ASSESSMENT OF THE IMPACT OF ENGINEERING MEASURES ON EFFECTIVE WATER DEMAND MANAGEMENT OF MOJO TOWN WATER SUPPLY SYSTEM, OROMIA, ETHIOPIA

A MASTER'S THESIS

By

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DEPARTMENT OF CIVIL ENGINEERING (WATER SUPPLY AND SANITARY STREAM)

ADDIS ABABA SCIENCE AND TECHNOLOGY UNIVERSITY

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A Thesis Submitted as Partial Fulfillment for the Degree of Master of Science in Civil Engineering (Water supply and Sanitary engineering)

to

DEPARTMENT OF CIVIL ENGINEERING (WATER SUPPLY AND SANITARY STREAM)

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Certificate

This is to certify that the thesis prepared by **Mr. Nagara Wakgari Futasa** entitled "**Assessment of the impact of engineering measures on effective water demand management of Mojo town water supply system, Oromia, Ethiopia"** and submitted as a partial fulfillment for the Degree of Master of Science complies with the regulations of the University and meets the accepted standards with respect to originality, content and quality.

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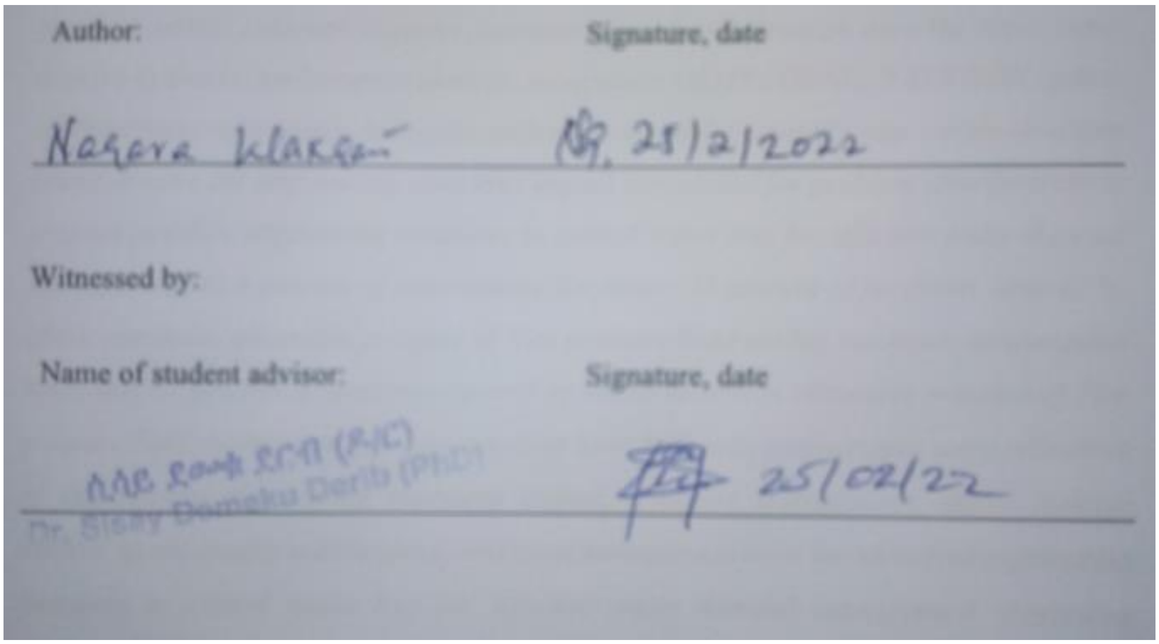
Declaration

I hereby declare that this thesis entitled "Assessment of the impact of engineering measures on effective water demand management of Mojo town water supply system, Oromia, Ethiopia" was prepared by me, with the guidance of my advisor. The work contained herein is my own except where explicitly stated otherwise in the text, and that this has not been submitted in whole or, in part, for any other degree or qualification.



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Abstract

Water loss management is the indicators of water supply structure management for effective water demand management. Water supply service scarcity was a reoccurring concern in the study area's water supply system, despite the presence of water supply source capacity. The objectives of this study are to evaluate water loss, to evaluate hydraulic performance and to indentify possible engineering measures to control water loss. The annual production and consumption were the secondary data collected for water loss analysis using water balance and top-down approach methods. Water source capacity, pipe size, service reservoir capacity, junction demand and elevation were the major input data for hydraulic performance analysis using waterGEMS CONNECT EDITION update 2. Water loss performance, hydraulic performance and the existing water supply structure situation were the engineering measures impact considered for problem identification to propose possible engineering measures to control water loss for effective water demand management. 46.4 percent of unaccounted for water, 19 percent of junctions covered by above maximum allowable pressure of 70m pressure head during maximum consumption hour, and 85 percent of junctions covered by above maximum allowable pressure of 70m pressure head during minimum consumption hour hydraulic performance were indicators of the existing engineering measures impact. Pressure management, active leakage control, speed quality maintenance, and asset management were the identified engineering measures to conrol water loss for effective water demand management. Regarding pressure management simulation using service reservoir and pressure regulating valve result, the implementation of adding service reservoir and pressure regulate valve to the existed distribution network make effective water demand management of Mojo town water supply system by reducing the mean maximum of pressure from 89m pressure head to 47m pressure head; reducing unaccounted for water from 46.4 percent to 27 percent; reducing real loss from 999,307m^3/year to 457,468m^3/year; and increasing domestic per capita consumption from 28.9l/c/d to 39.2 l/c/d. for effective water demand management, the system needs changing direct pumping system to combined system.

***Keywords**: - Water loss, Hydraulic performance, Service reservoir, Pressure reducing valve*

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Abbreviation and Acronyms

AMI	Advanced metering infrastructure
SCADA	Supper control and data acquisition
DCI	Ductile cast iron
DMA	District meter area
DXF	Drawing exchange format
GPS	Global position system
GTP-2	Growth and transformation plan-2
GWP	Global water partnership
HDPE	High-density polyethylene
IWA	International water association
IWA WLSG	International water association water loss specialist group
LPC	Leakage performance categories
LPCD	Liter per capita per day
MoWR	Ministry of water resource
MoWIE	Ministry of water irrigation and energy
MTWSSSE	Mojo town water supply and sewerage service enterprise
MTSESR	Mojo town socio-economic study report
NRW	Non-revenue water
OWWDG	Oromia water work design guideline
Pcc	Per capita consumption
PWC	Palestinian water commission
UFW	Unaccounted for water
uPVC	Un-plasticized polyvinyl chloride
WEDC	Water engineering development center
WDM	Water demand management
WRA	Water resource authority

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CHAPTER ONE

1 INTRODUCTION

1.1 Background of the study

Water is a very crucial resource for all living things. The availability of fresh water for human consumption is a world issue. Out of 100% of global water resources; freshwater content is only 2.5%; the rest percent is saltwater (UNESCO, 2012). Out of 100% freshwater, only 31.1% (44,800km^3/year) is the principal source of human activity (UNESCO, 2012). The quantity of freshwater shows the water to be human consume is limited. Water is a limited resource whose use is regulated to manage and conserve it. Also, water management is part and parcel of daily life activity (Nyende-Byakia et al., 2015).

Safe drinking water and adequate sanitation are very important for poverty reduction and sustainable development. Not only in the low-income countries, achieving universal access to safe and affordable drinking water and also, but universal access to adequate and equitable sanitation and hygiene is also a major challenge in many parts of the world. 884 million people in the world have lack of basic water services and 2.1 billion people who lack water accessible on premises, available when needed and free from contamination (UN, 2018). The provision of adequate and reliable water supply and sanitation service to the rapidly growing population is increasingly becoming a challenge facing many countries worldwide (Qdais, 2003). Ethiopia faces a range of challenges in water management, with levels of service provision for water supply and sanitation that are amongst the lowest in the world (Barron et al., 2007).

Water demand management is the tool that can sustain safe drinking water and adequate sanitation by making better use of the current existing supplies by proper structure management (WEDC, 2011). Water demand management is regarded with efficient use of water and can define as the adaptation and implementation of strategy by water utilities to influence water demand and better use of existing supplies rather than developing new ones to achieve: economic efficiency, social development, and social equity, environmental protection, sustainable water supply services and political acceptance (Qdais, 2003). Poor water demand management in a country of specified town brings scarcity of water supply services, health effects, economical poverty, environmental pollution, and political mismanagement (Qdais, 2003). Engineering measures based on technical and budgetary considerations tend to standardize in terms of component selection, materials, typical design, and installation in order to sustain the hydraulic aspects of the water supply system (Trifunovic, 2006). Proper engineering measures help to avoid problems in the operational management of water supply systems like the frequent interruption of supply, increased water and energy losses, shorter pipe or control lifetime, expensive maintenance of the system, and deterioration of water quality (Trifunovic, 2006). Besides, limited potable water production, water loss in the urban water supply is accounted for more than 50% of the supplies that mainly arise from: leakage of pipe, joints, and valves; overflowing service reservoirs; and waste of water through illegal connections and non-metered house connections (Desalegn, 2005). Water loss in water supply systems ranges from 15% to 30% in the developed world but elsewhere it is likely to range from 30% to 60% (WEDC, 2011). Water demand management aims to minimize loss and waste, protect the water resources, and use water efficiently and effectively (Perto & Richarch, 2018). This problem is abundant in Ethiopian cities, towns, and rural areas (Bhagat et al., 2019).

1.2 Statement of the problem

The Mojo town is one of the Oromia region towns whose population is rapid incremental year to year. Without insufficient water availability; the Mojo town's community exists under inadequate water supply and sanitation service. In the Mojo town, water loss and service delivery performance management were the main problem due to intermixed distribution network of direct pumping to distribution network, and the combined water distribution system. Providing water service from source to customers through the transmission line, distribution network, and flow management was inefficient in the Mojo town. The site has evident problems such as a concealed isolate valve in the earth, flooding, and solid materials. The study area's water supply system has a technical limitation in the form of a pipeline junction built beside natural drainage. The poor structural and operational measures water demand management where brings scarce potable water service.

Figure 1-1 Example of isolate valve situation

Where images "a" and "b" shows isolated valves entirely buried underground, "c" shows an isolate valve manhole filled with flood muck, and "d" shows an isolate valve with no roof covering and filled with solid solid materials.

Figure 1-2 Faulty pipe alignment, and junction setting on flood route

Where “a” shows unplasticed poly vinly chloride, “b” pipe surrounded concrete exist under scouring, “c” rainy season natural drainage, and “d” three direction valve junction and isolate valve manhole

1.3 Objectives of the research

1.3.1 General objective

The general objective of the study is to assess the impact of engineering measures on effective water demand management of the Mojo town water supply system, Oromia, Ethiopia.

1.3.2 Specific objectives

The specific objectives of the research are:

I. To evaluate the water loss in the Mojo town water supply system
II. To evaluate water distribution system hydraulic performance
III. To identify possible engineering measures to minimize water loss for effective water demand management

1.4 The research questions

I. Is the existing water demand management of Mojo town water supply system effective? How much acceptable water loss exists under the Mojo town water supply service?

II. Is the hydraulic performance of the water distribution system within the acceptable range?

III. Are there possible engineering measures to minimize the water loss and to have effective water demand management? which of engineering measures have the best effect on minimizing water loss to have effective water demand management?

1.5. Scope of the study

This research focuses on the assessment of engineering measures to effective water demand management through the evaluation of infrastructure performance analysis and pressure management for leakage reduction. It also focuses on the field to determine the physical performance of existing physically observable water supply facilities, to ensure that their installation follows the country's water supply design guidelines, and to determine what engineering measures are required to maintain the existing water scarcity and water supply infrastructure performance. Rather than focusing on water quality, the study concentrates on water quantity, which is influenced by infrastructural performance.

1.6. Significance of the study

The research has the potential to minimize domestic water scarcity, water loss and make a good operational performance by identifying the suit engineering measures for the Mojo town water supply system. The findings can assist water utility workers and experts in determining how infrastructure performance affects water delivery and what remedial steps should be taken to preserve infrastructure performance while successfully managing water demand.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 Water demand

Water demand is total water volume mobilized to meet different uses including water lost during distribution and uses (GWP, 2012). Under of water supply system, water demand is often monitored at supply points where the measurements include leakage as well as the quantity used to refill the balancing tank that may exist in the system. In this interim balancing tank, water demand agreed to equivalent the summation of water consumption and leakage. Furthermore, when supply is calculated without having interim water storage that means water directly goes to the water distribution network, water demand is equivalent to water production (Trifunovic, 2006). Population and water consumption estimates are the basis for determining the demand of a water supply and distribution system (Guyer et al., 2012).

Domestic water demand is the amount of water needed for drinking, cleaning, food preparation, bathing, laundry, toilet flushing, gardening, and other use in the living area (Butler & Memon, 2006). 50l/person/day is The minimum basic water requirement for basic four human needs: drinking water for survival, water for hygiene, basic water for sanitation, and for preparing food (Butler & Memon, 2006). Average quantity about 50l/c/d intermediate access within 5 minutes total collection time be assured for consumption, hygiene, sanitation and it is enough for low-level health concern (Howard & Bartram, 2003). Domestic water consumption varies according to the mode of service, climatic condition, socio-economic and other factors (MoWR, 2006). To increase safe water supply and upgrade the water service level of urban population dwellers, the second growth and transformation plan (GTP-2) explain per capita consumption by town level as follows (MoWIE, 2016):

Table 2- 1 Domestic per capita demand by town level

Town level	Population number	Per capita demand (l/c/d)
Rank-1	>1,000,000	100
Rank-2	100,000 – 1,000,000	80
Rank-3	50,000 – 100,000	60
Rank-4	20,000 – 50,000	50
Rank-5	<20,000	40

Source: (GTP-2, 2016)

Non-domestic water demands are public institutional and commercial demand, industrial demand, domestic animal demand, firefighting, and unaccounted for water. Commercial demands are like shop, hotel, restaurant, grocery, and commerce-related institutions demand. Public institutional demands are like health care, hospital, schools any public service government office. Industrial demands are the amount of water required for industrial processes which are expressed as the amount required per product per day. According to the ministry of water resources (2006) and according to Oromia water work design Guidelines (2008) except for small to medium scale industries, the big industry should have their water supply. In the absence of defined water demand for industries, institutional and commercial it is to assume 5%-10% of domestic water demand for small to medium industries and 20%-40% of domestic water demand for institutional and commercial water demand (OWWDG, 2008). Firefighting demand is taken by increasing the storage tank by 10% and unaccounted for water is 40%-25% of total water production from the start of the water project service until the end of the service period (MoWR, 2006).

2.1.1 Seasonal, day, and hour demand variation factor

According to the Ethiopian ministry of water resource, urban water supply design criteria (MoWR, 2006), water demand patterns are categorized as seasonal factors, peak day factors, and peak hour factors. Seasonal peak factors may vary between 1 and 1.2 representing the relative increase in average daily demand during the dry or hot months compared with average annual demands. Due to a demand cycle that is higher in one day of the week than other days, peak day demand adopted besides to Ethiopian urban water supply designer consultant it is the range between 1 and 1.3. the favorable optimal range for a specified area or urban may be selected due to the area water use habit through a week and the knowledge of the community and system operators. Water demand varies greatly during the day and the distribution system must be designed to handle the demand which is taken into account by the use of a peak factor.

Table 2-2 Peak demand factor

<table>
<tr><th>Population size</th><th>Maximum day factor</th><th>Peak hour factor</th></tr>
<tr><td><2000</td><td rowspan="3">1.3 -1.5</td><td>2.6</td></tr>
<tr><td>2000- 10,000</td><td>2.4 -2.2</td></tr>
<tr><td>10,000 – 50,000</td><td>2.2 – 1.8</td></tr>
<tr><td>50,000 – 80,000</td><td rowspan="2">1.2</td><td>1.7 - 1.8</td></tr>
<tr><td>>80,000</td><td><1.7</td></tr>
</table>

Source: (MoWR, 2006)

2.1.2 Climatic and socio-economic factor

As mentioned above on the topic of domestic water demand, the factors mostly which make water demand and consumption vary from place to place, country to country are climatic and socio-economic factors. According to the Oromia water work design guideline (2008); the climatic factors and socio-economic factors are as follows tables.

Table 2-3 Climatic water demand factor

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

Source: (OWWDG,2008)

Table 2-4 Socio-economic water demand factor

Group	Description	factor
A	Towns enjoying high living standards and with high potential for development	1.1
B	Towns have very 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: (MoWR, 2006)

2.2 Water demand management

Water demand can be managed along supply side, supply management, and along demand side, demand management. Supply-side demand management is the traditional view of water use considers water as a requirement that must be done through the process of adding water sources and demand-side water demand management is the new modern approach that improves the equitable, efficient and sustainable use of water from the existing water resource (Qdais, 2003). Water demand management is the comprehensive reforms and actions to optimize existing water supplies (Perto & Richarch, 2018). Water demand management seeks to encourage better use of existing water supplies through economical and efficient management before further increasing the supply (GWP, 2012). Demand management is the practical activities that acted through various water sectors and by different measures; the development and implementation of strategies aimed at influencing demand, to achieve efficient and sustainable use of scarce resources (Dziegielewski, 2003).

Water demand management is the key element of sustainable water resource strategy; which must be supported by accurate demand forecasts, that addresses micro-scale multi-component demands these can provide better information on which optimal decisions about water allocation can be made (WRA, 2010). From a practical viewpoint, water demand management includes a wide two interrelated activities: the improvement in technical efficiency of water use and the efficient allocation of available water among competing uses (Dziegielewski, 2003).

According to the meaning given by different authors; Water demand management has a relatively similar meaning with water conservation. But, their difference: water conservation is not massive as water demand management and its concern on water save at source and user rather at the delivery structure. Water demand management is the adaptation and implementation of strategy by a water institution to influence water demand and usage of water to meet: economic efficiency, social equity, environmental protection, sustainability of water supply and services, and political acceptability (WEDC, 2011).

2.2.1 Water demand management measures

Water demand management refers to the measures aiming at increasing technical, economic, social, environmental, and institutional efficiencies in all sectoral uses (PWC, 2016). Water demand management aims to encourage better use of available water before plans are made to increase supply or another source (GWP, 2012). According to the water Engineering development center, 2011 explanation Water demand management measures are grouped into four basic categories: structural and operational measures, economic measures, behavior modification, Legal & institutional measures. Structural and operational measures could be taken at the utility level to reduce water losses and practical equity water distribution. Reduce water loss and raise practically equitable water distribution; pressure management is one of the most practical and cost-effective of reducing real loss and delivering service in an adequate manner (Samir et al., 2017). The economic measure is the water resources fees and services charges; designing optimal water-conserving tariff and addressing the incentives to those who adopt the technical methods of water-saving (WRA, 2010). Awareness raising and public education programs for modifying the behavior of water consumers may be hand in hand with other water demand management measures for a more effective water demand management strategy (WEDC, 2011). Legal and institutional measures follow the regulatory in which it could ensure efficient water use concerning raw water abstraction, water distribution, and end-use stages (WEDC, 2011)

2.2.2 Water demand management implementation

Water demand management measures can be implemented through action aimed at water supply structure management (increase system efficiency at improving the efficiency of the water treatment process, reduction of system losses in transmission line and water distribution network, practical equity water distribution); end-use level management (promotion of more efficient water devices: toilet retrofit, shower retrofit, appliances in the customer's premises) and substitution of potable water sources (greywater reuse, rainwater harvesting, and stormwater use) (Rooijen et al., 2011). The major new productivity methodologies are any action that reduces the demand for freshwater through more efficient usage of water there, by influencing water balance to the benefit of mankind and life on earth (Pieter, 1994). According to the study done on water demand management – current water productivity technology and water management tool, at south Africa Nelson Mandela Metropolitan university by J Pieter (1994), the project service life and delivering the required quantity of water supply without demand management, it ended in a short time.

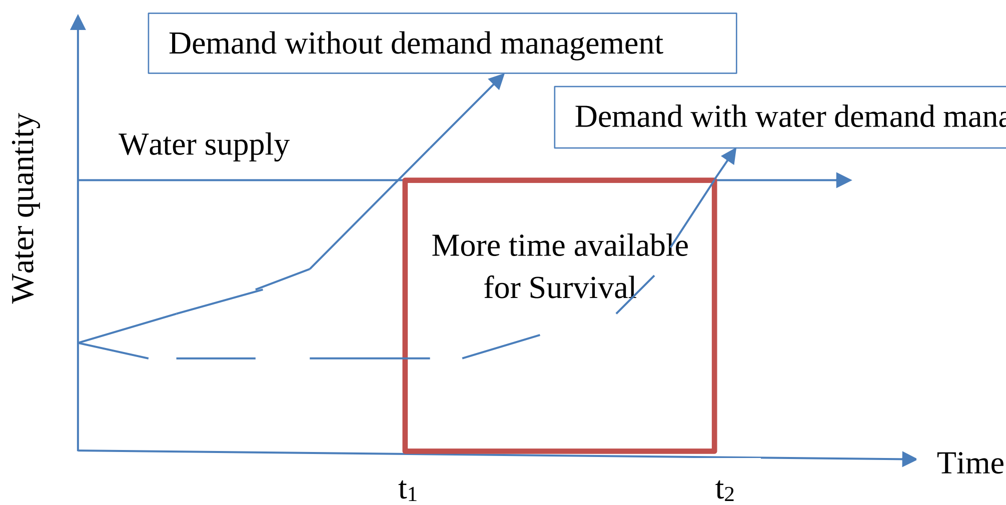


Figure 2-1 Water demand management implementation

(Source: Peter, 1994)

Where time is the water supply system service period, t_1 is the ended water supply system service period without giving service throughout service period due to due to improper water supply structure management, and t_2 is survival water supply system service period.

2.2.3 Reducing water demand by managing water distribution system

According to the study done on reducing water demand by managing water distribution system at the city of Cape town, South Africa (Perto & Richarch, 2018), water demand was reduced using advanced pressure management, advanced equipment to monitor water infrastructure and household flow regulators. Advanced pressure management was the implementation of automated pressure zone which adjust pressure remotely and force down consumption by throttling zones to the extent of partial supply if user behavior in the supply is high (Perto & Richarch, 2018). Advanced pressure management was contributing an average saving of 55 million liters per day. Advanced equipment to monitor water infrastructure was the implementations of robotic crawler which fitted with an on-board camera and remotely controlled water and sanitation infrastructure: identifying cracks, leaks and obstruction inside pipeline (Perto & Richarch, 2018). The implementation of robotic crawler in the city of Cape town was contributing reducing water losses at 16% in comparison to the national average of 36%. Household flow regulators were installed to restrict daily household consumptions and safeguard against the impact of leaks (Perto & Richarch, 2018). Household flow regulators implementation contributing sending warning letters to users with their municipal bills, which has an important impact on water use behavior (Perto & Richarch, 2018).

2.2.4 Pressure control for minimizing leakage in water distribution system

According to the study done on Pressure control for minimizing leakage in water distribution systems at the city of Alexandria, Egypt (Samir et al., 2017), pressure management was employed using fixed pressure reducing valves (PRVs). Pressure reducing valves maintain the pre-set downstream pressure regardless of the upstream pressure (Ulanicki & Abdelmeguid, 2008). The diameter and locations of PRVs altered in different scenarios with constant 20m pressure set up in all scenarios (Samir et al., 2017). Due to the pressure in the best scenarios dropped from the base scenario 29.93m to 19.997m, the leakage in the best scenario dropped from the base scenario 230m^3/day to 142m^3/day(Samir et al., 2017). Comparing with the calculated leakage in the base scenario, the use of the pressure reducing valve in the best scenario reduces the leakage by 37% (Samir et al., 2017)

2.2.5 Partitioning distribution network in to district meter area for active leakage control

According to the study done on DMA design and implementation at Halifax Nova Scotia, Canada (Morrison et al., 2007), the annual real losses prior to full DMA implementation (1999/2000) were 18,055,000m^3 and infrastructure leakage index was 6.4. As DMA implemented (March, 2005), the annual real losses were 8,101,000m^3 and infrastructure leakage index was 3.8. At the Halifax regional water commission, a DMA size decision was made based on acceptable leakage run time for three days. The ideal maximum DMA size was set at 150-200 hydrants, 2,500 customer connections or 30 km of water main (Macdonald & Yates, 2005).

2.3 Water loss

Water demand management is targeting water losses reduction and uses optimization for better satisfaction of the current and also for the future (PWC, 2016). The reduction of unaccounted for water is the key approach for urban water demand management (Deverill, 2001). Improving the efficiency of the water distribution system; a loss recorded on water distribution system is a headache it varies according to country and town but it can be as high as 40 or 50% (Deverill, 2001). To deal with this, water utilities have set up system diagnostic operations to detect and repair leaks and renew infrastructure (GWP, 2012).

2.3.1 Water loss analysis

2.3.1.1 Water balance for water loss analysis

Water loss is calculated by subtracting legitimate use from the system's input volume or net production using the water balance method. Water loss is defined as the difference between non-revenue water and un-billed authorized consumption (Sharma, 2008). Water loss is expressed as a percentage of net water production delivered to the system, as m^3/km of distribution mainline/day, as l/service connection/ day, and as l/service connection/day/m of average operating pressure (Carpenter et al., 2003). Unaccounted for water is expressed as the percentage of total water produced for the system and it arises as leakage, inaccuracy metering, overflowing of the reservoir, illegal connections, and legitimate unmetered use such as firefighting, street flushing, and reservoir washing (Bahre & Demeku, 2021). Unaccounted for water is a useful indicator of probable losses, but it may overestimate because the supply meter tends to under record consumption. In the United Kingdom, figures for unaccounted water tend to be unreliable because un-metered consumption has to be estimated and can be 10% in error (Desalegn, 2005). To avoid a wide diversity of formats and definitions related to water loss international water association water loss specialist group produced a commonly standard approach for water balance calculation focused on apparent loss calculation shown in table 2-10 (Vermersch et al., 2016).

Table 2-5 IWA WLSG apparent loss focused water balance

System input volume	Authorized consumption	Billed authorized consumption	Billed metered consumption	Revenue water
			Billed unmetered consumption	
		Unbilled authorized consumption	Unbilled metered consumption	Non-revenue water
			Unbilled unmetered consumption	
	Water losses	Apparent (commercial) losses	Unauthorized consumption	
			Customer metering errors	
			errors in the estimate of unmetered consumption	
			errors throughout the data acquisition process	
		Real (physical) losses	Leakage on transmission lines and distribution mains	
			Leakage and overflow at storage tanks	
			Leakage on service connections up customer meters	

Source: (Vermeersch et al., 2016)

2.3.2 Splitting water loss into apparent loss and real loss

Water loss can be classified as physical (real) water loss and commercial or non-physical or apparent water loss (Fikadu, 2018). There are two methods of splitting water loss into apparent loss and rea. The top-down approach and the bottom-up approach (Vermersch et al., 2016). The top-down down approach is the first estimation of apparent loss or current annual apparent loss, using a simple percentage of billed metered consumption (Vermersch et al., 2016). When there isn't enough well-defined data to analyze apparent loss, it accounts for 20% of overall water losses. (Sharma, 2008). Using a top-down approach, the current annual real loss is the difference between total water loss and current annual apparent loss. The bottom-up approach is the current annual real loss evaluated from the analysis of measured minimum night flows (MNF) and night day factors (Vermersch et al., 2016). Using the bottom-up approach, the subtraction of current annual real loss (CARL) from total water loss yields current annual apparent loss (CAAL).

2.3.2.1 Top-down approach

Apparent loss analysis

I. Guideline default percentage value

In the absence of adequate data and a proper methodology, most developed countries use default values. For example, the default value used in the developed countries: 1% unauthorized water consumption, and 4% meter inaccuracy for reference annual apparent losses (Vermersch et al., 2016). Guideline default percentage value for developing countries as follows (Mutikanga et al., 2011):

Table 2-6 Percentage of BMC for the apparent loss

Utility	un-authorized use of water	meterage and error			Meter reading, data handling, and billing errors
		age	error		
			with storage tank	direct supply	
City (> 100000 service connections)	10%	poor (>10 years)	28%	10%	poor (10%)

Medium towns (5000 – 50000 service connections)	2%	average (5-10 years)	20%	8%	average (6%)
Small towns (<5000 service connections)	0.5%	good (<5 years)	15%	5%	good (2%)

Source: (Mutikanga et al., 2011)

II. Metering accuracy test

The accuracy test of metering is highly significant when analyzing apparent loss. Determine the accuracy of a meter at any given time, it must be tested at various flow rates that approximate average customer water usage. The weighted meter accuracy is calculated by combining the proportion of water utilized at various flows with the meter accuracy at each flow (Ncube & Taigbenu, 2018).

$$Weighted\ average\ meter\ accuracy = (PTC_l * GAA_l) + (PTC_m * GAA_m) + (PTC_h * GAA_h) --- [2.1]$$

Where PTC_l - is the percentage of total consumption at the low flow rate. PTC_m - is the percentage of total consumption at the medium flow rate, PTC_h - is the percentage of total consumption at the high flow rate, GAA_l - group average test result accuracy at low flow, GAA_m - group average test result accuracy at medium flow and GAA_h – group average test result accuracy at high flow. The volume of flow counted during peak hour demand when many service connections exist on service giving is referred to as consumption at low flow rates and also, the volume of flow counted during minimum hour demand referred to consumption at a high flow rate. The flow rate can also be controlled with a gate valve and a service faucet.

III. Data mining

Data mining is a process of discovering valuable information from large amounts of data using computational techniques (Ncube & Taigbenu, 2018). The data mining method can be used to calculate apparent losses in a well-managed water supply system. The use of smart metering infrastructure, databases and information systems has provided an opportunity to apply data mining and computational intelligence in the analysis of water consumption (Ncube & Taigbenu, 2018).

$$Apparent\ loss\ estimate = \frac{0.7a + 4b + 5c}{100} ----[2.2]$$

Where a – average meterage, b - the estimated probability of meter under-registration, c- the probability of oversizing.

Water meterage, meter under-registration, and meter oversized data are required for apparent loss analysis utilizing data mining methods. Rather automatic, manual using data mining approach, it needs collecting data using customer water meter sample, field survey, and test.

2.3.2.2 Bottom-up approach

Real loss analysis

I. Reservoir water loss

Leakage on the reservoir can be measured using a reservoir drop test by closing the inlet and outlet valves and measuring the rate of fall of water level throughout the test (Desalegn, 2005).

$$Rate\ of\ leakage\ \left(\frac{m^3}{hr}\right) = \frac{(D1 - D2) * A}{t} --[2.3]$$

Where D1 – initial depth of the water (m), D2 – final depth of the water (m), A – the surface area of the reservoir (m^2), t- Hrs.

Overflow from reservoirs can be evaluated and managed by inspecting float valves.

II. Leakage in a distribution network

The night flow methods and the total integrated flow method can both be used to express leakage in a supply system. The night flow method expresses the leakage in a system in terms of a liter per property per hour. The total integrated flow method calculates the leakage in a system from total flows and not for a limited period. This method is the preferred method for calculating leakage (Desalegn, 2005). The total leakage flow is obtained from the following mass balance:

$$Q_l = Q_n - Q_d - Q_{cm} - Q_{cn} - Q_{op} - Q_{apl} --- [2.4]$$

Where Q_l – is the estimated leakage flow, Q_n – the night flow or the minimum recorded inflow to the area, Q_d – the estimate legitimated domestic use, Q_{cm} – legitimate commercial metered use, Q_{cn} – the estimated legitimate commercial non-metered use, Q_{op} – operational use by water company and Q_{apl} -apparent loss estimated.

In practice, the bottom-up approach is not applicable, and it requires a large investment, such as installing DMA per component; the top-down approach, on the other hand, is frequently based on approximations of apparent losses and is more cost-effective for analysis, but it counts more inaccuracy than component analysis (Vermersch et al., 2016). When a water distribution network is configured without a district meter area, a bottom-up strategy is impossible.

2.3.3 Water loss performance indicator

2.3.3.1 IWA recommended performance indicators

Table 2-7 IWA recommended performance indicator

Performance indicator	level	functions	Remark
The volume of NRW as % of a system input volume	basic	Financial: NRW by volume	Can be calculated from a simple water balance
Value of NRW as a percentage of the annual cost of running system	detailed	Financial: NRW by cost	Allows different unit cost for NRW component
Real losses as % of the system input volume	basic	The inefficiency of use of water resources	Unsuitable for assessing the efficiency of management of distribution system.

m^3/service line/ day, where the system is pressurized	basic	Orientational: Real losses	Best traditional basic performance indicator
Infrastructure leakage index	detailed	Operational: real losses	Ratio of CARL to UARL

Source: (Sharma, 2008)

2.3.3.2 Acceptable water loss

Acceptable water loss is the comparison of benefits from water loss reduction (community water service, utility income) and costs needed for water loss reduction action implementation (Desalegn, 2005). Acceptable water loss is expressed using unaccounted for water (UFW) level, Guideline for water loss level, and benchmark water loss level (Sharma, 2008). According to the UFW level, a water loss of 10% of input volume is acceptable, but it must be monitored and controlled. When it is between 10% and 25%, it is said to be in an intermediate state and could be reduced. It also exists on an issue of inspection, monitoring, and management; when it exceeds 25%; hence, it might be minimized. According to the guideline for water loss level which indicates basic operational performance indicator and benchmark water loss level; less than 250l liter per service connection/ day and less than 10000 liters per kilometer of mainline per day respectively shows the good conditions of the system. When they are between 250 and 450 liters per service connection per day, and between 10000 and 18000 liters per kilometer of mains, they indicate that the system is in average condition. They show that the system is in poor or bad condition when they exceed 450 liters per service connection per day and 18000 liters per kilometer of mainline per day.

Basic operational performance that guides permissible water loss, when less than 250 l/SC/day is considered a good condition and larger than 450 l/SC/day is considered a bad condition, should be constrained by less than per capita consumptions of 150 l/day and well-managed system(Sharma, 2008).

2.3.3.4 Infrastructure leakage index (ILI)

A better indicator that describes the quality of infrastructure management is ILI (Sharma, 2008). High and increasing water losses are indicators of ineffective planning, construction, and low operational maintenance activities of the water supply structure (Hamilton et al., 2006). The recommended terminology and methods of calculation of real and apparent losses and the infrastructure leakage index for international comparisons are indicated using best practice and agreed on terminology by the international water association, water loss task force (Hamilton et al., 2006). Infrastructure leakage index is the ratio of current annual real loss to unavoidable annual real (Frauendorfer & Liemberger, 2010). Infrastructure leakage index is the overall water supply structure performance indicator by showing water supply system management strategy, technical performance, the utility skilled manpower performance, billing efficiency performance, public participant to leakage management performance, and data management performance. Technical achievable low-level annual real losses are equal to the best estimates of unavoidable annual real losses (Vermersch et al., 2016). According to recommendations of the EU Reference document good practices leakage management on fit for purpose key performance index (KPI) for real losses, shown in the table below, have categorized suitability of different KPIs for real losses according to the operational purpose they are used (Vermersch et al., 2016):

Table 2-8 Good practice performance indicator for leakage, fit for purpose

Objective	The good practice performance indicator for leakage, fit for purpose						
	V/ year	Liter/ SC	m^3/km mains	Ltr/BP	%IV	%WS	ILI, with pressure
Set targets and track P for an individual system	Yes, for a large system	Yes*	Yes*	Yes, (UK)	no	no	Only if all justifiable pressure man. completed
TP comparisons of different systems	no	no	no	no	no	no	yes

Draw GC from single or multiple systems	no	no	no	no	no	no	Yes, together with other context factors
*Choose SC density > 20/km; if not choose mains; or base choice on country custom and practice							

Source: (Vermeersch et al., 2016)

Where V – volume, P- performance, SC- service connection, BP- billed property, IV- input volume, WS- water supply, ILI – infrastructure leakage index, TP- technical performance

After ILIs calculated haven calculated, they can be assigned to leakage performance categories (LPC) A to D (Vermersch et al., 2016). As explained in the table below, LPC bandwidths for low- and middle-income countries are twice as large as for high-income countries.

Table 2- 9 Infrastructure leakage performance categories

L&M ICs ILI range	H ICs ILI range	LPC	General description of LPCs A to D	Recommended actions for each LPC range	A	B	C	D
				pressure management option	yes	yes	yes	
< 3	< 1.5	A1	further loss reduction may be uneconomic unless there are shortages; careful analysis needed to identify cost-effective improvement	speed & quality repairs	yes	yes	yes	
				check economic intervention frequency	yes	yes		
3 to <4	1.5 - 2	A2		improve ALC	yes	yes	yes	
				Identify options for improvement		yes	yes	
4 to <6	2 - 3	B1	potential for marked improvements; consider pressure management, better ALC practices, & better network maintenance	asses economic leakage level	yes	yes		
6 to <8	3 to 4	B2		review burst frequency		yes	yes	
8 to <12	4 to 6	C1	poor leakage record; tolerable only if water is plentiful and	review asset management		yes	yes	yes

12 to <16	6 to 8	C2	cheap; even the analyze level and nature level and nature of leakage and intensity leakage reduction efforts	deal with deficiencies in manpower, training, and communications	yes	yes
16 to <24	8 to 12	D1	Very inefficient use of resources, leakage reduction programs imperative and high priority.	5-year plan to achieve next lowest band	Yes	Yes
>24	>12	D2		Fundamental peer review of all activities		Yes

Source: (Vermeersch et al., 2016)

Where: L & M ICs – low- and middle-income countries, H ICs – high-income countries

2.3.4 Causes of water loss

Water loss is caused due to leakage (burst, malfunction, and faulty construction) in transmission lines and distribution networks, customer meter error, and illegal connections (Farley, 2001). From one municipality to another municipality, from one location to another location water loss occurred vary due to pressure level in a distribution system, quality of construction, materials used, the nature of the soil, the age of the materials, the utility maintenance, and operation practice (Desalegn, 2005).

Operating and surge pressure

Pressure can affect system losses as pressure rises and pressure surges. The pressure rises in the system increase the rate of leakage. the rate of leakage from leaking pipes or faulty joints will increase with a rise in pressure causing the leak to appear sooner by making leak location and increasing the noise level of leaks (Farley, 2001). Pressure surges can damage pipelines and result in leakage which can cause considerable damage to people and the environment (Wichmann & Stellmacher, 2019).

Pipe age and material

Pipe age and materials are important factors contributing to the burst probability of pipes that as a result cause lots of water loss. Burst or breakage of pipe by minimum force pressure hurts operation system and service delivery of the system. (Desalegn, 2005).

Unauthorized connection, Customer meter inaccuracy, service reservoir seepage, overflow, and inappropriate service connection/ fitting installations are the causes of water losses.

2.3.5 Water loss monitoring

The traditional approach to leakage control known as a visual inspection has been a passive one, whereby the leaks repaired only it became visible (Morrison et al., 2007). According to the international water association's water loss task force the four basic leakage management for leakage detection are pressure management, active leakage control, speed, and quality repairs, and pipeline asset management and renewal (Christodoulou et al., 2010).

2.3.5.1 Identifying water loss using district meter area (DMA)

The development of acoustic instruments significantly improved the leakage control situation, allowing invisible leaks to be located as well. But, the application of such instruments over the whole large water network is expensive and time-consuming. To solve this problem, the solution is the permanent leakage control system whereby the network is divided into district meter areas (DMAs) supplied by a limited number of key mains on which flow meters are installed (Morrison et al., 2007). The segmentation of water distribution networks into manageable areas or sectors called leakage control zones (LCZs) has become a more and more pressing must-have phenomenon water companies struggling to achieve for a better non-revenue water (NRW) control and reduction strategy. These zones or district meter areas (DMAs) serve a variety of purposes like managing unaccounted for water (UFW), pressure regulation, asset management of water distribution system infrastructure like renewal planning, and equitable supply of water during scarcity scenarios (WEDC, 2011). The process of leakage management using DMA is a follow sequence (Farley, 2005).

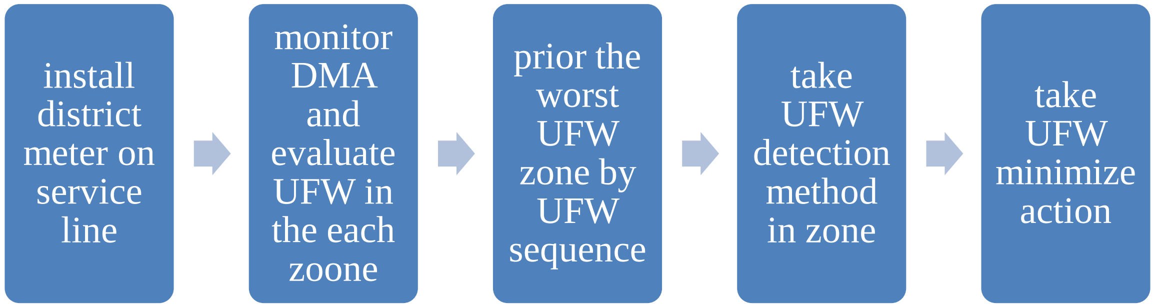


Figure 2-2 leakage management process using DMA

2.3.5.2 Identifying leaks through visual inspection

Identifying leaks through visual inspection is suitable only for reported bursts and leaks which typically have high flow rates and short run time before they are reported to utilities either by the general public or the water utilities' staff (Samir et al., 2017).

2.3.5.3 Identifying leaks using leak detection equipment

According to Stuart Hamilton and Ronnie McKenzie, 2014 water management and water loss books explanation: A widely used piece of equipment for many water utilities for leakage detection is a stethoscope or listening stick. The listening stick has an earpiece, used to listen to leaks in fittings and pinpoint the location of leaks. This technique is dependent on the ability of the engineer to hear the leak and no use of electronic equipment to enhance the sound. The technique is best suited for use on metallic pipelines between 75mm and 250mm with pressures above 10m (Hamiton & Mckenzie, 2014).

2.3.6 Real loss management

2.3.6.1 Unavoidable real loss analysis

Real loss cannot be eliminated. The lowest technically achievable annual volume of real losses for a well-maintained and well-managed system is unavoidable annual real loss (Carpenter et al., 2003). American water work association's leak detection and accountability committee recommended a 10% benchmark of UFW. Decrease urban non-revenue water from 39% to 20% for urban water supply utilities of the category of 1 to 3 in five year planning between 2016 to 2020 (MoWIE, 2016). Where service are connections metered close to street or property boundary, the corresponding system-specific equation for unavoidable annual real losses derived from component analysis assuming well-maintained infrastructure in good condition are: (Carpenter et al., 2003)

$$UARL\ (liters/day)\ =\ (18 * Lm\ +\ 0.8 * NC)\ * P\ - - - - [2.5]$$

If the supply system meets the density of service connections between 10 and 120 per kilometer of mains, customer meters between 0 and 30 meters from the edge of the street, and average operating pressure between 20 and 100 meters, the UARL conducts an analysis using the methods below (Sharma, 2008):

$$UARL\ \left(\frac{L}{day}\right) =\ (18 * Lm\ +\ 0.8 * NC\ +\ 25\ * Lp) * P - - - [2.6]$$

Where LM – length of main, NC- number of service connection, DC – density of connections, P – average pressure. For large systems with mixed pipe materials, the process to determine basic performance index for the operational management of real losses:(Carpenter et al., 2003)

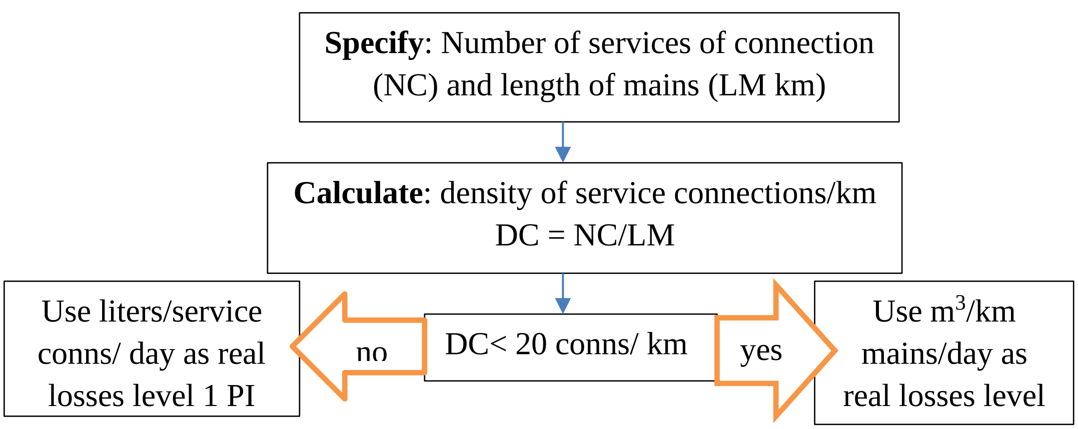


Figure 2-3 The process basic PI for the operational management of real losses

2.3.6.2 Avoidable real loss management action

Real or physical losses are influenced by many factors including soil conditions, quality of pipe materials, pressure regime, active and quality of repairs, leakage control system, and others (Hamilton et al., 2006). As the system ages, there is a natural rate of raise of real losses through new leaks and bursts some of which will not be reported to the utility (Carpenter et al., 2003). The four primary components of real loss management which manage real influences are pipeline and asset management, pressure management, speed and quality of repairs, and active leakage control, to locate un report leaks (Lambert, 2000).

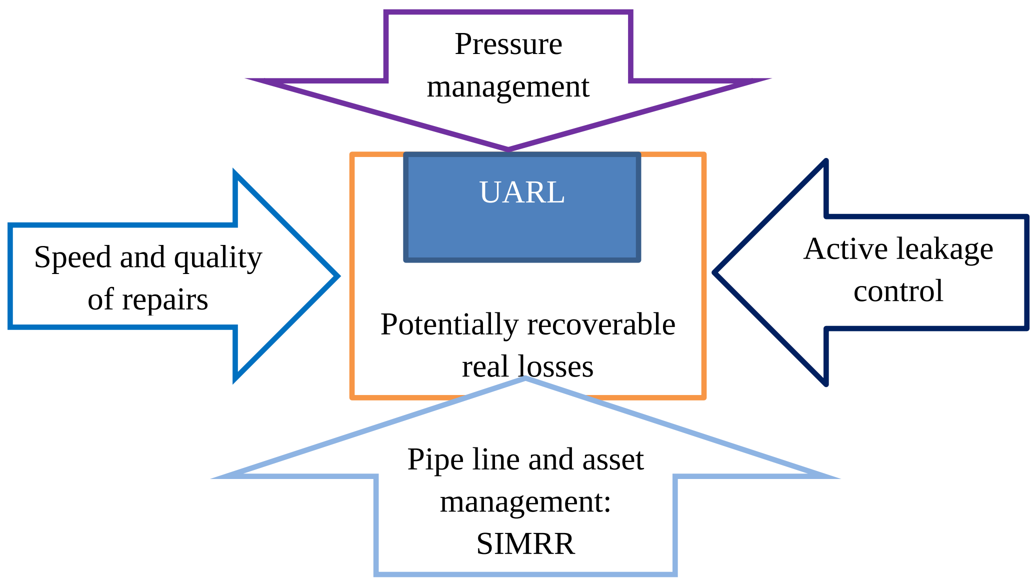


Figure 2-4 The four methods of managing real losses and infrastructure leakage index

2.4 Hydraulic performance

2.4.1 Software tools to analyze water distribution system hydraulic performance

Water modeling software is an effective tool to simulate and predict water transportation and behavior in the distribution system and this can contribute to saving system resources maintenance of water quality and satisfying demand (Awe et al., 2019). There is quite several software that model and analyze water distribution system. Among them some of the software are explained here below:

Water GEMs is robust, comprehensive, and easy-to-use water modeling and analysis software with advancements in system optimization, platform interoperability, and model building. And also, water GEMs is super-set of water CAD (Awe et al., 2019). Water GEMs have the advantage of a multi-platform workspace that allows for fire flow and water quality simulation, pipe flow, and pressure analysis in steady, extended period simulations, as well as transient or water hammer analysis. It also gives free academic authorization for a limited time. The disadvantage Water GEMs is not open-source software and it is commercially available software.

Water CAD is a sub-set of water GEMs. Water CAD is water distribution system modeling management and analysis software that has a range of functions that help to improve design productivity (Agunwamba et al., 2018). Because it is a subset of water GEMs, water CAD has a disadvantage over water GEMs in that it lacks local current user training for junior users. And also, it is commercial-like to water GEMS.

EPANET is a computer program that performs an extended period simulation of hydraulic and water quality behavior within a pressurized pipe network (Kumar et al., 2015). It is an innovation of the United State environmental protection agency and open-source hydraulic analysis software for the water distribution system. The same as water CAD, EPANET has a disadvantage over water GEMs in that it lacks local current user training for junior users.

Pipe flow expert is a commercially available software application for designing and analyzing complex water pipe distribution systems where flows must be balanced to solve the system (Awe et al., 2019). Aside from water GEMS and water CAD, the hydraulic simulation examined previously is little, which is one of the shortcomings of pipe flow expert software. As a result, the response of the existing interface is kept to a bare minimum.

General criteria for the selection of software to model water distribution networks are technical features, training or support manuals, user interface, integration with other software (such as GIS, CAD), cost, and the response of existing users (Clark, 2005). Water GEMs software currently contains a multi-plat form, a local trainee, and several responses from existing users.

2.4.2 Hydraulic performance analysis

A distribution system is represented in a hydraulic model as a series of links and nodes. Links represent pipes as pipes whereas nodes represent junctions, sources, tanks, and reservoirs (Clark, 2005). Hydraulic performance evaluation of a water distribution network can be defined as its ability to deliver a required quantity of water at all intended places within the city with reasonable sufficient pressure head and velocity to request the amount of water for various types of demand (Bahre & Demeku, 2021). Hydraulic performance evaluation is the process of simulating and hydraulic analysis of community water distribution network by computer software to save time for repeating iteration, computation of flow and pressure in pipelines, to understand hydraulic engineering principles to make (sound assumption, accurately input data field, and understanding model output) (Jalal et al., 2008). The computer programs for network hydraulic modeling distinguish between two general groups of input data: junctions and links. Junctions describe the source, nodes, and reservoir while links describe pipes, valves, and pumps (Trifunovic, 2006). The Water GEMS software package requires information on pipe diameters, pipe lengths, pipe roughness 'C', pump curves, different valve settings, tank cross-section information, tank elevation, nodal elevation, and much information. Nodes inputs are elevation and node demand. Tanks inputs are base elevation, minimum and maximum elevation, and diameter of the tank. Pumps inputs are elevation of the pump and pump curve, and Reservoir inputs are reservoir elevation, diameter, and depth (Fitaye, 2015). Any location at which water leaves the system can be characterized as demand on the system (Clark, 2005). The demand usually reaches a peak in the morning, half-day (launch time), and evening due to people being at home and preparing their meal, and also it reaches a minimum in the nighttime due to minimum water user (Fikadu, 2018). Water demand estimation and also inputs by a count of structures of different types using a representative consumption per structure, meter readings, and assignment of each meter to node and to general land use (Clark, 2005). Universal adjustment factor should be used to account for losses and other unaccounted water usage so that total usage in the model corresponds to total production (Agunwamba et al., 2018).

According to the Ethiopian ministry of water resources, water supply, and sanitation department, urban water supply design criteria (2006): urban water supply operating pressure was restricted by considering flow velocity in pipeline and head loss consideration pipe friction coefficient by pipe age and materials (MoWR, 2006). The operating pressure is restricted as table follow:

Table 2- 10 Water distribution network operating pressure in Ethiopia

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

Source: (MoWR,2006)

Where: minimum exceptional when distribution pipe near to reservoir by location and elevation. And also, in a small section of the distribution system that would require PRV. In the main raising pipe, 15m is the minimum. Maximum exceptional in a small section of distribution section which would require separate pressure.

The restricted Hazen - Williams C-coefficient:

Table 2- 11 Hazen-William's pipe C-coefficient

Types of material	uPVC	Steel	DCI/GI
New	130	110	120
Existing	100-110	90-110	100-110

Source: (MoWR,2006).

Restricted velocity: maximum less than 2m/s except in short section or transient and minimum 0.6m/s.

2.4.3 Calibration and validation

Calibration is the process of comparing the model result to field observation and if necessary adjusting the data describing the system until the model predicted performance reasonably agrees with measured system performance over a wide range of operating conditions (Fikadu, 2018). The two steps in model calibration are: determining why the model is in calibration and making the necessary adjustments to accept the model to the system (Walski, 2017). To obtain representative measurement points for model parameter calibration and to minimize the cost related to the measurement collection process, sampling design is the important solution (Kozelj et al., 2005). The minimum criteria for the hydraulic model calibration are as follow:

Table 2- 12 Criteria for hydraulic model calibration

Intended use	Level of detail	Type of time simulation	Number of pressure readings	Accuracy of pressure readings	Number of flow readings	Accuracy of flow readings
Long-range planning	low	Steady-state or EPS	10% of nodes	$\pm$ 5psi for 100% of reading	1% of the pipe	$\pm$ 10%
design	Moderate to high	Steady-state or EPS	5%-2% of nodes	$\pm$ 2 psi for 90% of readings	3% of the pipe	$\pm$ 5%
operation	Low to moderate	Steady state or EPS	10% -2% node	$\pm$ 2psi for 90% of readings	2% of pipe	$\pm$ 5%

Source: (Fikadu, 2018) & (USEPA, 2005)

The possible sources of error in model calibrations are pipe roughness, system demands, system information (elevations and pump curves), time, and measuring equipment (Trasky, 2008). The model validation is the step that follows calibration and uses an independent observed data set to verify that the model is well simulated (Clark, 2005). The model calibration is acceptable under which it satisfied the setting pressure calibration and validation criteria under the average level of error $\pm$ 1.5 average to $\pm$ 5 maximum and the model performance correlation coefficient (R^2) greater than 0.5 (Bahre & Demeku, 2021).

2.5 Water supply structure component economic life

Technical lifetime: the technical lifetime of a system component represents the period during which it operates satisfactorily in a technical sense. It is a wide range that mostly depends on the appropriateness of the choice and how the component has been maintained (Trifunovic, 2006).

Economic lifetime: The economic lifetime represents the period for which the component can operate before it becomes more costly than its replacement (Trifunovic, 2006).

According to the Ethiopian ministry of water resource (2006), urban water supply design criteria report the economic lifetime of water supply structure component as follow:

Table 2-13 Water supply component economic lifetime by MOWR

Items	Economic lifetime (years)
Borehole in hard rock, limestone	25, 15
Electromechanical equipment at pumping station and boreholes	10
Pipes: DCI, uPVC, steel pipes	40, 25, 30
Concrete water tank	50
Any civil engineering building	40
Treatment plant	50
Chemical dosing	10

Source: (MoWR,2006)

2.6 Possible engineering measures for water loss control

To sustain the hydraulic aspects of the water supply system, Engineering measures depending on technical and financial grounds tend to standardize regarding the choice of components, materials, typical design, and installation. And also, Proper engineering measures help to avoid problems in the operational management of water supply systems like the frequent interruption of supply, increased water and energy losses, shorter pipe or control lifetime, expensive maintenance of the system, and deterioration of water quality (Trifunovic, 2006).

2.6.1 Pipe materials selection, installation, repair, and replacement

A. Pipe material selection

There are three types of pipe materials depending on their resistance to backfill and shock load. Rigid, semi-rigid, and flexible (Trifunovic, 2006). Rigid pipes are cast iron (CI), asbestos cement (AC), and concrete. Semi-rigid are ductile iron (DI) and steel. Flexible pipes are polyvinyl chloride (PVC), polyethylene (PE), and glass-reinforced plastic (GRP). Pipes commonly used for water supply projects are ductile cast iron (DCI), steel, uPVC, high-density polyethylene, and galvanized iron (GI) (OWWDG, 2008). These types of pipe materials are selected depending on: characteristics of the soil, chemical nature of water, the comparative cost of alternative pipe, weather condition of the area, geologic formation of the pipe routing, expected pressure in the pipeline, and also types of crossing or fittings, (MoWR, 2006). Pipe materials selected for delivering water from source to customer must resist the following forces: (Kayombo, 1981). Temperature-induced expansion and contraction; external loads in the form of traffic, backfill, and their weight between supports; unbalanced pressure bends, contraction, and closure; water hammer; and internal pressure equal to a full head of water. According to Oromia water work design guidelines (2008); pipeline materials selected for water supply lines are: metal pipes (DCI/steel) where exposed above the ground for special sections such as drains or stream crossing and other cases; large diameter pipes (DN 400mm and above) could be metals (DCI /steel); HDPE or uPVC for distribution system with pipe diameters of DN 400 to 50mm; and Galvanized iron (GI) for service pipes of DN 2inch to 0.75inch (OWWDG, 2008).

B. Pipeline installation

According to the design guidelines of Oromia water work (2008) and Ethiopian ministry of water resources (2006) urban water supply design criteria, the pipelines for potable water delivering system laid: at roadside and verge of footpaths located at a minimum distance 90cm outside edge of the road or the roadside drain; distribution lines follow existing or planned road; a minimum distance of 1m maintained from fences and buildings to the verge; mains laid in trench have a minimum cover of 1m for pipes of DN 400 mm and smaller, 1.2m for pipes DN 400 mm and larger; minimum 1.2m depth cover under carriageways or road verges; 1.5m far and 30cm above from sewer line (OWWDG, 2008).

C. Pipe repair and replacement

As explained in title 2.10, the economic lifetime water supply structure component; component can operate before it becomes more costly than its replacement (Trifunovic, 2006). Pipe repair occurred during it shows leaks with its fitting rather than burst occurs.

2.6.2 Setting auxiliary equipment

A. Isolating Valve

Isolating valves installed at 1.5km distance on mainlines for washout requirement, connection to consumers, connection to other mains; at intervals, not more than 0.5km on secondary mains alongside tertiary mains to reduce the number of consumers affected by any failure in an artery and also on consumer pipeline will be provided at every branch connections, at the street junction, and where indicated by special requirement (MoWR, 2006).

B. Check valve

The check valve is the automatic valve closing during the backflow time (OWWDG, 2008). The best check valve closes at the moment when the forward flow stops or quickly closes before the reverse flow becomes large (El-turki et al., 2013).

C. air valve

Air valve of double orifice kinetic type DN 80 should be installed on mains of diameter DN 250 and above; DN 50 single orifice air vents should be installed on pipelines of smaller diameter and for larger pipes were only accumulated air has to be expelled (OWWDG, 2008). In general, an air valve will be installed as follow: between source and pump; downstream of the pump; at high points of vertical bends and over crossings; every 500m to 1000m on long pipeline sections with mild slope (MoWR, 2006).

D. washout line

Washouts located at the lowest point of mains or transfer pipelines near drains or streams with isolating valves and check valve installed on it (OWWDG, 2008).

E. Water meter

According to the Ethiopian ministry of water resources Urban water supply design criteria (2006), provision of metering at the location: at the outlet springs, at individual tube wells, at the outlets of the treatment plant, wellfield pumping stations, at the outlets of distribution reservoir, for all customers and public taps (MoWR, 2006). Installing a water meter in a district area serve for managing unaccounted for water, pressure regulation, asset management of water distribution infrastructure, equitable supply of water during scarcity scenarios (WEDC, 2011).

F. Pipe support

Concrete support known as thrust block for pipes is designed and constructed whenever the pipe is laid above the ground surface, where the foundation formation is not good, where the pipeline changes direction on the steep slope and either horizontally or vertically (OWWDG, 2008).

2.6.3 Setting pressure regulating facilities

In a pipeline excessive pressure may occur due to sudden flow and velocity change which can control by using a control valve closure scheme, check valve, surge relief valve, air venting procedure (air release and air vacuum valve), surge tanks, and air chambers (El-turki et al., 2013). Excessive pressure should be in an appropriate manner by using pressure-reducing valves (PRV), orifice (for transient flows), and pressure-breaking tanks (OWWDG, 2008). Pressure surges can damage pipelines and result in leakages which can cause considerable harm to people and the environment (Wichmann & Stellmacher, 2019). Three possible solution approaches in case of the transient analysis revealed unacceptable incidental pressures: modification of transient event, such as slower valve closure or flywheel; modification of the system, including other pipe material and another pipe routine; and application of anti-surge device (Pothof & Karney, 2012)

Pressure control utilizing pressure reduction valves is one of the most popular and suitable for most water utilities around the world. (Samir et al., 2017). According to Samir et al., 2017, the infrastructure leakage index ranges from 1 to 4, indicating leakage of less than 50 l/SC/day at average water pressure 10mH20, between 50 and 100 l/SC/day at average pressure 20mH20, between 100 and 150 l/SC/day at average pressure 30mH20, and between 200 and 250 l/SC/day at average pressure 50mH20. (Samir et al., 2017). i.e., leakage quantity increases as the system pressure increases. This is accomplished using a pressure regulating valve that maintains a constant pressure under varying input pressures and output flows. Pressure regulate valves powerful enough to control the flow of liquid when used with combination cylinders and hydraulic pumps (Talamini Junior et al., 2017)

2.6.5 Customer meter repair and replace

The main obstructions of real loss management and active leakage control in a well-managed water supply system are customer meter error and malfunction (Vermersch et al., 2016). When the level of metering loss is high, it requires a massive (extensive) replacement program, a specific or targeted replacement program, and a meter resizing program. Massive replacement age surveying criterion and the flow volume registered and also, targeted replacement depends on type criterion, technology, category of consumer, and rate of consumptions (Vermersch et al., 2016). Water meters need regular re-calibration and replacing after five to eight years of use (Deverill, 2001).

2.6.6 District meter area (DMA):

A world water utility concern for water loss and water demand management is employing a meter district in a water distribution network on a sub-main line to create zones without intermixing flow between zones. District meter areas (DMAs) are used for a variety of objectives, including water loss control, pressure regulation, asset management of water distribution system equipment, such as renewal planning, and equitable water supply during scarcity scenarios (Morrison et al., 2007). According to the explanation of the books of water demand management in the city of the future, the chapter of zoning tool for water distribution leakage control, the size of DMA depends on the acceptable leakage run time of three days which recommended the size of either 150-200 hydrants, or 2500 connections, or 30km of water mains (WEDC, 2011).

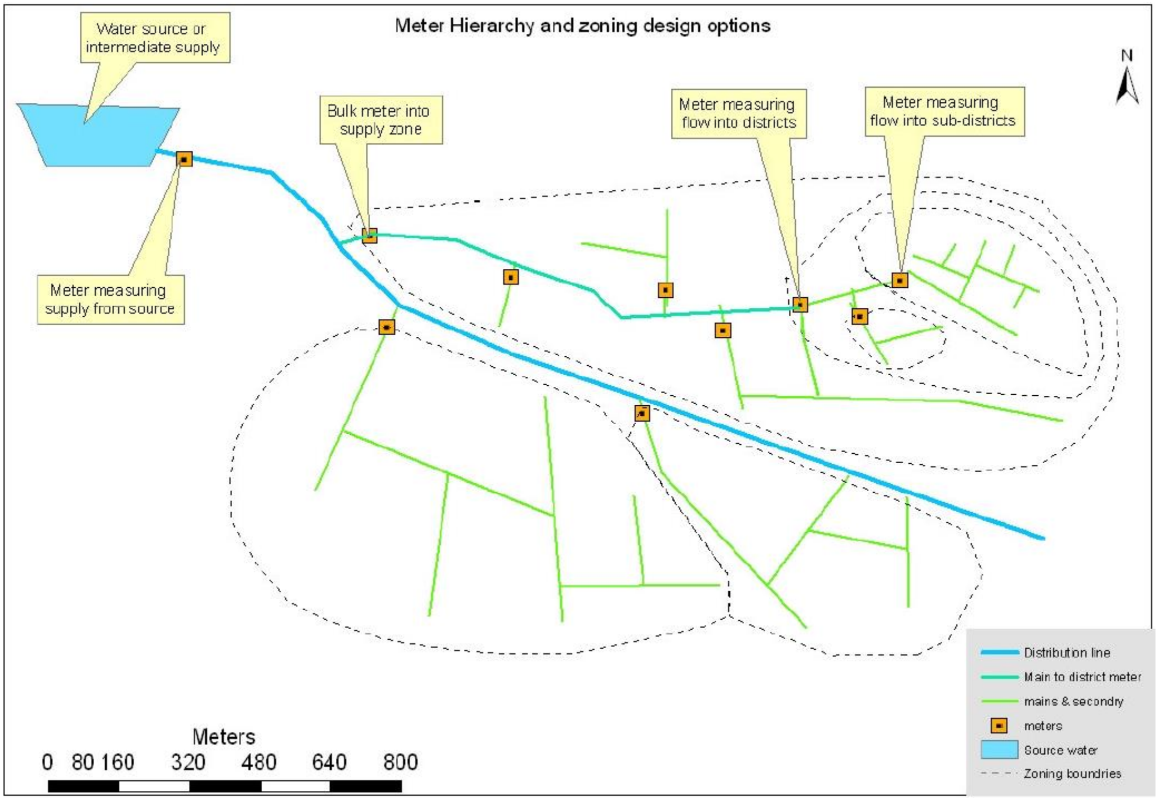


Figure 2-5 setting district meter area

Source: (WEDC,2011)

2.6.7 Float valve and reservoir water level indicator:

Float valve: most applicable direct float uses as water level in the tank increases, the float valve rises with. In the simple arrangement, the main valve open or close by the movement of float valve arm unit (Ragan, 2020)

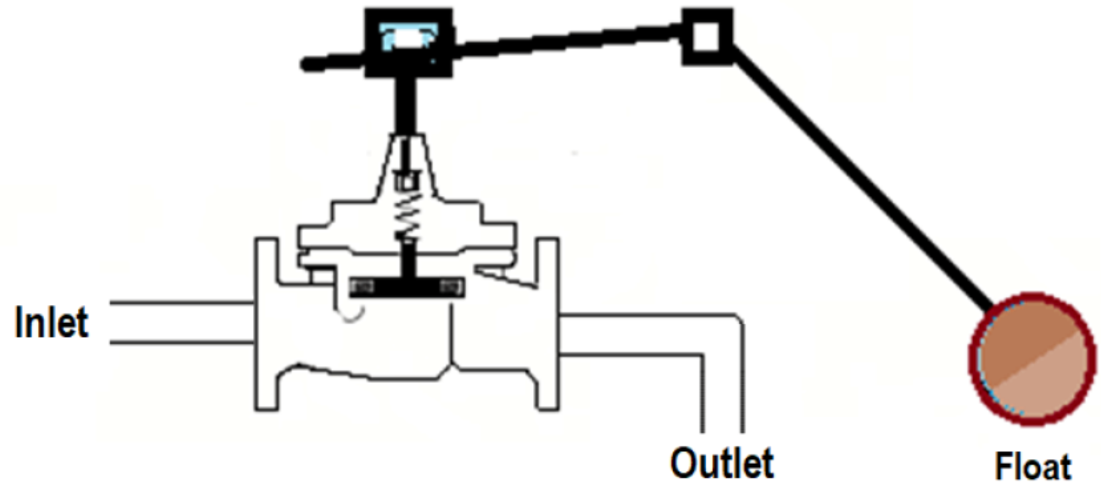


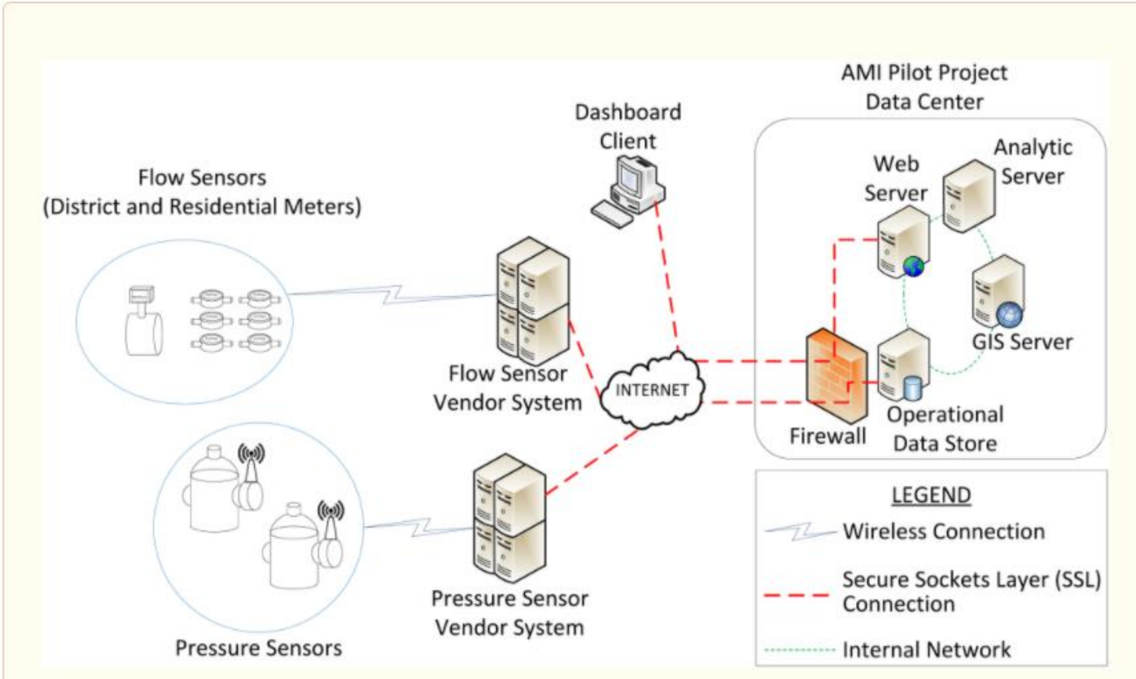
Figure 2- 6 Float valve

Source: (Ragan,2020)

Water level indictor: It is used to monitor the system by raising a rope on the external tank wall arranged structure when the reservoir water level is low and lowering it when the tank is full. And water level indicator as alarm when water level full and uses as transmitter to be off pumping especial on booster pump when tank water level is low (OWWDG, 2008).

2.6.8 Smart meter

Smart meters, Advanced metering infrastructure (AMI), and supper control and data acquisition (SCADA) are advanced engineering measures that permit two ways of communication between utilities and customers by integrating communication networks and data management systems. By enabling two ways of communication between meters and utilities, millions of data points are funneled in SCADA historians and pushed to the analytics platform (Innovyze, 2020). Smart meters and advanced metering infrastructures are meters with a direct connection sensor to the data center for sensing pressure, flows, and tank level data to manage leakage and non-revenue (Mudumbe & Abu-Mahfouz, 2015).



Source: (Innovyze, 2020)

Figure 2-7 Wireless water loss control engineering measures

Wireless water loss control applications are unable to implement without GIS-based distribution network management and internet access. They also require advanced technology and highly trained personnel in the fields of information technology.

2.7 Engineering measures for problem identification

Water supply system performance is defined as the state that the water distribution system fulfills its purposes within physical performance, hydraulic performance, and water quality performance (Van ZYL, 2014).

2.7.1 Physical performance

Physical performance means that the system components can function as intended and provide a barrier between the water in the system and external threats. Design and construction elements, as well as physical factors, contribute to physical performance issues. Design faults, defective materials, defective parts (improper design or construction), improper construction or repair, improper use (using components and materials outside their operating specifications), and cross-connections between the drinking and non-potable networks can all lead to contamination of the drinking water system. Excessive loads (water pressure, change in momentum, and water hammer), erosion, exposure to sunlight, corrosion, and cavitation are all key physical issues that impair the water supply system's physical performance.

According to the Ethiopian ministry of water resources (2006), urban water design criteria, there are four types of delivery from source to consumer. 1) Pure gravity system: the delivery system from source to service reservoir and service reservoir to the consumer by gravity. 2) Combined system: the delivery system from source to service reservoir by fixed-rate pumping and from service reservoir to consumer by gravity. 3) Mixed pumping and gravity: the delivery system from source to service reservoir by variable-rate pumping and from service reservoir to consumer part pumping and part gravity. 4) Pure pumping system: variable rate pumping only no storage reservoir. mixed pumping and gravity and pure pumping system requires multiple pumps for variable pumping and not be suitable for well field supplies unless intermediate collection or booster provided (MoWR, 2006). The system will be unable to work as planned, there will be a loss of water, and there will be a risk of contamination as a result of poor physical performance (Van ZYL, 2014).

Measuring physical performance: The system's physical performance can be measured in a variety of ways. Visual inspection of non-buried components for signs of degradation or failure is occurred. Opening sections of pipe can be used to inspect underground pipes and system components, and sounding techniques can be used to locate leaks in distribution systems (Karaa & Marks, 1990; Van ZYL, 2014).

2.7.2 Hydraulic Performance

Hydraulic performance is the ability of the distribution system to all user's demands (domestic, industrial, commercial, firefighting, etc.) and ensures desirable pressures, velocity, and water age in the system (Van ZYL, 2014). Excessive demands, system capacity decreases, negative pressure, pumping straight from the network, pressure transients, excessive pressure, and flow velocity are all causes of poor hydraulic performance. Inability to meet consumer demand, damage, pollution, depletion of disinfection residuals, and sediment accumulations are all consequences of poor hydraulic performance.

2.8 The effects of pressure management on effective water demand management

Water demand management measures can be implemented through action aimed at water supply structure management: increase system efficiency at improving the efficiency of the water treatment process and reduction of system losses in transmission line and water distribution network (Rooijen et al., 2011). Pressure management is one of the most important water demand management interventions that can be implemented by a water utility in its effort to reduce leakage (Mckenzie & Wegelin, 2009). In cases of decreases or increases, there will be a significant influence on the annual volume of both unavoidable and current real losses. For example, for the last 20 years, the Japanese have used the standard relationship that leakage rate varies with pressure$^{1.15}$ (Lambert, 2000), which means a 1 percent pressure change will typically change the average leakage rate by 1.15 percent.

$$ {}^{L_1}/_{L_O} = ({}^{P_1}/_{P_O})^{N_1} - - - [2.5] $$

Where; L_o - the existing leakage, P_o - existing pressure, P_1, and L_1 – the new pressure and leakage after appropriate engineering measures applied to the existing system respectively and N1 – is the exponent of leakage rate varies with pressure change.

The N1 value varies between 0.5 (for rigid pipes, which are insensitive to pressure variations) and 2.5 (for flexible pipes highly sensitive to pressure variations) (Fontana et al., 2017). Considering all the available data the best guidance for predicting N1 values for individual sectors is that N1 depends on pipe material and level of leakage, N1 for background or undetectable leakage being 1.5 for whatever pipe materials (Lambert, 2000).

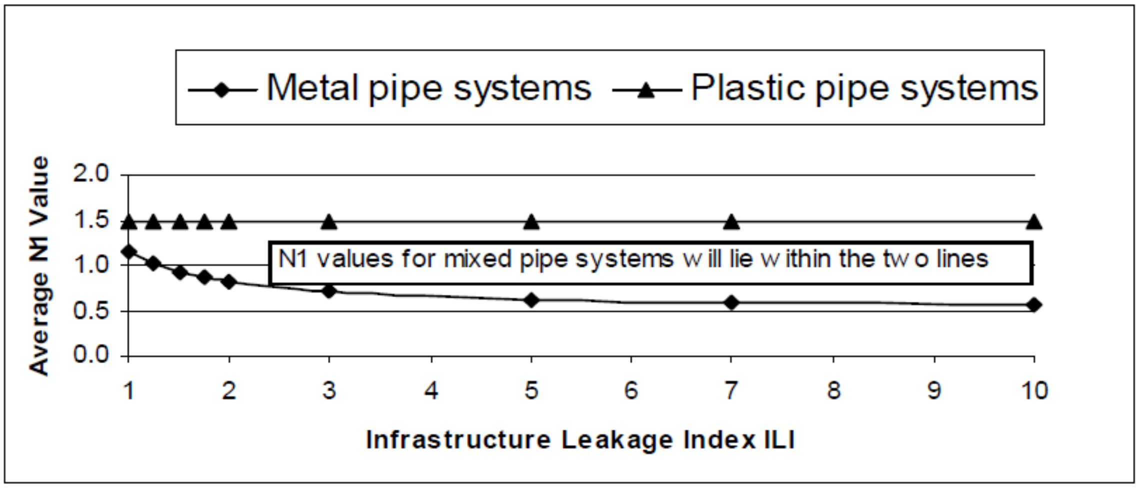


Figure 2-8 N1 predicting guidance for individual sectors

Source: (Lambert, 2000)

2.8.1 Pressure management using service reservoir

Service reservoirs in a water supply distribution system used: for balancing storage between constant inflow and variable outflow; maintains pressure levels by evening out the flow in a network; provide chlorine contact time for the inactivation of micro-organisms and as the reserve of water in the event of a power cut or treatment plant malfunction; provisions of emergency storage and ensuring water is available for firefighting (Van ZYL, 2014). The first, most important pressure management message is to avoid frequent pressure changes; wherever pump into reservoirs, not direct into distribution mains (Lambert, 2000).

2.8.2 Pressure management using pressure-reducing valve

The best practices suggest that pressure management is one of the most effective ways to reduce the amount of leakage in water distribution systems (Samir et al., 2017). A pressure-reducing valve (PRV) is useful to model leakage as a function of pressure and pipe length by developing different scenarios using hydraulic model performance analysis software (Samir et al., 2017). Pressure-reducing valves automatically reduce high inlet pressure to a steady and low level of downstream pressure (Nasrollahi et al., 2021).

CHAPTER THREE

3. MATERIAL AND METHODS

3.1 Description of the study area

3.1.1 Location

Mojo town is located in the East Shewa zone of Oromia regional state, the central part of Ethiopia, at a distance of 77.6 km from the capital city Addis Ababa. From the Ethiopia river basin, it is found in the Awash River basin, Mojo River tributaries of Awash River. Astronomically, Mojo is located at 8°38´30´´, 39°2´30´´ North West; 8°32´30˝, 39°2´30´´ South West; 8°38´30˝, 39°10´30˝North East and 8°32´30˝, 39°10´30˝South East.

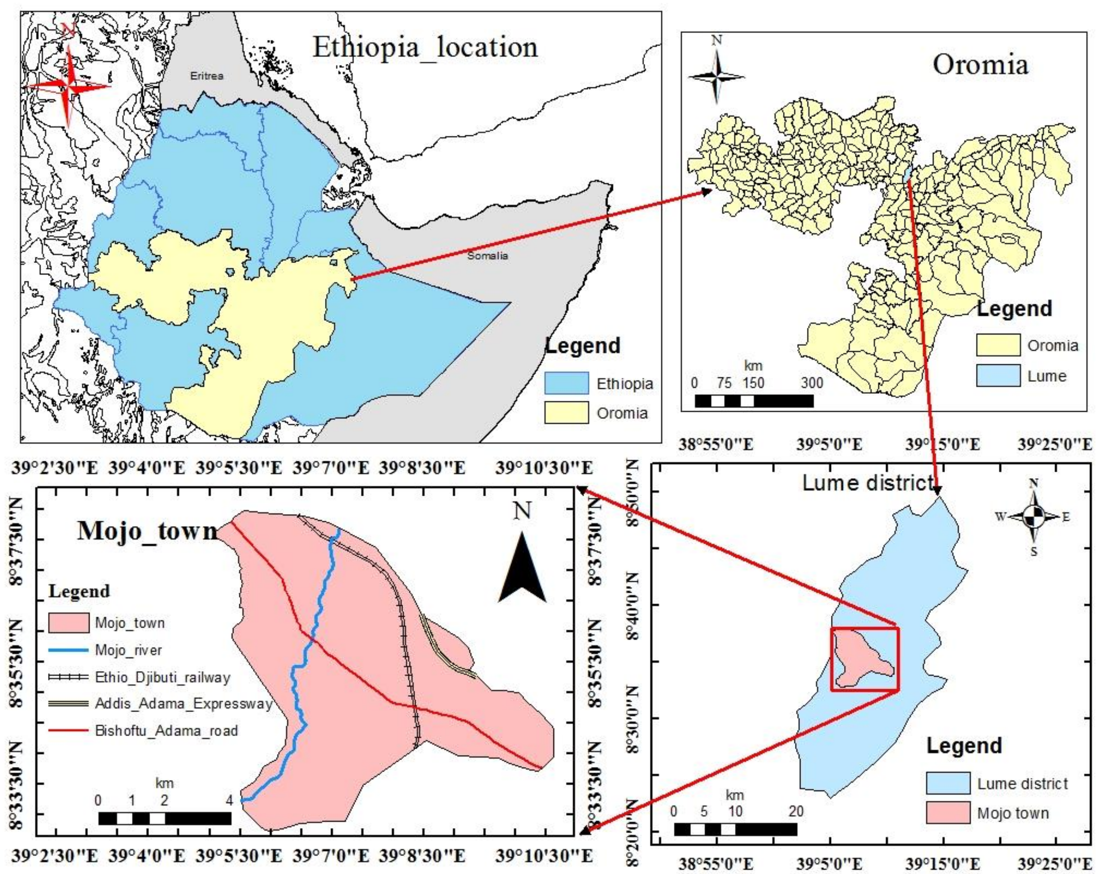


Figure 3-1 Location map of the study area

3.1.2 Climatic condition and topography

According to the Ethiopian meteorological data and World weather online, the mean annual temperature of the Mojo town exists on an average 18^{o}c in the rainy season in June - September, and 25^{o}c in the dry season from January – to April. the annual rainfall in the rainy season is averagely 550m. The town's elevation ranges from 1688 meters above sea level to 1891 meters above sea level.

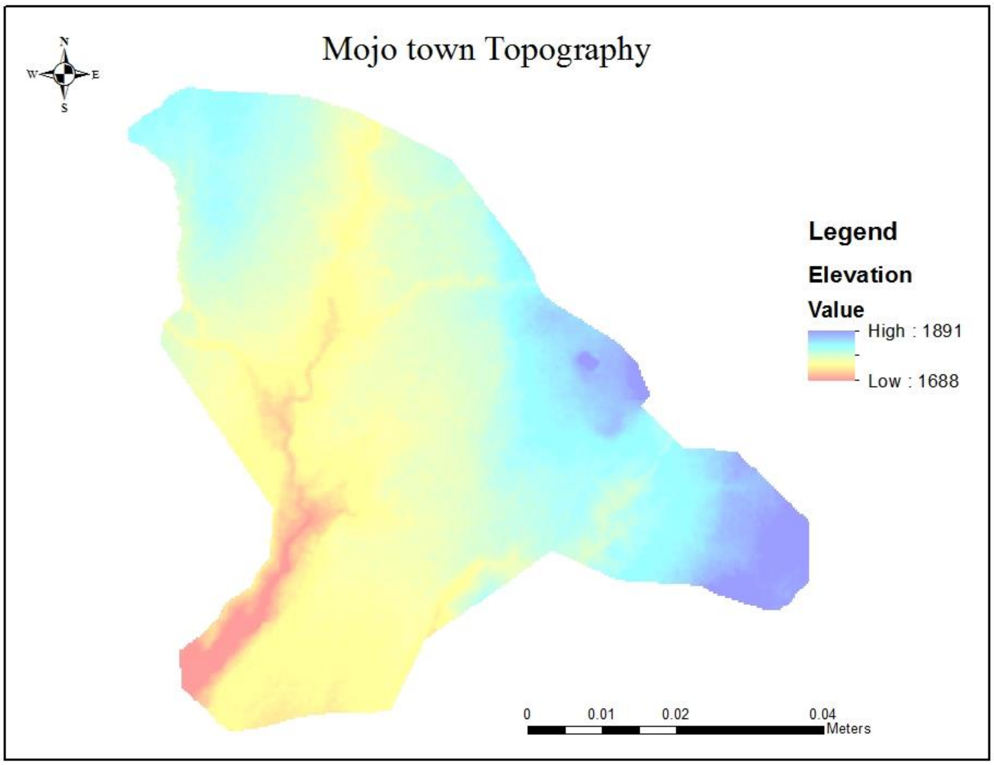


Figure 3-2 Mojo town topography

3.1.3 Socioeconomic condition

According to GTP-2, based on population size, in national urban rank, Mojo town exist on the spot transformation from the third category to the second category. And also, Mojo town is one of the industry zone towns as regional and national towns. According to Oromia urban planning institute explanation, in regional urban rank, Mojo town exist on transformation from the second rank to the first rank.

3.1.4 Population

According to the national central statistical agency, population census during 1994 and 2007, the Mojo town population was 21997 and 29547 respectively. Based on the CSA population census data, the population was forecasted by Oromia region urban planning institute to study the town socioeconomic level and the forecasted population in 2014 was 54447. for the town development plan for 2019 and 2024, the population size forecasted by the regional urban planning institute using a 6.3% growth rate based on population increment were 82670 and 100617 respectively. According to 2020 G.C the town municipality population count report, population size was 93264, whom male and female 45154 and 48110 respectively. Based on the population forecasted for the town development plan and the town municipal population census report in 2020, the actual population size was above the population size project. The main cause of population increment known by the municipality was the population migrant from the neighboring region and the town expanded to the neighbor rural border.

3.1.5 Water supply system

The water supply system for Mojo town is supplied from seven boreholes. The current total borehole yield is 141 l/s, based to field measurements taken from a meter on the borehole during the study. As indicated in Table 3-1 and Figure 3-1, the Mojo town water supply system was a mix of the direct and combined systems. The system has seven different types of water users. those are private, community, governmental, non-governmental, commercial, religious, and industry. Table 3-1 summarizes the characteristics of the existing water supply system in the years 2019/20 and 2020/21.

Table 3-1 Water delivery system in 2019/20 and 2020/21

Characteristics	**2019/20 year**	**2020/21 year**
Population size	93264	99328
Number of boreholes water source	6	7
Number of service reservoirs	2	2
Number of booster reservoir	1	1
Number of pump station	1	1
supply system	Interconnections of mixed: direct and combined	interconnections of mixed: direct and combined
Total length of raising + distribution main	104.569 km	105.116km
Number of hydrants	5	5
Total number of customer meters registered	10240	11547
the volume of potable water supply	1,875,210m^3	2,768,513m^3
the volume of metered billed	1,097,935m^3	1,351,546m^3
the volume of flat-rate billed	-	133,710m^3

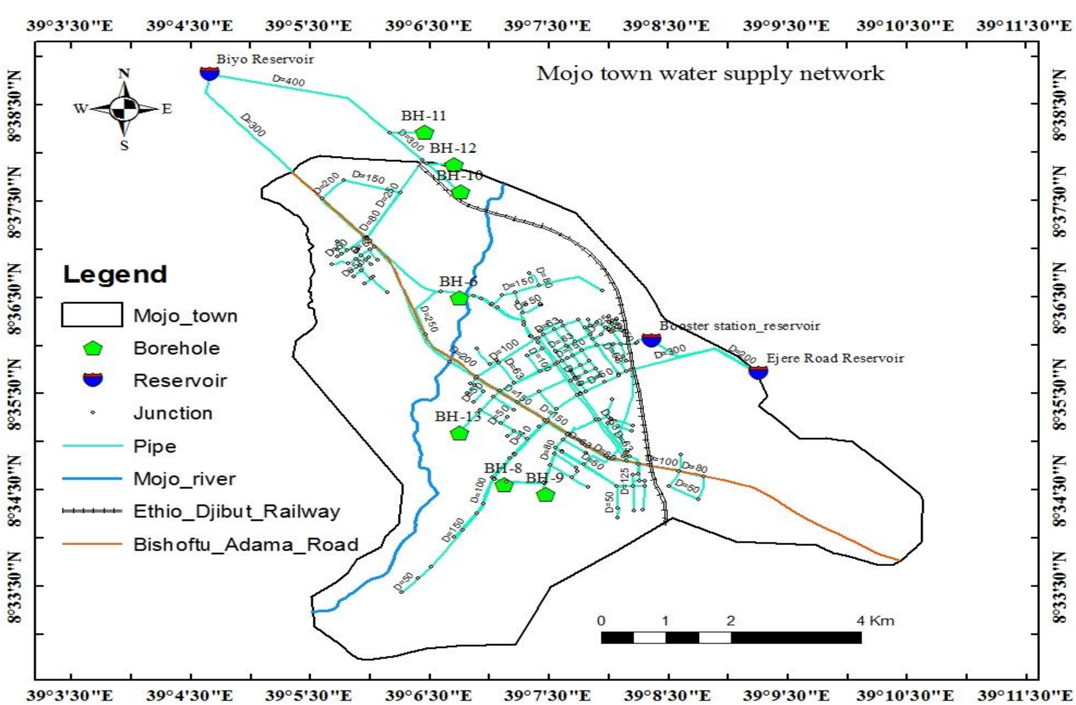


Figure 3-3 The existed 2020/21 water distribution network line.

3.2 Materials and tools

The materials and tools used for the study of assessment of the engineering measures to effective water demand management of Mojo water supply system, Ethiopia were summarized in Table 3-2.

Table 3-2 Materials and tools used

		Purposes
Materials	GPS	To collect locations of water sources, reservoirs, isolate valve and public tap
	Pressure gauge	To record field pressure data.
Tools	MS-excel	To concatenate the point survey data and analysis data in tabular format
	Google earth	To show the study area for preliminary study and distribution network preparation.
	Global mapper	To convert file format between Google earth, Auto CAD, water GEMS, and Arc GIS software. And also, prepare the study area contour.
	Auto CAD	To trim and extend the distribution network prepared on Google software
	Water GEMS	To analyze the hydraulic performance of the distribution network.
	ArcGIS 10.4	To prepare the study area location map and prepare population layer shapefile for water GEMS to automatic demand input.

The utility pressure gauge, which is a 16bar model, was used to capture field pressure data. its accuracy was checked with the same 16bar model.

3.3 Data collection

3.3.1 Data collection techniques

Interview, field observation, field measurement, and sampling were employed collect the data to assess the existing engineering contribution on effective water demand management of the Mojo water supply system.

A. Interview: Before the study began, and during data collection, an interview was conducted. The interview before the study started was made with the office expert to identify the town water supply system situations, available data, and statement of the problem which hypothesis the entitled research based on water demand management concept. During data collection, the interview was made with office experts for evaluating service delivery system management. The utility water loss detection method, maintenance method, customer water meter test method, water balance analysis trend, pressure management method, water meter, and pipe rehabilitation trend were all discussed during the interview with the office expert.

B. Field observation: Data gathered from field observations was utilized to focus on objective three, existing water supply structure situation, and possible measurements that exist and do not exist for all components. Flow control, pressure control, pipe type, pipe alignment, float valve, reservoirs accessories, flow meter, pressure gauge, isolation valve, air valve, and district meter area were the discovered structure. The structure that was discovered is shown in title 4.3.2 as a result of objective three.

C. Field measurement: field measurement was carried out on borehole yield capacity measurement and on sampled junction site to record actual pressure. Borehole yield capacity measurement was occurred on all seven boreholes, using water meter existed on borehole head and stopwatch clock to analyze the existing water demand management. the sampled junction actual pressure was recorded using the pressure gauge tool during maximum consumption hours at the morning (8:00 AM), and lunch time (12:00 PM) for hydraulic model calibration. The detail of sampling techniques is as below.

D. Sampling techniques: sampling was carried out to determine junction size applicable for simulated hydraulic performance calibration and to determine the default value for apparent loss analysis using a customer meter survey.

D1. The number of sample nodes and location: A sample of the pressure field measurement was taken following the instructions of the United States Environmental Protection Agency (2005). Pressure field measurements sample size was estimated by ranging low pressure less than 15m pressure head, moderate pressure between 15m and 70m pressure head, and high pressure above 70m pressure head using peak demand (8:00 A.M) hydraulic simulation. With 10% of low pressure 21 junctions, 5% of intermediate pressure 156 junctions, and 2% of high pressure 41 junctions, the sample size was estimated to be 11 sample sites. Two low-pressure samples, eight moderate-pressure samples, and one high-pressure sample were chosen. The positions were carefully picked, taking into account the calibration of the complete supply system (combined, direct, and mixed).

Table 3-3 Actual pressure measurement sample size

Detail level	Sample size
Low	10%
Meddle	5%
High	2%

(Source: U.S.EPA,2005)

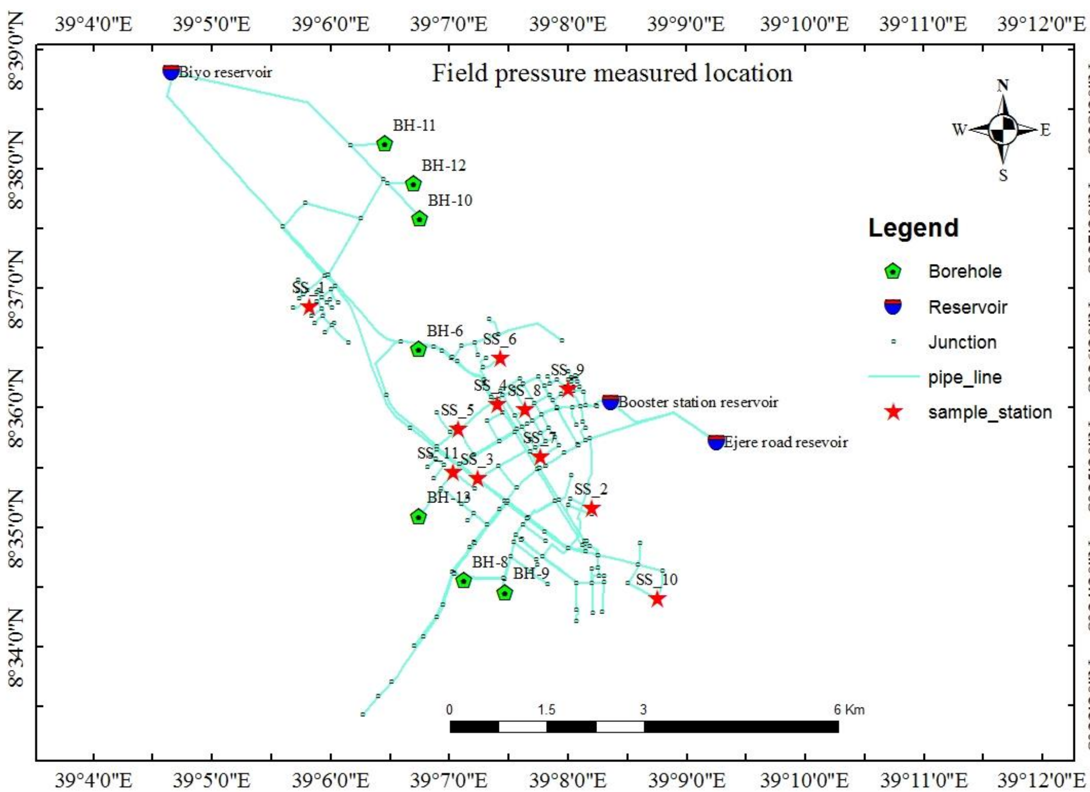


Figure 3-4 Location of observed pressure

D2. Customer water meter sampling: A sample of a customer's water meter was collected for apparent loss analysis and physical performance study. The sample size was determined using C.R. Kothari's (1990) sample size determination formula in the case of the finite population (customer water meter).

$$n = \frac{Z^2pqN}{(N-1)e^2 + Z^2pq} - - - [3.7]$$

Where n - customer meter sample size, Z – standard variant at given confidence level =1.96, p=sample proportion, if there is no previous study on the key parameters (P is taken as 50%), N= number of the customer water meter, q=1-p, e=the precision (5%).

$$n = \frac{1.96^2 * 0.5 * 0.5 * 11547}{(11547-1)0.05^2 + 1.96^2 * 0.5 * 0.5} = \frac{11089.7}{28.865 + 0.9604} = 372$$

Quota sampling from each kebele was utilized to choose the customer water meter sample, which was accomplished by dividing the sample size by the number of kebeles and selecting the quotas in each kebele using simple random sampling.

3.3.2 Data collected

Primary and secondary data were collected for quantitative and qualitative analysis to analyze the impact of engineering measures of the present water delivery system that make effective or inefficient water demand management.

3.3.2.1 Primary data

1. Borehole (BH) yield capacity (l/s): The sevens boreholes, coded BH-6, BH-8, BH-9, BH-10, BH-11, BH-12, and BH-13, have yield capacities of 7l/s, 3.25l/s, 7.13l/s, 30.3l/s, 25.5l/s, 36l/s, and 32l/s, respectively that collected from field measurement. This borehole yield capacity was gathered as part of a water demand balance analysis, which is a water demand management indication.

2. Sampled junction pressure (bar): the 11 sampled node, coded J-189, J-127, J-24, J-39, J-43, J-56, J-66, J-78, J-101, J-133, and J-172, have pressures of 1.8bar, 1.5bar, 6.6bar, 6.3bar, 6.5bar, 4.3bar, 5.5bar, 4.5bar, 2.8bar, 6.3bar, and 9.6bar respectively that recorded during Peak consumption hour (8:00AM). This junction pressure was gathered for hydraulic model calibration.

3**. Expert response**: The utility water delivery system's adapted pumping hours are 20 hours per day. Pumping rest for 4 hours in a day, starting at 1:00 A.M. and ending at 4:00 a.m., based on the night minimum consumption time. There is no timetable for filling the reservoir. During the night, the reservoir outlet valve is turned off. There are no other methods or schedules for leakage detection other than the public report using their cell phone and the utility field inventory twice a year. Pump off and reservoir outlet valve off action taken during maintenance since several isolate valves are inactive.

4. Customer water meterage and ways of use: as shown in Appendix-1 Customer water meter was discovered by age range less than 3 years old, 3-5 years old, 5-10years, greater than 10years, and techniques of usage with or without storage using classification by kebele. Using Quota sampling 124 samples were collected per kebele and simple random sampling in the kebele.

Table 3-4 Sampled customer water meter data collected

Sampled quantity in location	Water meter age (year)				Ways of use	
	< 3	3-5	5-10	>10	With storage	Without storage
01 kebele (no.)	19	39	42	24	18	106
02 kebele (no.)	23	47	46	8	26	98
03 kebele (no.)	22	53	35	14	29	95
Total	64	139	123	46	73	299

5. Auxiliary equipment: Field observation was used to determine which auxiliary equipment existed and which did not, as well as the principal water supply structure. A flow meter, gate valve, and check valve are installed in every borehole. Except for the high-capacity boreholes, the BH-6, BH-8, and BH-9 do not feature an air valve. Only BH-10 and BH-11 boreholes contain a pressure gauge; the others do not. All of the borehole heads have galvanized steel pipes. Except for the booster reservoir, nothing of the service reservoir has a minimum or maximum water level indicator. There is no float valve in any of the service reservoirs. Furthermore, no inlet valves or by-passes are used during reservoir cleaning or maintenance. The majority of isolated valves are buried and not functional. In a pipeline, there is no other flow control and pressure control valve.

3.3.2.2 Secondary data: Secondary data was obtained to evaluate the impact of engineering measures on water demand management using water loss analysis, hydraulic performance analysis, and infrastructure leakage index analysis. The secondary data collected based on data availability were: production (m^3), consumption (m^3), population number, service connection number, borehole hydrogeology (borehole depth (m), pump position (m), dynamic depth (m)), pipe diameter (mm), pipe length (m), pipe material (type), distribution line (image), pipe age (installation year), and the location map of the study area (AutoCAD format polygon). Population and town location were collected from the town municipality and all water supply data collected from the town water supply and sewerage enterprise.

Production data were collected using annual production and detailed monthly production as its availability. Data on consumption was gathered by customer type for per capita consumption analysis and by kebele for hydraulic performance junction demand input utilizing an automatic Thiessen polygon and demand loading proportion by consumption. Also, only annual consumption was collected for unaccounted water analysis and to examine the effect of connecting a new borehole line to an existing distribution line.

3.3.3 Data preparation

3.3.3.1 The town water distribution network

The town water distribution network was prepared using the existed pdf format, pencil and ruler aided format, common structure survey data, the office expert line location understands, the town plan visible on google earth, MS-excel, AutoCAD2007, global mapper218, and google earth pro software. All pipe information (diameter, material, and year of installation) was gathered from hard copy distribution line maps and expert knowledge. Appendix-2 details the steps involved in creating the integrated distribution map.

3.3.3.2 Contour of the study area

The research area's contour map was prepared for automatic junction elevation input using the waterGEMs software Trex tool. it used to analyze the hydraulic performance of the water distribution network using the waterGEMS CONNECT Edition Update 2 software. The research area's digital elevation model (DEM) was downloaded with a resolution of 12.5m x 12.5m from the Alaska satellite facility. Using the downloaded DEM and global mapper software, the contour map of the study area was generated by a 10m contour interval.

3.4 Water demand management analysis

3.4.1 Population forecasting

The population of the town is 29,547 out of which 14,355 male and 15,192 females (CSA, 2007). According to the municipality population count, the population of the town is 93,264 out of which 45,154 male and 48,110 female (Municipality, 2020). One of the significant population increases was the town's extension into the nearby rural area. The population number in 2020 was analyzed using the arithmetic method, geometric method, incremental increase method, and central statistical agency method, and compared to the municipality's counted population number to select the most conform population forecasting techniques for the 2021 population size analysis.

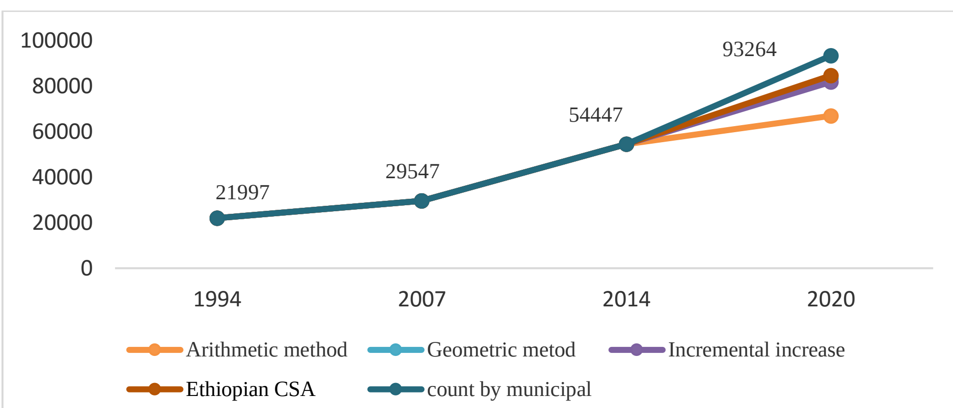


Figure 3-5 Population projection techniques

The Ethiopian CSA approach has a higher closest to the counted size than the other methods, as shown in Figures 3-5, and it is consistent with the population size prediction for 2021.

$$P_t = P_0 * e^{rt} ------------ [3.1]$$

Where P_t – projected population at a time 't', P_0 – base population, e – constant base of the natural logarithm, r- annual growth rate, and t – number of years.

Population count is only the authority of the national census statistical agency. Due to absence of the recent population counted from national census statistical agency, the population enumerated by the town municipal in February 2020 was used as the base population size for the forecast. By comparing the maximum growth rate reported by the national central statistical agency, the growth rate reported by regional urban planning institutes, and the analyzed growth rate in this study, the growth rate reported by regional urban planning institutes was used to forecast population size during the study period (2021). According to the estimations, the population of Mojo town is 99328 at the 2021 year.

3.4.2 Water consumption analysis

For the analysis of the existing water demand management is effective or not effective, using 2019/20 and 2020/21 consumption data collected, and the domestic per capita consumption was analyzed using the general per capita consumption analysis method.

$$per\ capita\ consumption\ (lcd) = \frac{annual\ consumption\ volume\ (m^3) * 1000l}{pupolation\ number\ in\ study\ year * 365\ days} --[3.2]$$

The consumption data for 2020/21 was in two categories: metered billed consumption and unmetered billed consumption. For the 2020/21 water consumption analysis, the total water consumption was the summation of annual metered billed consumption and annual unmetered billed consumption.

A. Water consumption by customer type

2019/20 and 2020/21, the total customer meter including public tap was 10240 and 11547 respectively. The total consumption by customer type in each budget year was summarized in Table 4-5.

Table 3-5 2019/20 and 2020/21 water consumption by customer type

Subscriber type	2019/20 consumption (m^3/year)	consumption percentage (%)	2020/21 consumption (m^3/year)	consumption percentage (%)
private	764623	69.6	1043448	70.3
Community	3250	0.3	4699	0.3
Commercial	224871	20.5	292430	19.7
Industrial	68697	6.3	84074	5.7
Governmental	20506	1.9	39910	2.7
NGO	9033	0.8	9950	0.7
Religious	6955	0.8	10745	0.7
Total	1097935	100	1485256	100

As the utility water consumption management system: private is house connection and yard connection; community is yard share connection and public tap connection; the rest are non-domestic customer subscribers. As seen in the table, a large proportion of the town's residents uses the home connection and yard connection service modes. The minimum percentage of the residential uses yard share connection and public tap connection. Residential consumption was 69.9% in the 2019/20 budget year, with per capita consumption of 22.6 l/c/d. in the 2020/21 budget year; the residential consumption was 70.6% with 28.9l/c/d. from 2019/20 budget year to 2020/21 budget year water supply service in the Mojo town was raised by 6.4l/c/d.

B. Water consumption by kebele (Cluster)

Water consumption by kebele was analyzed for hydraulic performance analysis junction demand input using proportion by the consumption. For junction demand input non-domestic consumption was merged with domestic consumption and explained by l/c/day. the study town the majority of non-domestic users exist in 02 and 03 kebele.

Table 3-6 Water consumption by kebele non-domestic with domestic

Cluster name	2019/20 year			2020/21 year		
	population (no.)	consumption (m^3/year)	per capita (l/c/d)	population (no.)	consumption (m^3/year)	per capita (l/c/d)
01 kebele	29,556	215,125	19.94	31477	257,546	22.4
02 kebele	32,780	431,616	36.07	34912	710,147	55.7
03 kebele	30,928	451,194	39.97	32939	517,437	43
Total	93,264	1,097,935	95.98	99,328	1,485,130	121.1

3.4.2 Water production analysis

The existing seven boreholes yield capacity, in a day, using cubic meters per day, was analyzed by summations of in liter per second and multiplied by unit change factor and pumping hour factor for the research area's water supply system to analyze the existing water supply capacity.

$$production\ \left({m^3}/{day}\right) = \sum boreholes\ yield\ \left(\frac{l}{s}\right) * 3.6 * \frac{pumping\ hours}{24\ hours} - [3.3]$$

The demand balancing capacity was assessed as the ratio of total production volume (excluding non-domestic demand) to total population number to explain the existing source capacity by domestic consumption in liters per day. Non-domestic demand was calculated using non-domestic consumption analyzed under water consumption by customer type, which accounts for average 30% of total consumption in 2019/20 and 2020/21. In addition, unaccounted for water was accounted for according to the national plan, with non-revenue water accounting for 20% of the system input volume. The domestic demand factor is set at 0.5, excluding non-domestic demand and UFW.

$$Domestic\ demand\ balance\ capacity\ \left(\frac{\frac{l}{c}}{d}\right) = \frac{production\ (m^3) * 1000ltr * Df}{population\ number} --[3.4]$$

Where D_f – Domestic demand factor.

The water production capacity of the existed water source, boreholes studied and recorded during the study period, data collection were as follow:

Table 3-7 Borehole production capacity in 2020/21

Code	BH-6	BH-8	BH-9	BH-10	BH-11	BH-12	BH-13	total
Yield (l/s)	7	3.25	7.13	30.3	25.5	36	32	141.18
Production (m^3/day)	504	234	5134.36	2181.6	1836	2592	2302	10165

Using 20 hours of pumping per day and the study period population size of the study town, the existing well capacity of the study town water supply sources can offer 51 l/c/d domestic and 50 percent of supply non-domestic services including UFW. The recorded annual water production delivered to the system by the water utility in the past five budget years was as the following table:

Table 3-8 Annual production delivered to the system

Year	2016/17	2017/18	2018/19	2019/20	2020/21
Production (m^3)/year	1,753,752	1,797,541	1,814,682	1,875,210	2,768,513

(Source: MTWSSSE, 2021)

During 2020/21, only 74.6% of the borehole production capacity was delivered to the system due to the electric power intermittent and supply pump off for distribution network line maintenance.

3.5 Water loss analysis

Water loss was analyzed as total water loss, apparent loss, real loss and non-revenue water. Total water loss was calculated using the general water balance equation and production and consumption data collected over five years to determine the effects of adding a new borehole line to the existing system without upgrading the distribution network. The top-down approach method was chosen for partitioning total water loss into apparent and real loss, as stated under literature reviewed in sub-title 2.3.1.

Water loss performance was analyzed using UFW percentage to determine whether existing system water loss was acceptable or not acceptable, and using the basic operational performance indicator (liter per service connection per day) and water loss reduction benchmark (liter per km length of mainline per day) to determine whether structural components were in good, average, or bad condition, according to the worldwide water loss level guideline.

3.5.1 Unaccounted for water analysis

In unmetered authorized consumption insignificant on the result of total water loss, total water loss equal with unaccounted-for water. The total water loss or unaccounted for water-focused on the five-year analysis and uses a definite water balance method; total input volume and total volume consumed.

A. Annual UFW

Five years of production and consumption Data was collected to analyze yearly UFW. As described above, the annual UFW is used to analyze the system's effects on each other, as well as whether the existing water loss is acceptable or not acceptable. And also, to analyze the impact of water loss on water demand management.

$$Unaccounted\ for\ water\ \% = \left(\frac{TSV - TACV}{TIV}\right) * 100 - - - - - - - - [3.5]$$

Where TIV- total supply volume (produced) in analysis year, TACV – total authorized consumption volume in the analysis year

B. Monthly UFW

Based on the available data collected, mothy accounted for water analyzed for two years. i.e., 2019/20 and 2020/21. Monthly UFW analysis was chosen to assess the worst season for water supply structure and to assess management differences in a month and a season. Also, to determine the causes of seasonal variations.

$$Monthly\ UFW = \frac{MS - MC}{MIV} * 100 - - - - - - - - - - - [3.6]$$

Where MS- monthly supply, MC – monthly consumption

3.5.2 Apparent loss analysis

Based on utility willingness and system capability, the % of billed metered consumption approach was chosen for apparent loss. According to Mutikanga et al., (2011), a standard apparent loss analysis applying default values in developing countries is dependent on utility service connection capacity, as mentioned in sub-title 2.3.2.1. For intermediate utilities with service connections between 5000 and 50000, such as Mojo water utility, metering error accuracy varies from a maximum of 20% to a minimum of 8%. According to Sharma, (2008), apparent loss which contain water theft, metering inaccuracy, and data acquisition error, accounts for 20% of billed metered consumption. 10% metering inaccuracy was assessed using the study area water meterage and ways of use. To analysis apparent losses, 12% metering inaccuracy was adopted for the Mojo town water supply system by considering international water associations Sharma, (2008) recommendations and the assessed metering inaccuracy using the standard default for developing countries. The additional percentage default value for the study town was the billed unmetered consumption estimation error, which was calculated as 10% of the billed unmetered consumption volume.

I. Customer water meterage

From customer water meter samples taken and studied by the range age of 1 to 3 years, 4 to 5 years, 5 to 10 years, and above 10 years; as the analysis result shows, 55% of the sample were aged below 5 years and 45% of the sample were above 5 years.

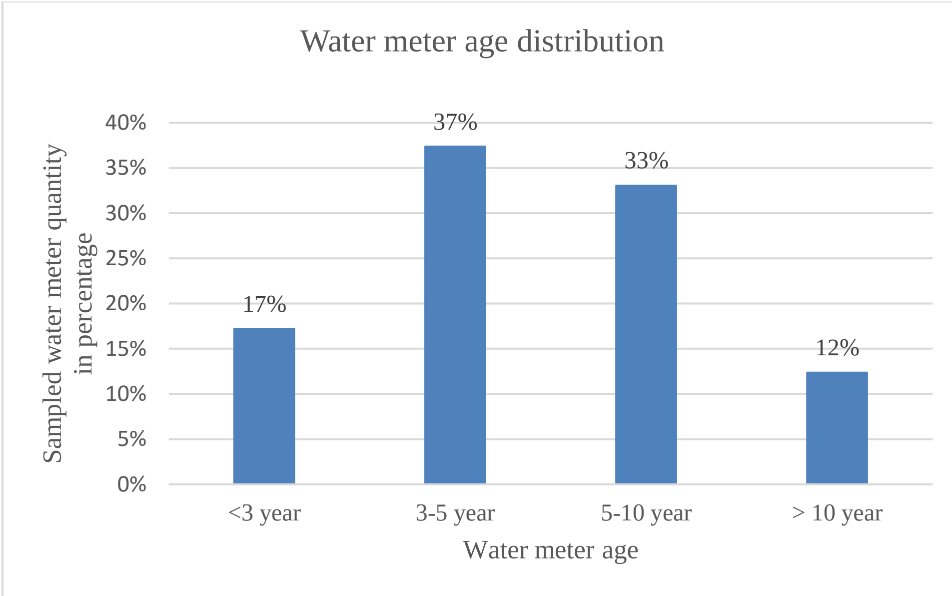


Figure 3-6 Sampled customer water meterage

II. Customer water meter use system

From the taken sample, 20% of customers use a water meter with a storage tank and 80% used a direct supply system or without a storage tank. According to Mutikanga et al., 2011, the apparent loss is large in old and uses with storage tanker water customer meter.

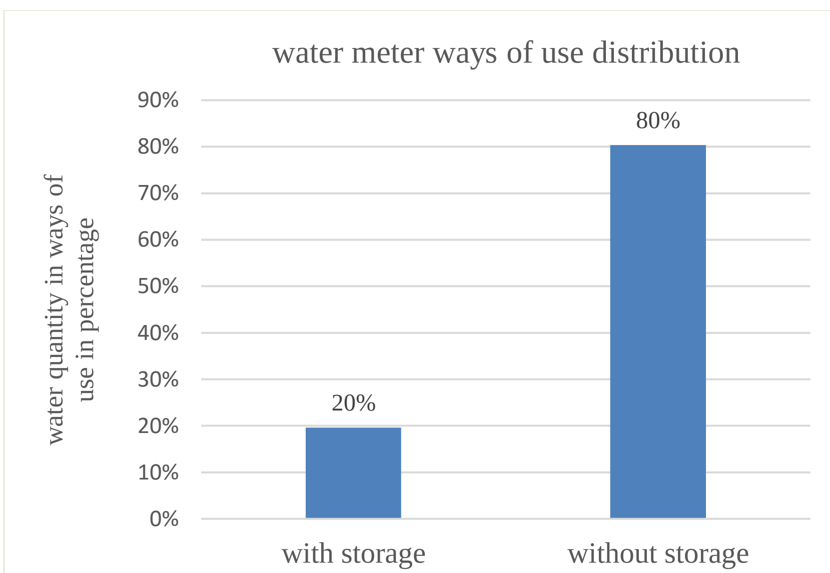


Figure 3-7 Sampled customer water meter use system

3.5.3 Real loss analysis

The real loss was analyzed to evaluate basic operational performance, water loss reduction benchmark, and infrastructure performance. According to the top-down approach, the real loss is the difference of apparent loss from total water loss.

$$Real\ loss\ \left({m^3}/{year}\right) = \ TWL\ \left({m^3}/{year}\right)(-\ Appent\ loss\left({m^3}/{year}\right)\ --[3.10]$$

Where: TWL – total water loss.

Using the analyzed real loss as input, real loss benchmark analyzed in liter per km length of main per day. it is good if less than 10,000 liter/km/day or bad if greater than 18,000 liter/km/day. and also, basic operational performance is analyzed using real loss in liter/number of service conditions per day. it is good if less than 250 l/SC/day and bad if greater than 450 liters per service connection per day.

According to, Sharma (2008), performance indicators of water losses in the distribution system, basic operational performance indicator methods are selectable for the system per capita consumption of fewer than 150 liters per day. according to Vermeersch et al., (2016), basic operational performance using liter per service connection is selectable if the water supply system is properly managed and service connection density greater than 20 per kilometer of main. Also, the local practice should be considered. The Mojo town water supply structure is intermixed of direct and combined supply system; new supply line fitted to pipe line greater than 25 years old. Both methods were selected for real loss performance indicator evaluation and comparison.

3.6 Mixed water supply system hydraulic performance analysis

Using the water supply characteristics for 2020/21, the hydraulic performance of the existing Mojo town water delivery system was investigated.

3.6.1 Distribution network setup and physical data entering procedure

WaterGEMS CONNECT edition update 2 was selected for the dual system water supply, hydraulic performance analysis. The main purposes of performance analysis are to assess the existing system and to evaluate the integration of the hydraulic systems with the existing engineering measures to sustain improvement supply systems or effective water demand management. The existing data, the procedure for distribution network setup as follow:

First step: skeletonizing the distribution network using google earth software by warrant of collected surveying data of common structure, existing pdf format distribution network map, pencil and ruler aided distribution network map and the pipeline location understanding of the institution senior expert. The procedure skeletonizing on Google earth software and entering to water GEMS is shown in Appendix-2.

Second step: entering distribution network using model builder tool and locating the important elements like water source (reservoir), pump, junctions, pipe, tanks and other were located.

Third step: as explained in data collection sub-title 3.3.2 and indicated in Appendix-6, data collected for a data entering procedure were pipe data, junction, and pump curve data. pipe inputs are diameter (mm), roughness factor or material (type), length (m); pump curve (pumping discharge Vs pumping head based on the pump power and the borehole yield capacity), tanks (cross-sectional information, minimum and maximum water level). junction inputs are: entering elevation from the prepared study area contour map and water demand allocations.

3.6.1.1 Pump definition

The data for the pump curve was entered using a standard 3-point pump definition type. The borehole yield on field measurement was the maximum operating discharge in liters per second, and the design discharge was the maximum daily consumption proportion of 2020/21 with water loss consideration. The discharge shut-off is zero. The maximum head at shut-off discharge was 1.5 times the design head. design head is the summation of static and dynamic head. Static head is the difference of pump position elevation from service reservoir elevation. Dynamic head is the friction or the head loss in a pipeline analyzed using the Hazen William friction head analysis. The minimum head at maximum operating discharge was two-thirds of the design discharge.

$$H_{dy} = 10.67\ \frac{LQ^{1.85}}{D^{4.87}C^{1.85}} - - - [3.17]$$

Where: H_{dy} – dynamic head or head loss (meter of water column), L-length of transmission pipe that the summations of suction pipe and delivery pipe (m), Q – the study period well yield based flow in the pipeline (m^3/s), D - diameter of the pipeline (m), C- existing Hazen Williams coefficients.

3.6.2 Junction demand allocation procedure

First step: demand input method was selected based on water consumption data available. There are three methods of automatic demand inputs using water GEMS software. Those methods are point load, area load, and population proportion or land use method. Point load method needs geoinformation referenced water meter data and the area load method is applicable when consumption data is not known in the service area. because the consumption data by cluster (kebele) service area was available load estimation by population proposition was selected for the study analysis.

Second step: preparing the service area Thiessen polygon on water GEMS using node layer selection those selected by selection set tool and polygon boundary layer prepared by shapefile on ArcGIS ArcMap software.

Third step: preparing population layer by using the utility water consumption bill management cluster (kebele) on ArcGIS ArcMap software by shapefile format. Editing the attribute of the prepared population layer by population density type field.

Fourth step: entering the calculated consumption data of the clustered (kebele) area as demand data. Entering the water loss factor (global multiplier); analyzed by the following formula:

$$Water\ loss\ factor\ = \frac{production\ (m^3\ )}{consumption\ (m^3)} - - - - [3.18]$$

On the last; $Junction\ demand\ \left(\frac{l}{s}\right) = P * pcc\ (\frac{\frac{l}{c}}{s}) * df$ ----[3.19]

Where: P- population, PCC- per capita consumption, df – demand factor

The demand of each junction was entered using the first step until the fourth step. The population layer was created using ArcGIS software and a shapefile format based on the Mojo town water supply utility's water consumption management approaches. To create a demand load service area layer, the Thiessen polygon was created using water GEMS software and selected junction. The Thiessen polygon overlaid with the population layer to allocate junction demand in kebeles to kebeles. Water loss factor was distributed uniformly to all junctions by the waterGEMs program language global multiplier.

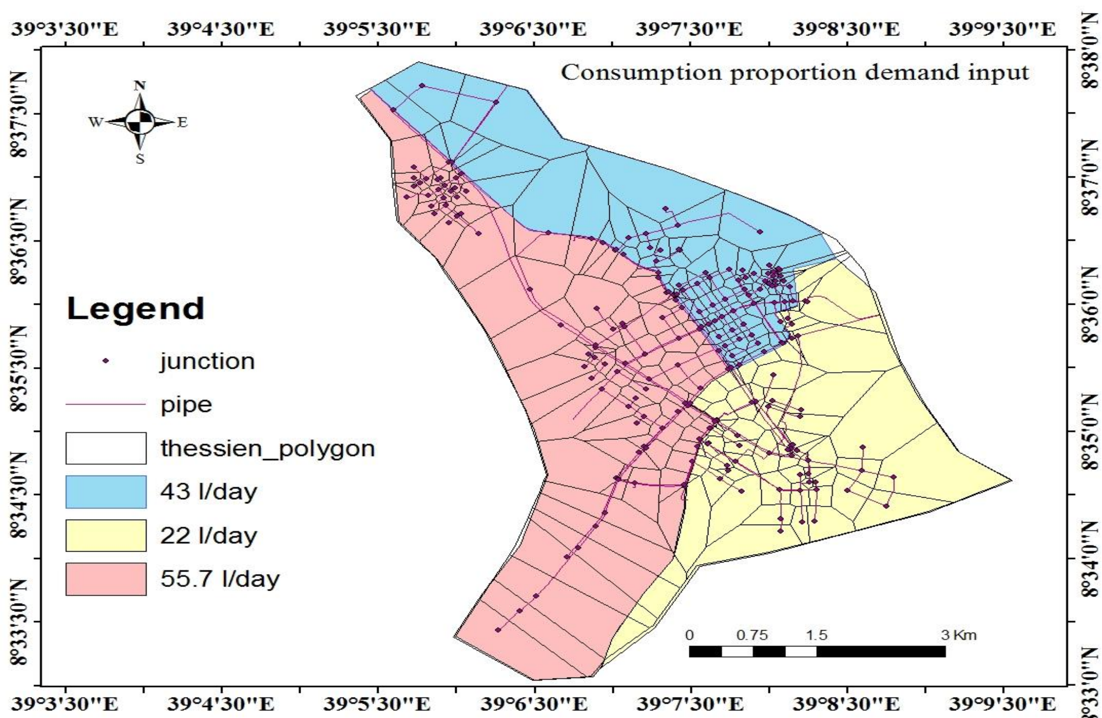


Figure 3-8 Automatic consumption proportion demand input

In addition to customer type, the study town's water consumption management strategies were clustered, with local names of 01 kebele, 02 kebele, and 03 kebele. Using non-domestic consumption by the percentage of domestic, the total consumptions in kebeles by domestic expression as were 22.4l/c/d, 55.7l/c/d, and 43l/c/d in 01 kebele, 02 kebele, and 03 kebele respectively.

Hydraulic pattern

Because the extended period simulation is preferable to steady-state simulation to understand hydraulic performance (pressure, velocity, head loss) variation in water demand variation in a day; extended period simulation was selected and analyzed for the Mojo town mixed water supply system hydraulic performance analysis. Water demand pattern in a day was taken as the water demand pattern adapted as national and also as regional with the peak hour demand determined by the town population size. The peak hour demand of the town was 1.6 multipliers and lower hour demand 0.4 multiplier.

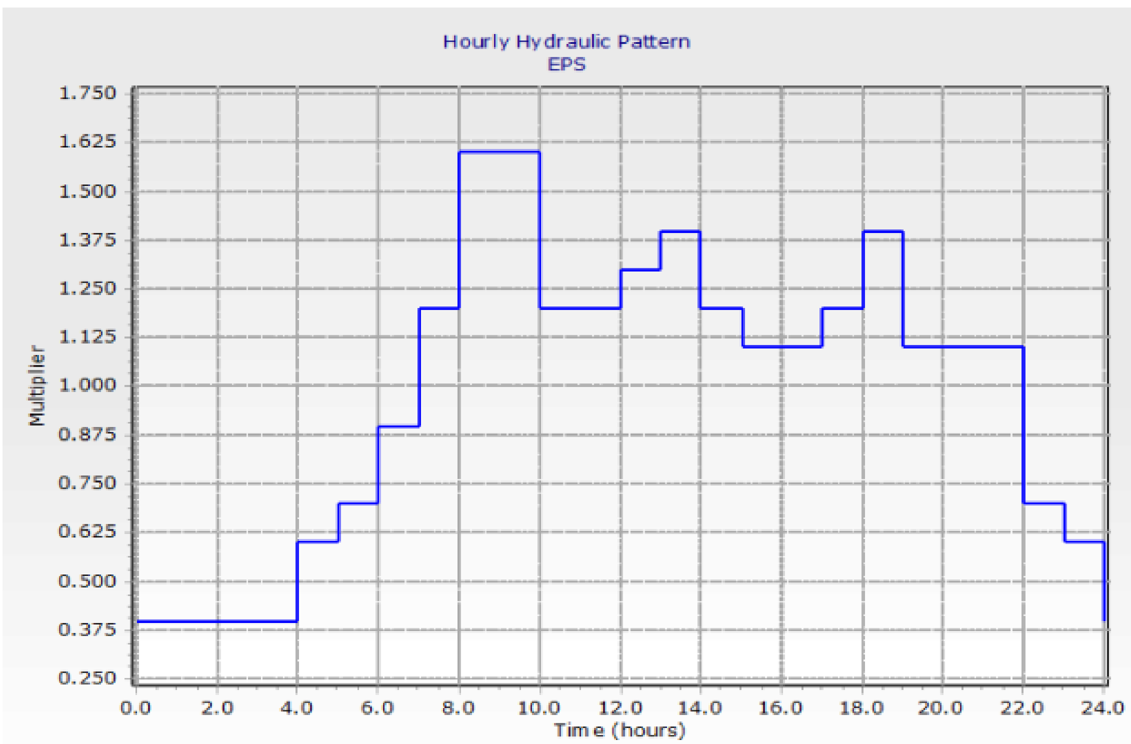


Figure 3-9 Mojo town hourly water demand pattern

Source: (OWWDG,2008)

3.6.3 Hydraulic performance calibration and validation

As stated in the sampling data collection section, the hydraulic model was calibrated using the pressure gauge instrument actual pressure observed. Because direct connections to supply main junctions were dependent on utility willingness and also costly, measurements were made closer to supply mains at private dwelling faucets and public taps. The positions were carefully picked, taking into account the calibration of the complete supply system (combined, direct, and mixed).

The statistical average difference error and correlation coefficient (R^2) were assessed using the following formula to validate the simulated pressure, which was based on the pressure recorded on site.

I. Average difference error

$$average\ difference\ error = \frac{observed\ value - simulated\ value}{total\ number\ of\ sample} - -[3.20]$$

The calibration is validated under which it satisfies average difference error, average $\pm$1.5m pressure head to $\pm$5m pressure head.

II. Correlation coefficient

$$R^2 = \frac{\sum S - S_{mean} * \sum O - O_{mean}}{\sqrt{\sum (S - S_{mean})^2 * \sum (O - O_{mean})^2}} - - - [3.21]$$

Where: S- the simulated pressure, O- field measured pressure, S_{mean}- the sampled average simulated pressure, and O_{mean} – the average field measured pressure.

3.7 Possible engineering measures identification to control water loss and for effective water demand management

The existing water loss, hydraulic performance, and water supply structure situation were the indicators of the existing engineering measures impact on effective water demand management. Infrastructure leakage index is the indicators of the action to be taken to well-maintain the existing water supply system for effective water demand management.

Pressure management application using the identified engineering equipment is the indicators of the effect of engineering measures for effective water demand management.

3.7.1 Infrastructure leakage index analysis (ILI)

The length of the main in kilometers, the average service pipe length, and the number of service connections were all data collected from the utility for the infrastructure leakage index analysis. The data analyzed for ILPI analysis, were current annual real loss, average pressure, and unavoidable annual real loss.

The study used the international water association water loss specialist group technique, ILI is the ratio of CARL, and UARL. ILI shows leakage performance categories depending on the level of the countries. According to the infrastructure leakage performance categories, a developing country's ILI is two times that of a developed country's unit ILI. Depending on this concept, The ILI score for poor and middle-income nations was used to examine the study's infrastructure leakage performance categories.

$$ILI = \frac{CARL}{UARL} - - - - - - - [3.22]$$

Where: ILI - infrastructure leakage index, CARL- current annual real loss, UARL- unavoidable annual real loss.

$$UARL\left(\frac{\frac{liter}{NC}}{day}\right) = \left(18 * \frac{LM}{NC} + 0.8 + 25 * \frac{LP}{NC}\right) * P - - - - [3.23]$$

Where: LM – length of main (km), NC – number of service connection, LP – total length of service connections from the edge of the street to customer meters in kilometers, P- average pressure per pipe segment.

3.7.2 Existing water supply structure situation

The study area's water supply structure was examined utilizing the national water supply design guideline. It was primarily focused on the structure's situation (system type, pipe alignment, pipe material selection, pipe size, pipe economic lifetime, reservoir capacity, and auxiliary equipment (reservoir equipment, borehole equipment, and valves) at the site, as well as a comparison to national water supply design guidelines.

3.7.3 Evaluation of the effect of pressure management for effective water demand management

The effects of pressure management using setting pressure regulating facilities (service reservoir or pressure regulate valve) to effective water demand management; evaluated by pressure leakage proportion.

$$ {}^{L_1}/_{L_O} = ({}^{P_1}/_{P_O})^{N_1} --- [3.24] $$

Where; L_o - the existing leakage, P_o - existing pressure, P_1, and L_1 – the new pressure and leakage after appropriate engineering measures applied to the existing system respectively and N_1 – is the exponent of leakage rate varies with pressure change. Using the predicting guidance (Fig.2.8) and the existing pipe materials, the N_1 value was taken as 1.2.

Leakage reduction is the result difference of leakage after pressure management from leakage before pressure management. Water demand management is the result of leakage management.

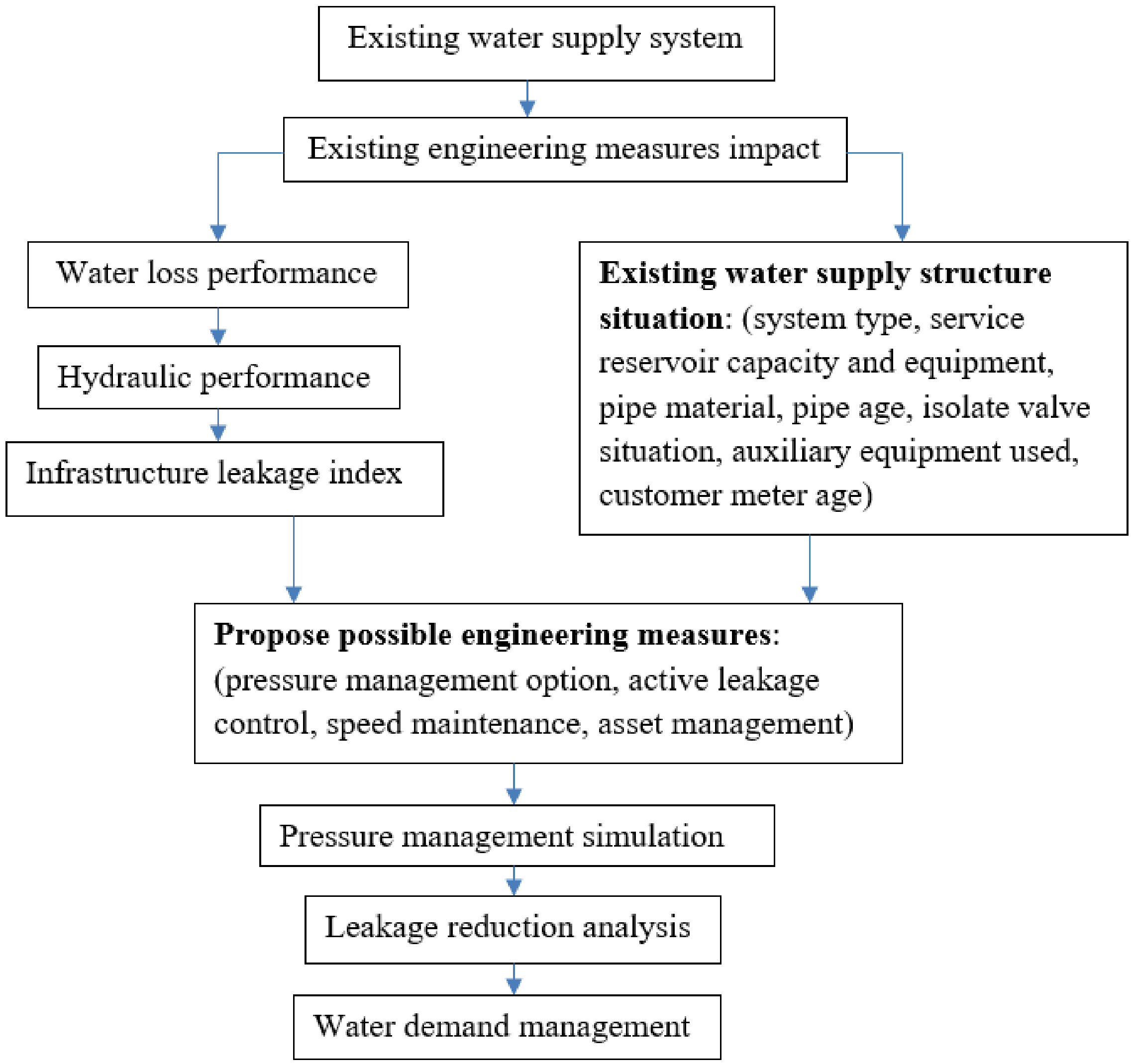


Figure 3-10 Impact of the engineering measures on effective water demand management

CHAPTER FOUR

4. RESULT AND DISCUSSIONS

4.1 Water loss

4.1.1 Unaccounted for water

A. Annual UFW

Unaccounted water entire the town by yearly variation was analyzed for five years (2016/17 -2020/21). The existed UFW in all five years was above the acceptable limit of 25%. i.e., it needs reduction measures. Due to the water source borehole number increased without distribution network update and direct fitted to the existed distribution line, unaccounted for water were increased dramatically from 32.6 percent to 41.6 percent and from 41.6 percent to 46.4 percent.

Table 4-1 Annual unaccounted for water

year	production (m^3/year)	consumption (m^3/year)	UFW (m^3/year)	UFW %
2016/17	1,753,752	1,162,612	591,140	33.7
2017/18	1,797,541	1,208,975	588,566	32.7
2018/19	1,814,682	1,223,681	591,001	32.6
2019/20	1,875,210	1,097,935	780,172	41.6
2020/21	2,768,513	1,485,256	1,283,257	46.4

The graphical representation of unaccounted for water management efficiency in a year is as follow:

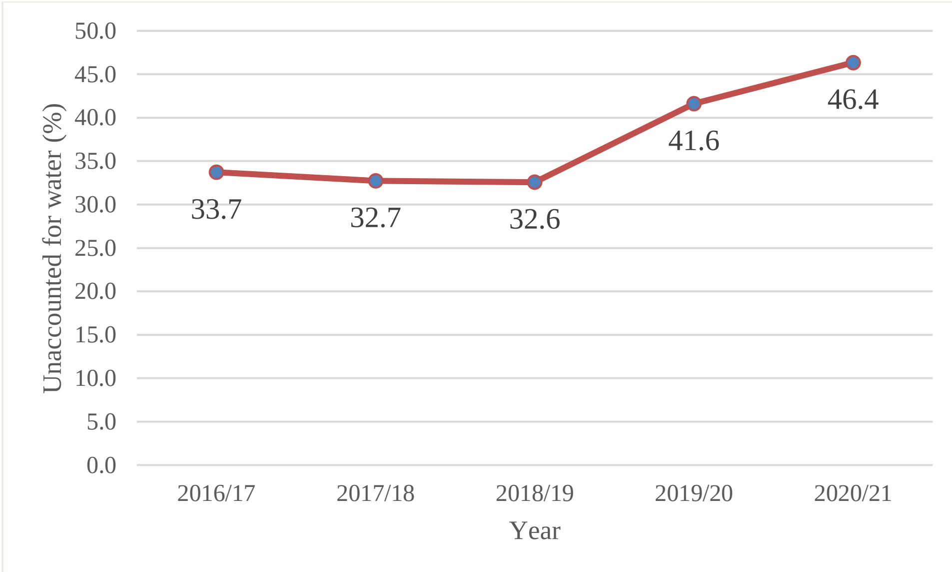


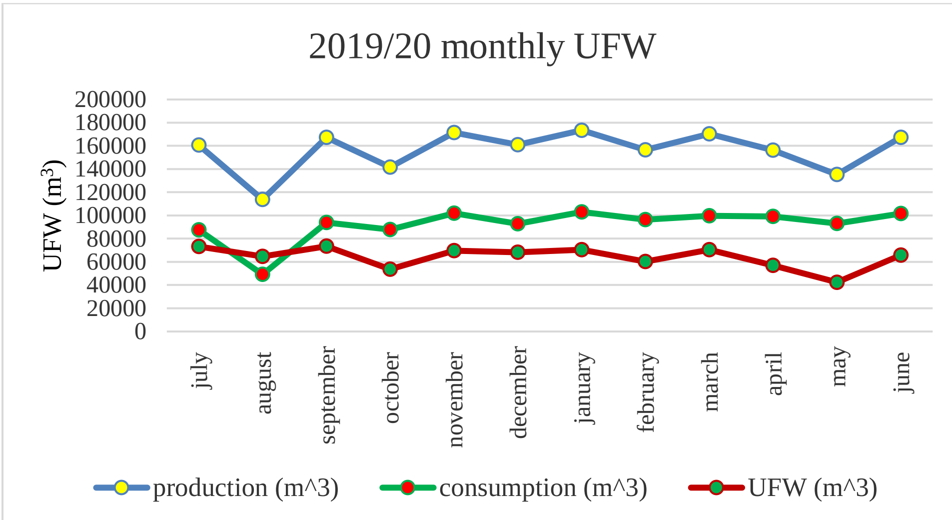
Figure 4-1 Annual unaccounted for water

B. Monthly UFW

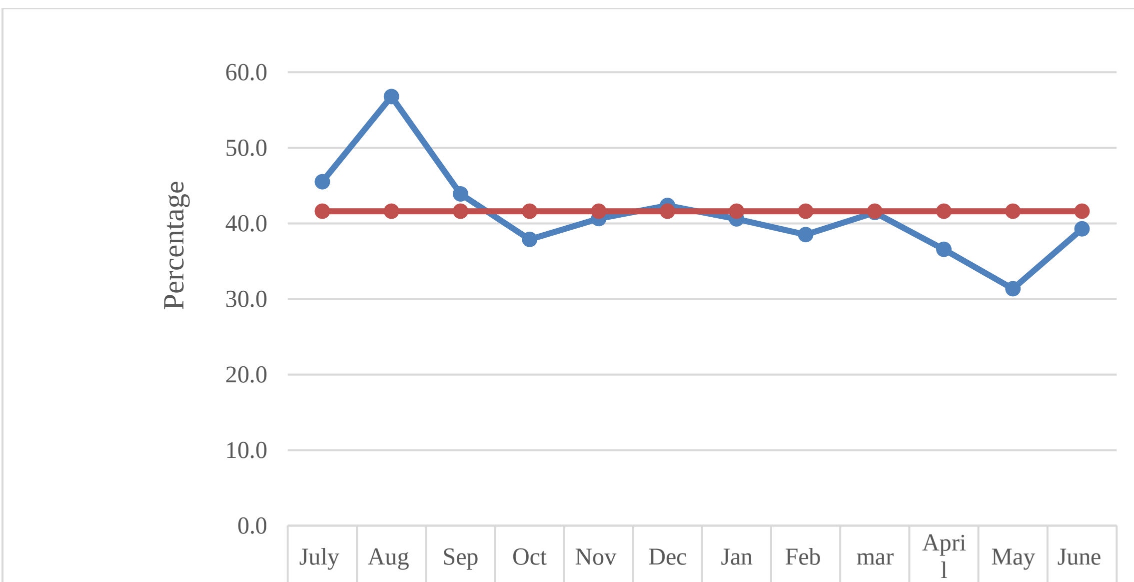
Monthly unaccounted for water was analyzed in two different years (2019/20 and 2020/21) by monthly production and consumption recorded data. Both in two years amount of unaccounted-for water higher than consumption amount in the rainy season. the key methods of the research area's water supply institution's, leak detection system, was community, or public report by their thinking and their own cell phone. During the rainy season; pipe burst or leak flow mixed with flood and its identification difficult for dwellers to report leak or burst to water supply institution. Water use is directly proportional to water production, as demonstrated in both research years. During high-production seasons, there was also high consumption, and during low-production seasons, there was also low consumption. i.e., when output is poor, the community suffers as a result of water scarcity. But water loss was varied: in the months of low production and consumption, water losses were high, and also it continues by maximum value throughout the year. This demonstrates that there is an issue with the active leakage control system, as well as the lack of monitoring and water balance analysis.

I. 2019/20 monthly unaccounted for water

A. Using volume per month



B. Percentage per a month

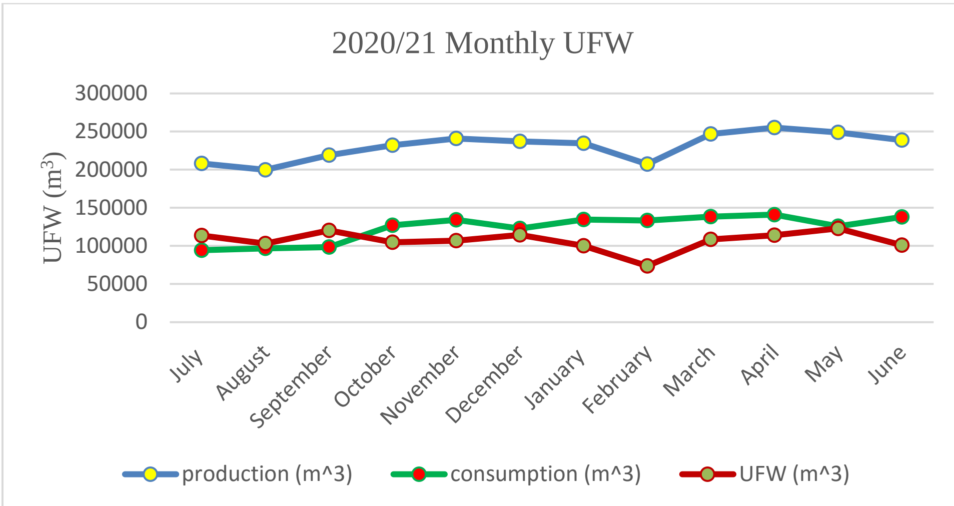


	July	Aug	Sep	Oct	Nov	Dec	Jan	Feb	mar	April	May	June
Monthly UFW %	45.5	56.8	43.9	37.9	40.6	42.4	40.6	38.5	41.4	36.5	31.3	39.3
Average UFW%	41.6	41.6	41.6	41.6	41.6	41.6	41.6	41.6	41.6	41.6	41.6	41.6

Figure 4-2 2019/20 Monthly UFW using volume and %

II. 2020/21 budget year monthly UFW

A. using m^3/month



The reorded unbilled authorized consumption in the study area is only 270m^3 per yer. It is only 22.5m^3 per a month. Relative to water loss per a month, unbilled authorized per a monlth is insignificant in unaccounted for water.

B. Using percentage per month

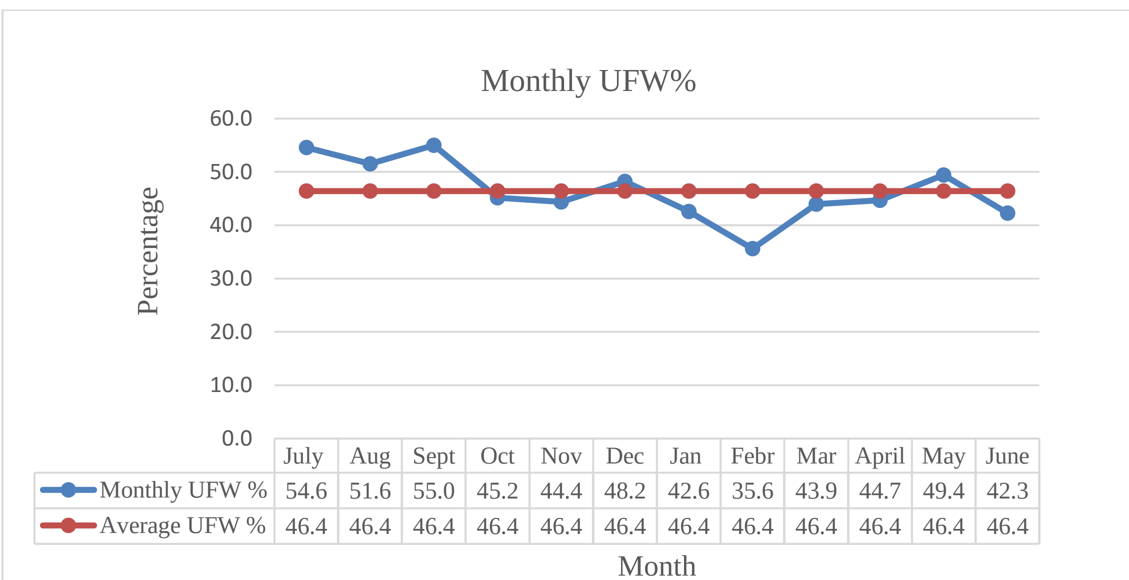


Figure 4-3 2020/2 monthly UFW using volume and %

4.1.2 Water loss component

Using a top-down method, total water loss was divided into apparent loss and loss. Per apparent loss component, apparent loss was analyzed using default % guideline.

A. Apparent loss

Using the common standard default for developing nations, 219,587 m^3/year and 283,680 m^3/year were studied, with 2% MBC unlawful consumption, 12% MBC metering inaccuracy, 6% MBC data acquisition process error, and 10% BUC.

Table 4-2 2019/20 and 2020/21 Apparent losses

Parameter	Measurement	Value (m^3/year)	
		2019/20	2020/21
Unauthorized consumption	2% of MBC	21,959	27,031
Metering inaccuracy	12% of MBC	131,752	16,2185
Data acquisition process error	6% of MBC	65,876	81,093
Billed unmetered consumption estimation error	10% BUC	-	13,371
Total (m^3/year)		219,587	283,680

Where: MBC – billed metered consumption, BUC – billed unmetered consumption

B. Real loss

The real loss is the subtractions of apparent losses from total water loss. The sum of consumption at reservoir cleaning, well field guard consumption, and utility internal use equals total unbilled authorized consumption.

Table 4-3 2019/20 and 2020/21 Real losses

Parameter	Value (m^3) in a budget year	
	2019/20 G.C	2020/21 G.C
Total input volume (IV) (m^3)	1,875,210	2,768,513
Total billed metered consumption (BMC) (m^3)	1,097,935	1,351,546
Total billed unmetered consumption (BUC) (m^3)	-	133710

Total unbilled authorized consumption (UAC) (m^3)	270	270
Apparent loss (AL) (m^3)	219,587	283,680
Real loss (RL) (m^3/year)	**557,418**	**999,307**

Non revenue water

Non-revenue is the summation of unaccounted for water and authorized unbilled consumption. Also, it is the subtraction of billed authorized consumption from total input volume.

Table 4-4 2019/20- and 2020/21 non-revenue water

Parameter	Budget year	
	2019/20	2020/21
Total water loss (m^3/year)	780,172	1,283,257
Total authorized unbilled consumption (m^3/year)	270	270
Non-revenue (m^3/year)	**780,442**	**1,283,527**

4.1.3 Non revenue and real loss performance

I. 2019/20 Non-revenue and real loss performance

The total length of the mainline of the utility water supply system during the 2019/20 budget year was 104.569 km and its service connection was 10240 customer meters and 5 hydrants

Table 4-5 2019/20 Water loss performance

Performance indicator	Parameter	Calculated value (CV)	Acceptable limit		Remark
			explanation	value	
Basic financial performance	Non-revenue (m^3/year)	780,442	<=20% IV according to EGTP-2	375,042	51.9% >AL
Basic operational performance	liter/SC/day	149	Good condition	<250	Good condition

Real loss reduction benchmark	liter/km of LM /day.	14,604.5	Good condition	<10,000	Average condition

Where: WL – water loss

According to the IWA performance indicator guideline, there are good and average circumstances for computed real loss performance. However, it is not excellent, it is negative, depending on the national financial performance strategy and the per capita consumption obtained by the society. As a result, a reduction is required.

II. 2020/21 Non-revenue and real loss performance

The total length of the mainline of the utility water supply system at the 2020/21 work budget year was 105.116 km and its service connection was 11,547 customer meters and 5 hydrants.

Table 4-6 2020/21 Water loss performance

Performance indicator	Parameter	Calculated value (CV)	Acceptable limit (AL)		Remark
			explanation	value	
Basic financial performance	Non-revenue (m^3/year)	1,283,527	<=20% IV according to EGTP-2	553,702.6	56.8% >AL
Basic operational performance	liter/SC/day	237	Good condition	<250	Good condition
Real loss reduction benchmark	liter/km of LM /day.	26,040	Good condition	<10,000	bad condition

Where: - IV – input volume water,

According to the IWA performance indicator guideline, real loss reduction benchmarks greater than 18,000 l/km/day were in poor condition. Furthermore, it is bad, depending on the national financial performance strategy and the society's per capita consumption. As a result, engineering measures are required to reduce the issue.

III. Temporal non-revenue and real loss performance comparison

Table 4-7 Temporal non-revenue and real performance comparison

Performance indicator	parameter	2019/20		2020/21	
		calculated value	acceptable limit comparison	calculated value	acceptable limit comparison
Basic financial performance	Non-revenue (m^3/year)	780,442	51.9%> AL	1,283,527	56.8% >AL
Real reduction benchmark	liter/km/day.	14,604	Average condition	26,040	Bad condition
Basic operational performance	liter/SC/day	149	Good condition	237	Good condition

Where: SC- service connection, km – length of the mainline measurement

Based on comparisons of both the 2019/20 and 2020/21 fundamental financial and operational performance, as well as the real loss reduction benchmark, the 2020/21 water loss performance is in bad condition. In 2020/21, there is a significant real loss of more than 88 liters/SC/day and 11,436 l/km/day as compared to 2019/20. Reduction measure is more important to improve effectively the user water service and the utility income.

4.2 Mixed water supply system hydraulic performance

4.2.1 Extended period simulation hydraulic performance

A. Peak hour consumption hydraulic performance

I. **Pressure**

The Mojo town dual water supply system water distribution network hydraulic performance was analyzed by classified into peak flow condition and minimum flow condition. The distribution network was classified using a pressure contour browser, to indicate which area is high, medium, and low areas. The result of pressure using the estimated average daily demand during peak hour consumption is summarized in Table 4.8. 9.4% of junctions failed to satisfy desirable minimum pressure during peak hour consumption. 18.8% junctions were exceeded the maximum allowable pressures of 70m.

as the special 0.9% was above 100m of pressure. 71.6% of junction exists in allowable pressure minimum 15m and maximum 70m. as shown in Figure 4-4, the red and green colored pressure contour indicates the pressure out of the allowable maximum pressures of 70m. the blue color indicates the pressure head exists in the standard range.

Table 4-8 Peak hour flow pressure head

Pressure range in (m)	Junctions in peak hour consumptions (no.)	% coverage
<15	21	9.4
15-70	156	71.6
70-100	39	17.9
100-150	2	0.9
Total	218	100.0

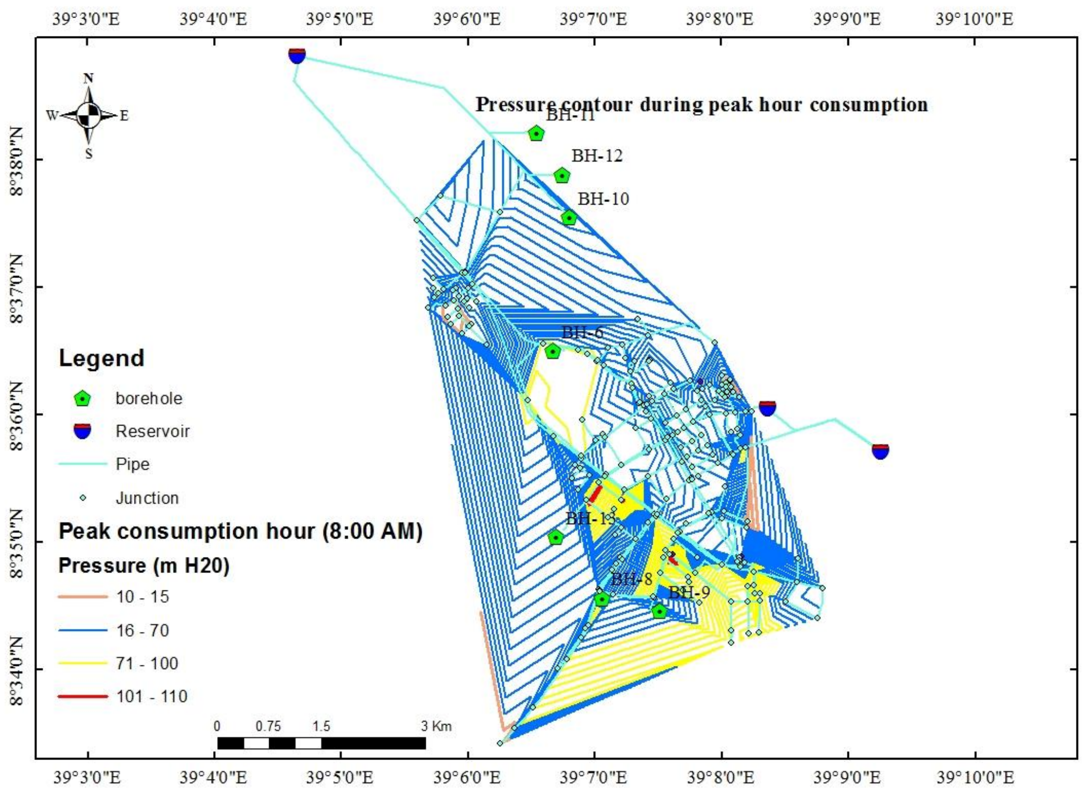


Figure 4-4 Peak hour consumption pressure contour

II. Velocity

The result of velocity using estimated average daily consumption during peak hour consumption is summarized in Table 4-9. 62.4% of the pipe flow velocity failed to satisfy the desirable minimum velocity of 0.5m/s during peak consumption. 37.5 % pipe flow velocity exists in the range of allowable 0.5 m/s to 2m/s. no pipe flow velocity exceeded the maximum allowable velocity of 2m/s. on the loop system network layout, as illustrated in Figure 4-5, a velocity of less than 0.2m/s and also less than 0.5m/s occurred. The majority of the loop network pipe velocity exist below 0.5m/s due to the flow in many direction. According to the Ministry of water resources (2006), urban water supply design requirements, in case of loop system it is acceptable.

Table 4-9 Peak hourly consumption pipe flow velocity

Velocity range in (m/s)	pipe (number)	% coverage
<0.2	78	29
0.2 - 0.5	90	33.5
0.5 - 2	101	37.5
2- 2.5	0	0
Total	269	100.0

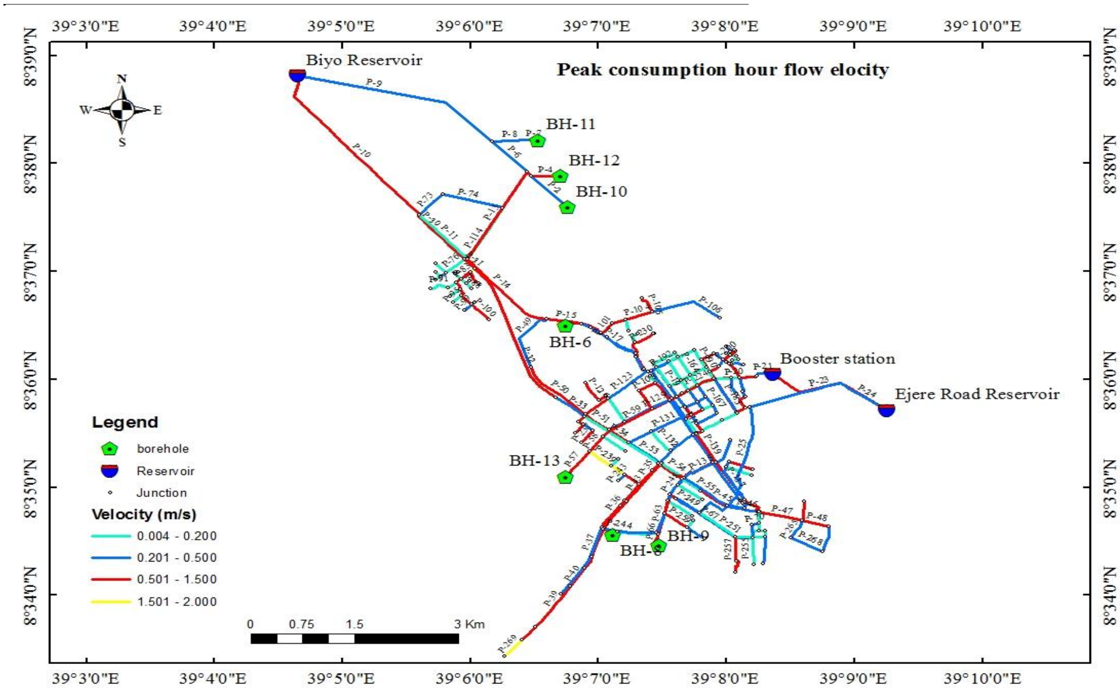


Figure 4-5 peak hour consumption flow velocity

III. Head loss

The result of head loss using estimated average daily consumption during peak hour consumption is summarized in Table 4-10. The result analyzed depend on the acceptable range less than 5m in a one km main length and un-acceptable greater than 5m in a 1km pipe length. 37.2% of the pipe flow head loss gradient failed to satisfy desirable head loss of less than 5m/km. In case of direct pumping head loss is acceptable until 10m per km lenth as special case.

Table 4-10 Peak hour flow head loss

head loss range in (m/km)	pipe (number)	% coverage
< 5	169	62.8
> 5	100	37.2
total	269	100

B. Minimum hourly consumption pipe flow hydraulic performance

I. Pressure

The result of pressure using the estimated average daily demand during minimum consumption hours is summarized in Table 4-11. There are no junctions that failed to satisfy minimum pressure during minimum hourly (5:00 AM) consumption. 85.3% junctions were exceeded the maximum allowable pressures of 70m. as the special 17.9% was above 100m of pressure. 14.7% of junction exists in allowable pressure minimum 15m and maximum 70m. as shown in Figure 4-6, the red and green colored pressure contour indicates the pressure out of the allowable maximum pressures of 70m.

Table 4- 11 Minimum hourly flow pressure head

Pressure range in (m)	Junctions in minimum consumptions hour (no.)	%
<15	0	0.0
15-70	32	14.7
70-100	147	67.4
100-150	39	17.9
Total	218	100.0

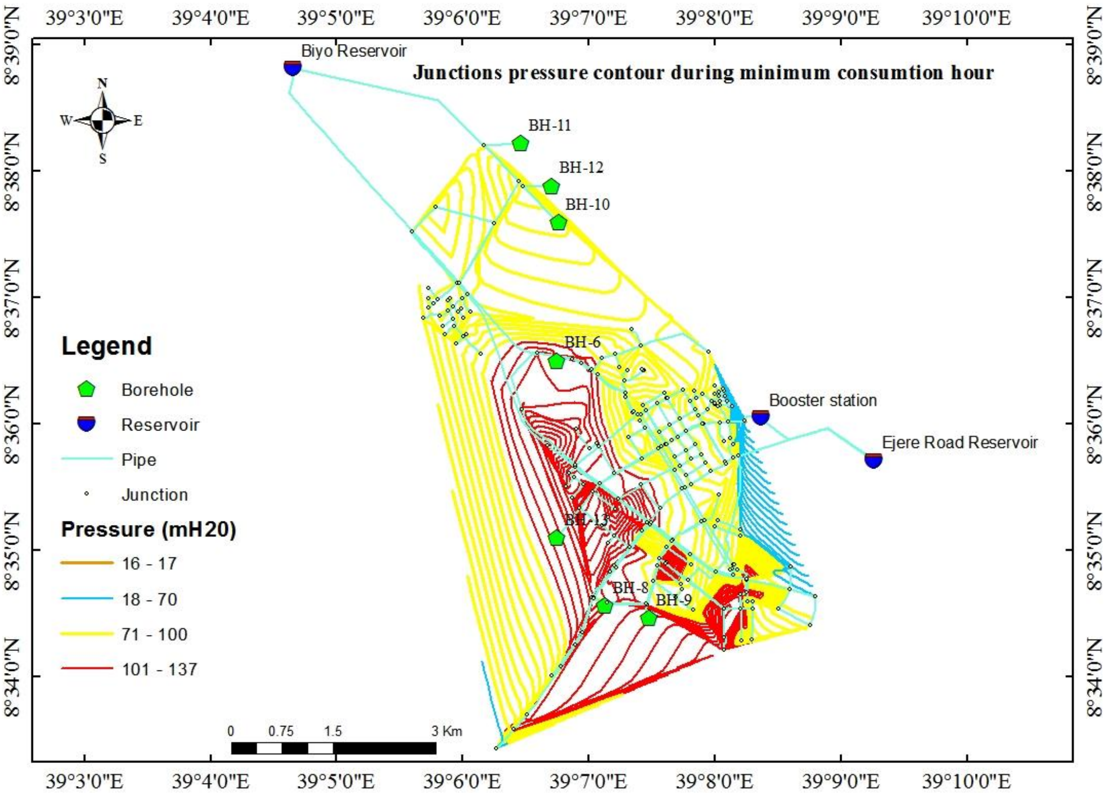


Figure 4-6 Minimum consumption hour pressure contour

II. Velocity

The result of velocity using estimated average daily consumption during minimum consumption hour is summarized in Table 4-12. 16% of pipe flow velocity existed in the range of allowable minimum velocity of 0.5 m/s and maximum velocity of 2m/s. no pipe flow velocity exceeded the maximum allowable velocity of 2m/s. Because the flow in the pipe is minimal during minimum consumption and the loop network flow is in various directions, 84.1 percent of the pipe flow velocity failed to meet the desired minimum velocity of 0.5 m/s. on the loop system network layout, as illustrated in Figure 4-7, a velocity of less than 0.2m/s and also less than 0.5m/s occurred. according to the Ministry of water resources (2006), urban water supply design requirements, in loop system it is acceptable.

Table 4- 12 Minimum hourly consumption pipe flow velocity

Velocity range in (m/s)	pipe (number)	%
<0.2	127	47.2
0.2 - 0.5	99	36.8
0.5 -2	43	16
2-2.5	0	0.0
Total	269	100.0

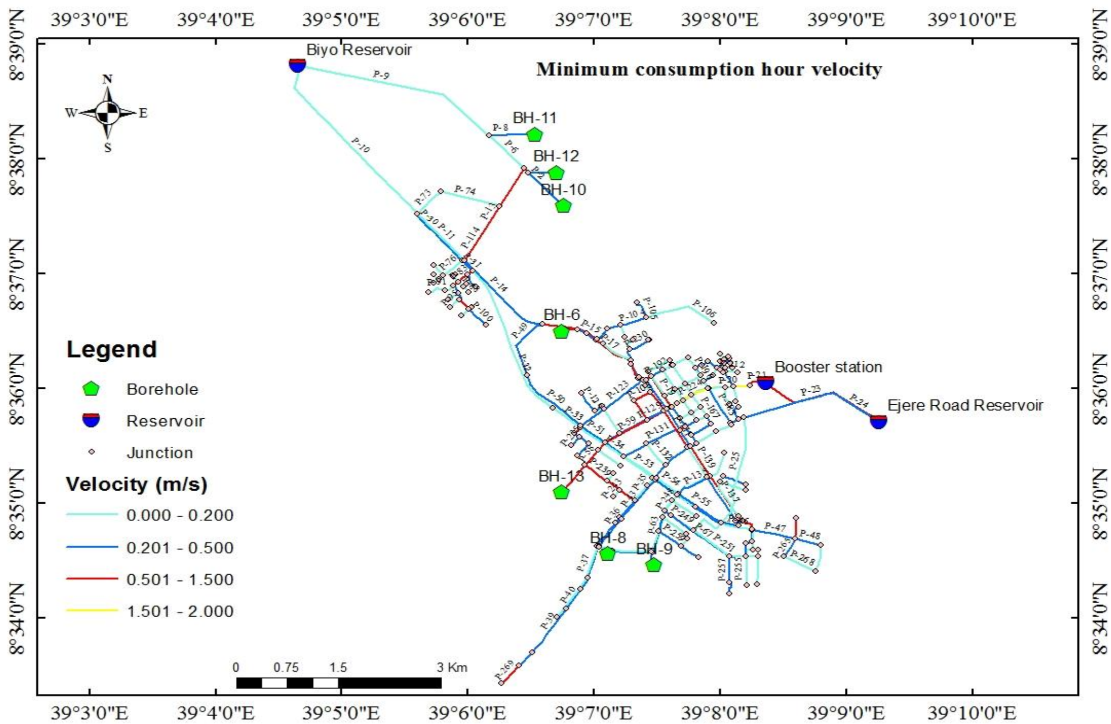


Figure 4-7 Minimum consumption hour flow velocity

III. Head loss gradient

The result of head loss using estimated average daily consumption during minimum hourly consumption is summarized in Table 4-13. The result analyzed depend on the acceptable range less than 5m in a one km main length and un-acceptable greater than 5m in a 1km pipe length. As shown in table 4.13, 20.8% of the pipe flow head loss gradient failed to satisfy the desirable head loss gradient of less than 5m/km.

Table 4-13 Minimum hourly consumption pipe flow head loss

Head loss gradient range in (m/km)	pipe (number)	%
<=5	213	79.2
>5	56	20.8
total	269	100

4.2.2 Calibration and validation

A. Calibration and validation using average difference error

Using pressure gauge tool, field measurements were taken on 11 junctions, 5% of the total nodes. At the public tap and the private tap, the junction pressure was recorded. The simulated pressure was validated using calibration and validation criteria based on the pressure recorded at the location, and the simulated pressure average difference error exist in the acceptable range between average minimum $\pm$1.5 to average maximum $\pm$5.

Table 4-14 Simuated pressure calibration based on measured pressure

Time (Hrs.)	label	measured pressure head (m)	simulated pressure head (m)	difference pressure error (m)
	J-189	18	12.3	5.7
	J-127	15	7.8	7.2
	J-24	66	65	1
	J-39	63	61	2
8:00AM	J-43	65	67	-2
	J-56	43	38	5
	J-66	55	50.3	4.7
	J-78	45	43.4	1.6
	J-101	28	22.4	5.6
	J-133	63	56.3	6.7
	J-172	96	104.8	-8.8
Total sample difference error				28.7
average				2.609

The calibrated simulated pressure is within acceptable range. However, because the junction demand input technique was population consumption proportion, the majority of the simulated pressure head is lower than the observed pressure head. Because the customer water meter in the study area is not geo information based, the point load method was not used.

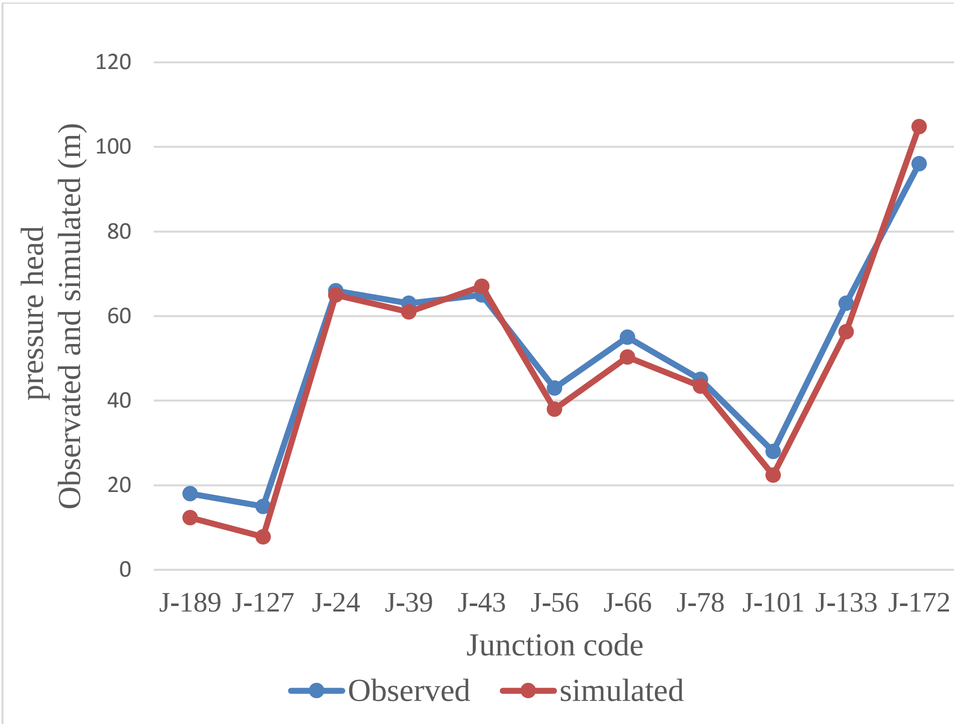


Figure 4-8 Observed and simulated pressure fitness

B. Calibration and validation using correlation coefficient evaluation

Coefficient of determination (R^2) which ranges between 0 and 1, describes the degree of co linearity between simulated and measured data using the proportion of the variance in the measured data, which explained by the model with higher values indicating less error variance. The evaluated R^2 is 0.9896 and it greater than 0.5 shows the model simulated result is co linear with the measured data and the model result is acceptable.

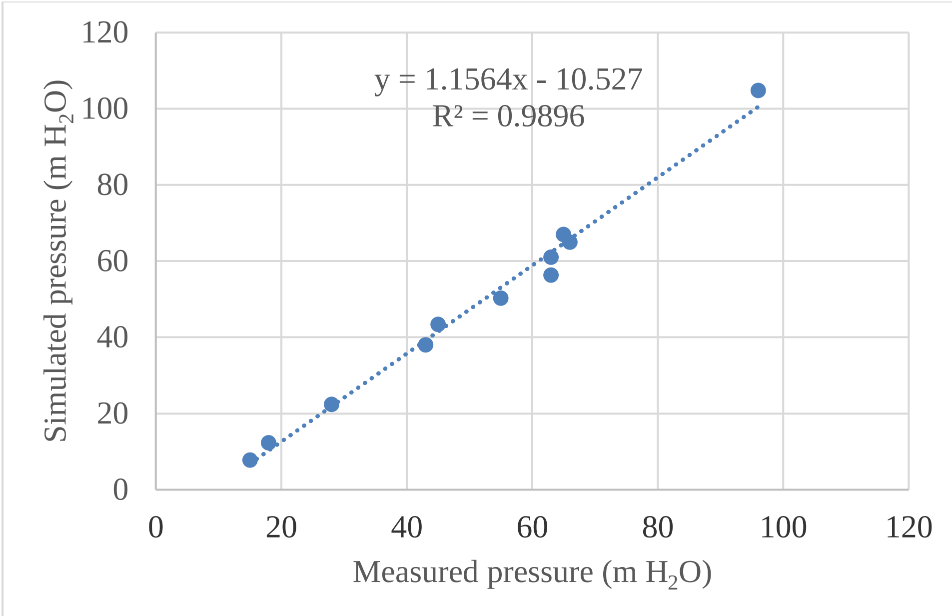


Figure 4-9 Correlated plot during peak hour consumption for validation

As the main source of water loss during peak hour consumption, 18.8% of junctions were over the maximum acceptable pressure of 70m H_20, while 85 percent of junctions were above the maximum allowable pressure of 70m H_20 during the minimum consumption hour. Existing pressure management is one of the technical solutions to be implemented for effective water loss and water demand management.

4.3 Identification of possible engineering measures to control water loss for effective water demand management.

4.3.1 Infrastructure leakage index

Infrastructure leakage index is the function of current annual real loss and unavoidable annual real loss. According to international water association water loss specialist group explanation, the Mojo town water supply system infrastructure leakage performance during the 2020/21 exists under the categories of A2. i.e., the ratio of 2020/21 real loss to an unavoidable real loss in that year is 3.35. the average operating pressure was calculated using average summations of mean peak hour consumption pressure head and minimum hour consumption pressure head, multiplied by the number of junctions divided to pipe number.

Table 4-15 2020/21 infrastructure leakage performance index

Current annual real loss (CARL)	unavoidable annual real loss (UARL) (lit/SC/day)				ILI = $CARL/UARL$
	LM (km)	LP (km)	SC (no.)	average operating pressure (m) = [((49+89)/2) *218]/269	
999307 m^3/year	105.116	138	11552	56	**3.35**
237 lit/SC/day	70.7				

Where: LM-length of main, LP- length of the main pipe, SC- number of service connection, and ILI – infrastructure leakage index.

Infrastructure leakage performance falls into the A2 category, indicating that the water supply system infrastructure requires corrective action employing pressure management, active leakage control, and speed quality maintenance for effective water demand management.

4.3.2 Water supply structure situation

4.3.2.1 Water supply system type

The existing system is a mixed system, as shown in Figure 4.10, direct pumping to the service reservoir and direct pumping to the distribution network. The distribution system from borehole 11 is a combined system with use of non-return valve, and the distribution system from the other boreholes (BH-10, 12, 13, 6, 8, and 9) is direct to the distribution network. According to the Ethiopian Ministry of Water Resources, Urban Water supply design Criteria (2006), direct pumping from boreholes to customer point without intermediate collector or booster station is impossible. From the concept of physical performance, direct pumping from boreholes to customer points is known as improper use. The system requires a direct connection to a booster reservoir, an additional intermediate collector, or an additional reservoir to overcome the limitation of above permited pressure accourrence at the junctions feed from the direct pumping susyem and chlorine contact hour absence.

4.3.2.2 Pipe material selection and alignment

DCI, GI, HDPE, and uPVC pipe materials are employed in Mojo's water delivery systems, as indicated in Figure 4.10. As shown in Table 4-16, plastic pipes (HDPE and uPVC) account for 88.2 percent of service delivery. Metal pipes make up only 11.8 percent of the total: DCI and GS/GI.

Table 4-16 Pipe material used in Mojo town water supply system

material	length (m)	% coverage
Ductile cast iron (DCI)	5765	5.5
Galvanized iron/ steel (GI)	6568	6.3
High-density poly ethylene (HDPE)	34092	32.5
unplasticized polyethylene (uPVC)	58587	55.8
Total	105012	100.0

Because metallic pipes were chosen for raising the main pipe and plastic pipes were chosen for the distribution network, the pipe material selection that existed in the study region water supply systems made a good engineering contribution. The buried HDPE and uPVC are unaffected by the meteorological conditions of Mojo town, which include moderate air. Corrosion resistance is excellent in plastic pipe materials. They're also long-lasting, lightweight, sturdy, and cost-effective.

Metal pipes, either galvanized steel or ductile cast iron, should be used for pipe alignment above ground and at unique sections such as crossing drains/streams. But, the Selection of unplasticized polyvinyl chloride cross rainy season natural drain and installation at drained bed was one of the weak engineering methods noticeable in the study town water supply system. In addition, a connection with an isolating valve and a manhole near to the drain existed.

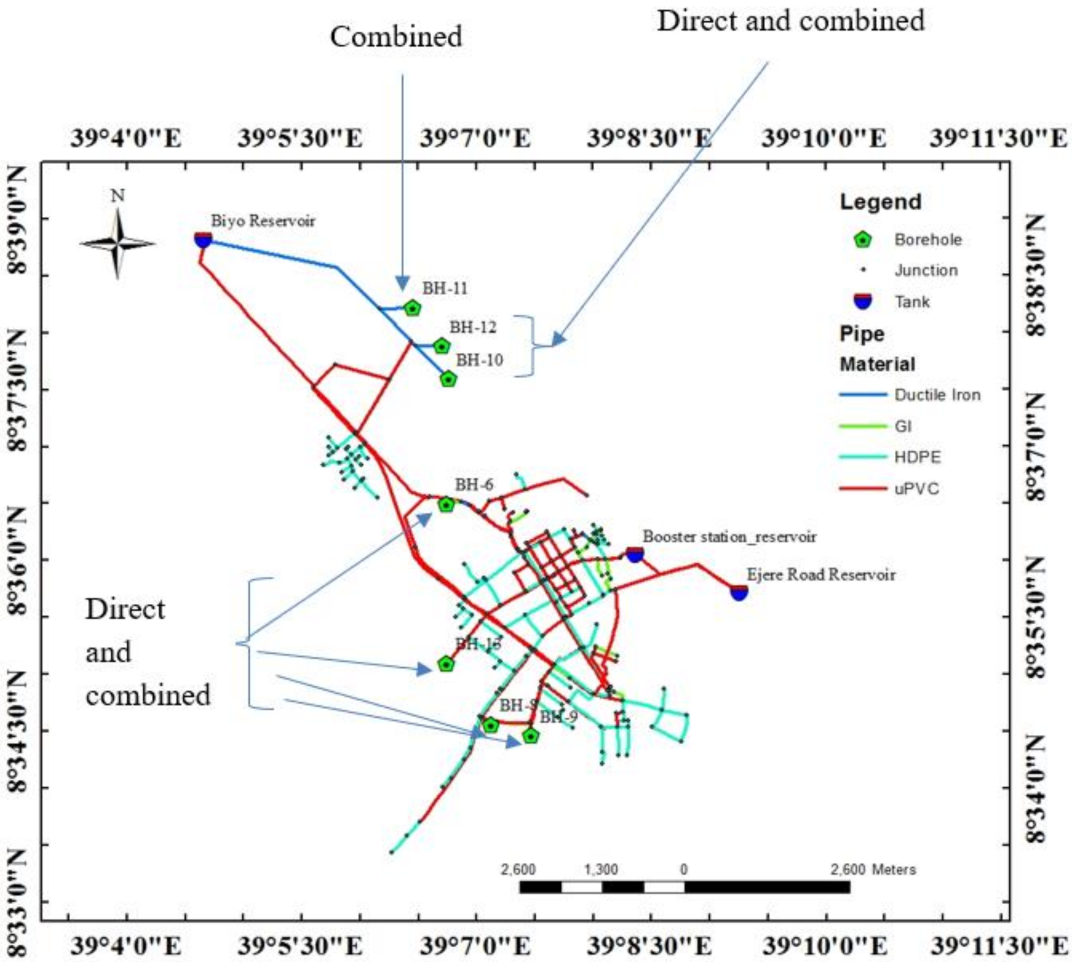


Figure 4-10 System type and pipe material used in Mojo town water supply system

Install ductile cast iron pipe instead of a uPVC pipe, and invert siphon method with upper concrete surrounds instead of drain bed level alignment. Also, changing the location of the junction manhole is one of the possible solutions to the problem.

4.3.2.3 Pipe age

Table 4-17 summarizes the findings of the existing pipe age analysis in the study town water supply system. As shown in the table, 61% less than 10 years age, 15% between 10- and 20-years age, and 24% of the existed pipe age were above 25 years.

As shown in Figure 4-11, the red-colored pipeline indicated above 25 years old and as shown in figure 4-10, the red-colored pipeline material is unplasticized polyvinyl chloride.

From the existed above 25 years aged pipe, 90.1% are uPVC and 8.9% galvanized steel. According to the ministry of water resources of Ethiopia's urban water supply design criteria (2006), the economic life of uPVC is until 25years, not above. At the same age, pipeline disruption was visible in the study town water supply system due to road construction and they clocked by end cup.

Table 4-17 Existed pipe age coverage

Pipe age (year)	Pipe (no.)	% coverage
<=10	165	61
<=20	40	15
>25	65	24
Total	269	100

The solution is to replace the study area's water supply system's polyvinyl chloride pipes, which are over 25 years old.

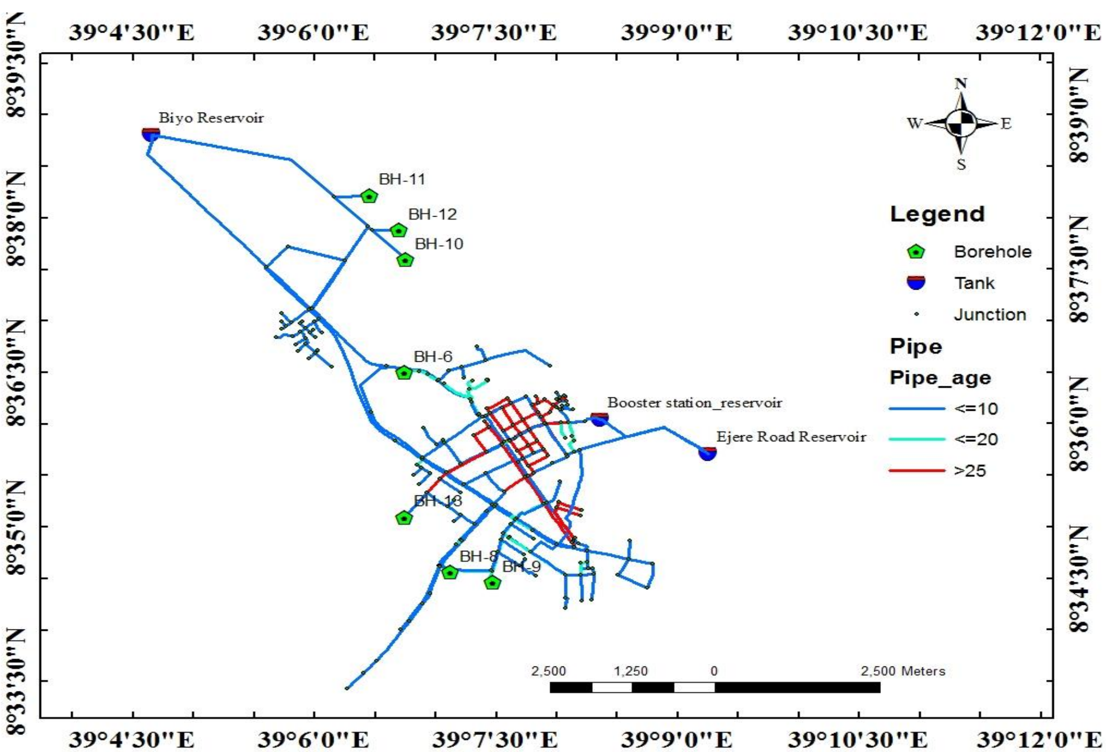


Figure 4-11 pipe age

4.3.2.4 Pipeline valve

The valves utilized in the pipeline of the water supply system of the research area were: isolate valve, check valve, and washout or drain valve was identified based on field observations and office data collection. No spacing of valves and no pressure regulating valves in the pipeline. From the field observation, major isolate valves are installed at the junction branch.

The number of isolation valves existing in a pipeline was unknown. Many of the existing isolation valves had soil, muck, solid waste, road, and solid course material covering them. The absence of an isolation valve manhole cover, as well as a utility monitoring difficulty, have had a greater impact on this isolation not working. This performance demonstrates that the most significant consequence of water loss is a lack of speed and quality maintenance. The problem can be solved by appropriately locating the isolate valve and installing appropriate isolate valves, as well as manhole cleaning, maintenance, and the building of an open surface roof.

4.3.2.5 Reservoir

I. Demand balance capacity

According to the capacity of the examined boreholes given in topic 3.4.2, the existing water production capacity can provide 51 l/c/d to each residential unit in the area. The existed total reservoir capacity, including the booster reservoir, is 1300m^3. According to the reservoir capacity analyzed in appendix-5, the existing reservoir capacity is inefficient for delivering the existing production capacity of 423m^3 per hour.

Depending on the existed source capacity, an Additional 600m^3 new reservoir size and 1300m^3 existing reservoir size can balance the study town water demand of 51 l/c/d.

II. Service reservoir equipment

Both service reservoirs were reinforced concrete, circular, and ground-level reservoirs, based on the building materials, shape, and location. In the research town water supply system's service reservoirs, input pipe, outlet pipe, wash outlet pipe, overflow pipe, ladder, air vent, manhole, outlet pipe gate valve, and wash outlet gate valve were employed as reservoir accessories. Inlet pipe gate valve, water level indicator, float valve, and bypass not installed. float valve or a water level indicator installation is required for overflow water loss management.

III. Booster reservoir equipment

The booster reservoir was a reinforced concrete, circular, and ground-level reservoir, depending on the building materials, shape, and location. Inlet, outlet, wash outlet, overflow pipe, ladder, air vent, manhole, outlet pipe gate valve, and wash outlet gate valve, as well as minimum and maximum water level indicators, were all included. The minimum water level indication avoids cavitation in pumps, while the maximum water level indicator prevents overflow.

IV. Booster Pump station equipment: power source of the booster pump is the national electric power. two centrifugal booster pumps of identical size were fitted, one for operation and the other for standby. Both of them work by shift. Both pumps have their motor, hydraulic controller, and electrical controller. A suction pipe controller and a discharge pipe controller were included in the hydraulic control. Both suction and discharge pipes are steel pipes. suction or inlet pipe equipped with gate valve and discharge pipe equipped with check valve, gate valve, air valve, pressure gauge, and water meter. From the findings of the research, the existing pumping station was supplied with solid engineering recommendations and contributions, and it is cost-effective to keep it running as-is. The issue at hand was power outages, which resulted in an erratic supply system. In the event of an electric power failure, there is no reserve diesel generator available. the standby diesel generator is the solution for the issue,

4.3.2.6 Borehole equipment

The borehole pumps received their drive power from the national electric power grid. All boreholes were equipped with a gate valve, a check valve, and a water meter. Boreholes with a high yield (BH-10, BH-11, BH-12, and BH-13) additionally were equipped with an air valve or vacuum breaker, as well as a pressure gauge and a withdrawal branch for quality testing. At boreholes (BH-6, BH-8, and BH-9) there is no air valve or vacuum breaker at the borehole pipeline summit point. The lack of a standby diesel generator and a borehole water level monitor, as well as the same issue at all boreholes. standby diesel generator, borehole water level indicator, and vacuum breaker at boreholes (BH-6, BH-8, BH-9) are necessary to function as intended.

4.3.2.7 Metering

I. District meter area

There was only one pipeline meter (district meter area) installed at the second main branch. To examine the water balance, 248 customer water meters were fed from the flow pass in the district meter area. It does not meet the required minimum of 500 customer water meters in comparison to the district water meter. Relocating the current DMA, adding up to 500 customer water meters, and disconnecting pipelines that interfere with water balance assessments are all things that are being considered. Designing DMA is a viable engineering HT to overcome uneconomical water loss for the remaining 11047 customer water meters.

II. Customer water meter

According to apparent loss analysis on title 3.5.2, 55 percent of customer water meters are less than five years old, while 45 percent are more than five years old. Out of 45 percent above five years old, 12 percent above 10 years. According to a phone discussion with a utility customer, customer water meters older than ten years have not changed since they began providing service. Investigating the customer's water meter for metering errors and replacing those that are incorrect, with age being the most important consideration, particularly for those above the age of ten.

4.3.3 Identification of possible engineering measures using the existing engineering measures impact

Pressure management, active leakage control, speed quality maintenance, and asset management engineering measures were identified based on the output of water loss performance, hydraulic performance, infrastructure leakage performance categories, and the existed water supply structure situation. Installing a raising main and service reservoir for demand, and pressure balance. Installing district meter areas (DMA), pressure control valves, maintaining isolating valves, and old pipe rehabilitation to upgrade the distribution network. Rehabilitating aging customer water meters can help reduce apparent losses (above 10 years old). The main causes of maximum hydraulic pressure were direct pumping from the borehole to the distribution network and minimum elevation around the study area of Mojo River. Pressure management methods include pipe size optimization, pressure control valves, and the use of a service reservoir. In addition to pressure balance, a service reservoir is required to meet demand, and chlorine contact hour should be improved. Service reservoir is the first option to regulate pressure using changing direct pumping system to combined system. The majority of the pipe flow exists within the minimal range of velocity, as determined by minimum and peak hour usage. Pipe size reduction increases pipe friction and raises head loss. So, pipe size optimization is not necessary measures. As a result, the pressure control valve is the second option to control the maximum pressure due elevation factor.

4.3.4 Pressure management application to conrol water loss for effective water demand management

4.3.4.1 pressure management using service reservoir

During minimum consumption hours, the pressure occurrence in a pipeline may affect the system due to causes high background leakage rate and pipeline burst. As the existed mixed water supply system hydraulic performance analyzed using the study area minimum consumption hour in a topic 4.2.1B, 85.3 percent of 218 junctions were exceeded the maximum allowable pressure head of 70m. this 85.3% above allowable maximum pressure head occurred due to direct connection from the borehole (source) to distribution main without service reservoir or intermediate chamber application and elevation variation. The result of pressure using the estimated average daily demand during minimum consumption hour (5:00 AM) and service reservoir application before a feed to distribution main is summarized in Table 4-18.

Table 4-18 Pressure management using service reservoir

pressure range in (m)	Junctions in minimum consumption hours (no.)	%
<15	1	0.5
15 - 70	135	61.9
71 - 100	76	34.9
> 100	6	2.8
total	218	100

Due to using a service reservoir in a distribution network before a feed to distribution main, the 85.3% above allowable maximum pressure head of 70m junctions reduced to 37.7%. 47.6% junctions were added to the allowable pressure range between 15m to 70m of pressure head.

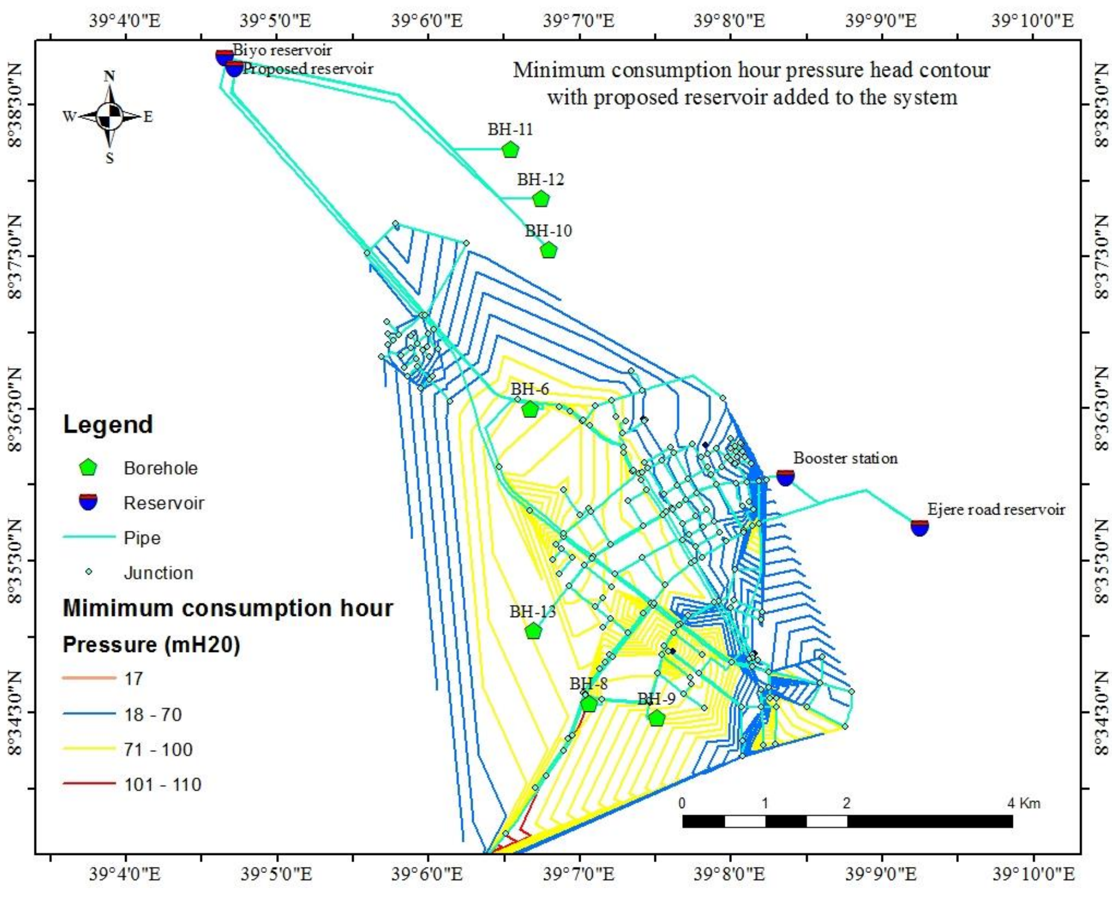


Figure 4- 12 The system pressure contour with service reservoir added to the system

4.3.1.2 Pressure management using pressure regulating valve (PRV)

As summarized in Table 4-19, the left 37.7% above the allowable pressure head of 70m from the use of service reservoir before a feed to distribution main were eliminated by using pressure regulate valve setting on distribution mainline.

Table 4-19 Minimum consumption hour pressure management using PRV

pressure range in (m H20)	Junctions in minimum consumption hours (no.)	%
<15	0	0.5
15 - 70	218	100
71 - 100	0	0.0
> 100	0	0.0
total	218	100

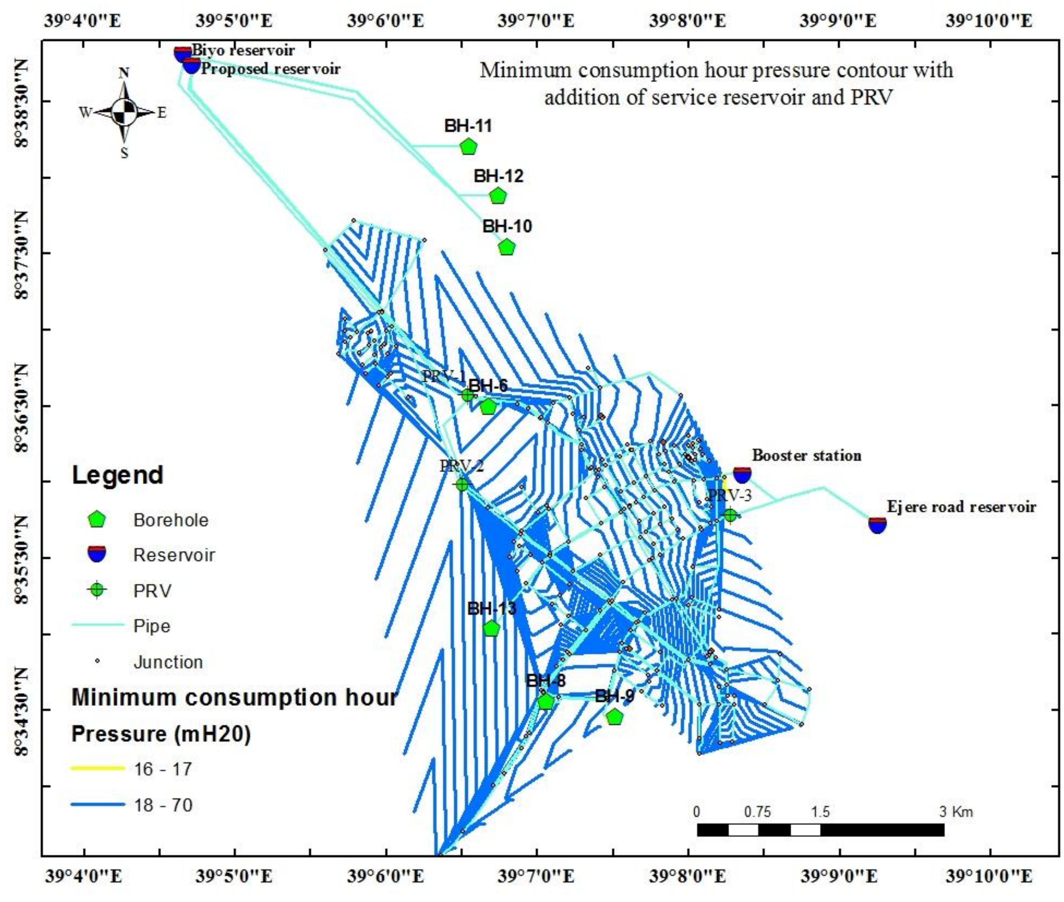


Figure 4-13 Minimum consumption hour pressure management

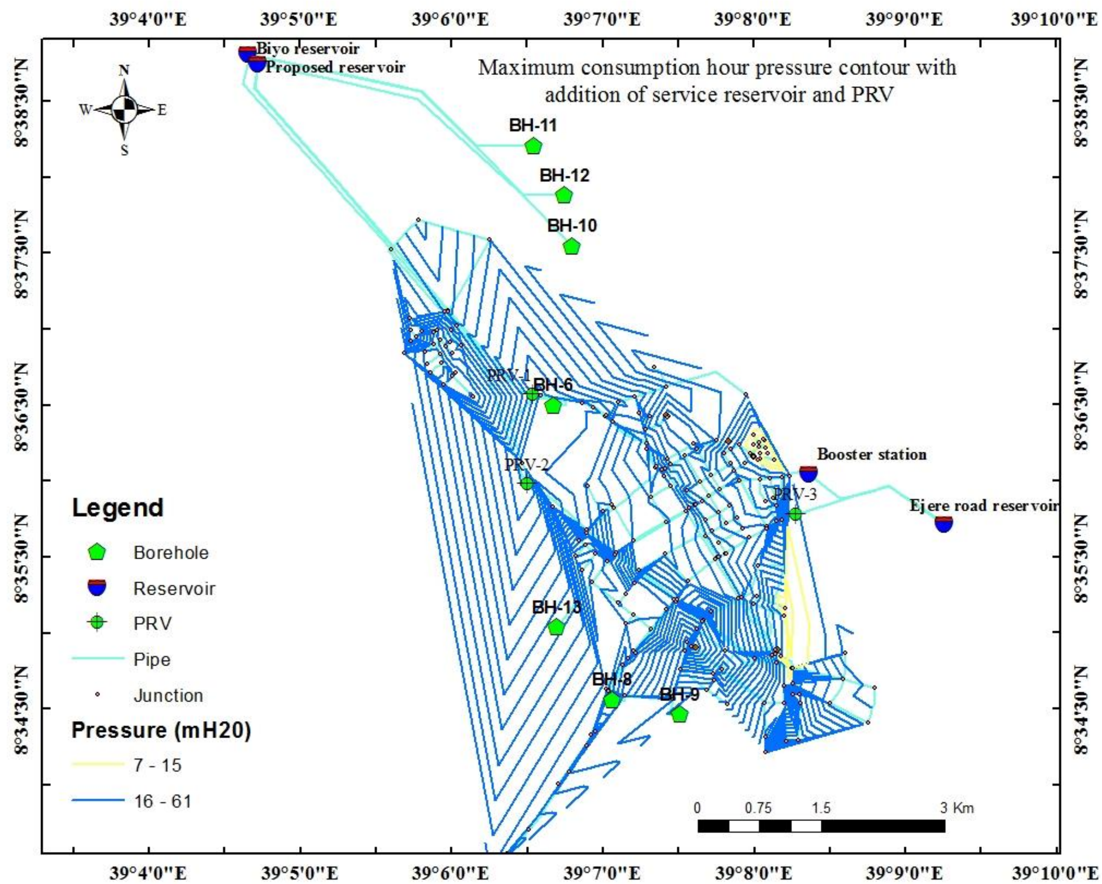


Figure 4-14 Maximum consumption hour pressure management

4.3.5 Leakage reduction

The major effect of pressure management in a water supply system is to reduce the leakage (background or burst) rate in a pipeline system. as shown in figure 4-18, the pressure head average using minimum consumption hour (5:00 AM) simulation of the existed system, after the proposed reservoir added and after both proposed reservoir and PRV added were 89.25m, 67.4m and 46.54m respectively. Using 1.2 exponents of leakage rate varies with pressure change, the reduced leakage by using the proposed engineering measure (service reservoir and pressure regulate valve) is 464460m^3/year

Reduced leakage in cubic meter per year equal to the leakage before reservoir and PRV applied minus leakage after reservoir and PRV applied. it is the substractions of 464460m^3/year from 999307m^3/year. it is 1465.3m^3/day, 13.94m^3/km main line/day and 126.85 liter/service connection/day.

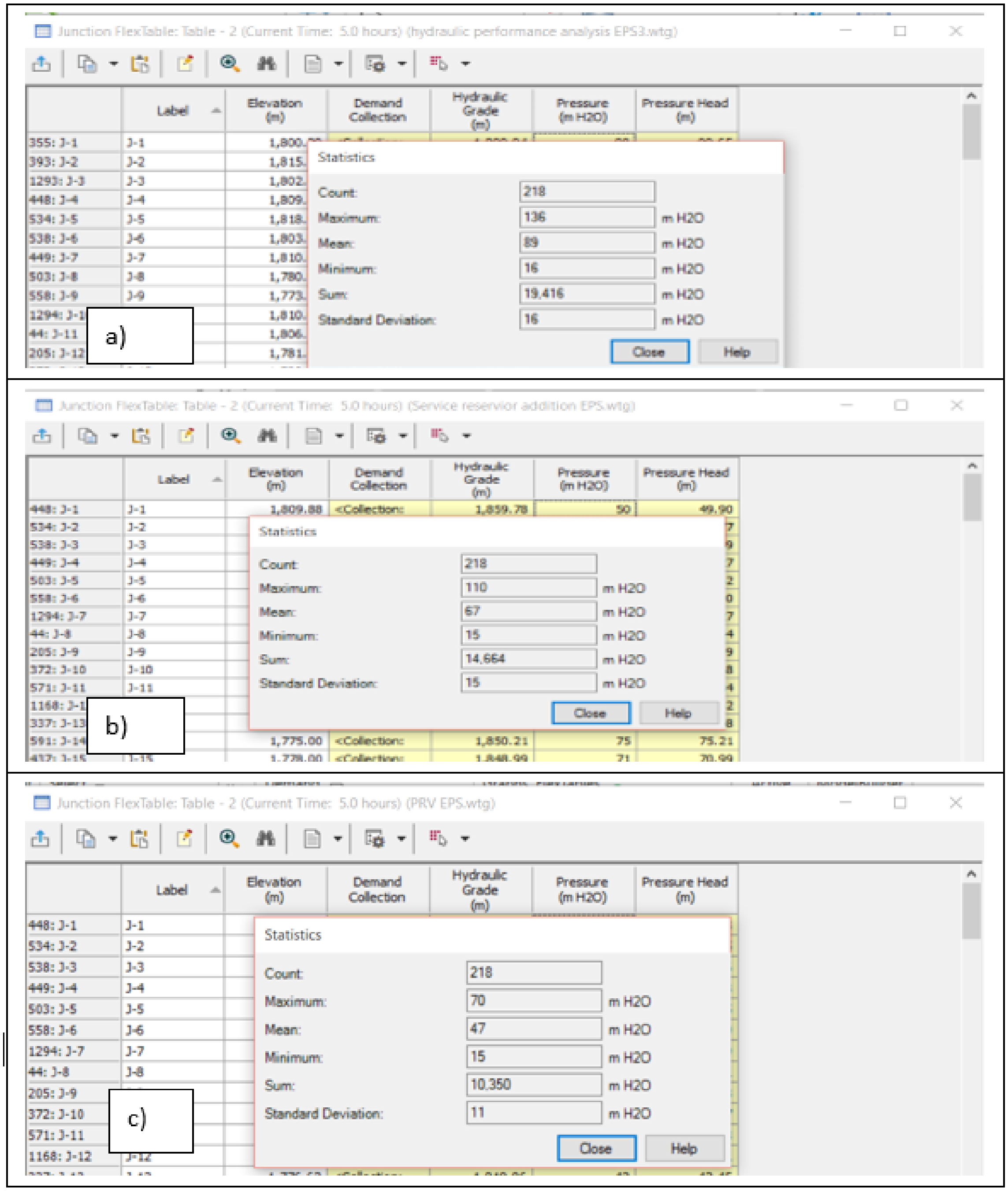


Figure 4-15 Pressure head statistics comparison

a) the existed system pressure head statistics b) pressure head statistics after proposed service reservoir added c) pressure head statistics after proposed service reservoir and PRV added.

Unaccounted for water reduced is the substractions of leakage reduced from unaccounted for water before pressure managed. It is 748410 m^3/year in volume per year and 27 pencent in percentage of input volume. 27 percent of unaccounted for water is near to the acceptable 25 percent unaccounted for water for developing countries. To make accepatable water loss in the study area water supply system, additional to pressure management; the proposed engineering measures using the existing engineering measures impact, active leakage control and speed quality maintenance are important measures.

4.3.6 Water demand management

Water demand management with proper use of possible water supply structure engineering measures application is the function of leakage management. Water demand managed was the value of leakage reduced (saved volume from loss). i.e., 534,847m^3/year. as it analyzed on topic 3.4.2, during the 2020/21 work budget year, the existed system was delivered 28.9 l/c/d domestic consumption and 30% of total consumption, non-domestic consumption. By using 30% total consumption, non-domestic consumption, the proposed engineering measures reduces the leakage and raises water demand by 10.33l/c/d; from 28.9l/c/d per capita consumption to 39.2 l/c/d per capita consumption.

CHAPTER FIVE

5. CONCLUSION and RECOMMENDATION

5.1 Conclusion

On the existed water supply system, water demand didn’t manage due to leakage existing above the real loss benchmark and basic operation performance existed out of a good condition. Pumping directly from a borehole to a customer point has an impact on infrastructure performance and water demand management. The study town's present water supply capacity can provide 51 liters per day for domestic use and 50 percent of the capacity for non-domestic use, including unaccounted for water. Regardless of source capacity, water service in 2020/21 was 28.9 liters per day for domestic use and 30 percent for non-domestic use. 46.4 percent of the water delivered to the system was recorded as unaccounted for water due to apparent losses of 283,680m^3/year and real losses of 999,307m^3/year. The water distribution system was above the allowable maximum pressure of 70m pressure head during both the peak consumption hour (8:00 AM) and the minimum consumption hour extended period simulations, with 18.8 percent junctions in peak consumption (8:00 AM) and 85.3 percent junctions in minimum consumption (5:00 AM).

As it discovered using the output of infrastructure leakage performance index and physical performance; the main reasons for Mojo town water supply system water demand management problems were: improper use of direct pumping from the borehole field to the distribution network; the inefficient capacity of the existed service reservoir, old pipe and recent pipe connected, absence of space valve and pressure regulating valve in a pipeline, absence of service reservoir water level indicator, physically damaged and buried isolate valve, absence of district meter area in a pipeline, and presence of old customer water meter especially those above 10 years old.

Changing from a direct to a combined pumping system, maintaining isolate valves, upgrading the distribution network, old pipe rehabilitation, district meter area, pressure regulates valve, reservoir water level indicator, and old customer water meter rehabilitation are all necessary appropriate measures for the existing water supply system to node balance pressure, minimize water loss and effectively manage water demand.

Regarding pressure management simulation using service reservoir and pressure regulating valve result, the implementation of adding service reservoir and pressure regulate valve to the existed distribution network make effective water demand management of Mojo town water supply system by reducing the mean maximum of pressure from 89m pressure head to 47m pressure head; reducing unaccounted for water from 46.4% to 27%; reducing real loss from 999307m^3/year to 457468m^3/year; and increasing per capita consumption from 28.9l/c/d to 39. l/c/d.

5.2 Recommendation

This study identifies the following recommendation for successful water demand management under the Mojo water supply system, as well as effective water supply system design, construction, and operation as a regional and national consideration:

1. To National and Regional water supply system client, consultant, and contractor

- According to the ministry of water resources (2006) urban water supply design criteria, direct pumping from the well-field to the user for drinking is impossible without intermediate chamber collection or booster station. But The action existed in the Mojo water supply system. it brings service delivery without quality consideration, above allowable pressure, and ineffective water demand management. Due to this reason, guideline consideration is important for implementation.
- Water demand management is unsuccessful when a water supply system is designed without a district meter area and pressure zone. As a result, it is preferable to build a water supply system with DMA and pressure zone.
- The existing national guideline for urban water supply design is over 15 years old. It is preferable to update by integrating DMA and the economic life of the customer's water meter.

2. To Mojo town water supply and sewerage service enterprise

- As evidenced by research field observations, an isolating valve has been hidden by soil erosion, and an existing manhole has been filled with solid course materials and solid garbage. This event demonstrates the absence of a distribution line monitor and speed quality control. Inspection and monitoring of distribution lines, on the other hand, are critical measures for effective water demand control.
- The monthly unaccounted for water analyses reveal that there was a peak in unaccounted for water during the rainy season and a year-round maximum unaccounted-for water. As a utility, this demonstrates poor leak identification and monthly water balance analysis. As a result, as a utility, paying close attention to water balance analysis and leak identification is critical.
- The old customer water meter has a dark color which is not suitable for reading and scours land due to leakage visible under of it. Water meter error affects the utility income and brings complexity to leakage location detection. So, water meter age consideration is very important.
- Making customer water meter code using geoinformation system (GIS) based is very important for management and accuracy of the future research study.

3. For the future research study

- In this research, considering the utility willingness apparent loss was analyzed using the conventional default value method. If it is analyzed using customer meter accuracy test and data mining methods, it gives a more accurate apparent loss value.
- If the system is applicable for district analysis, real loss assessment per district and component is more significant to pinpoint the cause of water demand management problem, to identify effective water demand management reforms and actions.
- Due to the data availability, population proportion junction base demand allocation was selected. But, if GIS-based customer water meter consumption data available, the point load method is may more accurate for the system hydraulic performance analysis.

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Appendix-1 Questionnaires

A. questionnaires with the enterprise expert

1. what water sources does, Mojo drinking water supply system uses?
2. How was the water production recorded?
3. For how many hours in a day does pumping occur?
4. How does inactive customer subscriber bill their consumption?
5. What water demand management, water supply structure management, efficient operation, and water loss management concept has the utility experience adapted?
6. In what situation the distribution network pipe, fitting, and controls are maintained and replaced?
7. What is the difference between burst and leakage?
8. How is the water supply structure management performance evaluated as the office implementation experience?
9. What is the impact of the main challenges to implementing proper water supply structure management and water loss management as the Mojo town water supply service utility?
10. What the old pipeline and old customer meters negative impact on proper water supply structure management?
11. What is the concept of Engineering measures in the water supply system?
12. What best engineering measures to effective water demand management are adapted in the water supply structure management?
13. How do you express the goodness of engineering measures (flow control and pressure control) method to water loss management? Is it adapted to providing safe, clean, and adequate water service in your town?
14. How does the community report the visible water loss when they/he/she sees it?
15. How the public education is giving on water demand management to community behavioral modification?

B. Customer water meter inventory

Customer meter inventory sheet

Remember:

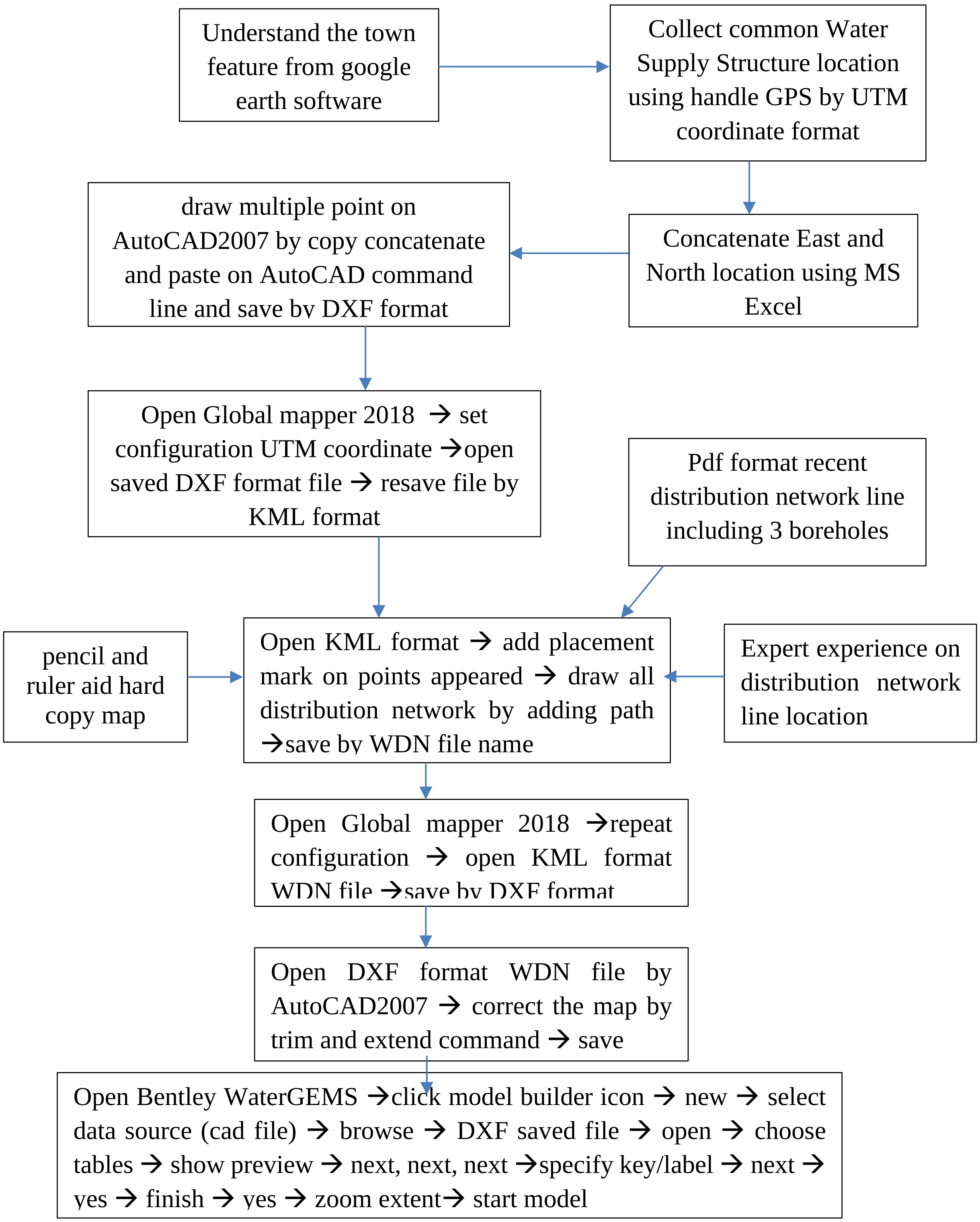
I. Location: A. 01 kebele B. 02 kebele C. 03 Kebele

II. customer meter age: A. 1-3 years B. 3-5 years C. 5-10 years D. >10 years

III. Customer meter use system: A. with storage tank B. without storage tank or direct supply

S.no.	contract no.	location (I)	customer meter age (II)	customer meter use system (III)	S.no.	contract no.	location (I)	customer meter age (II)	Customer meter use system (III)
1	C000379	B	A	B	19	C02-200165	B	D	A
2	C000459	..	A	B	20	C02-200237	..	C	B
3	C000562	..	B	A	21	C02-200324	..	C	A
4	C000708	..	B	B	22	C02-200357	..	C	B
5	C000867	..	C	B	23	C02-200372	..	B	B
6	C001013	..	A	B	24	C02-200382	..	D	B
7	C001552	..	B	B	25	C02-200425	..	C	A
8	C001288		A	B	26	C02-200472	..	C	B
9	C001219	..	C	A	27	C02-200474	..	C	B
10	C001474	..	B	A	28	C02-200500	..	C	B
11	C002569	..	C	B	29	C02-200548	..	B	B
12	C002034	..	C	B	30	C02-200614	..	C	B
13	C002210	..	A	B	31	C02-200615	..	C	B
14	C002607	..	A	B	32	C02-200708	..	B	A
15	C002690	..	C	A	33	C02-200737	..	D	B
16	C02-200068	..	D	B	34	C02-200758	..	C	B
17	C02-200070	..	C	A	35	C02-200[illegible]	[illegible]	C	B
18	C02-200134	..	C	B	36	C02-200830	[illegible]	[illegible]	A

Appendix -2 The town water distribution network map preparation



Appendix-3 Population forecasting

1. Area and Population of Mojo town within kebele at 2020

Kebele		Area (hector)	Household	base population
01		973.68	4225	29556
02	02	1477.44	4566	32780
	03	810.46	4635	30928
Total		3261.58	**13426**	**93264**

Kebele 0l, 02 – municipality administration system,

kebele 01, 02, 03 – WSSSE water consumption management system

2. Growth rate analysis

Growth rate sated by the town socio-economic study report (2014) maximum, middle and minimum 6.3, 5.5 and 4.3 respectively. The current growth rate is as follow:

Year	1994	2007	2014	2020
Population	21997	29547	54447	93264
Increment		7550	24900	38817
Growth rate/year		2.6%	12%	11%
Average growth rate			8.5%	

The existing growth rate is much greater than the growth rate stated by CSA as the country's plan and greater than the maximum assumed by the socio-economic study report. The main cause of the population increments was commonly town expansion and suddenly migrant from the neighbor regional state. The minimum accepted growth rate for this study was the maximum growth rate sated by the regional urban planning institute, 6.3%.

3. population forecasting

Year	No. of year	Base population	Growth rate	Projection
2020	0	93264	0.063	93264

2021	1	93264	0.063	99328

Appendix-4 Detail hydraulic performance: pressure and velocity

Table 5-1 junction elevation, max. consumption hour and min. consumption hour pressure head.

Label	Junction elevation (m)	Minimum consumption hour (5:00am) pressure head (m)	Maximum consumption hour (8:00am) pressure head(m)
J-1	1,800.29	99.66	63
J-2	1,815.36	84.57	47.51
J-3	1,802.63	97.2	60.42
J-4	1,809.88	79.27	46.6
J-5	1,818.92	70.17	37.26
J-6	1,803.95	84.95	51.25
J-7	1,810.00	79.36	46.36
J-8	1,780.90	106.38	72.13
J-9	1,773.29	102.87	70.22
J-10	1,810.00	79.37	46.37
J-11	1,806.34	75.95	34.37
J-12	1,781.65	93.96	59.31
J-13	1,789.69	85.55	49.54
J-14	1,810.00	76.05	29.53
J-15	1,779.16	96.69	62.87
J-16	1,776.62	107.49	49.89
J-17	1,775.00	109.09	50.98
J-18	1,778.00	106.02	44.72
J-19	1,765.00	112.37	27
J-20	1,767.59	110.69	28.62
J-21	1,776.97	109.74	74.63
J-22	1,776.65	108.71	71.93
J-23	1,780.00	104.28	66.27
J-24	1,780.00	103.7	65.1
J-25	1,780.34	103.1	64.12
J-26	1,784.90	98.51	59.41
J-27	1,805.37	77.13	36.03
J-28	1,803.00	76.06	69.11
J-29	1,801.00	78.26	72.03
J-30	1,802.00	77.88	73.93
J-31	1,804.15	75.48	70.61

J-32	1,801.00	77.39	68.04
J-33	1,802.00	64.27	10.99
J-34	1,805.36	76.92	35.35
J-35	1,790.00	90.23	47.18
J-36	1,786.70	92.96	54.07
J-37	1,785.50	80.41	56.67
J-38	1,785.40	86.2	61.45
J-39	1,785.59	86.01	61.26
J-40	1,783.35	89.81	64.95
J-41	1,782.72	92.81	68.17
J-42	1,780.16	103.57	64.78
J-43	1,777.60	106.27	67.4
J-44	1,776.12	107.52	67.84
J-45	1,775.86	107.39	66.29
J-46	1,768.26	114.24	70.44
J-47	1,770.05	107.05	74.86
J-48	1,771.43	105.2	72.75
J-49	1,773.01	103.13	70.5
J-50	1,782.94	84.48	58.33
J-51	1,782.91	84.42	57.92
J-52	1,780.73	84.99	52.69
J-53	1,776.79	88.7	55.58
J-54	1,783.98	81.24	47.16
J-55	1,783.66	81.55	47.4
J-56	1,788.13	76.01	37.99
J-57	1,788.30	75.77	37.5
J-58	1,785.38	97.59	58.26
J-59	1,786.12	96.1	56.43
J-60	1,787.39	93.12	53.4
J-61	1,787.37	92.34	53.35
J-62	1,787.35	92.37	53.37
J-63	1,782.37	99.92	60.75
J-64	1,780.00	102.59	63.44
J-65	1,788.94	90.81	51.69
J-66	1,787.51	91.47	50.14
J-67	1,788.67	90.1	47.98
J-68	1,787.07	91.03	47.54
J-69	1,787.21	90.44	46.05
J-70	1,788.21	89.36	44.86
J-71	1,785.65	91.88	47.31
J-72	1,784.96	92.57	47.97
J-73	1,792.23	85.29	40.68

J-74	1,789.49	88.05	43.48
J-75	1,791.14	86.39	41.81
J-76	1,791.21	91.04	51.31
J-77	1,792.53	85.01	40.54
J-78	1,789.68	87.88	43.38
J-79	1,792.60	85.02	40.6
J-80	1,789.71	87.92	43.52
J-81	1,788.39	89.24	44.85
J-82	1,791.50	86.3	42.18
J-83	1,789.26	88.63	44.68
J-84	1,790.79	87.23	43.56
J-85	1,793.03	84.99	41.31
J-86	1,792.71	85.53	42.38
J-87	1,788.89	90.69	51.27
J-88	1,794.18	85.85	46.44
J-89	1,797.95	82.51	42.87
J-90	1,796.23	85.98	46.29
J-91	1,795.35	86.87	47.13
J-92	1,796.00	84.29	81.79
J-93	1,797.49	79.93	35.58
J-94	1,797.69	81.16	59.32
J-95	1,796.84	80.1	33.91
J-96	1,798.54	78.14	30.98
J-97	1,797.78	79.68	36.17
J-98	1,800.71	75.55	27.01
J-99	1,801.38	75.03	26.85
J-100	1,802.92	73.21	23.96
J-101	1,804.17	71.89	22.37
J-102	1,806.35	68.37	13.98
J-103	1,805.16	68.93	12.28
J-104	1,804.40	69.47	11.97
J-105	1,805.12	68.98	12.32
J-106	1,802.04	71.96	14.95
J-107	1,804.94	69.17	12.58
J-108	1,808.12	66.6	12.2
J-109	1,805.14	71.02	21.79
J-110	1,805.92	70.21	20.85
J-111	1,806.10	70.02	20.65
J-112	1,808.59	67.53	18.15
J-113	1,810.32	65.72	16.05
J-114	1,790.42	70.55	50.19
J-115	1,819.48	58.12	13.64

J-116	1,821.01	15.91	14.97
J-117	1,800.00	77.6	33.11
J-118	1,798.86	78.74	34.26
J-119	1,798.94	78.36	32.79
J-120	1,801.23	76.38	33.02
J-121	1,804.69	72.93	30.24
J-122	1,799.83	79.8	68.56
J-123	1,790.00	97.89	59.15
J-124	1,796.30	91.12	50.72
J-125	1,796.28	85.22	40.77
J-126	1,797.43	84.12	39.84
J-127	1,807.39	69.37	7.75
J-128	1,808.97	72.56	28.2
J-129	1,808.17	107.58	76.99
J-130	1,811.70	61.69	34.19
J-131	1,808.39	69.41	57.91
J-132	1,810.00	67.1	53.05
J-133	1,805.45	71.4	56.44
J-134	1,806.07	71.03	56.95
J-135	1,800.28	79.29	74.19
J-136	1,800.84	78.71	73.54
J-137	1,800.16	118.41	88.78
J-138	1,798.72	119.97	90.39
J-139	1,797.29	121.69	92.22
J-140	1,793.84	85.6	80.03
J-141	1,793.66	85.55	79.15
J-142	1,793.06	126.24	96.93
J-143	1,796.09	121.12	84.26
J-144	1,792.27	124.25	84.91
J-145	1,787.53	92.2	87.69
J-146	1,783.91	95.29	88.84
J-147	1,784.27	94.9	88.36
J-148	1,783.82	95.03	87.35
J-149	1,786.06	91.13	77.4
J-150	1,789.32	87.18	70.99
J-151	1,786.36	90.82	77.07
J-152	1,786.28	92.87	86.26
J-153	1,784.90	136.49	109.15
J-154	1,780.00	98.67	90.3
J-155	1,775.44	105.65	49.73
J-156	1,767.96	112.49	54.25
J-157	1,777.56	101.44	37.93

J-158	1,776.71	101.76	36.33
J-159	1,776.01	101.77	33.86
J-160	1,780.00	97.32	27.72
J-161	1,780.00	100.24	44.85
J-162	1,783.41	96.74	43.19
J-163	1,788.08	92.05	38.45
J-164	1,784.58	95.57	42.39
J-165	1,787.53	92.61	39.41
J-166	1,784.85	95.31	44.44
J-167	1,773.83	115.27	59.52
J-168	1,773.77	115.07	58.37
J-169	1,772.26	123.27	76.94
J-170	1,775.04	120.44	73.99
J-171	1,780.00	129.26	106.22
J-172	1,780.00	127.56	104.34
J-173	1,778.33	129.06	105.26
J-174	1,776.58	106.38	44.65
J-175	1,777.00	105.31	41.24
J-176	1,773.41	107.6	38.81
J-177	1,780.00	102.04	36.98
J-178	1,780.00	101.09	32.58
J-179	1,805.25	80.26	31.24
J-180	1,807.49	76.97	24.16
J-181	1,801.86	81.04	22.61
J-182	1,809.38	73.67	15.78
J-183	1,810.00	73.05	15.13
J-184	1,810.00	73.04	15.12
J-185	1,805.61	77.42	19.45
J-186	1,806.11	76.92	18.94
J-187	1,804.35	78.67	20.67
J-188	1,809.32	73.08	12.85
J-189	1,809.55	72.81	12.4
J-190	1,802.95	79.36	18.75
J-191	1,807.05	74.6	11.6
J-192	1,807.40	74.24	11.22
J-193	1,805.50	75.54	10.33
J-194	1,804.64	76.34	10.96
J-195	1,785.00	93.57	19.45
J-196	1,790.00	88.47	13.97
J-197	1,791.00	87.57	13.42
J-198	1,781.00	95.56	14.16
J-199	1,810.00	75.98	29.23

J-200	1,810.00	75.94	29.02
J-201	1,809.20	76.74	29.8
J-202	1,810.00	75.93	28.97
J-203	1,807.25	78.71	31.9
J-204	1,780.23	95.38	60.72
J-205	1,789.20	84.33	42.11
J-206	1,784.94	92.59	48.02
J-207	1,773.26	103.81	89.66
J-208	1,776.83	99.82	84.16
J-209	1,774.08	102.04	84.48
J-210	1,797.06	78.04	41.51
J-211	1,770.00	105.37	85.06
J-212	1,763.58	111.11	88.34
J-213	1,804.09	114.25	84.54
J-214	1,796.15	81.46	36.98
J-215	1,751.48	132.73	76
J-216	1,779.54	90.16	62.26
J-217	1,806.69	75.52	33.65
J-218	1,800.79	78.77	73.62

Table 5-2 Diameter, min. consumption hour and max. consumption hour flow velocity

label	Diameter (mm)	Minimum consumption hour (5:00AM) flow velocity (m/s)	Maximum consumption hour (8:00AM) flow velocity (m/s)
P-1	250	0.3	0.45
P-2	250	0.3	0.45
P-3	250	0.47	0.62
P-4	250	0.47	0.62
P-5	300	0.53	0.75
P-6	300	0.16	0.2
P-7	250	0.23	0.39
P-8	250	0.23	0.39
P-9	400	0.15	0.27
P-10	300	0.3	0.57
P-11	300	0.18	0.12
P-12	300	0.5	0.65
P-13	250	0.99	0.78
P-14	300	0.49	0.63
P-15	150	1.08	1
P-16	200	0.43	0.16
P-17	150	0.92	0.45
P-18	150	0.91	0.42

P-19	150	0.83	0.24
P-20	150	1.66	0.68
P-21	200	0.92	0.34
P-22	200	0.66	0.67
P-23	200	0.66	0.67
P-24	300	0.21	0.5
P-25	300	0.18	0.42
P-26	80	0.84	1.3
P-27	80	0.84	1.3
P-28	100	0.39	0.49
P-29	100	0.36	0.43
P-30	150	0.49	1.23
P-31	150	0.44	1.11
P-32	150	0.19	0.54
P-33	150	0.12	0.38
P-34	150	0.02	0.16
P-35	80	0.06	0.52
P-36	80	0.49	1.12
P-37	100	0.17	0.4
P-38	100	0.33	0.75
P-39	150	0.25	0.58
P-40	150	0.32	0.74
P-41	150	0.39	0.89
P-42	150	0.44	1.02
P-43	200	0.22	0.51
P-44	200	0.22	0.51
P-45	200	0.29	0.66
P-46	150	0.19	0.44
P-47	100	0.37	0.85
P-48	80	0.25	0.57
P-49	250	0.26	0.44
P-50	200	0.35	0.55
P-51	150	0.44	0.66
P-52	150	0.35	0.52
P-53	150	0.18	0.28
P-54	150	0.06	0.16
P-55	100	0.24	0.45
P-56	150	0.88	1.11
P-57	150	0.88	1.11
P-58	150	0.64	0.68
P-59	100	1.4	1.44
P-60	80	0.48	0.76

P-61	80	0.48	0.76
P-62	80	0.2	0.12
P-63	80	0.05	0.24
P-64	80	0.01	0.32
P-65	125	0.35	0.51
P-66	125	0.35	0.51
P-67	125	0.34	0.49
P-68	125	0.28	0.35
P-69	125	0.27	0.33
P-70	125	0.27	0.32
P-71	125	0.27	0.31
P-72	80	0.65	0.75
P-73	200	0.1	0.25
P-74	150	0.11	0.28
P-75	80	1.23	1.17
P-76	100	0.08	0.19
P-77	100	0.05	0.12
P-78	50	0.09	0.2
P-79	50	0.04	0.09
P-80	50	0.03	0.08
P-81	80	0.71	1.62
P-82	50	0.38	0.86
P-83	80	0.55	1.26
P-84	63	0.03	0.06
P-85	50	0.03	0.06
P-86	50	0.06	0.13
P-87	50	0.02	0.03
P-88	50	0.03	0.07
P-89	80	0.51	1.16
P-90	63	0.09	0.2
P-91	63	0.06	0.15
P-92	80	0.45	1.02
P-93	63	0.02	0.06
P-94	80	0.42	0.97
P-95	63	0.08	0.18
P-96	100	0.41	0.66
P-97	63	0.59	1.35
P-98	63	0.05	0.12
P-99	63	0.13	0.29
P-100	63	0.39	0.9
P-101	150	0.31	0.71
P-102	150	0.29	0.66

P-103	150	0.01	0.02
P-104	150	0.26	0.6
P-105	80	0.45	1.03
P-106	150	0.09	0.21
P-107	100	0.58	0.61
P-108	100	0.54	0.52
P-109	100	0.01	0.02
P-110	80	0.81	0.73
P-111	80	0.92	0.27
P-112	63	0.16	0.43
P-113	63	0.26	0.35
P-114	80	0.12	0.2
P-115	63	0.29	0.67
P-116	63	0.26	0.6
P-117	40	0.51	1.16
P-118	40	0.29	0.67
P-119	40	0.14	0.33
P-120	63	0.03	0.61
P-121	63	0.05	0.67
P-122	80	0.03	0.06
P-123	100	0.22	0.28
P-124	63	0.35	0.8
P-125	63	0.29	0.65
P-126	63	0.23	0.53
P-127	63	0.08	0.05
P-128	80	0.27	0.44
P-129	80	0.26	0.33
P-130	100	0.37	0.46
P-131	80	0.36	0.41
P-132	80	0.15	0.15
P-133	80	0.39	0.42
P-134	80	0.49	0.46
P-135	50	0.41	0.43
P-136	50	0.14	0.31
P-137	63	1.02	1.17
P-138	80	0.2	0.46
P-139	63	0.66	0.67
P-140	80	0.32	0.09
P-141	80	0.4	0.4
P-142	80	0.21	0.26
P-143	63	0.25	0.46
P-144	63	0.02	0.1

P-145	80	0.21	0.4
P-146	80	0.18	0.42
P-147	80	0.12	0.28
P-148	80	0.04	0.09
P-149	40	0.42	0.97
P-150	100	0.24	0.25
P-151	100	0.24	0.53
P-152	80	0.38	0.82
P-153	80	0.21	0.48
P-154	80	0.4	0.73
P-155	80	0.27	0.49
P-156	80	0.1	0.17
P-157	80	0.05	0.09
P-158	100	0.02	0.03
P-159	100	0.01	0.03
P-160	80	0.03	0.03
P-161	80	0.05	0.1
P-162	80	0.02	0.04
P-163	63	0.05	0.12
P-164	63	0.01	0.08
P-165	80	0.12	0.15
P-166	80	0.15	0.27
P-167	63	0.18	0.32
P-168	63	0.2	0.38
P-169	63	0.24	0.46
P-170	63	0.65	1.04
P-171	63	0.32	0.59
P-172	63	0.02	0.03
P-173	80	0.17	0.31
P-174	80	0.19	0.33
P-175	80	0.11	0.17
P-176	63	0.05	0.04
P-177	63	0.04	0.03
P-178	100	0.05	0.1
P-179	63	0.17	0.31
P-180	63	0.11	0.21
P-181	80	0.11	0
P-182	75	0.1	0.52
P-183	63	0.19	0.88
P-184	50	0.35	1.49
P-185	50	0.43	1.81
P-186	50	0.08	0.12

P-187	50	0.12	0.02
P-188	50	0.18	0.12
P-189	50	0.23	0.22
P-190	100	0.02	0.04
P-191	100	0.07	0.07
P-192	63	0.2	0.25
P-193	63	0.3	0.69
P-194	63	0.27	0.62
P-195	63	0.25	0.58
P-196	63	0.21	0.48
P-197	63	0.17	0.4
P-198	50	0.24	0.57
P-199	50	0.07	0.2
P-200	50	0.25	0.58
P-201	50	0.43	0.98
P-202	50	0.4	0.91
P-203	50	0.02	0.04
P-204	50	0.25	0.57
P-205	50	0.13	0.29
P-206	50	0.02	0.04
P-207	50	0.17	0.37
P-208	50	0.33	0.73
P-209	50	0.14	0.32
P-210	50	0.03	0.07
P-211	50	0.02	0.05
P-212	50	0.09	0.21
P-213	40	0.01	0.24
P-214	40	0.04	0.31
P-215	25	0.24	1.07
P-216	40	0.12	0.27
P-217	150	0.05	0.09
P-218	150	0.04	0.08
P-219	150	0.06	0.12
P-220	150	0.07	0.13
P-221	200	0.03	0.07
P-222	200	0.03	0.05
P-223	200	0.01	0.01
P-224	150	0.02	0.05
P-225	150	0.12	0.1
P-226	63	0.4	0.91
P-227	63	0.39	0.89
P-228	63	0.1	0.23

P-229	63	0.24	0.55
P-230	50	0.24	0.55
P-231	63	0.04	0.09
P-232	40	0.26	0.6
P-233	63	0.52	1.18
P-234	63	0.47	1.06
P-235	50	0.39	0.89
P-236	63	0.2	0.47
P-237	50	0.27	0.62
P-238	50	0.08	0.19
P-239	63	1.05	1.8
P-240	50	0.06	0.14
P-241	63	0.93	1.51
P-242	63	0.76	1.14
P-243	50	0.19	0.44
P-244	100	0.12	0.27
P-245	100	0.15	0.34
P-246	100	0.25	0.57
P-247	150	0.12	0.28
P-248	150	0.11	0.26
P-249	50	0.14	0.31
P-250	80	0.09	0.21
P-251	50	0.05	0.12
P-252	80	0.04	0.09
P-253	80	0.07	0.16
P-254	80	0.08	0.17
P-255	50	0.05	0.11
P-256	50	0.11	0.26
P-257	50	0.32	0.74
P-258	50	0.24	0.55
P-259	50	0.29	0.66
P-260	50	0.2	0.47
P-261	50	0.02	0.05
P-262	50	0.03	0.06
P-263	63	0.03	0.06
P-264	50	0.55	1.26
P-265	50	0.2	0.46
P-266	50	0.1	0.23
P-267	100	1.88	1.01
P-268	50	0.09	0.21
P-269	50	0.73	1.67

Appendix-5 Reservoir capacity analysis

Time (Hrs)	consumption pattern	water consumption (m^3)	Cumulative water consumption (m^3)	water production (m^3)	Cumulative water Production (m^3)	difference (m^3)
1	0.4	169.2	169.2	423	423	253.8
2	0.4	169.2	338.4	423	846	507.6
3	0.4	169.2	507.6	423	1269	761.4
4	0.6	253.8	761.4	423	1692	930.6
5	0.7	296.1	1057.5	423	2115	1057.5
6	0.9	380.7	1438.2	423	2538	1099.8
7	1.2	507.6	1945.8	423	2961	1015.2
8	1.6	676.8	2622.6	423	3384	761.4
9	1.6	676.8	3299.4	423	3807	507.6
10	1.2	507.6	3807	423	4230	423
11	1.2	507.6	4314.6	423	4653	338.4
12	1.3	549.9	4864.5	423	5076	211.5
13	1.4	592.2	5456.7	423	5499	42.3
14	1.2	507.6	5964.3	423	5922	-42.3
15	1.1	465.3	6429.6	423	6345	-84.6
16	1.1	465.3	6894.9	423	6768	-126.9
17	1.2	507.6	7402.5	423	7191	-211.5
18	1.4	592.2	7994.7	423	7614	-380.7
19	1.1	465.3	8460	423	8037	-423
20	1.1	465.3	8925.3	423	8460	-465.3
21	1.1	465.3	9390.6	423	8883	-507.6
22	0.7	296.1	9686.7	423	9306	-380.7
23	0.6	253.8	9940.5	423	9729	-211.5
24	0.4	169.2	10109.7	423	10152	42.3
the volume of balancing supply and demand = max 6hrs + min 21hrs						1479
storage required for firefighting = 10% of balance storage						147.9
storage required for emergency supply =15% of balance storage						221.85
Total storage required						1848.75
recommended capacity						1900

Appendix-6: Article status

A. Article status

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B. Article linkage

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