Source; Bentley Water GEMS: user manual, 2008)

2.9.1 Non-Revenue water

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

$$NRW = System\ Input\ Volume - Billed\ Authorized\ Consumption \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (2.9)$$

Unaccounted-for-water also expressed as a percentage and, has generally evaluated as the amount of water produced minus the metered customer use divided by the amount of water produced and multiplied by 100 (EPA, 2010).

$$Unaccounted\ for\ water = \frac{(Water\ Produced - Metered\ Water\ Used)}{Water\ Produced} * 100 \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (2.10)$$

2.9.2 Physical / Real loss

‘Physical losses, sometimes called ‘real losses’, are the annual volumes lost through all types of leaks, bursts, and overflows on mains, service reservoirs and service connections up to the point of customer metering. So, utility managers must be verifying the physical loss assessment of town’s water distribution system’ (Farley et al., 2008).

2.9.2.1 Leakage from transmission and distribution mains

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

2.9.2.2 Leakages from storage tanks and reservoirs

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

2.9.2.3 Leakage on service connections up to the customer's meter

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

2.9.3 Apparent or Commercial loss

Commercial loss referred to as apparent loss. Apparent losses can amount to a large volume of water than physical losses and often have a greater value, since reducing apparent losses increases revenue, whereas physical losses reduce production costs. For any profitable utility, the water tariff will be higher than the variable production cost and sometimes up to four times higher. Thus, even a small volume of apparent loss will have a large financial impact' (Farley et al., 2008).

Commercial loss also consists of unauthorized consumption, all types of metering inaccuracies and data handling errors. It also includes water that is consumed but not paid by the users (Farley et al., 2008).

In the developing countries, metering inaccuracies (mainly under recorded problem) and illegal users of water within the distribution system is the common problem of water losses. Whereby, they contribute large coverage to apparent losses, so the level of these losses was one of the significant concerns in developing country water distribution systems (Dighade et al., 2014).

2.9.4 The infrastructure leakage index (ILI)

2.9.4.1 Performance indicator for physical loss

As per Farley, et al., 2008; The Infrastructure Leakage Index (ILI) is an excellent indicator of physical losses. Thus, the International Water Association (IWA) developed the index, and the American Water Works Association (AWWA) and Water Loss Control Committee (WLCC) were recommending this indicator. Therefore, ILI described as the ratio of Current Annual Volume of Physical Losses (CAPL) to Unavoidable Annual Real Loss (UARL).

$$ILI = \frac{CAPL}{UARL} \qquad (2.11)$$

Where, the ILI has no units and thus facilitates comparisons between utilities and countries that use different measurement units. According to IWA, Unavoidable Annual Real Loss (UARL) is also called the Minimum Achievable Annual Physical Losses (MAAPL); and its formula have been converted to a format using pre-defined pressure for a practical use as follow (Farley, 2008).

In general, the concept of Infrastructure Leakage Index is identifying how well a distribution network is managed to control physical losses. Therefore, according to Farley, et al., 2008; the ILI target matrix shows the expected level of physical losses of countries at differing levels of network pressure. Hence, the water utility experts can use the matrix to guide further network development and improvement.

Table 2.3: Physical loss target matrix; World Bank Institute

Technical performance category	Rank	ILI	Physical Losses (Liters/connection/day) (When the system is pressured) at an average pressure of:				
			10	20	30	40	50
Developing Countries	A	1-4	<50	<100	<150	<200	<250
	B	4-8	50-100	100-200	150-300	200-400	250-500
	C	8-16	100-200	200-400	300-600	400-800	800-1000
	D	>16	>200	>400	>600	>800	>1000

(Source; from Farley, et al., 2008)

Category-A (Good): Further loss reduction may be uneconomic and careful analysis needed to identify cost-effective improvements.

Category-B (Potential for marked improvements): Consider pressure management, better active leakage control, and better maintenance.

Category-C (Poor): Tolerable only if water is plentiful and cheap, and even then intensify NRW reduction efforts.

Category- (Bad): The utility is using resources inefficiently and NRW reduction programmers are imperative.

2.10 Non-Revenue Water Management and Controlling Methods

Non-Revenue water control program should be flexible and modified to the specific needs and characteristics of town's water supply system. Accordingly, there are three major components to an effective water loss controlling program (EPA, 2010): Water Audit is an assessment of the distribution, metering, and accounting operations of the water utility and uses accounting principles to determine how much water is being lost and where. This method used to compare and evaluate with consideration of all issues and concerns the public water system faces.

Typical steps in water audit include; determining flows into and out of the distribution system based on estimates or metering, calculating the standard performance indicator values and assessing water loss standing by comparing these values with ranges of values from audits from other water utilities, Assessing where water losses appear to be occurring based on available metering and estimates, Analyzing data gaps (e.g., determining if more information is necessary to make comparisons and an informed decision), considering options and making economic benefit comparisons of potential actions, and selecting the appropriate interventions.

Intervention is a process puts the options selected into action. While, more than one action may be selected and it should be based on budget constraints, public benefit, and priority of other scheduled capital improvements.

Therefore, intervention can include:

- Metering assessment, testing, or a metering replacement program,
- Detecting and locating leaks,
- Repairing or replacing pipe, and maintenance programs and changes, administrative processes or policy changes.

Evaluation is part of the method consists of identifying the success of the audit and intervention process. The evaluation should answer questions such as:

- How often should we repeat the Audit, Intervention, and Evaluation process?

- ❖ Is there another performance indicator we should consider?
- ❖ How did we compare to the last Audit, Intervention, and Evaluation process?
- ❖ How can we improve performance?

2.11 Challenges of Non-Revenue Water

- ❖ Addressing NRW is the responsibility of managers across the water utility, including finance and administration, production, distribution, customer service, and other departments. But, most of developing countries have the problem of infrastructure and establishing operational procedures to begin tackling NRW. Hence, water utilities face greater challenges including (Farley 2008);
- ❖ Outdated infrastructure
- ❖ Poor operations and maintenance policy, including ineffective record-keeping systems inadequate technical skills and technology
- ❖ Greater financial constraints, including unsuitable tariff structure and or revenue collection policy
- ❖ Political, cultural, and social influences
- ❖ A higher incidence of commercial losses, particularly illegal connections,
- ❖ Withdrawing water supply and environmental pollution

2.12 Causes of Water Loss

Leakage is usually the major source of water loss in developing countries, but this is not always the case in developing or most of developed countries, where illegal connections, customer meter reading inaccuracy, unauthorized consumption and, data handling and accounting errors are often more significant and the reasons of total water losses in the water distribution system (Farley, 2008).

2.13 Leakage Management in Water Distribution Network

All water supply networks are made of assets, these are the physical components of the system and it includes; pipe, reservoirs, pumps, valves, hydrants, and other components that make up the system (EFC, 2006). Good asset management involves a combination of technical, financial and engineering practices while achieving the least cost and risk over the life cycle, and meeting service standards for customers (WUAM et al., 2013).

In general, urban water distribution networks are one the most valuable parts of the public infrastructure. The water utilities should be trusted with the responsibilities of managing and expanding them for current and future generations (Coelho et al., 2013).

2.13.1 Infrastructure asset management

'Infrastructure asset management is one of a key topic towards compliance with the performance requirements in water supply and distribution networks. Systems that fully adopt infrastructure asset management principal can achieve many of benefits' (Alegre 2013) & (Coelho et al, 2013). Therefore the water organizations need to put plans, policies, and strategies for sustainable management of the systems, and they should be responding to the need for;

- Improving the sustainable use of water and energy
- Promoting adequate levels of service and strengthening long-term service reliability,
- Managing service risk, taking into account users' needs and risk acceptance,
- Extending service life of existing assets instead of building new, when feasible,
- Upholding and phasing in climate change adaptations, and
- Improving investment and operational efficiency in the organization and Justifying investment priorities in a clear, straightforward and accountable manner.

2.13.2 Leakage controlling strategy

Many of developing country water utilities face the challenge of controlling their unacceptably high levels of Non-Revenue Water. Accordingly, large volume of this total water loss lead to increasing the cost of delivered water, and money for investment and productivity (WUAM, 2013).

Therefore, to reduce these losses the water utility should consider;

- Expand distribution networks to manage with demand growth
- Setting maintenance plan and strategy in order to reduce physical failures,
- Financing for the timely replacement and maintenance of existing assets,

- Identifying the life span of assets, and
- Raise their service levels 24 hours a day and 7 days a week, to meet the required outlook

3. MATERIALS AND METHODS

3.1. Description of the Study Area

3.1.1. Location

Durame is the Zonal Town in Southern Regional State of Ethiopia, which located between 7°14′82'' N latitude and 37°53′89’’E longitude with an elevation of 2101 m.a.s.l. The boundaries of Durame Town are, rural peasant associations of Dega-Kedida in the North, Bezena Benara in the South, Teza-Gerba in the East and Zato Shodera in the West directions, and it covers a total area of 55.59 km^2 with estimated total population of 83000 (2011 and 93,390 in 2012, Durame Town Municipality). The town is located about 275 kms from Addis Ababa, which is situated on the Addis Ababa-Hulberag-Halaba-Durame Town main road.

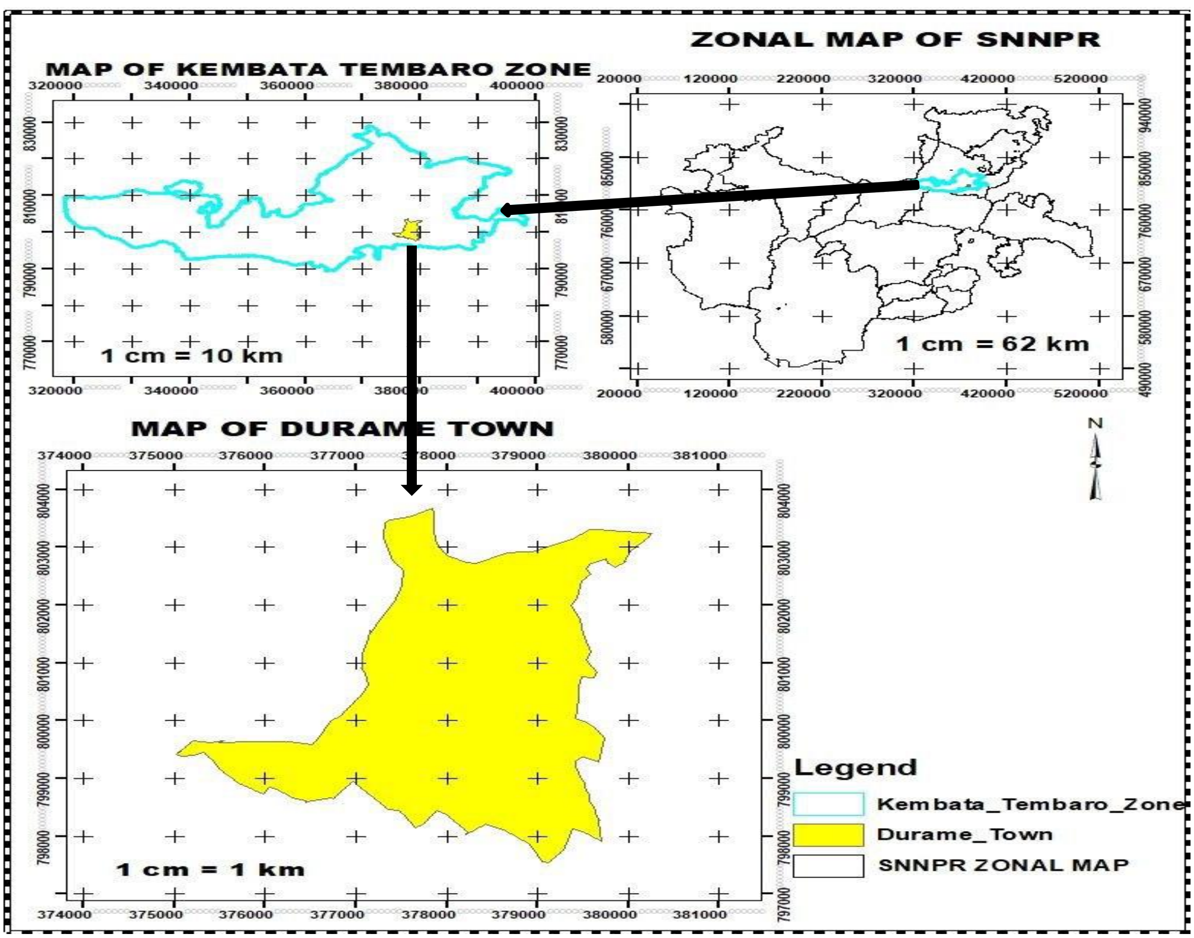


Figure. 3. 1: Location of Durame Town

According to the Ethiopian Central Statistical Agency (CSA), the last census population data of Durame Town was 24,472 for the year 2007 G.C. Durame Town is 275 km far from Addis Ababa (the capital of the country), accessed through Addis Ababa_Butajira_Halaba Durame road. It can be accessed from Hawassa via the road Hawassa_Shashemene_Alaba_ Durame road; 125 km.

3.2 Climate

3.2.1. Temperature

The climate in the study areas is influenced mainly by altitude. Influence of the altitude, the pressure and air flow pattern determine the tremendous difference in climate of the area. The altitude of Durame Town ranges between 1500 m to 2800 m. a. s. l and the specific study area has elevation range of 2000 to 2500 m. a. s. l. According to the map from National Meteorology Service Agency the daily mean temperature (°C) in the study area ranges from (17.5-27 °C.)

Temperature generally decreases with an increase in altitude; in contrast rainfall increase with an increase in altitude. The climate of the area in general fall under the temperate climatic zone or based on the Ethiopian Traditional way of agro-climatic zoning it can be grouped as weina dega climatic zone, Ethiopian meteorological service agency, 1976 and Bayissa.A, 2003 (table 3.1).

Table 3.1: Mean annual temperature with altitude, from Bayisa, (2003).

Altitude (m. a. s. l)	Mean annual Tempe. (°C)	description
Above 3,300	10 or less	Cool
2,300-3,300	10-15	Cool Temperate
1,500-2,300	15-20	Temperate
500-1,500	20-25	Warm Temperature
Below 500	25 and above	Hot

(Source: from Bayisa.A, 2003)

Table. 3. 2: Adjustment due to climatic conditions

Mean annual Temp.(°C)	Description	Altitude	Factor
<10	Cool	>3300	0.8
10--15	Cool Temperate	2300--3300	0.9
15--20	Temperate	1500--2300	1
20--25	Warm Temperate	500--1500	1.3
25 and above	Hot	<500	1.5

(Source: from Kembate Tembaro Zone Geo-Spatial annual report, 2020)

3.2.2 Rainfall

Ethiopia receives rainfall mainly from two sources: - From the Atlantic equatorial westerly's and from the southerly and easterly Indian Ocean air currents. The regional variation and amount of rainfall is generally determined by two factors: -Direction of moisture is bearing seasonal winds and Elevation (Altitude).

In the area of study mainly two rainy seasons are experienced:-

1. **Small Rains**:-Last from February to May during which air currents bring rains from the Indian Ocean. In the areas of study mean annual rain fall gradually decreases toward the east. According to the Cumulative Mean Annual Rainfall (mm) map of National Meteorolog y Service Agency the mean annual Rainfall in the study area ranges from (800-1381) mm.

2. **The main rainy season this is Keremt**:-Last from June to September during which the Atlantic equatorial westerly's provide the main rain fall to the western highlands of the area. This summer rainfall show July and August peak.

3.2.3 Topography and Drainage

The topography of Kembata Tembaro Zone is particularly characterize by highest elevated Hmbericho Mountain of 3058 m above the mean sea level and with varies undulating topographic situations in its surrounding and plain topography in lowlands.

While, Durame Town is the zone town in which the average elevation of the study area is 2000 m with maximum peaks reaching up to 2500 m above the mean sea level. The area is characterized by dissected plateaus, undulating landforms, deep gorges and plains towards lowlands. In rural area the surface drainage system of the area is due to uplifted Hmbericho Mountain, where radial drainage system is formed and radiate to different direction from mountain. The south western parts of mountains are drains towards Omo river basin and the rest parts drains towards Billate river basin. But its urban drainage system is managed according to the plan and policy of Ethiopian urban drainage ruling system.

3.3 Socio-Economic Activity of the Town

3.3.1 Economic activities

Durame is one of economically important town in the region. Zingible and Coffee is the main foreign currency earning cash crop, and they covers large area in and around the town. While, various categories of manufacturing and trade services at varying scales are the main sources of livelihood of the residents of the town. These operations mainly consist of activities like hotel services, fuel stations, garages, coffee hulling and pulling, woodworks, metal works, grain mills, shop keeping, grain and chat retailing, sales of charcoal and fire woods, and many unclassified other trades.

Table 3.3: Socio-Economic factors adapted from 34 towns water supply project

Group	Description	Factor
A	Towns Enjoying high living standards and with very high potential for development	1.1
B	Towns having a very high potential for development but lower living standards at present	1.05
C	Towns under normal Ethiopian conditions	1
D	Advanced Rural Towns	0.9

(Source: from Socio-Economic activity of 34 towns water supply project, 2008)

3.4 Description of Existing Water Supply System

During my assessment of the water supply System as per information is obtained from Durame Town water service office, the design of Durame Town the first small spring in (2001G.C) and six bore hole in (2006 G.C) was constructed by World Vision Ethiopia and by Federal Water Fund before 19 and 14 years ago respectively. Currently, house holders collect their total water needs from this water supply system and from additional new drilled functional bore holes using directly through public taps and private connections.

3.4.1 Water potential source

The study area is characterized by two main rainy season's which is kiremt and Small Rains are experienced. Due to this reason the town is not widowed with adequate flow of streams. But, the small spring near Hambericho mountain and other six drilled bore holes from the other available surroundings sources in the area that can be utilized as a source of water supply for the town. So, the current potential source of water for Durame Town is from Hambericho spring and the currently functioned drilled bore holes.

The first spring was developed in 2001(G.C) at Zato Shodera Kebele. According to the Bore hole data, currently functioned are drilled in 2006 (G.C) by World Vision Ethiopia at Teza Gerba kebele yielding 7, 8.5, 5 3.5, 7.1 and 5.8 l/sec and but the other currently not functioned additional 4 boreholes are drilled by Federal Water Fund in a year 2015 (G.C), but out of the four bore holes one is filled with iron content. Currently, the only potential surface water source used for Durame Town water supply system is the Hambericho small spring from east location and the drilled bore holes in out of the town from different locations of the Durame Town. From the existing sources analysis, the current functioned and in addition to new future served sources we have a total source to be considered within this study have a total yield of 38.9 l/sec.

Table 3.4: Summary of water supply existing potential sources parameters

Sources	Location	Q (l/s)
Spring 1	Zato Shodera kebele near Hmbericho mountain	2
BH1	Bezena Benara Soda area	7
BH2	Bezena Benara kebele Gashara area	8.5
BH3	Wota field-1	3.5
BH4	Wota field-2	7.1
BH5	Wota field-3	5.8
BH6	Near Zato Shodera kebele	5
Total yield		38.9

(Sources: from Durame Town water supply existing sources, 2020)

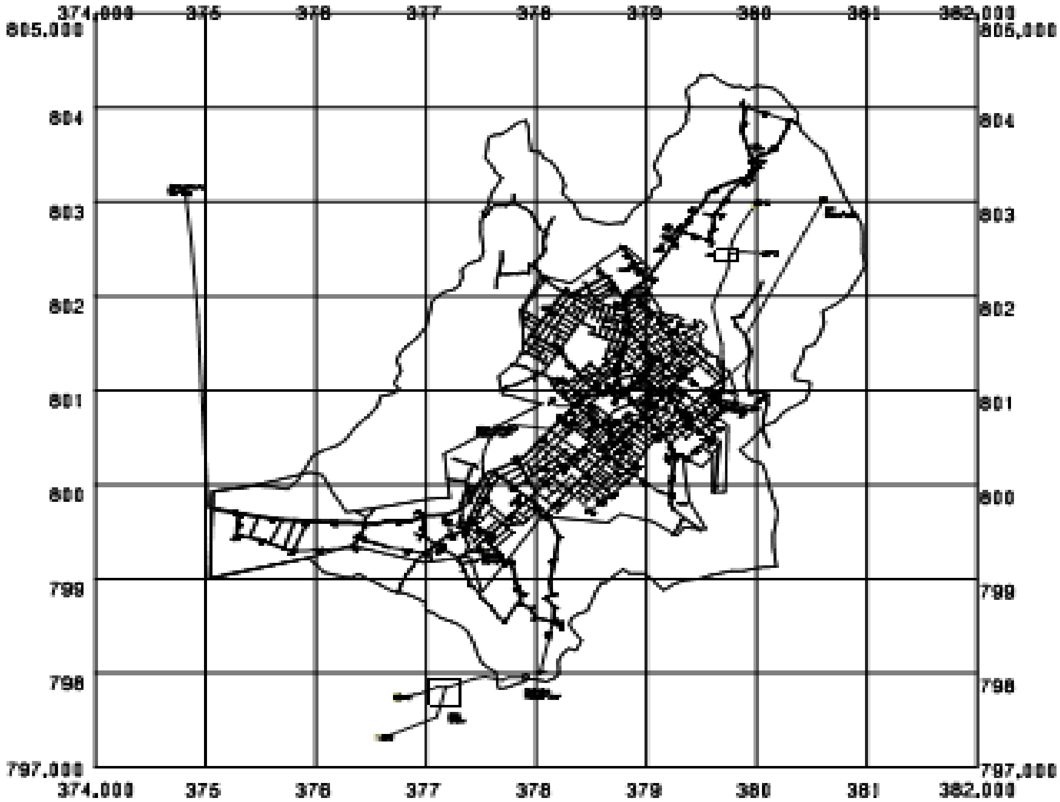
The analysis is depending on the total yield from each source and the served population of the town. For this analysis and modeling of Durame Town Water Supply system the population size is based on the data of Ethiopian Central Statistic Agency's (CSA-2007) population data as a base and which updated by SNNPRS. The population size of Durame Town is increased too much fast in between (2019 GC) and (2020 G.C). This is due to the fact that the inhabitants from the nearby kebele (Teza Agara, Abonsa, Fulasa Deketa & Bezena Benara) migration & inclusion of part of the kebeles in to the towns master plan, migration from other regions for investment, settlement and unexpected institutions like Durame Campus and Industrial Zones in 2018 G.C.

3.4.2 Components of water supply system

3.4.2.1 Transmission system (spring and boreholes)

Water flowed from the spring by gravity is transmitted through DN75 DCI pipe up to 50 m^3 sandwich masonry service reservoirs located relatively at elevated area of the town near Durame Preparatory School and the total length of the DN75 DCI transmission pipe is also 3,339 m. The transmission main of near Gidano River BH1 is 125 mm DCI pipe and a total length of 1,635 m up to KMG site 100 m^3 masonry sandwich reservoir.

The two BH of Wota field-1 and wota field-2 are collected by DN150 uPVC up to booster station site and then pumped to using pipe of length 1667 m up to 300 m^3 Concrete service reservoir at KMG site. Bezena Benara kebele Gashara area BH4 is pumped to Booster station site using DN100uPVC pipe and then pumped to Danshe site 300 m^3 Concrete service reservoir using DN 125 uPVC pipe of length 2,064 m. Bezena Benara Soda site Transmission is 4” GI pipe up to Dongicho Health Center reservoir site.



(Source: from Kembata Tembaro Zone WME Design of, 2019)

Figure 3.2: Layout of Durame Town water distribution system

3.4.3 Overall analysis of water demand and supply coverage

A water supply system capable of supplying sufficient quantity of portable water is necessary for city or town. In order to estimate as correcting as possible, the total demand of a particular community, all demands must consider. Generally speaks in design the water supply scheme. For a town, it is necessary to determine the total quantity of water required for various proposes.

Some of the factors that affect water demand are:-

- **Climatic condition:-**Water consumption during winter is more than summer. During winter no rain and more water used for all activities and also consumption of drinking and bath increased.

- **Size of the town:-** Generally, the demand of water per head will be more on big city than that in small city. In big cities lot of water is required for maintaining clean and health environments while in small towns more or less small.
- **Culture of people:-** High class community uses more water due to their better standard of living and high economic status. Middle class people uses water at average rate and for poor people a single water tap may be sufficient for sever al families.
- **Cost of water: -** If cost of water is high, the water demand will be less .Hence the rate at which water is supplied to consumer may affect the rate of demand.
- **Quality of water:-** A water work system having good facility and portable water supply will be more popular with consumers.
- **Industries: -** more water is used in highly industrial city.
- **Pressure in the distribution system: -** There would be of great importance in the case of localities having number of two or three storied buildings. Adequate pressure would mean an uninterrupted and constant supply of water.
- **System of supply:-** The system of water may be continuous or intermittent. In continuous system water is supplied all 24 hours. While in the case of intermittent system water is supplied for hours of the day only results in some reduction in the consumption. This may be due to decrease in loss and other waste of full use.
- **Method of charging: -** In a town where meters are used less quantity of water will be used than in towns without meters in their system. A metered supply ensures minimum of waste as the consumer because of they know there is payment as per consumption.

Water consumption for various purposes is divided under the following categories.

- Domestic demand
- Public demand
- Commercial & industrial demand
- Fire demand
- System loss
- Institutional water demand

- ❖ Animal water demand

3.4.3.1 Domestic water demand

It includes water finished to in- house purpose such as drinking, cooking ablution washing u tensil, washing clothe, washing toilets, watering animal. The amount of water used for domestic purpose varies depending on the life style living standard, climate mode of service and all the price of water and affordability of users.

3.4.3.2 Demand computation by mode of service

The following are the most common mode of domestic service in Ethiopia and are used in Durame Town.

1. House connected tap users (HTU)
2. Yard connected user (YU)
3. Yard shared connected user(YSU)
4. Public tap uses (PTU)

- ❖ Average demand by mode of service

According to the current standard of the country the third level towns like Durame have the following per capital demand and the calculated average demand is shown in table below.

Average Demand for Town = Number of Population *Per Capita Demand

Table 3.5: Domestic water demand for different categories of consumer

N	Connection Type	Stage 1	Stage 2
1	Private house connection	50 l/c/day	70 l/c/day
2	Private yard connection	25 l/c/day	30 l/c/day
3	Private yard shared	30 l/c/day	40 l/c/day
4	Public tap user	20 l/c/day	25 l/c/day

(Source: from Design criteria of MWR, 2006)

3.4.4 Non-domestic demand

Non-Domestic demand is a quantity of water required for various non- domestic needs.
Non – domestic demands are:-

- ❖ Institutional demand

- Commercial demand
- Industrial demand
- Fire demand

Table 3.6: Institutional and Commercial demand

S/N	Institutions	Consumption
1	Restaurants	10 l/Seat
2	Boarding school	60 l/Pub
3	Day schools	5 Lit/Pub
4	Public offices	5 l/employee
5	Workshop/shops	5 l/employee
6	Mosques & Church	5 l/worshipper
7	Abattoir	150 l/cow
8	Hospitals	50-75 l/bed
9	Hotels	25-50 l/bed
10	Public Bath	30 l/visitor
11	Public latrines	20 lit/seat

(Source: Design Criteria Guide Line of MWR, 2006)

3.5 Water Supply Coverage

3.5.1 Mode of services

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

3.5.2 Population distribution by mode of services

The percentage of population served by each mode of services is varying with time. This variation is caused because of the changes in living standards, improvement of the service level, changes in building standards and capacity of the water supply service to expand. According to the CSA 2007 census shown in table 3.7 below, the overall domestic water supply coverage of Durame Town was indicate 85.1%.

The greater number of the populations was served their water need from both shared yard taps and public fountains found in different villages of the town are covers 20% and 39% , respectively. The remained 8% population was obtained water from their bounded privet yard connections during that service period. While, remain 10% of the population was got water for their day to day activities from the unprotected surface water (river and streams), and rain water harvesting technique which come during the rainy seasons.

Table. 3.7: Summarized Durame Town domestic water supply coverage.

Modes of Service	Durame Town		
	Coverage (%)	Service Connections	Served Population
(House Connection) (HC)	8.1	242	3,588
Traditional Source Users (TSU)	10	234	3,122
Yard Connection Private (YCP)	20	986	1,280
Yard Connection Shared(YCS)	8	742	1,224
(Public Fountains) (PF)	39	3324	2,258
Total	85.1	5,578	24,472

(Source: CSA, 2007 statistical census document)

3.6 Materials

3.6.1 Source of data

The Durame Town water supply system have no organized data specially for old distribution network, hence all relevant primary data of main and distribution pipe line network has been collected. Both primary and secondary source of data was involved. For the study, the primary data were obtained from pressure reading, elevation surveying, and structure of the questionnaire and by made of discussion with water utility staff members to obtain additional relevant information on the subject matter. While, secondary data were collected from different literature reviews, design report, the town water supply service office existing documents and annual reported papers.

3.6.2 Equipment's used

GPS (GarminH72) was used to collect the required elevation data during pressure reading. Pressure readings were done using pressure gauge which is commonly taken in the selected points of distribution system. The EasyGPS Software was used to download the collected data in gpx format for analysis and Arc GIS v10.2 software to process digital elevation model (DEM) for water shade delineation & other data exchanging purpose. Finally, Bentley Water GEMS V8i (Select Series 5) software was used for hydraulic analysis.

3.6.3 Hydraulic modeling

Model is something that represents things in the real world. Computer model uses mathematical equations to explain and predict physical events. Modeling of water distribution systems can allow determining system pressure and flowing rate under a variety of different conditions without having to go out and physically monitor the system (Dawe, 2000).

3.6.3.1 Hydraulic modeling capabilities

- Places no limit on the size of the network that can be analyzed
- Computes friction head loss by Hazen Williams, Darcy Weisbach, or Chezy Mannin g formula
- Includes minor head losses for bends, fittings, etc.
- Models constant or variable speed pumps computes pumping energy and cost
- Allows storage tanks to have any shape (i.e., diameter can vary with height)
- Considers multiple demand categories at nodes, each with its own pattern of time variation

Table 3.8: Hydraulic modeling parameters

Node parameters	Link parameters
Elevation	Length
Base Demand (nominal or average demand)	Diameter
Initial Quality (water quality at time zero)	Roughness Coefficient
Actual Demand (total demand at current time)	Flow Rate and Velocity
Pressure	Head loss (per 1000 feet (or meters) of pipe)

(Source: Design Criteria Guide Line of MWR, 2006)

3.6.3.2 Water GEMS software

Pipe sizing and hydraulic calculations for all distribution pipe network, collectors as well as transmission mains analyzed by using Bentley Water GEMS, the latest version of Water CAD, by applying Hazen Williams' formula. Water GEMS is used to model, design and calibrate the input data resulting undertaking of pipe sizing, pressure and velocity determination of the pipe system for any scale water supply project. (Water GEMS V8i, User manual, 2008).

3.6.3.3 ArcGIS software

ArcGIS, was used to display the overlapped shape file of the distribution network on the topographic map of the town. While, Microsoft Excel sheet were used to organize elevation data, to calculate a repeated work of nodal base water demand requirement of distribution network simulation and for manual pressure validation work. (ArcGIS: User Manual, 2006).

3.7 Methods

3.7.1 Preliminary data collection

In research work data collection is the most significant part. In order to accomplish this work, the data were gathered with regard to the situation of exiting water supply and demand, necessary input parameters of model simulation, water losses and leakage management trend in the system. The data collection techniques were done by conducted a field visit to Durame Town on January 08, 2020. Data were obtained from town water service office, design report of existing town water supply system, field observation, and from interview or check lists.

The summarized collected data were presented as below;

3.7.2. Data organization

Depend on the guide line of water GEMS manual the collected primary and secondary data organized on excel sheet. Depend on easy to identify the problem, topography and technical point of view all the pipe network system divided in to two zones which have a total of have 129 nodes.

- Zone-1. From balancing reservoir to 800 m^3 service reservoirs has 32 junction,
- Zone-2. From 800 m^3 ground reservoir to the down part of the reservoir.

The tabulated data includes the pipe length, elevation, pipe size, pipe material type and the demand of each junction, base, minimum, initial and maximum elevation of the two reservoirs and the maximum, average and minimum head and discharge of the pump.

3.7.2.1 Transmission main and distribution pipeline network system

The distribution and transmission main line consists of branching system and supplying water through public fountain and yard connections by pumping and gravity means. The transmission main transmit clear water simultaneously into the distribution network and service reservoir. The existing distribution network is DCI, HDPE and galvanized steel pipes varying from DN150 mm to smaller sizes of tertiary galvanized pipes. A distribution pipe was made in 2012 GC by Federal water fund.

During distribution gravity main, secondary and tertiary HDPE pipes ranging DN150 mm to DN50mm were laid. According to the socio economic data it was witnessed that the TWSSE has no appropriate operation layout of the distribution lines. Also, there is no appropriate record on the length, type and conditions of the distribution pipe lines. Except the HDPE pipes laid during the transplantation project due to asphalt work project, some part of the existing distributions are old and need complete replacement. However, the HDPE pipes laid during expansion project will be incorporated in the future improvement. Water is distributed to a customer’s having private connections and public taps. Most of the private connections and public tabs are not working at the moment due to water shortage. All taps are equipped with water meter and gate valves.

The rising main transmit clear water simultaneously into the distribution network and service reservoir. It has a HDPE pipe of DN 75 mm and DCI transmission pipe has a length of 7000 m. As observed from the drawn distribution layout; there was one flushing device (wash out valve), one air release, and one pressure reducing device was installed in transmission line at chain-age to connect a low pressure area of the town. As per information obtained from the town water service office; the existing distribution pipe line was DCI and uPVC pipe type with DN75 mm to DN 150 mm. In general, the pipe categories in Durame water distribution system were HDPE, DCI, GI and uPVC ranges from DN 75 to DN 150 of PN 10 and PN 16.

Table 3.9: Summarized quantity of pipe material in distribution system

Pipe Type	Length(m)	Coverage in System (%)
HDPE	1900	12.41
DCI	7000	62.78
uPVC	2888	24.81
Total	11788	100

(Source: Durame Town water supply service office, 2020)

3.7.3 Town water reservoirs

There are different reservoirs in different place with in different service storage capacities. From that the 50 m^3 trapezoidal reservoirs is located near in Durame preparatory school, which is very old and it needs complete demolishing to construct new reservoir and two 300 m^3 concrete services reservoirs were constructed in 2019 GC by Water fund credit budget are capable to serve for the future as storage. The other two 100 m^3 are old reservoirs.

However, they will be rehabilitated and continues to serve in the future as storage to command the lower zone of the town as it is relatively at higher position than the 300 m^3 storage service reservoir. The 75 m^3 reservoir is also newly constructed at KMG site and it can serve in the future. Also there is one new 100 m^3 reservoir constructed at Industrial Collage area and serves in the future. Finally all this storage service reservoirs has a capacity of 850 m^3 old storage service reservoirs which is currently on functioning and 175 m^3 storage service reservoir structures which will be considered for up to 2040 GC of new future design of Durame Town water supply system respectively. Currently total capacity of reservoir is 1125 m^3.

The water developed from the spring and bore holes are not treated at the site and is flowed to directly to the reservoirs. The reported water treatment result is only being done at the reservoir every fifteen days with chlorine tablets. This kind of treatment method is applied to disinfect the pipeline and to treat the water distributed to the customer via the distribution pipes.

3.7.3.1 Existing reservoirs capacity

The capacities of reservoirs in the water supply system were determined using different methods. The most appropriate and economical approach of determining storage volume of reservoir is the 24 hours supply demand simulation mass curves.

In order to develop such type of curves, it requires reliable recorded historical data of hourly water demand figures of the town. But, in the absence of such type of data, to determine the size of reservoirs, it was adopted the commonly practiced in many water supply systems and based on the urban water supply design criteria of the ministry of water resources; it was used for sizing the reservoir volume as one third of the maximum daily demand.

Therefore, as per the design criteria of the FDRE; MoWIE, the maximum day factor usually varies between 1.0 and 1.3. Hence, a maximum day factor of 1.15 was adopted for assessing the maximum day water demand and reservoirs capacity for Durame Town and applied it corresponding to the total average day demand of a particular year (2020).

0.3*4326.19 to 0.5 *4326.19 = 1,297.857 m^3/day to 2,163.095 m^3/day

3.7.3.2 Flow metering

Water meters are installed at the outlet of the reservoirs and also there are water meters at the bore holes outlets.

3.7.3.3 Power supply units

There is power supply service in the town. The water utility was served power from hydroel ectric power plant and used with its own transformer which is provided by EELPA. The water distribution system was operated for 24 hours of its design period. But, there is no standby generator for distribution system during power failure is occurring.

3.7.3.4 Elevation data

One of the best significant requirements to simulate the hydraulic characteristics of water in distribution system is setting elevation. Elevation data for existing line and expansion area in the town were served in the field using surveying instrument, global position system

(GPS). In appendix-A, all the assigned node elevation data were listed with coordination system.

Table 3.10: Elevation data for service reservoir site

Description	Coordinates		
	X	Y	Z (Elevation, msl)
Existing Service Reservoirs			
Spring1	374923	800287	2061.78
BH 1	376909	797810	1970.68
BH 2	376784	797389	1967.45
BH 3	380606	803030	2083.63
BH 4	380146	803009	2088.97
BH 5	380238	802482	2090.78
BH 6	377493	799305	2022.04

(Source: from Durame Town Design Report, 2020)

3.7.4 Operating pressure

Pressures are measured throughout the water distribution system to monitor the level of service and to collect data for use in calibration. Pressure readings are commonly taken at water distribution mains also at hose bibs, and home faucets. Optimum operating pressure is one of the most important parameter in design of water supply system.

In the design of this water distribution system, criteria of minimum and maximum operating pressures of 10 m and 70 m H_2O respectively with the higher value on the higher margin are used. This range of pressure is the basis for the design of the distribution system for the areas that are going to be fed by the proposed reservoirs. In exceptional cases the operating pressure may rise above 70 m of water head based on the topographic condition (Bentley, 2008).

3.7.5 Velocity of flow

As per MWIE design manual 2006, water velocities shall be maintained at less than 2 m/sec, except in short sections. Velocities in small diameter pipes (<DN100) may need even lower limiting velocities. Similarly, according to the manual a minimum velocity of 0.6 m/sec can be taken, but for looped systems there will be pipelines with sections of zero velocity.

Generally, velocity of flow is one of the important parameter in design of water supply system. So it is analyzed that, the transmission and distribution network system is mostly based on the following velocity intervals:

- ❖ Minimum velocity in all pipes= 0.3-0.6 m/s
- ❖ Maximum velocity in distribution mains ≤ 2.0 m/s
- ❖ Maximum velocities in major transmission main & Collectors ≤ 2.5 m/s
- ❖ Although the velocity of flow in some distribution pipes may fall below the minimum stated value due to the looping effect in the distribution network, it is practical to consider up to 0.3 m/s to make the system most efficient and bring sustainable supply in those areas having such problems.

3.7.6 Selection of pipe materials

Thus, appropriate pipe material particularly GS pipes and HDPE pipes.

- ❖ C= coefficient of hydraulic capacity = 120 for Galvanized Iron Pipes(GI), 130 for Ductile Cast Iron(DCI) and 150 for High Density Polyvinyl Chloride (HDPE) pipes
- ❖ 0.3 to 2 m/s velocity range is considered
- ❖ Water distribution through water points for rural at operation with house connection later in towns
- ❖ All water points assumed to be operated at the same time
- ❖ The supply mains and distribution system are aligned in accordance with the following criteria mainly for reasons of protection, safety, ease of maintenance and avoidance of pressure due to traffic and other live loads.
- ❖ As far as possible the mains are laid on the side of carriageways, footpaths and pave ment.
- ❖ Mains laid in soil should have a minimum cover of 0.80 m for GI pipes and 0.9 – 1.2 m for HDPE pipes, as the condition of the location for DCI pipes, but the maximum cover will not exceed 2 m.

3.7.7 Base water demand data

To estimate the current water demand of each node in the distribution network, it was necessary following the steps below;

Step one: Assigning the total population of the town

In the distribution network, population is the important data to assess water demand. Facts show that there are different population forecasting methods which are used for estimating the current or future population of a given town, but the results of the methods are vary from one to the other due to considering parameters of each method. To predict the population of a town, it is necessary knowing factors affecting the population distribution, size and growth rate.

In Ethiopia, the major factors that influences on the changes in population figure are births, death and migration. All these factors are influenced by family planning practice, war, natural disasters, development of the towns and the socio-economic activities in and around the towns. Therefore, for this study; based on the historical figures, assumptions considered (available of data) and to be precise, the CSA population estimation method was selected from the different population projection methods. Finally, the population figure of the town was assessed in (2020 G.C).

Table 3.11: Durame Town population growth, south regional state

Year(GC)	2006-2010	2010-2015	2015-2020	2020-2025	2025-2030	2030-2035	2035-2040	2040-2045
Growth rate (%)	4.82	4.82	4.8	4.8	4.8	4.05	3.7	3.77

(Source: CSA, 2020 national statistical census document, South region, Durame)

$$\mathrm{Pn} = \mathrm{Po} * e^{nr}$$

Where: Pn = is projected population

Po = Base population figure

E = Constant e, the base of natural logarithm

R = Growth rate and,

N= Number of year

Step two: Identification of number of houses around each supply node

For this study, the town topographic map was obtained and bought from Ethiopian Mapping Authority, with the scale of 1:50,000 and within fixed meter contour interval. In ArcGIS this topographic map was displayed and the town distribution network map which was drawn in Bentley Water GEMS was exported in to ArcGIS shape file and overlapped it in the topographic map of the town .But, this is done by following the Non-Revenue Water calculation steps for the scenario of water supply distribution in each block customers meters. Therefore, the number of houses nearby each node was physically counted from the overlapped map and assigned to every node in the network by considering the actual condition of the residents in the town.

Step three: Assigning number of peoples in each supply node

The current average number of person in each house (person per housing unit) was obtained from the revised design report of the town population projection and taken 5.5 of the average number. The total number of houses in the town was identified by dividing the total population to the average number of person in the town. Therefore, in the opened Microsoft Excel sheet, all the 129 nodal junctions in the system and the number of houses assigned for each node were entered respectively. Then, in the third column the number of a ssigned houses in each node was converted in to the number of people, by multiplying the a verage number of person in each house of the town.

$$\text{Number of supply node} = number\ of\ house\ assigned\ by\ that\ node * Average\ number\ of\ people\ in\ each\ house \ldots\ldots\ldots \quad (3.1)$$

Step four: Assigning average day water demand of Durame Town

For assessing the average water demand of the town, deterministic water demand estimation method was used. Hence, the per capital water consumption of the town was calculated by using the annual water consumption recorded data and projected total population figure during (2020). Therefore using equation below it was assessed.

$$\text{Per capital Consumption } (lc/d) = \frac{\text{Annual Consumption}(m^3 1000l/m^3)}{\text{Total Population}*365} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. \quad (3.2)$$

Therefore, the average water demand of the town was calculated by multiplying the per capital demand with the estimated number of population as follow.

$$Q_{Avg} = \text{per capital Water Consumption } * \text{ Total population of the town} \ldots.\ldots\ldots\ldots\ldots\ldots\ldots.. \quad (3.3)$$

Step five: Assigning base water demand in each supply node

Once the average day water demand of the system was determined, to calculate base water demand for the particular supply node the following equation was used (Bhadbhade, 2009).

Base water demand for supply node =

$$\frac{Pop'n\ Served\ by\ that\ node}{Total\ Pop'n\ of\ the\ town} * Avg\ day\ water\ consumption\ of\ the\ town \ldots\ldots\ldots\ldots \quad (3.4)$$

3.7.8 Demand multiplier factors

For modeling, peak hour demand scenario was adopted. Demand for each supply node was performed by taken demand multiplier factors of 24 hour flow duration and computed with assessed base water demand. For this study by considering the peak flow time, minimum flow condition and the actual condition of population served from the system ; the demand multiplier factors were adopted data obtained from the regional water, mine bureau. Peak factor and patterns for energy and demand multiplier factors were listed in table 3.12 below.

Table 3.12: Proposed hourly peak factor

Population	Peak Hour Factor
0 -50,000	2
50,001 – 100,000	1.8
101,000 & above	1.6

(Source: from FDRE, MSWR, 2006)

3.7.9 Hydraulic design

Applying Hazen William formulas' that is used for flow computation.

$$V = 0.85C * R^{0.6} * S^{0.54} \ldots\ldots\ldots\ldots \quad (3.5)$$

Where V = flow of velocity in (m/sec)

C = Coefficient of hydraulic capacity

R = Hydraulic mean depth in (m)

by substituting and re-arranging

$$S = \frac{HL}{L} \ldots\ldots\ldots\ldots (3.6)$$

Where HL =head Loss in (m) L = Length of pipe in (m)

$HL = L * (0.00212D - 4.87Q^{1.85})$ ……………………………………………………… (3.7)

D = diameter of pipe in (m)

Q = Discharge (flow through the pipe) m^3/sec

Empirical formula for determination of economic pipe diameter is: -

$D = 1.22\sqrt{Q}$ ………………………………………………………………………………… (3.8)

Where D = Economical Pipe Diameter in (m)

Q = Flow through the pipe (m3/sec)

Note. Velocity through the pipe ranges b/n 0.5 to 2.5 m/sec (Tomas et al., 2003).

3.7.10 Roughness coefficients for pipeline

The Hazen-Williams equation was developed for the action of friction at the pipe wall, because its formula uses a pipe carrying capacity factor. Higher C-factors represent smoother pipes (with higher carrying capacities) and lower C-factors describe rougher pipes (Tomas et al., 2003). The value of roughness coefficient, C-factor is depending on pipe materials and its age; this effect can be shown in Appendix E-, (Tomas et al., 2003).

Table 3.13: Hazen-Williams Roughness Coefficients C Value

Pipe status	Pipe Type		
	UPVC/HDPE	Steel	DCI/GI
New pipe	150	120	130
Old pipe	100-120	90-110	100-110

(Source: from Tomas et al., 2003)

3.7.11 Network simulation

To build and simulate the hydraulic model, Bentley Water GEMS Vi8 series stand- alone, graphical editor water distribution modeling software was used. The water distribution network map was obtained from the town water service office, Which was prepared as blue print drawn map report of distribution network and it was drawn in water GEMS drawing pane with physical observation. The network simulation was taken extended periods by consideration of hourly demand variation pattern over 24 hour flow duration analysis work. For this study, the network operational set-up was done by system international; SI unit and the project liquid were taken water at 20°c. The other model input were taken and carried out as mentioned below;

- Coordinate ………………….. X-Y
- Setting………………………… Pressure
- Tank level…………………….. Elevation
- Drawing Scale………………... Scaled
- Annotation Multiplier………... Adjusted for report visibility

3.7.12 Model calibration and validation

All 14 sampling points were selected after the computed model was simulated and knowing the pressure variation area (pressure zone) in the town water distribution network. The method of pressure readings was done by using pressure gauge commonly taken both at high and low pressure zone of the selected points in distribution network; such as clear water pump stations, bore hole outlets, service reservoir or tank, public fountains and different end user taps (like; customers, institution and commercial tap points).

The computed parameters of a model and actual field observation are not always has the same value. Therefore, before discussion about the simulated model results, the entire model data quality must be analyzed by calibration and validation technique.

Model validation: The model validation work was taken manually using the correlation coefficient equation (R^2) method and it were described and represent graphically in figures 4.9 and 4.10 as shown below.

$$R^2 = \frac{\sum(x-\bar{x})(y-\bar{y})}{\sqrt{\sum(x-\bar{x})^2 \sum(y-\bar{y})2}} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \quad (3.9)$$

Where: R^2= correlation coefficient, X and Y are the observed and computed pressure values, and X and Y are mean value of observed and computed pressure, respectively.

Model Calibration: is a process of adjusting the model input data until its results become closely approximate to the measured field data. Whereby, it used to obtain approach, realistic and acceptable results. Therefore, in this study the model data quality analysis was done by comparing and calibrating the computed pressure data with the observed one.

The method of pressure readings was done during (Jan 08, 2020) using pressure gauge commonly taken both at high and low pressure zone of the selected points in distribution network; such as Bore hole outlets, clear water pump stations, tanks or service reservoirs, public fountains and different end user taps (like; customers, institution and commercial tap points). However, the observed pressure data was taken a total of fourteen samples for both peak demand and low demand/night flow time analysis. All sampling points were selected after the computed model was simulated and knowing the pressure variation area (pressure zone) in the town water distribution network. According to (Tomas, et al., 2003), the calibration process was performed by adjusting sensitive parameters related with flow; like pipe roughness coefficient and water demand until it was become within the acceptable limit of 85% of field test measurements (it should be within ± 0.5 m or ± 5 % of the maximum head loss across the system, whichever is greater) and then finally it was validated manually using the correlation coefficient (R^2) method using Microsoft Excel sheet.

3.7.13 Pump efficiency description

The pump efficiency which operate currently in the system was calculated manually after validating the model. For each computing time (a total of 24 hour) value of the water power produced by the pump and which converted in to electric power form (watt) were collected from the model outputs presented in (Appendix-G). Therefore, to calculate these pump efficiency; the maximum value of 36.22 KW water power was obtained from the computed model result and used as the value of water power delivered from the pump. While, the value of pump power (the brake horsepower of the pump) in the system was obtained from the pump shaft during field observation, (Jan, 14, 2020). And based on the observed pump data, the brake horsepower on the pump shaft is 75 KW. Therefore, using these data and equation below the current pump efficiency for Durame water supply system was calculated;

$$ep = \frac{(Water\ Power\ \ out)}{(Pump\ Power\ \ in)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.10)$$

Where ep = pump efficiency (%) …………………… (Source: Tomas, et al., 2003)

3.8 Water Loss Assessment

The water loss assessment has made by using expression in terms of percentage of (UFW),loss per kilometer of main pipes and loss per number of connections. In order to identify the total loss of water in the study area, the total volume of water supplied to the network distribution system was compared with the actual water consumption. In this case, yearly aggregated production and consumption data was obtained from DWSS for four years from 2017 up to 2020 and used to compute NRW.

Moreover, the apparent and real loss computed by using IWA water loss task force developed equation by providing input data of length of mains pipe line, number of service connection, total length of service connection from the edge of the street to customer meters. Unlike the water consumption, the water production of Durame Town was only recorded at the town level. Hence, the water loss analysis in Durame was assessed at the town level based on the percentage of Non-Revenue Water which obtained from the total production and actual consumption figure.

3.8.1 Non-Revenue water

As per data obtained from the town water service office, water production and consumption (billed water volume) in the system was identified. Using this data and equation below; the total Non-Revenue Water (NRW) in the system was calculated for each recorded year.

Non-Revenue Water (NRWM) = System in put volume – Billed Authorized Consumption (BAC)

During field visit, it was also observed that the utility do not have any recorded data related with average leak flow, number of reported bursts and average leak duration; due to these physical loss in the main was assessed base on the available data, and it was adopted by considering the minimum achievable annual physical losses (unavoidable annual real loss) in the system (Farley, et al., 2008).

$$UARL\ \left(\frac{litters}{day}\right) = (18 * Lm + 0.8 * Nc + 25 * Lp * \ P \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.11)$$

Where Lm = mains length (km);

N_c = number of service connections; P = average pressure (m)

L_p = total length of Private pipe, property boundary to customer meter (km); and

3.8.2 Real and Apparent loss assessment

Non-revenue or total loss is the sum of real loss and apparent loss. However, for this study unable to compute total real loss due to limitation of night hourly consumption data, instead of this unavoidable real loss is computed by using IWA expression. The expression is good indicator of loss and this study focus on two different areas which have relatively high and low average pressure.

3.8.3 Unavoidable average real losses (UARL)

It is recommended that the calculation of the UARL in liters/service connection/day is based on the following form of equation. This recognizes separate influences of real losses from length of mains (Lm in km), number of service connections (NC), total length of service connections from the edge of the street to customer meters (Lp in km), and average pressure (P in meters) when the system is pressurized.

$$\mathrm{UARL} = (\mathrm{A} \times \mathrm{Lm}/\mathrm{Nc} + \mathrm{B} + \mathrm{C} \times \mathrm{Lp}/\mathrm{Nc}) \times \mathrm{P}\ (\mathrm{Liters}/\mathrm{service\ connection}/\mathrm{day})$$

The appropriate values for A (18), B (0.80) and C (25) the equation and its parameters A, B, C are based on statistical analysis of international water supply system data.

3.9 Assessment of Leakage Management Practice

The water distribution leakage management practices of the town water service office were assessed based on the management, technical and financial; plan, policy and strategies. Good leakage management practice is one of the ways of reducing water loss in the distribution system. Hence, field visits were made to identify the leakage in the system and its managing processes.

During field observation, discussions were conducted with town water supply service personnel to obtain information on the common failure of system, financing mechanis ms, and the maintenance culture and cost drivers of maintenance. Finally, the collected data were analyzed and presented in the next chapter.

3.9.1 Water utility organization

A strong management and organizational setup is the main reason for good leakage management practice in the water supply services. Based on the policy and guidelines of the regional administration, the town water supply service manager can organize, directs and administers the overall activities of the water service unit.

3.9.2 Operation and maintenance practice

Mostly the operation system was controlled by the utility guards (non-skilled man power) who protect the intake and treatment plant site. Maintaining, removing and replacing of both customers service line and bulk flow meters were done when the utility was reported or requested by the customers during failure is appear. The overall maintenances of system components were not checked by schedule and it was maintained during failure or damage is occurring.

4. RESULTS AND DISCUSSION

4.1 Water Demand Estimation and Supply Coverage

Assessing and sizing of system components such as water pumping station, reservoirs, and transmission and distribution pipe line is used to estimate the expected water demand of the town. To identify the gap between demand and supply the three ways are followed, first by using annual domestic consumption identifying the level of per capita demand per person, second computing the per capita demand by level of connection and third by following the design criteria calculating the overall demand coverage.

4.1.1 Forecasting population

The demand of water for a particular town is proportionally related with the population to be served. From Ethiopian CSA report, which is carried out in year 2007, the population of Durame Town was indicated that 24,472 and it was used as a base population for current estimation. According to CSA, the regional level annual growth rate for urban population (2020) was allocated as 4.8%.

Using the above CSA (2007) census data as a base, and applying exponential population for ecasting method, the current 2020 population figure for Durame Town was presented in table 4.1 below.

Table 4.1: Durame Town projected population figure (2020-2040)

Description	Unit	2020	2021	2025	2030	2035	2040
Growth rate - Urban	%	4.80%	4.80%	4.80%	4.05%	3.65%	3.25%
Population -Urban	No	90,390	94729	113,074	134,444	154,760	171,365

(Source: from CSA 2011 document, South Region; Durame)

Therefore, regarding to the above table 4.1; the forecasted population figure of Durame Town was 171,365 during (2040). So projecting this future population is useful to compare the current town's population with regard to supply coverage by knowing how much difference is found in between both periods.

4.1.2 Average daily per capita water consumption

To evaluate the amount of water consumption, the annual water consumption is converted to average daily per capita consumption using the population data of the town and the number of domestic connection per family has been also used to analyze the level of connection. The per-capita water consumption for various demand categories varies depending on the size of the town and the level of development. In Durame, because of the growth of the socio-economic activity in both governmental and private sectors, there was the high water demand in the town. Using the annual water consumption and population figure in (2020), the average per capita consumption of the town was identified.

The annual domestic consumption amount in 2020 is 905,802.1m^3 and the current total population of the town was adopted as 90,390, there for by using the above expression the average daily per capita consumption became 27 l/per capita/day.

According to Wallingford HR (2003) which is reviewed by Dessalegn (2005) a minimum quantity of 25 l/c/day domestic water supply categorized as basic level of service which is higher than the average domestic consumption of the town. In addition, according to the standard set by MWR, 2006 for third level town like Durame the per capita per day is 60 liter, there for ,the current coverage is only satisfy one third of the demand.

4.1.3 Level of connection per family

Level of water connection per family is one mechanism to evaluate the level of water coverage. The total number of connection or water meter with in the town are about 5578 that among these 3912.58 are for domestic users, according to the census of the 2011 average family size of 5.5 is used for calculating the average number of connection per family. The level of water connection as per the above expression became 0.6; this implies that the current connection coverage is only 60%.

4.1.4 Domestic and Non-domestic water demand

Domestic water demand includes water finished to in- house purpose such as drinking, cooking, washing utensil, washing clothe, washing toilets, watering animal. The amount of water used for domestic purpose varies depending on the life style living standard, climate mode of service and all the price of water and affordability of users.

Non-Domestic demand is a quantity of water required for various non- domestic needs.

Table 4.2: Summary of Domestic and Non-Domestic demand

S/N	Item	Demand (m^3/day)
1	Domestic demand	3912.58
2	Non-Domestic demand	978.14
3	30% loss (for scheme having 8 year service)	436.2
4	Total Sum	4326.92

(Source: from Durame Town water supply service office, 2020)

4.1.5 Average Demand computation by mode of service

The percentage distribution of population for each mode of service as shown in table below;

Table 4.3: Primary water source of sample house hold

N	Connection Type	% of Population by mode of Service	Number of population per service(2020)
1	Private house connection	1.66	1,500
2	Private yard	38.7	35,000
3	Private yard shared	1.11	1,000
	Sub total	41.47	37,500
4	Public tap user	58.52	52,890
	Total		90,390

(Source: from Durame Town water supply service office, 2020)

According to the current standard of the country the third level towns like Durame have the following per capital demand and the calculated average demand is shown in table below.

Average Demand for Town = Number of Population *Per Capita Demand

Table 4.4: Analysis of average day demand

S/N	Type of Connection	Number of population per service(2020)	Per Capital Demand(l/c/day)	Average Demand(l/c/day)	Average Demand(m^3/day)
1	Private house connection	1500	60	90,000	90
2	Private yard	35000	30	1,050,000	1050
3	Private yard shared	1000	40	40,000	40
4	Public tap user	52890	20	105,7800	1057.8
	Total			2,237,800	2237.8
					4475.6

(Source: from Durame Town water supply service office, 2020)

According to national standard, Durame town water supply system connection in above table 4.3 the public tap users of per capita demand is very small when compared with other types of connections

4.1.6 Demand variations

A. 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 fact 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.

B. 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.

C. Peak hour factor

Water demand varies greatly during the day. The distribution system must be designed to cope with the peak demand, which is taken into account by the use of a peak hour factor.

Table 4.5: Population vs. MDDF and PHF

Population	MDF	PHF
0 – 20,000	1.3	2
20,010 - 50,000	1.25	1.9
50,001 and above	1.2	1.8

(Source: from Durame Town water supply service office, 2020)

Table 4.6: Result of MDD and PHD

	Average day demand (ADD)	Maximum day demand (MDD)	Peak hour demand (PHD)
Demand Type	4326.92 m^3/day	6392.304 m^3/day or 64 l/sec	9588.456 m^3/day or 72 l/sec

(Source: from Durame Town water supply service office, 2020)

4.1.7 Reservoirs capacity and storage requirement

Operational reservoirs should be provided to command a distribution system, located at elevations providing the required pressure for water flow within the system. Types of reservoirs are one old trapezoidal and cylindrical types of reservoir are the ground level type (GLR) and one elevated water tank type (EWT). Whenever the local topographical conditions permit, ground level reservoirs are preferable. Ground level reservoirs will be usually be of solid block masonry or reinforced concrete, cylindrical or rectangular but under special circumstances may be of glass reinforced plastic (GRP).

Elevated water tanks will be cylindrical or conical in reinforced concrete (MWR, 2006).The study area has two ground level type and one elevated circular reinforced concrete reservoir. The situation of Reservoir location should maintain the desired pressure range in the supply network. Possible future extension of the storage capacity should be taken in to consideration when selecting a site. The equipment's of all reservoirs should be provided with inlet, outlet, drainpipe, overflow pipe, water level indicator, manh ole ladder, ventilation pipe, lightening conductor except the old 50 m^3 rectangular reservoir.

In order to provide for security of supplies above the need for balancing purposes it is recommended that the minimum total reservoir storage capacity be in the range of 30% to 50% of the average daily demand (MWR,2006). The study area has one 800 m^3 reservoir, one50 m^3 reservoir, one 75 m^3 and one 200 m^3 elevated reservoirs which have a total of 1125 m^3 reservoirs. To check this capacity enough or not the total average day demand as per computed above are 4326.19 m^3/day and the level of existing storage computed as:-

Their for, the storage capacity of existing reservoir has been on the recommended range and they were found in good condition and able to serve for extra years except the old reservoir as shown in figure 4.2, it which needs new maintenance or design because, it is in dangerous case and it has the reconstruction is the only solution and so, far the scheme is safe regarding to storage capacity.

4.1.8 Hydraulic model performance result in water distribution network anaysis system

Water distribution systems include main distribution line, distribution storage and distribution piping. The hydraulic performance was in terms of the pressure it is10% is out above 70 m head. The specified requirements in terms of velocity the result shows that the velocity in percentage less than 0.59 m/s is 85% and greater than 2 m/s is 1%.

4.1.8.1 Pressure and velocity in the system

The frictional head losses in the distribution network are computed using the Hazen - Williams formula. The distribution network has been carried out utilizing the Water GEMS computer program. The peak hour demand analysis is conducted considering the maximum day demand of 6392.304 m^3/day and peak hour factor of 1.8; whereas the minimum demand analysis carried out considering its flow factor and a maximum daily demand of the system. It can be seen that the available minimum head at most nodes also, except at few nodes which are located at lower level of the town, there are no nodes that will be submitted to a pressure higher than 70 m during minimum demand. However, for such nodes a pressure reducing valve is recommended at node as they are supplied from o ne main outlet gravity pipe from existing service reservoirs.

Regarding the model output, high pressure has been found at low demand and high velocity occurred at the period of peak demand. As shown on filed visit the pipe size of the existing distribution system is installed not corresponding with the demand, same place there is high demand with low pipe size like at P-22, 44 mm diameter and low demand with high pipe size at P-64,144 mm diameter. As a result high velocity occurred for smaller pipe size and low velocity occurred for larger pipe size with low demand.
The result shows that the pressure is high at morning time. The discharge and velocity is flow within the same line of action.

4.1.8.2 Water distribution network analysis

In the modern water supply system, clear water shall be delivered to the service reservoirs directly through the transmission main and which is completely isolated from the distribution system. But, existing Durame Town water supply system different sites are constructed before 19 years ago specially Hambricho spring it was old system and as it was the old system; water is pumped and wasted in every lines and it needs continual maintenance and from the other sites water is simultaneously pumped into the service reservoir and distribution. So, the impact of this network configuration and the capacity of distribution system components were described as below.

4.1.8.3 Distribution main line

Regards the topography of Durame Town, the locations of nodes in the water distribution line is in close proximity to each other. The maximum and minimum water pressures in the distribution system were 70 m and 10 m head around treatment plant service reservoir. According to the design criteria of the FDRE; MoWIE, the maximum and minimum water pressure in the distribution system is 80 m and 15 m, respectively. Beside these comparisons; the current Durame Town existing water distribution network around the reservoir was operating out of the recommended limitation.

So the town's maximum and minimum water pressure should be modified between in its true range. This is because of; water is delivered to the distribution main simultaneously by pumping and gravity means (old system), and the system were served beyond its design life. For safe Hydraulic system recommended value of pressure and velocity by (MWR, 2006) is at distribution system the pressure must be not less than 10m and more than 70 m head ,the minimum velocity not less than 0.6 m/s and the maximum velocity should be less than 2 m/s.

4.1.8.4 Pressure variation in the distribution system

Variation of water pressure in the distribution system is mainly because of hourly fluctuation of water demand. As shown in figure 4.4 and figure 4.5 below; the water pressures in Durame Town water distribution system were a function of this factor. Variation of elevation difference in most part of the town has also an impact for the rising and reduction of water pressure in the network. Therefore, during peak demand time most part of the network was disconnected from the system and wide residential area of the town were not getting water.

4.1.8.5 Negative pressure

The Negative Pressure shown at the model output indicate two things, the first, due to high demand along the smaller pipe size, there is high friction loss which is more than the ground elevation difference and the second is due to poor operation when the pipe fill by air and the hydraulic pressure less than the atmospheric pressure negative pressure will occurred as a result collapsing of pipe happened. Situations that give rise to negative pressures should always be avoided.

Hence, pressure in the distribution system is one of the factors for intermittent water supply. For this study, all negative pressure presented in appendixes indicated; the system was disconnected during peak demand time and water was not reaching to customers. Whereby, these was mainly as a result of; there is demand concentration (greater demand than the design demand), inadequate pipe capacity (small diameter), and availability of residences on higher ground of the Town.

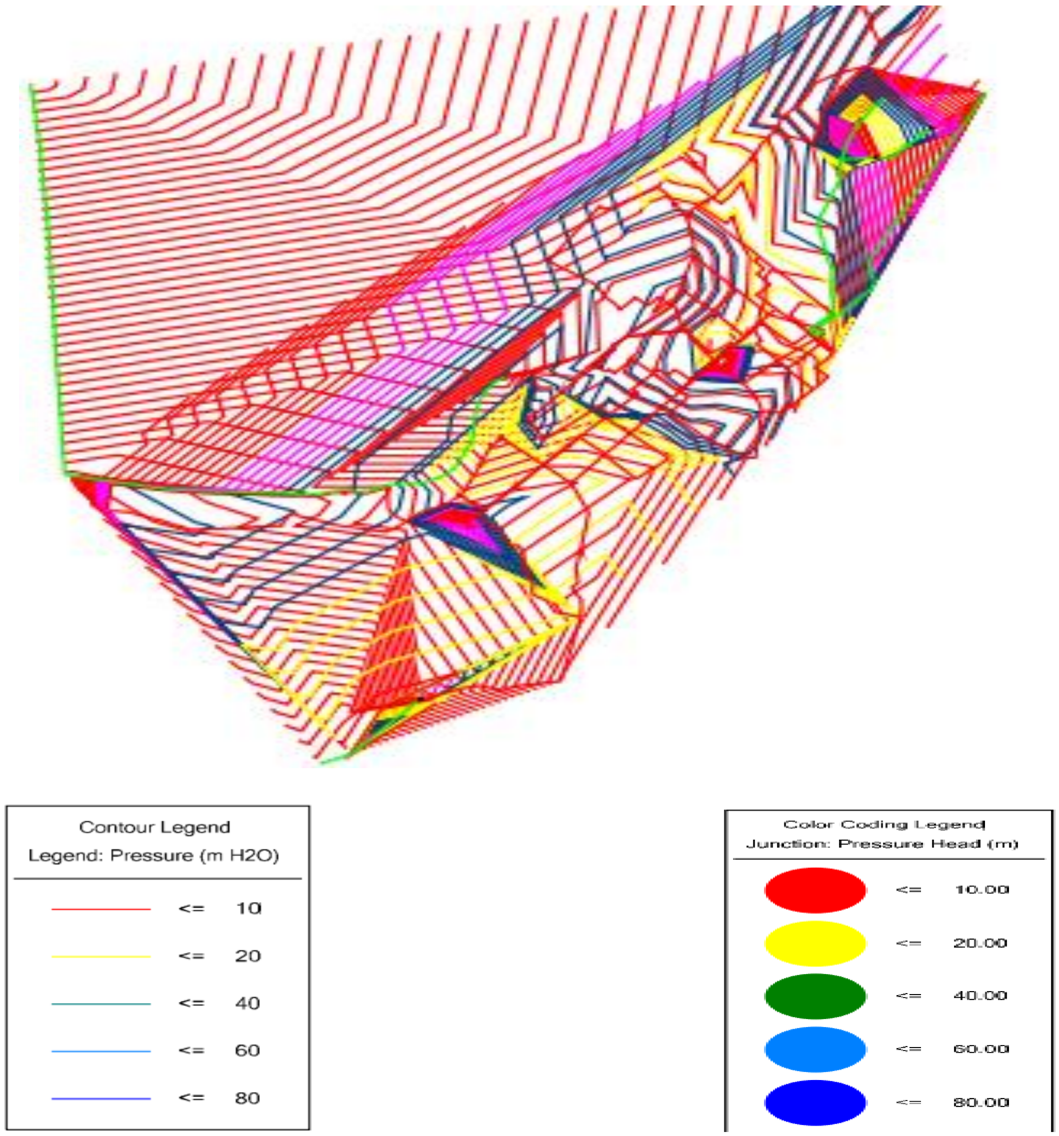


Figure 4.4: Water distribution network during peak demand time

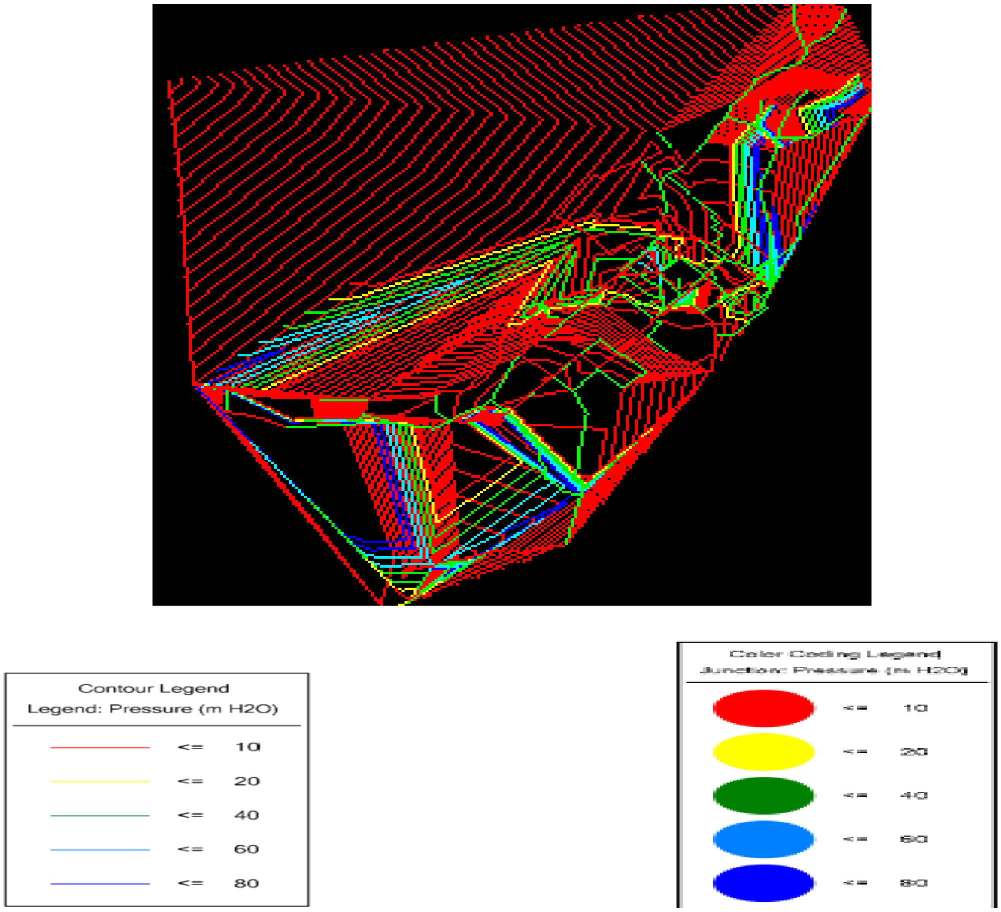


Figure 4.5: Water distribution network during night flow/low demand time

4.1.9 Hydraulic model calibration and validation

Pressure measuring was done during (Jan 08, 2020) using local instrument by adjusting time both at high and low pressure zone of the selected points in distribution network; such as Bore hole outlets, clear water pump stations, tanks or service reservoirs, public fountains and different end user taps (like; customers, institution and commercial tap points).In the modern time, water utilities have able to analyze the status of their existing water supply system using hydraulic models.

But, for assuring the entered water distribution model inputs data accuracy; the computed model results have been compared with the actual observed field conditions of study area. (Bentely, 2008) states that the average difference of ±1.5 m to a maximum of ± 5.0 m for a good data set and ± 3.0 to ± 10 m for a bad data set would be a reasonable target. As shown in figure 4.6 and 4.7 below; during the comparison of measured pressure value with the simulated one, gaps were recorded up to 1.8 m head and it was out of the pressure standard and limitations suggested by (Bentely, 2008). Therefore, the computed pressure value of both scenarios, during peak demand time and low demand time (night flow) were calibrated until the result was approach to the observed pressure value.

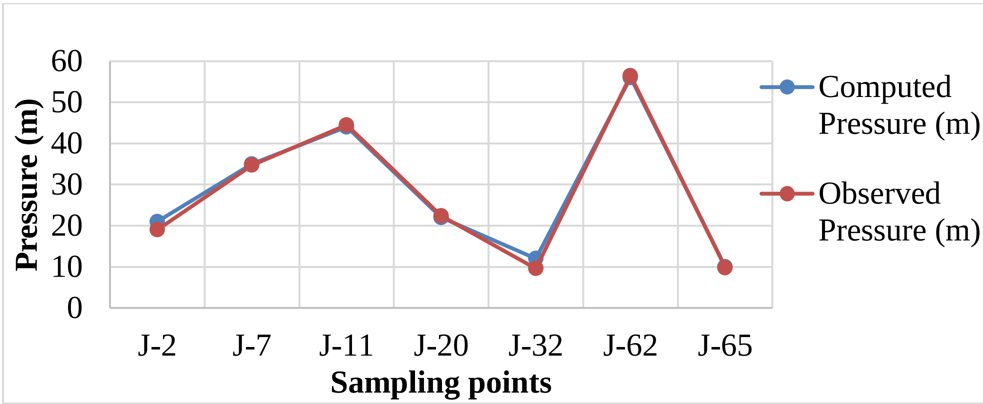


Figure 4.6: Computed and observed pressure value during peak demand time

Figure 4.7: Computed and observed pressure value during night flow (low demand time)

As per pressure criteria 85% of the computed model results should become within ±0.5 m head of the observed field conditions. Hence to assure the acceptable level of calibration, the two most commonly used model input parameters; pipe roughness coefficients and junction demand data were adjusted.

Therefore, during model calibration; C-factor was used 150 for PVC, 120 for GI and average value of 109 for DCI pipe. While, as per discussion with the water utility manager, in Durame the maximum hour water demand is happen during Saturday morning specially

@2*am* local time and Sunday evening @1*am* local time because, most people use water for bathing, washing and cooking purpose in this continual days. Accordingly, demand adjustment was undertaken by adopting multiplier factors in reasonable way (a maximum and minimum of 2 and 0.3, respectively) and demand concentration also adjusted based on actual condition of the town. With regard to these, time series representations of the calibrated pressure head difference were presented as Appendix-F.

The model validation work was taken manually using the correlation coefficient equation (R^2) method and it were described and represent graphically in figures 4.8 and 4.9 as shown below.

As shown in figure 4.8 and 4.9; it explain the results of correlation value (R^2) for both peak and low demand time was represent as 98.94% and 99.85%, respectively. The coefficient of determination (R^2) value was 0.9985, it indicates that observed and simulated relation is strongly as values tend to 1. Thereby, the calibrated pressure value was validated within the recommended standard.

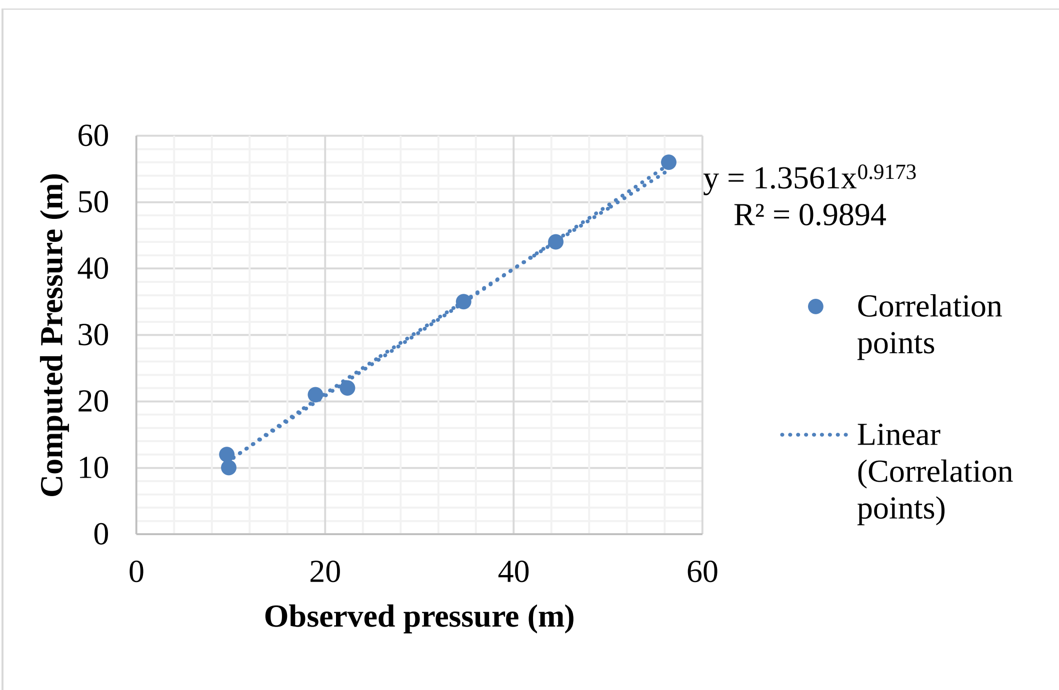


Figure 4.8: Correlated plot during pressure calibration for peak demand time

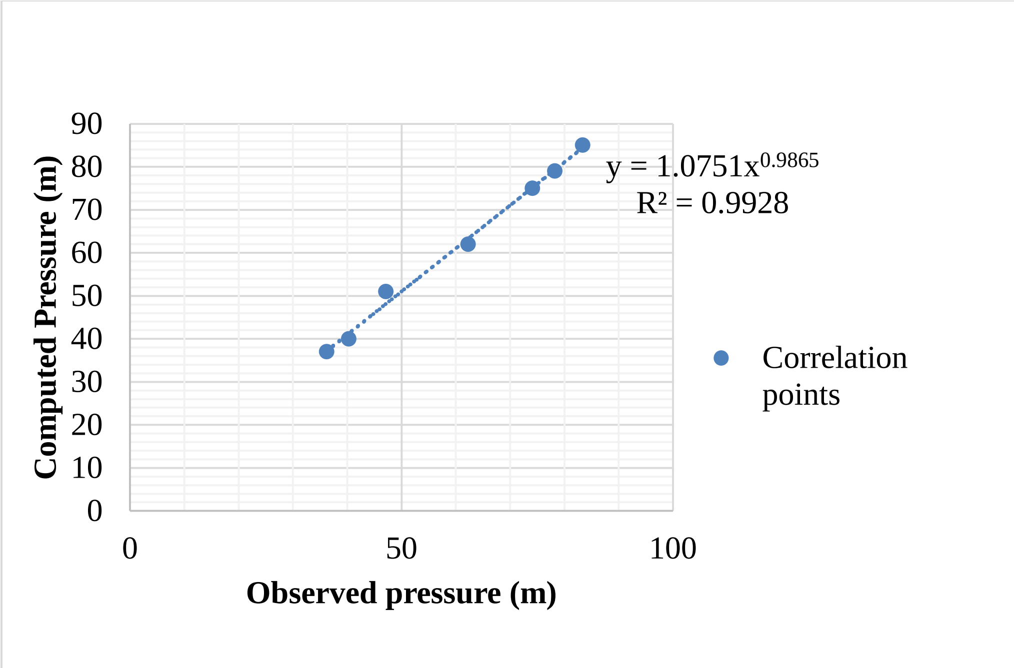


Figure 4.9: Correlated plot during pressure calibration for low demand time (night flow).

4.1.10 Difference of calibration based on error

The degree of accuracy varies depending on the size of the system and the amount of field data and testing available to the modeler. (Bentley , 2008) states that the average difference of ±1.5 m to a maximum of ± 5.0 m for a good data set and ± 3.0 to ± 10m for a bad data set would be a reasonable target. This is in terms of comparing the observed versus the calculated pressure and heads in the system.

Finally, the average error as shown in Appendix-G, computed values are within an average error of 0.9985 m pressure simulated to observed values. Hence, the model is acceptable calibrated which is satisfied the setting pressure calibration and validation criteria under average level (average +1.5 m to the maximum +5 m).

4.2 Water Loss Assessment

The water loss assessment has made by using expression in terms of percentage of (UFW), loss per kilometer of main pipes and loss per number of connections.

4.2.1 Non-Revenue water

Non-revenue water is the difference between the volumes of water put into a water distribution system and the volume that is billed to customers.

NRW comprises three components: physical (or real) losses, commercial (or apparent) losses, and unbilled authorized consumption.

- ❖ Physical losses comprise leakage from all parts of the system and overflows at the utility's storage tanks. They are caused by poor operations and maintenance, the lack of active leakage control, and poor quality of underground assets.
- ❖ Commercial losses are caused by customer meter under registration, data handling errors, and theft of water in various forms.
- ❖ Unbilled authorized consumption includes water used by the utility for operational purposes, water used for firefighting, and water provided for free to certain consumer.

The water loss assessment has made by using expression in terms of percentage of (UFW), loss per kilometer of main pipes and loss per number of connections. To compute the level of apparent loss due to limitation of data, the real loss is calculated as per level of connection and length of pipe and the apparent loss became the difference between total loss and real loss.

A. Total Water loss expressed as percentage

Four year production and consumption data of the study area used to compute the total loss as shown in table 4.6 below by using this expression.

$$\text{NRW}\ (\%) = \frac{\text{Production} - \text{metered use}}{Production} * 100\%$$

Table 4.7: Percentage of Non-Revenue water

Year	Prod.(m^3/year)	Cons. (m^3/year)	Loss. (m^3/Year)	Loss (%)
2017	208,350	139,965	68,385	32.82
2018	297,570	201,705	95,865	32.21
2019	338,270	228,377.1	109,892.9	32.48
2020	392,490	264,596.5	127,893.5	32.58
Yearly Average	309,170	208,660.9	100,509.1	32.50

(Source: from Durame Town water supply service office, 2020)

B. Water loss expressed as per number of service connection

The total number of connection in the study area is 5578, the water loss per connection computed by using this expression.

Water loss = Annual total loss *1000/(Number of connection*365)

From four years computed total annual water loss the last year value 127,893.5m^3/year used

Water loss = 127,893.5*1000/(5578*365) = 60.64 lit/connection/day

C. Water loss expressed as per length of pipe

Expressing water loss as per kilo meter of main pipe is one way to indicate the loss. The total length of pipes of size 25 mm to 150 mm is 72.45 km and these total pipe length used to express the water loss.

Table 4.8: Summary of pipe length by age category

S/N	Age Category	Pipe Length(m)	% from total	Remark
1	10 Years and less	37324.00	53.53	Distribution
2	10 to 20 years	13452.00	15.25	Distribution
3	20 t0 30 years	21674.00	31.22	Gravity main line
	Total length	72450	100.00	

(Source: from Durame Town water supply service office, 2020)

$$\text{Water loss} = \frac{\text{Annual loss}}{\text{length in Km} * 365} = \frac{127{,}893.5}{72.45\text{ km} * 365} = 4.83\text{ m3/km/day}$$

4.2.2 Real and Apparent loss assessment

Non-revenue or total loss is the sum of real loss and apparent loss. However, for this study unable to compute total real loss due to limitation of night hourly consumption data, instead of this unavoidable real loss is computed by using IWA expression. The expression is good indicator of loss and this study focus on two different areas which have relatively high and low average pressure.

4.2.2.1 Unavoidable average real losses (UARL).

It is recommended that the calculation of the UARL in liters/service connection/day is based on the following form of equation. This recognizes separate influences of real losses from length of mains (Lm in km), number of service connections (NC), total length of service connections from the edge of the street to customer meters (Lp in km), and average pressure (P in meters) when the system is pressurized.

$$\mathrm{UARL} = (\mathrm{A} \times \mathrm{Lm}/\mathrm{Nc} + \mathrm{B} + \mathrm{C} \times \mathrm{Lp}/\mathrm{Nc}) \times \mathrm{P}\ (\mathrm{Liters/service\ connection/day})$$

The appropriate values for A (18), B (0.80) and C (25) the equation and its parameters A, B, C are based on statistical analysis of international water supply system data.

Location 1. From 300 m^3 balancing reservoir to 800 m^3 KMG service reservoir.

There for, Length of mains (Lm in km) = 32.21km
Number of service connections (Nc) = 1250
Total length of service connections from the edge of the street to customer meters
average 10m for individual (Lp in km) = 20 m*1250 = 25,000 m = 25 km
Average pressure (P in meters) = 64 m from model pressure result of these pipe line

$\mathrm{UARL} = \left(\mathrm{A} \times \frac{\mathrm{Lm}}{\mathrm{Nc}} + \mathrm{B} + \mathrm{C} \times \frac{\mathrm{Lp}}{\mathrm{Nc}}\right) \times \mathrm{P} = \left(\frac{18*52}{1250} + 0.8 + \frac{25*25}{1250}\right) * 85$ = 174.122lit/ service connection.

Annual volume UARL = 174.122 Lit *365 = 63554.71 lit/connection/year.

Total annual UARL in all connection = 63554.71 lit/connection/year*1250 connection = 79, 443.387 m^3/year.

Location 2. From 800 m^3 ground reservoir to the down part of service reservoir.

There for, Length of mains (Lm in km) = 40.24 km.

Number of service connections (N c) = 4328.

Total length of service connections from the edge of the street to customer meters is average 10m for individual (LP in km), 20 m*4,328 = 86,560 m = 86.56 km.

Average pressure (P in meters) =18m, from model result for distribution system.

$\mathrm{UARL} = \left(\mathrm{A}\,\mathrm{x}\frac{\mathrm{Lm}}{\mathrm{Nc}} + \mathrm{B} + \mathrm{C}\,\mathrm{x}\frac{\mathrm{Lp}}{\mathrm{Nc}}\right)\mathrm{x}\,\mathrm{P} = \left(\frac{18*40.24}{4328} + 0.8 + \frac{25*86.56}{4328}\right)*8$ = 26.28lit/service connection/day.

Annual volume UARL = 26.28lit *365 = 9,592.2 lit/connection/year

Total annual UARL in all connection = 9,592.2 lit/connection /year*4328

$= \frac{9{,}592.2\ \mathrm{lit}}{\mathrm{connection}}/\mathrm{year} * 4328\ \mathrm{Connection}$ = 41,515 m^3/ year

Total sum from location 1 and 2 = 120,958 m^3/year

Apparent loss = NRW-UARL

NRW of 2020 used from the Appendix D4 is 127,893.5 m^3/year

Apparent loss = 127,893.5 m^3/year-120,958 m^3/year = 6,935 m^3/year

From the above descriptions, apparent loss was low in volume, and it covers 1.76% of total volume of water losses in Durame town water distribution system. While, physical losses were also contribute unconsidered volume of loss in the system and it covers 30.81% of total NRW.

4.3 Leakage Management Practice

Good leakage management practice is one of the ways of reducing water loss in the distribution system. The leakage management trends of Durame Town water supply service office were assessed and described as;

4.3.1 Water utility organization

A strong management and organizational setup is the main reason for good leakage management practice in the water supply services. According to Durame Town water service office, the total numbers of staff members including supporting sections are 38 in number. Of these seven are professionals from technique section (one electrician, one electromechanical, three plumbers and two operators). Based on the policy and guidelines of the regional administration, the town water supply service manager can organize, directs and administers the overall activities of the water service unit. While, regard to the leakage management view, the operation and maintenance section of the utility is the most important section in the water supply service and the head of this section shall be accountable to the manager of the water service office. The main activities of this section are direct, coordinate and control the water supply operations and maintenances work including production, distribution, leakage and laboratory units.

4.3.2 Operation and maintenance practice

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

4.3.2.1 Operation system

There is no operators dwelling and permanent operators who control the above assets in the treatment plant site.

- ❖ Mostly the operation system was controlled by the utility guards (non-skilled man power) who protect the intake and treatment plant site.
- ❖ Recently there is no controlling and operating mechanism at service reservoir due to empty of water in the tank.
- ❖ There is the practice of recording treated water production and consumption figures by the utility experts.

4.3.2.2 Maintenance practice

The overall maintenances of system components were not checked by schedule and it was maintained during failure or damage is occurring.

- ❖ Valves and all accessories installed in the main were not properly used and maintained regularly (greasing and cleaning). Such as, the PRV in the distribution network was not functional all the operation time, and it made challenges to limit pressure in the network.

- ❖ No functional controlling check valves within the network. Accordingly, when failure is happened; the utility were maintaining the system by switching off the pumps operating at pumping station. Whereby, the technique group forced to stay until the water pressure was reduced in the system.
- ❖ Maintaining, removing and replacing of both customers service line and bulk flow meters were done when the utility was reported or requested by the customers during failure is appear.
- ❖ The other collected information was due to limitation of budget; when failure is occur at customer's service line, the customers forced to supply all the necessary accessories. Accordingly, until the required accessories is purchased and replace d a considerable quantity of water is lost.

4.3.3 Financial analysis

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

4.3.3.1 Water tariff policies

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

As per the town water service office, the tariff structure is adopted as flat and graded rate. Table 4.9 shown that; public fountain users are charged flat rate i.e. the same rate for all consumption. While, house and yard connection users are charged progressive rate polices i.e. the tariff rate increases with the consumption volume of water.

Table 4.9: Durame Town water tariff system

Block	Consumption range (m^3/month)	Tariff (ETB)/m^3
1	0-5	8.00
2	6-10	9.00
3	10-30	9.5
4	Above30	10.30

(Source: from Durame Town survey result, 2020)

From Table 4.9, taken the average unit price of 8.16 birr and financially in between four years the water utility were collect 1,702,672.944 birr from authorized revenue water. But, similarly in between four years, the authority was lost an estimated of 820,154.286 birr as result of total water loss.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The water supply of acceptable quality and adequate quantity of distribution is one of the basic needs of human beings, but the provisions of potable water distribution for Durame Town is not efficient.

- ❖ The situation is getting worse due to the population growth, spatial expansion of the town and number of technical and management problem which outstripped its ability to supply sufficient water for its inhabitants.
- ❖ Existing water distribution system of Durame Town was established for an estimated population of 24,472. But, as compared with the current projected population figure of 90,390, it was totally served beyond the design period too much high demand of water supply in the town.
- ❖ Finally, for various demand categories, the current average per capita water consumption of Durame Town was found as 70 l/c/d. besides comparing the maximum water demand of the town (6392.304 m^3/day), in contrast the size of existing infrastructural components such as Bore holes, clear water reservoir, pumping station transmission and distribution pipes leads to supplying water intermittently and insufficiently. Thereby, but not in everywhere in the system water pressure in the distribution network system observed that were not performing within the proposed maximum and minimum design criteria set by MoWIE.
- ❖ In general, Durame Town water distribution network system were exposed to large volume of water loss especially in high pressure zone areas, while during high demand time mostly residences found in dense population and higher level of the town were not received and they served continuous water from the system in town water distribution network system.

5.2 Recommendations

Depending on the analyzed findings the following recommendations were mentioned to Durame Town existing water supply distribution system:

- ❖ As the current water demand and supply in the town is much greater of the daily water production of the system, so it is necessary revising the design and rehabilitates the water distribution system by functioning new non functioned reservoirs improving their capacity and replacing the new pumps with the required hydraulic performance. For distribution network, need fixing air release valve for low pressure zone and controlling the water flow by gate valve for location of high pressure zone.
- ❖ Pipe rehabilitation decision should be involved by considering that most of existing pipe is still an asset for the utility, it is most advisable and recommended installation of a parallel main line with larger diameter than replacement of the old line. While, cleaning and removing deposits from the old pipeline walls should be also advised to improve flow through the pipeline and restoring lost carrying capacity in the mains.
- ❖ To control and protect the pump sets against high pressure conditions and broken pipe situations, the pump station should be provided with new switchboards.
- ❖ To control and minimize risks related with variation of pressure, water hammer and back water flow; it was advised installing the necessary valves and accessories in the water distribution system.
- ❖ During water loss analysis, it was observed that large amount of water were lost as result of Illegal connection and data handling problems. So that, the water authority should be provides customer awareness programs and should be encouraged to report illegal connections, and regulations should be in place to penalize the water thieves and while, the town water utility should be improved their data handling system by supporting with computerized recording technology.
- ❖ During leakage management practice, the water authority should be pay enough attention and put forward dimensions on water leakage management strategies through the processes with involving engineering approaches and international experiences.

- In general, management of Durame Town water supply distribution system totally needs improvement to maximize the overall performance of the utility. Locally there are a number of water supply services that operate efficiently with the principle of cost recovery and full autonomy. Visiting such institutions would behelpful to improve the performance of the board, management and staffs. Timely updating, upgrading and acquaintance of staffs with innovative ideas, new technologies with clear purpose and change is a tool for improvement of the utility.

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APPENDIXES

APPENDIX A: ELEVATION DATA FOR SERVICE RESERVOIRS

Description	Coordinates		
	X	Y	Z (Elevation, a.m.s.l)
Existing Service Reservoirs			
Spring1	374923	800287	2061.78
BH 1	376909	797810	1970.68
BH 2	376784	797389	1967.45
BH 3	380606	803030	2083.63
BH 4	380146	803009	2088.97
BH 5	380238	802482	2090.78
BH 6	377493	799305	2022

APPENDIX B: POPULATION AND DEMAND PROJECTION OF DURAME TOWN (2020-2040)

Description		Year Phase I				Year phase II	
	Unit	2020	2021	2025	2030	2035	2040
Population Growth Rate		4.80%	4.80%	4.80%	4.05%	3.65%	3.25%
Projected/Forecasted population	No	90390	94729	113074	134444	154760	171365
Population Percentage Distribution by Mode of service							
HTU	%	1.66	2.81	7.41	13.16	18.91	24.66
YTU	%	38.72	40.05	45.37	52.02	58.67	65.32
NTU	%	1.11	1.2	1.56	2.01	2.46	2.91
PTU	%	58.51	55.94	45.66	32.81	19.96	7.11
Total	%	100	100	100	100	100	100
Population served by							
HTU	No	1500	2661	8378	17692	29264	42258
YTU	No	35000	37940	51303	69939	90800	111937
NTU	No	1000	1133	1760	2697	3801	4980
PTU	No	52890	52994	51633	44115	30895	12189

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	260.6	884.74	2,038.13	3,652.20	5,679.43
YTU	m3/d	1,680.00	1,839.33	2,585.67	3,692.79	5,012.14	6,447.59
NTU	m3/d	57.6	66.39	109.98	181.78	274.8	384.41
PTU	m3/d	2,030.98	2,061.43	2,111.59	1,914.25	1,417.71	589.77
Total Domestic Demand	m3/d	3,912.58	4,227.75	5,691.98	7,826.95	10,356.84	13,101.20
	l/s	45.28	48.93	65.88	90.59	119.87	151.63
Socio- Economic Factor		1	1	1	1	1	1
Climatic Factor		1	1	1	1	1	1
Adjusted Domestic Water Demand (ADD)	m3/d	3,912.58	4,227.75	5,691.98	7,826.95	10,356.84	13,101.20
	l/s	45.28	48.93	65.88	90.59	119.87	151.63
Non Domestic Water Demand							
Small Scale Industrial Water Demand	m3/d	-	-	-	-		-
(small industries 5% of ADD)	l/s	-	-	-	-		-
Commercial & Instituti onal Water Demand with allowance of small scale industry (25% of ADD)	m3/d	978.14	1,056.94	1,423.00	1,956.74	2,589.21	3,275.30
	l/s	11.32	12.23	16.47	22.65	29.97	22.75
Livestock Water Deman d (0% of ADD)	m3/d	-	-	-	-	-	-
	l/s	-	-	-	-	-	-
Total Demands	m3/d	4,890.72	5,284.68	7,114.98	9,783.69	12,946.05	16,376.50
	l/s	56.61	61.17	82.35	113.24	149.84	174.38
Unaccounted for Water (UFW) “non-revenue-water” UFW (15-25% of TAD)	m3/d	733.61	819.13	1,245.12	1,956.74	2,912.86	4,094.12
	l/s	8.49	9.48	14.41	22.65	33.71	43.59
UFW (15-25%)		15	15.5	17.5	20	22.5	25
Average Day Water Demand	m3/d	5,624.33	6,103.81	8,360.10	11,740.43	15,858.92	20,470.62
	l/s	65.1	70.65	96.76	135.88	183.55	217.97

Max Day Factor		1.2	1.2	1.2	1.2	1.2	1.2
Max Day Demand	m3/d	6,749.19	7,324.57	10,032.12	14,088.51	19,030.70	24,564.75
	l/s	78.12	84.78	116.11	163.06	220.26	261.57
	m3/hr	281.22	305.19	418	587.02	792.95	1,023.53
Peak Hour Factor		1.8	1.8	1.8	1.8	1.8	1.8
Peak Hour Demand	m3/d	10798.71	11719.31	16051.39	22541.62	30449.12	39303.6
	l/s	124.99	135.64	185.78	260.9	352.42	418.51

APPENDIX C: WATER LOSS ANALYSIS TABLE

Year	Prod.(m^3/year)	Cons (m^3/year)	Loss (M3/Year)	Loss (%)
2017	208,350	139,965	68,385	32.82
2018	297,570	201,705	95,865	32.21
2019	338,270	228,377.1	109,892.9	32.48
2020	392,490	264,596.5	127,893.5	32.58
Yearly Average	309,170	208,660.9	100,509.1	32.50

APPENDIX D 1: DISCHARGE YIELD FROM DIFFERENT POTENTIAL SOURCES

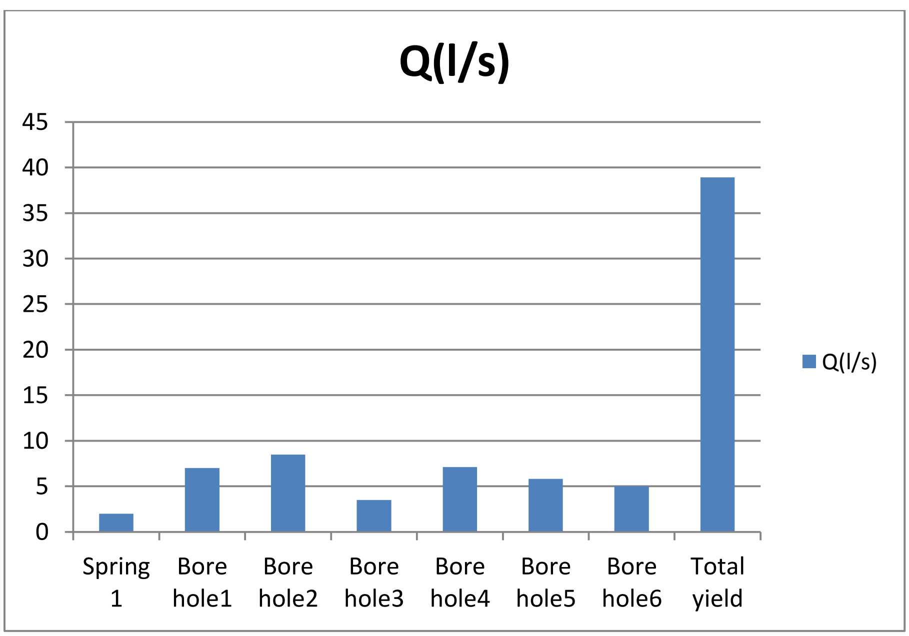

Figure 4.1: Discharge yield from different potential Sources

APPENDIX D 2: HAMBERICHO SPRING OLD SERVICE RESERVOIR

Figure 4.2: Hambericho spring old service reservoirs (source; field observation, Jan 08, 2020)

APPENDIX D 3: PATTERNS FOR HOURLY DEMAND MULTIPLIER FACTORS

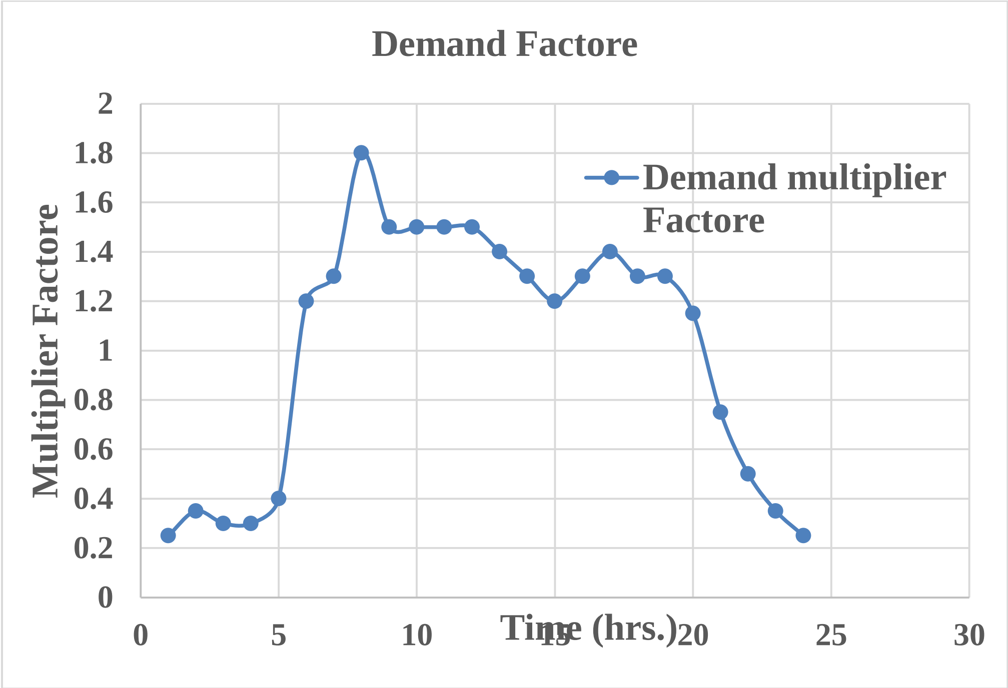

Figure.4.3: Patterns for hourly demand factors

APPENDIX D 4: WATER LOSS CATEGORIES

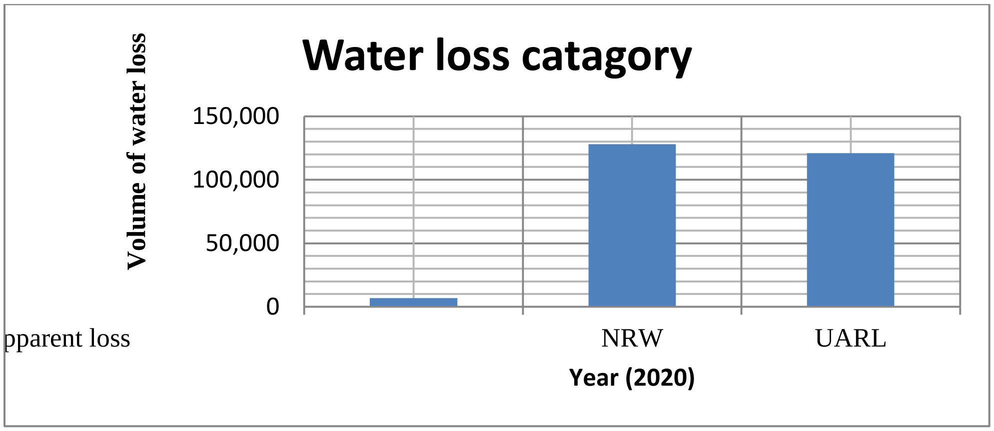

Figure 4.10: Water Loss Categories

APPENDIX E: ROUGHNESS COEFFICIENT, C-FACTOR FOR DIFFERENT PIPE MATERIAL

Types of pipe material	1.0 In (2.5cm)	3.0 in (7.60cm	6.0 in (15.2cm)	12 in (30cm)	24 in (61cm)	48in (122cm)
Uncoated cast iron-smooth and new		121	125	130	132	134
Coated cast iron- smooth and new		129	133	138	140	141
30 years old						
Trend1- sever attack		100	106	112	117	120
Trend2- moderet attack		83	90	97	102	107
Trend3- appriciable attack		59	70	78	83	89
Trend4-sever attack		41	50	58	66	73
60years old						
Trend1- sever attack		90	97	102	107	112
Trend2- moderate attack		69	79	85	92	96

Trend3-appreciable attack		49				78
Trend4- sever attack		30	39	48	56	62
100years old						
Trend1- sever attack		81	89	95	100	104
Trend2-moderate attack		61	70	78	83	89
Trend3-appreciable attack		40	49	57	64	71
Trend4- sever attack		21	30	39	46	54
Miscellaneous						
Newly scrounged mains		109	116	121	125	127
Newly brushed mains		97	104	108	112	115
Coated spun iron-smooth and new		137	142	145	148	148
Old take as coated cast iron-of the same age						
Galvanized iron smooth and new	120	129	133	134	137	142
Wrought iron-smooth and new	129	137	142	145	148	148
Coated steel-smooth and new	129	137	142	145	148	148
Uncoated steel-smooth and new	134	142	145	147	150	150

APPENDIX F 1: TIME SERIES REPRESENTATION OF PRESSURE VALUE FOR PEAK DEMAND

Item	Sample taken point	Location			Measured time	Computed Pressure (m)	Observed pressure (m)	Errors (m)
		X(m)	Y(m)	Z(m)				
1	J-2	377,392.03	799,559.93	2,049.95	2:00	21	19	2
2	J-11	379,449.88	802,893.59	2,117.44	3:00	44	44.49	-0.49
3	J-20	378,988.41	802,166.52	2,161.24	3:30	22	22.4	-0.4
4	J-32	380,292.37	803,862.89	2,117.33	4:00	12	9.6	2.4
5	J-62	378,425.64	800,938.11	2,117.73	4:30	56	56.46	-0.46
6	J-65	378,689.72	799,899.65	2,060.36	5:00	10	9.8	0.2
7	J-126	378,333.73	800,278.58	2,089.41	5:30	35	34.71	0.29
R^2= 0. 9894								

APPENDIX F 2: TIME SERIES REPRESENTATION OF PRESSURE VALUE FOR LOW DEMAND

Item	Sample taken point	Location			Measured time	Computed pressure (m)	Observed pressure (m)	Errors (m)
		X(m)	Y(m)	Z(m)				
1	J-2	2,049.95	377,392.03	799,559.93	2:00	51	47.17	3.83
2	J-11	2,160.79	379,294.43	801,127.78	2:30	62	61.14	0.86
3	J-20	2,117.44	379,449.88	802,893.59	3:00	37	37.29	-0.29
4	J-32	2,161.24	378,988.41	802,166.52	3:30	79	78.26	0.74
5	J-62	2,117.33	380,292.37	803,862.89	4:00	85	84.26	0.74
6	J-65	2,117.73	378,425.64	800,938.11	4:30	40	38.4	1.6
7	J-126	2,060.36	378,689.72	799,899.65	5:00	75	75.35	-0.35
R^2= 0.9985								

APPENDIX G: PUMP RESULT: CALCULATED WATER POWER (KW)

Time (Hr.)	Calculated Water Power (kW)	Time (Hr.)	Calculated Water Power (kW)
0:00:00	36.22	13:00:00	27.16
1:00:00	36.22	14:00:00	35.12
2:00:00	36.22	15:00:00	0.00
3:00:00	36.22	16:00:00	0.00
4:00:00	0.00	16:00:00	31.33
5:00:00	0.00	17:00:00	19.33
6:00:00	31.12	18:00:00	32.42
7:00:00	19.55	19:00:00	18.47
8:00:00	29.43	20:00:00	32.42
9:00:00	0.00	21:00:00	0.00
10:00:00	0.00	22:00:00	0.00
11:00:00	31.4	23:00:00	31.15
12:00:00	19.2	24:00:00	19.32

APPENDIX H: JUNCTIONS (NODAL) PRESSURE RESULT; FOR PEAK DEMAND TIME

Label	Demand Calculated(l/s)	Hydraulic Grade (m)	Pressure (m H2O)
J-	0.01	2,152.86	35
J-1	0.01	2,070.07	22
J-2	0.01	2,070.87	21
J-3	0.01	2,158.76	41
J-4	0	2,157.28	40
J-5	0	2,177.29	45
J-6	0.01	2,172.06	47
J-7	0.01	2,196.21	35
J-8	0.01	2,195.06	33
J-9	0.01	2,155.30	38
J-10	0.01	2,181.30	63
J-11	0.01	2,161.78	44
J-12	0.01	2,161.24	44
J-13	0.01	2,185.01	41
J-14	0.01	2,186.43	54
J-15	0.01	2,163.89	22
J-16	0.01	2,162.69	38
J-17	0.01	2,129.37	10
J-18	0	2,071.77	20
J-19	0.01	2,075.59	37

J-20	0.01	2,183.48	22
J-21	0.01	2,165.69	48
J-22	0.01	2,181.94	20
J-23	0.01	2,175.44	20
J-24	0.01	2,178.67	18
J-25	0.01	2,187.09	68
J-26	0.01	2,167.78	42
J-27	0.01	2,188.93	70
J-28	0.01	2,184.29	66
J-29	0.01	2,172.11	22
J-30	0.01	2,152.12	35
J-31	0.01	2,191.77	38
J-32	0.01	2,129.38	12
J-33	0.01	2,172.52	10
J-34	0.01	2,075.17	10
J-35	0.01	2,083.50	27
J-36	0.01	2,190.97	38
J-37	0	2,167.24	47
J-38	0.01	2,185.95	47
J-39	0.01	2,187.66	36
J-40	0.01	2,023.88	10
J-41	0.01	2,053.28	33
J-42	0.01	2,156.55	16
J-43	0.01	2,193.63	24
J-44	0.01	2,191.38	25
J-45	0.01	2,163.29	46
J-46	0.01	2,062.32	38
J-47	0.01	2,046.08	26
J-48	0.01	2,191.15	42
J-49	0.01	2,097.89	15
J-50	0.01	2,084.77	40
J-51	0.01	2,168.91	18
J-52	0.01	2,052.97	32
J-53	0	2,196.05	58
J-54	0	2,196.63	56
J-55	0.01	2,154.09	11
J-56	0.01	2,164.09	46
J-57	0.01	2,193.46	44
J-58	0.01	2,181.05	45
J-59	0.01	2,069.49	23
J-61	0.8	2,020.98	10
J-62	0	2,335.50	56
J-63	0.01	2,023.47	10

J-64	0.01	2,070.71	14
J-65	0.01	2,070.37	10
J-66	0.01	2,066.16	31
J-67	0.01	2,155.23	38
J-69	0.01	2,192.90	53
J-71	0.01	2,165.78	48
J-73	0.04	2,180.40	15
J-74	0.07	2,182.96	51
J-81	0.07	2,101.07	51
J-83	0.08	2,100.44	37
J-85	0.09	2,007.90	36
J-98	0. 09	2,072.58	43
J-103	0.07	2,100.95	47
J-112	0.06	2,160.43	51
J-113	0.05	2,170.73	59
J-114	0.04	2,161.61	47
J-115	0.02	2,154.94	40
J-116	0	2,194.38	66
J-125	0	2,145.63	68
J-126	0	2,146.69	35
J-127	0.01	2,189.77	56
J-128	0.01	2,190.86	58
J-129	0.01	2,174.31	48

APPENDIX I: JUNCTIONS (NODAL) PRESSURE RESULT; FOR NIGHT FLOW/ LOW DEMAND TIME

Label	Demand (Cumulative) (ML)	Elevation point(m)	Pressure (m H2O)
J-		2,197.36	80
J-1	0.02	2,100.93	80
J-2	0.04	2,100.95	51
J-3	0.04	2,100.52	49
J-4	0.04	2,100.47	47
J-5	0.002	2,100.10	47
J-6	0	2,197.92	73
J-7	0.01	2,198.77	78
J-8	0.01	2,198.73	36
J-9	0.02	2,197.44	48
J-10	0.02	2,197.25	57
J-11	0.01	2,199.65	62
J-12	0	2,197.63	80
J-13	0	2,198.40	75
J-14	0.04	2,198.45	66

J-15	0.04	2,197.72	56
J-16	0.04	2,197.68	53
J-17	0	2,196.61	77
J-18	0	2,100.98	79
J-19	0	2,101.10	63
J-20	0.004	2,177.35	37
J-21	0.003	2,176.59	36
J-22	0.03	2,176.30	36
J-23	0.02	2,198.09	42
J-24	0.02	2,198.19	37
J-25	0	2,198.17	70
J-26	0	2,197.80	72
J-27	0	2,198.53	78
J-28	0	2,198.35	80
J-29	0	2,197.98	77
J-30	0	2,197.30	80
J-31	0	2,198.59	78
J-32	0	2,196.61	79
J-33	0.01	2,198.00	35
J-34	0.01	2,101.09	36
J-35	0.01	2,101.36	45
J-36	0.01	2,198.18	45
J-37	0	2,197.81	78
J-38	0	2,198.39	59
J-39	0	2,198.47	47
J-40	0	2,099.44	85
J-41	0	2,100.39	80
J-42	0.07	2,172.52	32
J-43	0.08	2,198.10	29
J-44	0.07	2,198.10	32
J-45	0.02	2,197.55	80
J-46	0.02	2,100.68	76
J-47	0.02	2,100.16	80
J-48	0	2,198.57	70
J-49	0	2,101.82	18
J-50	0.02	2,101.40	56
J-51	0.02	2,197.88	47
J-52	0	2,100.38	80
J-53	0	2,198.63	61
J-54	0	2,198.67	68
J-55	0	2,172.44	69
J-56	0	2,197.69	58
J-57	0.03	2,198.61	49
J-58	0.03	2,198.23	62
J-59	0.03	2,100.91	54
J-61	0	2,099.35	80
J-62	0	2,335.50	85

J-63	0	2,099.43	76
J-64	0	2,100.95	44
J-65	0.03	2,000.04	40
J-66	0.02	2,100.80	66
J-67	0.02	2,197.40	80
J-69	0	2,198.66	58
J-71	0	2,197.68	80
J-73	0. 06	2,198.25	47
J-74	0	2,198.30	66
J-81	0.02	2,101.07	80
J-83	0	2,100.44	80
J-85	0	2,007.90	153
J-98	0.01	2,072.58	80
J-103	0.05	2,100.95	78
J-112	0.06	2,172.64	67
J-113	0.09	2,450.77	67
J-114	0	2,172.68	58
J-115	0	2,172.47	57
J-116	0.06	2,198.11	32
J-125	0.02	2,197.09	80
J-126	0.02	2,007.12	75
J-127	0.02	2,198.53	69
J-128	0.02	2,198.44	66
J-129	0.02	2,198.00	71

APPENDIX J: PIPE RESULT; DURING PEAK DEMAND TIME

Label	Length (Scaled) (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Head loss Gradient (m/km)
p-1	65	110	HDPE	130	7.17	0.33	0. 45
P-2	555	110	HDPE	130	8.34	0.35	0.197
P-3	570	65	HDPE	130	8.13	0.35	1.324
P-4	229	63	HDPE	130	8.13	0.21	1.84
P-4-1	528	45	HDPE	130	7.21	0.67	14.922
P-5	250	75	HDPE	130	7.23	0.68	8.442
P-5	153	110	HDPE	130	9.08	0.93	9.612
P-6	87	75	DCI	130	9.12	1.52	17.058
P-7	577	63	HDPE	130	7.12	0.71	11.171
P-8	450	40	HDPE	130	1	0.59	13.2
P-9	418	31.5	HDPE	130	0	0.01	0.007
P-10	588	31.5	HDPE	130	1	0.92	39.813

P-11	261	75	HDPE	130	2	0.49	4.594
P-12	286	63	HDPE	130	1	0.47	5.105
P-14	246	100	DCI	130	7	0.9	10.124
P-15	624	125	DCI	130	7	0.58	3.415
P-16	2,064	125	DCI	130	8	0.65	4.173
P-17	333	125	DCI	130	8	0.65	0
P-19	362	200	uPVC	130	11	0.97	5.137
P-21	31	50	uPVC	130	11.40	0.96	23.402
P-22	375	44	uPVC	130	9	0.76	28.69
P-23	113	63	HDPE	130	6	1.9	22.512
P-23	301	25	uPVC	130	5.22	0.48	15.569
P-24	238	40	DCI	130	5.21	0.76	21.166
P-24	272	50	DCI	130	5.43	0.74	15.688
P-24	339	100	DCI	130	7	0.85	9.038
P-25	90	63	uPVC	130	2	0.53	6.535
P-26	217	100	GI	130	6	0.76	7.39
P-27	183	90	DCI	130	6	0.96	12.667
P-28	1,333	75	DCI	130	3	0.65	7.716
P-29	123	50	DCI	130	2	1.22	39.672
P-30	74	25	HDPE	130	0	0.83	43.609
P-30	102	40	DCI	130	2	1.27	55.515
P-32	2,293	110	DCI	130	8	0.79	7.039
P-33	111	150	Ductile Iron	130	12	0.67	3.651
P-34	1,577	150	Ductile Iron	130	12	0.67	3.651
P-35	193	75	HDPE	130	4	0.98	16.294
P-35	106	50	Ductile Iron	130	1	0.47	6.682
P-36	163	75	HDPE	130	4	0.81	11.617
P-36	786	62.5	Ductile Iron	130	1	0.3	2.254
P-37	114	50	Ductile Iron	130	1	0.72	14.809
P-38	419	40	Ductile Iron	130	1	0.7	18.465
P-39	46	63	Ductile Iron	130	2	0.74	11.826
P-44	240	75	HDPE	130	3	0.61	6.904
P-46	184	32	HDPE	130	0	0.49	12.168
P-47	193	40	HDPE	130	1	0.85	26.458

P-47	536	40	HDPE	130	1	1.1	42.269
P-49	749	25	HDPE	130	0	0.53	18.753
P-49	622	25	HDPE	63	0	0.12	4.644
P-50	230	40	HDPE	130	1	0.53	10.8
P-51	1,283	25	DCI	130	0	0.12	1.304
P-52	757	63	HDPE	130	3	0.9	17.361
P-53	524	63	HDPE	63	1	0.62	55.995
P-54	264	45	HPE	130	0	0.18	1.314
P-54	398	50	HDPE	130	2	0.94	24.449
P-54	685	63	HDPE	130	2	0.67	10.059
P-55	402	40	HDPE	130	1	0.9	29.057
P-56	298	75	DCI	130	-1	0.74	9.644
P-56	345	63	HDPE	130	2	0.71	11.131
P-57	544	50	HDPE	130	1	0.76	16.592
P-57	978	25	DCI	130	0	0.29	6.09
P-58	283	90	HDPE	130	3	0.49	3.664
P-59	193	40	DCI	130	1	0.95	32.184
P-60	474	45	HDPE	130	1	0.77	19.252
P-61	282	125	DCI	130	14	1.13	11.791
P-62	393	125	DCI	130	13	1.04	10.066
P-62	597	40	HDPE	130	1	0.41	6.709
P-63	193	90	HDPE	130	8	1.21	19.612
P-63	277	40	DCI	130	1	0.66	16.552
P-64	227	144	HDPE	130	1.22	0.61	5.528
P-64	317	150	DCI	130	1	1.05	29.721
P-65	406	40	DCI	130	2.31	0.88	27.898
P-66	204	50	DCI	130	2.31	0.73	15.403
P-66	561	150	uPVC	130	14	0.77	4.724
P-67	186	125	HDPE	130	11	0.91	7.823
P-67	344	37.5	HDPE	130	1.12	0.78	24.15
P-68	293	90	HDPE	130	3.04	0.91	11.46
P-68	1,199	90	HDPE	130	3.04	0.88	10.929
P-69	402	50	DCI	130	1.34	0.57	9.57
P-69	1,240	90	HDPE	130	-2	1.15	17.98
P-70	295	75	HDPE	130	1	0.54	5.471
P-74	335	32	DCI	130	1	1.07	52.282
P-75	65	75	HDPE	130	0. 21	1.02	17.697
P-76	148	100	DCI	130	1.1	1.44	23.96
P-77	91	125	HDPE	130	1.4	1.11	11.4

P-78	274	100	DCI	130	10	1.29	19.605
P-79	114	75	DCI	130	11	1.6	20.815
P-80	112	75	DCI	130	6.22	1.42	12.685
P-81	106	50	HDPE	130	3.45	1.36	28.41
P-82	387	50	DCI	130	3.45	1.29	24.142
P-83	157	40	DCI	130	11	0.71	18.685
P-84	127	110	HDPE	130	10	1	10.951
P-85	80	25	DCI	130	0	0.78	38.881
P-85	320	125	DCI	130	11	0.91	7.809
P-86	403	20	DCI	130	0	0.31	9.085
P-87	24	40	HDPE	130	2	1.49	74.273
P-131	610	40	HDPE	130	1	0.52	10.643
P-132	750	25	HDPE	130	0	0.13	1.419
P-133	478	32	HDPE	130	1	0.98	43.943
P-134	292	75	DCI	130	2	0.5	4.66
P-135	603	75	DCI	130	4	0.84	12.217
P-137	273	37.5	DCI	130	1	0.7	19.92
P-139	271	40	HDPE	130	1	0.71	18.867
P-140	282	63	HDPE	130	0.82	0.98	20.277
P-144	317	150	DCI	130	1	1.05	29.721
P-145	77	50	DCI	130	2	1.01	28.095
P-146	218	32	HDPE	130	1	0.79	29.658
P-147	238	40	DCI	130	0.76	0.89	28.377
P-148	230	32	HDPE	130	0	0.52	13.62
P-149	367	32	DCI	130	1	1.06	51.007
P-156	207	32	DCI	130	1	0.9	37.515
P-A	27	90	HDPE	130	7	1.03	14.581
P-B	36	90	HDPE	130	7	1.08	15.925
P-C	41	90	HDPE	130	7	1.16	18.047
P-C	3,293	75	DCI	130	8	1.77	49.081
P-E	96	40	DCI	130	1	0.64	15.378
P-F	1,041	100	DCI	130	7	0.89	9.908
P-G	116	90	DCI	130	7	1.17	18.41
P-G	123	40	GI	130	1	0.64	15.379
P-H	148	90	DCI	130	8	1.28	21.845
P-H	55	40	GI	130	2	1.27	55.515
P-I	3,339	75	DCI	130	8	1.77	49.081
S1	261	100	DCI	130	7	0.85	0
S2	210	125	DCI	130	7	0.58	0

S3	446	100	DCI	130	6	0.76	0
S4	215	80	DCI	130	7	1.41	0.08

APPENDIX K: PIPE RESULT; DURING LOW DEMAND TIME/NIGHT FLOW

Label	Length (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (l/s)	Velocity (m/s)	Head loss Gradient (m/km)
p-1	65	110	HDPE	130	12.44	0.33	0.045
P-2	555	110	HDPE	130	14.22	0.35	0.197
P-3	570	65	HDPE	130	14.20	0.35	1.324
P-4	229	63	HDPE	130	8.32	0.21	0.84
P-4-1	528	45	HDPE	130	7.45	0.11	0.68
P-5	250	75	HDPE	130	6.04	0.11	0.471
P-5	153	110	HDPE	130	4.12	0.15	0.309
P-6	87	75	DCI	130	1	0.24	1.191
P-7	577	63	HDPE	130	0	0.11	0.359
P-8	450	40	HDPE	130	0	0.09	0.424
P-9	418	31.5	HDPE	130	0	0	0
P-10	588	31.5	HDPE	130	0	0.14	1.279
P-11	261	75	HDPE	130	0	0.08	0.148
P-12	286	63	HDPE	130	0	0.07	0.164
P-14	246	100	DCI	130	1	0.9	5.124
P-15	624	125	DCI	130	2.23	0.58	3.415
P-16	2,064	125	DCI	130	8	0.65	3.174
P-17	333	125	DCI	130	8	0.65	0.32
P-19	362	200	uPVC	130	5	0.16	0.188
P-21	26	20	HDPE	130	17.44	0.03	4.58
P-22	375	125	HDPE	130	1	0.12	0.186
P-23	113	63	HDPE	130	1	0.3	2.201
P-23	301	25	uPVC	130	0	0.15	1.763
P-24	238	40	DCI	130	0	0.2	1.865
P-24	272	50	DCI	130	11.78	0.18	1.112
P-24	339	100	DCI	130	12.07	0.86	1.543
P-25	90	63	uPVC	130	15.21	0.13	0.473
P-26	217	100	GI	130	6.12	0.77	0.75
P-27	183	90	DCI	130	4.12	0.15	0.407
P-28	1,333	75	DCI	130	0	0.1	0.248
P-29	123	50	DCI	130	0	0.19	1.275
P-30	74	25	HDPE	130	0	0.09	0.747

P-30	102	40	DCI	130	0	0.2	1.783
P-32	2,293	110	DCI	130	0	0.01	0.001
P-33	111	150	Ductile Iron	130	12	0.66	3.527
P-34	1,577	150	Ductile Iron	130	12	0.66	3.526
P-35	193	75	HDPE	130	0	0.11	0.283
P-35	106	50	Ductile Iron	130	1	0.33	3.563
P-36	163	75	HDPE	130	0	0.08	0.173
P-36	786	62.5	Ductile Iron	130	1	0.21	1.202
P-37	114	110	Ductile Iron	130	0	0.06	0.13
P-38	419	40	Ductile Iron	130	0	0.09	0.383
P-39	46	63	Ductile Iron	130	0	0	0
P-44	26	20	HDPE	130	17.44	0.03	4.58
P-46	184	32	HDPE	130	0	0.1	0.635
P-47	193	40	HDPE	130	0	0.13	0.864
P-47	536	40	HDPE	130	0	0.17	1.358
P-49	749	25	HDPE	130	0	0.08	0.578
P-49	622	25	HDPE	63	0	0.02	0.149
P-50	230	40	HDPE	130	0	0.08	0.347
P-51	1,283	25	DCI	130	0	0.02	0.049
P-52	757	63	HDPE	130	0	0.14	0.558
P-53	524	40	HDPE	63	0	0.1	1.799
P-54	264	45	HPE	130	0	0.03	0.043
P-54	398	50	HDPE	130	0	0.14	0.696
P-54	685	63	HDPE	130	0	0.11	0.323
P-55	402	40	HDPE	130	0	0.13	0.762
P-56	298	75	DCI	130	0	0.11	0.295
P-56	345	63	HDPE	130	-1	0.11	0.358
P-57	544	50	HDPE	130	0	0.12	0.533
P-57	978	25	DCI	130	0	0.06	0.377
P-58	283	90	HDPE	130	0	0.07	0.105
P-59	193	40	DCI	130	0	0.16	1.231
P-60	474	45	HDPE	130	0	0.12	0.619
P-61	282	125	DCI	130	2	0.18	0.395
P-62	393	125	DCI	130	2	0.17	0.336
P-62	597	40	HDPE	130	0	0.06	0.216

P-63	193	90	HDPE	130	1	0.19	0.63
P-63	277	40	DCI	130	0	0.11	0.619
P-64	227	144	HDPE	130	0.02	0.12	18.22
P-64	317	50	DCI	130	0	0.16	0.955
P-65	406	40	DCI	130	0	0.14	0.886
P-66	204	50	DCI	130	0	0.11	0.495
P-66	561	150	uPVC	130	2	0.12	0.152
P-67	186	125	HDPE	130	-2	0.14	0.251
P-67	344	37.5	HDPE	130	0	0.12	0.776
P-68	293	90	HDPE	130	1	0.14	0.368
P-68	1,199	90	HDPE	130	1	0.14	0.351
P-69	402	50	DCI	130	0	0.02	0.015
P-69	1,240	90	HDPE	130	1	0.18	0.578
P-70	295	75	HDPE	130	-2	0.08	0.152
P-74	335	32	DCI	130	0	0.17	1.701
P-75	65	75	HDPE	130	1	0.17	0.641
P-76	148	100	DCI	130	2	0.26	0.981
P-77	91	125	HDPE	130	2	0.18	0.387
P-78	274	100	DCI	130	2	0.21	0.68
P-79	114	75	DCI	130	1	0.25	1.348
P-80	112	75	DCI	130	1	0.23	1.085
P-81	106	50	HDPE	130	0	0.21	1.565
P-82	387	50	DCI	130	0	0.2	1.427
P-83	157	40	DCI	130	0	0.12	0.679
P-84	127	110	HDPE	130	2	0.16	0.365
P-85	80	25	DCI	130	0	0.12	1.209
P-85	320	125	DCI	130	-2	0.14	0.259
P-86	403	20	DCI	130	0	0.12	1.558
P-87	24	40	HDPE	130	0	0.23	2.404
P-131	610	40	HDPE	130	0	0.08	0.344
P-132	750	25	HDPE	130	0	0.02	0.042
P-133	478	32	HDPE	130	0	0.15	1.404
P-134	292	75	DCI	130	0	0.08	0.143
P-135	603	75	DCI	130	1	0.14	0.425
P-137	273	37.5	DCI	130	0	0.11	0.625
P-	271	40	HDPE	130	0	0.06	0.201

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P-140	282	63	HDPE	130	1	0.17	0.805
P-144	119	63	HDPE	130	1	0.18	0.847
P-145	77	50	DCI	130	0	0.17	1.003
P-146	218	32	HDPE	130	0	0.12	0.927
P-147	238	40	DCI	130	0	0.14	0.956
P-148	230	32	HDPE	130	0	0.09	0.584
P-149	367	32	DCI	130	0	0.17	1.642
P-156	207	32	DCI	130	0	0.14	1.206
P-A	27	90	HDPE	130	1	0.16	0.469
P-B	36	90	HDPE	130	1	0.17	0.512
P-C	41	90	HDPE	130	1	0.18	0.577
P-C	3,293	75	DCI	130	8	1.77	0.0566
P-E	96	40	DCI	130	0	0.1	0.494
P-F	1,041	100	DCI	130	0	0	0
P-G	116	90	DCI	130	1	0.18	0.591
P-G	123	40	GI	130	0	0.1	0.593
P-H	148	90	DCI	130	1	0.2	0.702
P-H	55	40	GI	130	0	0.2	0.56
P-I	3,339	75	DCI	130	8	1.77	0.039
S1	261	100	DCI	130	7	0.86	0
S2	210	125	DCI	130	7	0.58	0
S3	446	100	DCI	130	6	0.77	0
S4	215	80	DCI	130	7	1.41	0.04
S6	29	110	DCI	130	0	0.01	0