PERFORMANCE ANALYSIS OF RURAL WATER SUPPLY SYSTEM: CASE STUDY OF GORDAMA AREA, ALETA WONDO WOREDA

MSc THESIS

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PERFORMANCE ANALYSIS OF RURAL WATER SUPPLY UNDER DIESEL POWERED PUMPING SYSTEM: CASE STUDY OF ALETA WONDOWOREDA

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ADVISOR'S APPROVAL SHEET

This is to certify that the thesis entitled as " Performance Analysis of Rural Water Supply : Case Study of Gordama area ,Aleta Wondo woreda, Sidama Region Ethiopia" has been approved by the Department of Water Resource and Irrigation Engineering for partial fulfillment of the degree of Master of Science with specialization in 'Water Resource Engineering and Management' and has been carried out by Habte Thomas Abraham, under my/our supervision. Therefore, I/we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

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We, the undersigned, members of the Board of Examiners of the final open defense by Habte Thomas Abraham have read and evaluated his thesis entitled “Performance Analysis of Rural Water Supply System: Case Study of Goradama area, Aleta Wondo Woreda, Sidama Region Ethiopia”, and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree of master’s science.

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DECLARATION

I declare that this thesis is my own original work and all sources of materials used for this thesis have been duly acknowledged. I solemnly declare that this thesis is not submitted to any other institution anywhere for the award of any academic degree, diploma or certificate.

Habte Thomas Abraham ...……………………......

Signature .....…………………………...........

Place: Institute of Technology, Hawassa University.

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ABSTRACT

Unplanned water distribution is the key problems of many water authorities in developing countries including Ethiopia. This research was conducted to carry out the assessment of the water distribution network of Gordama area existing water supply system which is located in Aleta Wondo Woreda of Sidama Region of Ethiopia. To examine the hydraulic performance of the water distribution network, water GEMs modeling was adopted. Accordingly, different reviewed reports, percentage of Non-Revenue for water, and discussions with water service personnel were conducted to collect and analyze water losses. The result showed that the current maximum water demand in Gordama is estimated at 5`11.06 m3/day, while small reservoirs capacity and low pump efficiency were observed in the area water distribution networks. According to simulated results; maximum pressure heads were examined 329.39m and 112.78m on conveyance and main distributions lines respectively. But minimum pressure heads were computed 2.30m and 0 m on the two types of lines. Further, the analyzed water losses result in Gordama area indicates that about 19.5% was Non-Revenue Water. However physical and apparent losses contributed small amount as compared with non-revenue loss. Apparent losses cover 8.84% of total loss and 10.66 % was physical losses. Accordingly, for the case of Gordama community non-revenue losses are more significant; Most of residences were not satisfied on the Gordama area. In general, rising in water demand, small capacities of existing infrastructures and large volume of water loss leads to intermittent water distribution in Gordama area water service. Therefore, it is significant to rehabilitate and improve the water distribution system capacities and provide more attention to water losses reduction policies and strategies as appropriate intervention measures.

KEYWORD: Water GEMs modeling, water loss, Gordama, Aleta Wondo Woreda, Sidama Region

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Lists of Abbreviations and Acronyms

ADD	Average Day Demand
AWWA	American Water Works Association
BoFED	Bureau of Finance and Economic Development
CAPL	Current Annual Volume of Physical Losses
CSA	Central Statistical Agency
CSAE	Center Study for African Economy
DCI	Ductile Cast Iron
DN	Nominal Diameter
DW	Deep Well
EC	Ethiopian Calendar
EFC	Environmental Finance Center
EGL	Energy Grade Line
EPA	Environmental protection Agency
FAO	Food and Agriculture Organization
FDRE	Federal Democratic Republic of Ethiopia
GEMS	Geo-spatial Engineering Modeling System
GI	Galvanized Iron
GIS	Geographic Information System
GPS	Global Position System
GS	Galvanized Steel
GTP	Growth and Transformation Plan
HDPE	High-Density Polyethylene
HH	House Hold
ILI	Infrastructure Leakage Index
IWA	International Water Association
KW	Kilowatt
LTU	Livestock Tropical Unit
MAAPL	Minimum Achievable Annual Physical Losses
MDD	Maximum Day Demand
MDGs	Millennium Development Goals

MoWE	Ministry of Water and Energy
MoWR	Ministry of Water Resources
NGOs	Non-Governmental Organizations
NRC	National Research Council
NRW	Non - Revenue Water
PF	Public Fountain
PHD	Peak Hour Demand
PRV	Pressure Release Valve
PVC	Poly-Vinyl Chloride
RC	Reinforced Concrete
SNNPRS	South Nations Nationalities and People's Region
TLU	Tropical Livestock Unit
TSU	Traditional Source Users
UARL	Unavoidable Annual Real Loss
WC	Water Closet
WDN	Water Distribution Network
WHO	World Health Organization
WLCC	Water Loss Control Committee
WUAM	Water Utility Asset Management
YCP	Private Yard Connection

1. INTRODUCTION

1.1 General background

Water has a profound influence on human health & wellbeing. At a very basic level, a minimum amount of water is required for consumption on a daily basis for survival. However, access it may be restricted by low coverage, poor reliability, insufficient quantity, poor quality and excessive cost relative to the ability and willingness to pay. From the global fact stated in 2019, 71% of the global population (5.3 billion people) used a safely managed drinking water service that is, one located on premises, available when needed, and free from contamination. Of the total population, 90%of the global population (6.8 billion people) used at least a basic service. A basic service is an improved drinking-water source within a round trip of 30 minutes to collect water. Again, 785 million people lack even a basic drinking water service, including 144 million people who are dependent on surface water. As entities show, globally at least 2 billion people use a drinking water source contaminated with faces. Contaminated water can easily transmit diseases such diarrhea, cholera, dysentery, typhoid, and polio resulting 48500 diarrheal deaths each year. By 2025, half of the world's population will be living in water-stressed areas. The least developed countries will be ranked, 22% of health care facilities have no water service, 21% no sanitation service, and 22% no waste management service (WHO, 2019).

Worldwide and in Ethiopia significant number of children are dying each day because of lack of safe drinking water (MoWE, 2017). In the world fresh water that has been used for household purpose is not more than 3% of the natural available. The groundwater resources potential of Ethiopia is estimated to be about 40 billion cubic meters. Groundwater has been used as the main source of water supply since the 1970s for the main cities, towns and dispersed rural communities across the country, where provision of reticulated surface-water schemes is often expensive because of initial project construction costs and poor water quality (Haile A,Mengistu B, Demile&Tamiru A, 2019). Though the country possesses a substantial amount of water resources little has been developed for drinking water supply, hydropower, agriculture and other purposes (MoWR, 2012).

Access to water and sanitation in Ethiopia is among the lowest in the world and needs greater effort and resource to access these basic rights for the society (Manaye, S., 2009). Access to safe water is considered as a basic human need and basic human right to all people. Yet this basic right remains unrealized for large majority of people in developing Countries (Manaye, S., 2009).

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

There are number of factors, which in one or another way affecting the poor accessibility of water supply and basic sanitation. Drinking Water is become contaminated with fecal matter due to inadequate protection of the source, unhygienic practices of the community at the source and household level (Berhanu.A, 2015). So that giving a great attention for improving sanitation is the key to achieve health-related Millennium Development Goals (MDGs). However, it remains one of the biggest development challenges in all developing Countries but worst in Ethiopia. For consideration of above effects, water must be schematically supplied on the bases of demand from reliable source with appropriated lifting mechanisms.

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

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

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

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

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

Analysis of urban water supply system should provide to enough water supplies for human being and livestock consumption, for industrial and other uses in terms of coverage, quantity, reliability and acceptable quality taking the existing and future realities of the Gordama area to consideration. According to Aleta-Wondo Woreda water supply service office, the most common problems in the Gordama water supply system were shortage of water, interruption of water supply, break and burst of pipe, low coverage of water. Therefore, this research work was prepared to evaluate the performance of existing water supply distribution system in the Gordama using hydraulic model WaterGEMSV8i.

1.2 Statement of the problem

In a well-designed water schemes, there should be a good consideration of reliable water source, satisfaction of current and future water demand, water taking system, storage, conveyance and distribution system. Mostly problems that occurred in developing countries are intermittent, erratic pressure is not acceptable, inequalities in service provision between the rich and the poor, high rate of water losses from the distribution systems, population growth and urbanization, growing urban water demand, Infrastructure is aging and deteriorating (Welday, B.D, 2005). A serious problem arising from intermittent supplies, which are generally ignored, is the associated high levels of contamination. This occurs in networks where there are prolonged periods of supply interruption due to system pressures that are negligible or zero .Moreover, public utilities often face financial challenges due to a combination of low tariffs, poor services, poor consumer records, and inefficient billing and collection practices (Kimey, V, 2008).

In the case of Aleta Wondo Woreda, the sources do not satisfy the demand of present and future population and the distribution system do not cover the completely commanded part of the woreda with targeted beneficiaries. In other words, the water supply coverage needs assessment. There are villages in the woreda, which are out of the reach of distribution pipes, and villages with distribution pipes but unavailability of water most of the time. In addition to insufficient water supply coverage, high water loss and water quality issues are the major challenges of Aleta Wondo water supply system. As the water lost, is non-revenue water it has also economic effects because the water utility is losing revenue. High leakage also contributes to water quality risks due to possible infiltration of contaminated especially in areas of poor sanitation.

1.3 Objective of the study

1.3.1 General objective

To assess the performance of Gordama area of water supply system, water demand and supply, water loss and leakage management practice.

1.3.2 Specific objectives

- To determine water demands and evaluate the yield of existing water sources of Gordama area , Aleta Wondo woreda
- To conduct hydraulic performance analyses of the water supply network
- To estimate different losses in the water supply network and investigate leakage management practices of the area

1.4 Research question

The research questions that will be addressed in this research are:

- Is the existing water supply satisfied current demand for Goradama Area?
- How is the performance of the water supply system of Goradama Area?
- What are the possible remedial measures needed for the water supply system of Gordama area?

1.5 Significance of the study

The findings of the study are significant for the following reasons: Assess the existing Gordama water supply system and gather information in order to conduct a needs assessment and provide a starting point for potential water supply improvement projects. Give technical support for donors, stakeholders, government and NGOs who committed in assisting to improve Gordama water supply with financial and technical support in the area of rural water supply can use the research outcomes as reference for their objective. The research findings can also initiate other researchers to further study the delivery of similar rural water supply services. Can make designers and researchers to use such water modeling software to manage data easily, design and represent the results in a short period of time Support service providers plan maintenance and treatment schedule.

1.6 Scope of the research

The objective of this research is to present the fundamental concept of hydraulics applied to Gordama water supply network, in order for municipal officials of the town to a better evaluation and decision making of water distribution and delivery systems. Therefore, the research work was limited to assess the water distribution network (from clear water well to distribution end point) of Gordama water supply system in Sidama region of Ethiopia and it mainly focus and was assess to identify the hydraulic performance and the factors for intermittent supply, water loss, and leakage management strategy of the town water distribution system. This was achieved with hydraulic modeling (Water GEMS software), water loss analysis, and by made of discussion with the town water utility personnel to gather relevant information in the subject area. But, due to lack of enough budgets, chemical reagents, resources/ logistics and distance of the study area from laboratory these research exclude the water quality analysis in the distribution system of the Gordama study area.

1.7 Structure of the thesis

Chapter One: The introduction part contain about the background of the research, statement of problem, general objective, specific objectives, research questions, and the scope and limitations of the research.

Chapter Two: It contains different reviewed literatures which were related to the objective of this research work.

Chapter Three: It includes explanation about description of the study area and detail descriptions of the methodology work.

Chapter Four: It contains the results and discussion part of research outputs.

Chapter Five: Is the conclusion and recommendation part of the research work

2. LITERATURE REVIEW

2.1 Introduction

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

WHO defines drinking water as safe if and only if there are no significant health risks during the scheme's lifespan and when it is consumed? Open field defecation, animal waste, plants, economic activities (agricultural, industrial, and commercial), and even waste from residential areas, as well as the flooding situation in the area, are all potential contaminants that pose a threat to drinking water quality. Any water source is vulnerable to contamination, particularly older water supply systems, hand-dug wells, pumped or gravity-fed systems (including treatment plants, reservoirs, pressure break tanks, pipe networks, and delivery points (Haylamichael, I., & Moges, A., 2012). Drinking water starts its journey within catchments, and is subsequently purified at treatment plants and delivered through distribution systems. Before being delivered to the distribution system, it must meet the highest quality standards.

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

Water utilities are facing the high level of water loss in their distribution networks. ‘For many utilities, reducing loss should be the first option to pursue when addressing low service coverage levels and increased demand for piped water supply. But, expanding water distribution networks without addressing water losses will only lead to a cycle of waste and inefficiency’ (Frauendorfer, R. & Liemberger, R., 2010). However, there is no simple solution to reduce water losses in the distribution system especially in the developing world, it should be involving improvements not only regar to the water system, but also required a change in attitudes (WUAM, 2013). In addition ‘understanding how leakage are currently performing and collecting relevant data, and turning it into useful information for planning and good information systems’ are essential to water loss reduction polices (Farley, M., Wyeth, G., Ghazali, Z.B., Istandar, A. & Singh, S., 2008).

However, there is no simple solution to reduce water losses in the distribution system especially in the developing world, it should be involving improvements not only regard to the water system, but also required a change in attitudes (WUAM, 2013). In addition ‘understanding how leakage are currently performing and collecting relevant data, and turning it into useful information for planning and good information systems’ are essential to water loss reduction polices (Farley, M., Wyeth, G., Ghazali, Z.B., Istandar, A. & Singh, S., 2008).

2.2 Water demand

In order to design water supply scheme for a particular section of the community, it becomes imperative to first of all to evaluate the amount of water available and the amount of water demanded by the public. In fact, the first study is to consider the demand, and the second requirement is to find sources to fulfill that demand. However, many times a compromise is sought between the two.

2.2.1 Classification in category of demands

It is very difficult to assess the quantity of water required for a town due to many variable factors which would affect water consumption. However, there are certain empirical formulae used to get fairly an accurate idea of estimation about the water consumption. This involves estimating the population of the service area, per capita water consumption and those factors that affect the water demand. Thus, intelligence and foresightedness of the designer plays an important role in ascertaining the quantity of water. Before planning a water supply scheme, it is the engineer’s duty to examine carefully the various types of water demand for a city/town. The demands have been classified in the following categories;

Domestic water demand
Institutional water demand
Commercial water demand

Industrial water demand

Livestock water demand

Fire water demand

Unaccounted for water

Domestic water demand: is the quantity of water required for various domestic usages. This includes water requirements for drinking, cooking, bathing, washing, flushing toilets, lawn sprinkling, gardening and other household sanitation purposes in private buildings. The amount of domestic water consumption per person shall vary according to the living conditions of the consumers.

For the purpose of estimation of total requirements of water, the demand is calculated on an average basis expressed in litres per capita per day (lpcd). This figure represents the average daily amount of water required per person during period of a year in normal or drought conditions. It generally is estimated by dividing the annual-average daily water consumption by the total population served.

$$\text{Average per capital demand} = \ Q/(P * 365 \dots\dots\dots\dots\dots..\dots\dots\dots\dots.\dots\dots..(2.1)$$

Where; Q: total annual volume of water in litters, and

P: population of the area

Institutional water demand: is the quantity of water required for various public utility purposes of public buildings; such as city hall, jails, schools, health facilities as well as water used for public services, including street washing, park irrigation, etc is considered public water use. This quantity will certainly vary with the nature of the city and the number of institutions present in it.

Commercial water demand: these establishments include hotels, office buildings, shopping centres, service stations, movie houses, airports, and the like. The commercial water demand depends on the type and number of commercial establishments. The commercial water demand is mostly about 10-20 %t of the total water demand. However, the water demand in various types of commercial establishments may vary greatly.

Industrial water demand: Generally, large industries develop their own water supply systems. Only small industries purchase water and, therefore, imposed water on local municipal systems. Industrial water demand may be estimated on the basis of proposed industrial zoning and of the types of industries most likely to develop within the city.

Livestock water demand: In many rural and some urban localities, livestock are important asset for households livelihood. For this reason, the demand for livestock watering shall be assessed for any water supply project during the socio-economic survey for decisions if this would be part of the public water supply system. It has to be considered if and only if alternative water source is not available with 2 km radius (MOWE, 2022)

Unaccounted for water demands: this includes the water lost in leakage due to bad plumbing or damaged meters, stolen water due to unauthorised water connections, and other losses and wastes. This unaccounted system losses and leakages can be reduced by careful maintenance and universal metering.

Fire water demand: is the quantity of water required for fighting a fire out-break and will be particularly essential for high value of district of commercial centres, stores, etc. The water demand for fire fighting must be considered in the design of water distribution systems. Although the annual volumes of water required for fire fighting are small, the demand may be exceedingly high during the periods of need. Therefore, fire flows must be considered while designing the water distribution systems. The fire hydrants are installed on water mains at an appropriate locations thus to brought fires under control with very high pressure. This quantity is normally obtained on the basis of certain empirical formula and is directly a function of population of the

2.2.2 Baseline demands

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

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

Criteria for domestic per capita demand

Contextualizing experiences of other countries with similar socio economic development to Ethiopia and to best extent considering the overarching national plans and global commitments, the unit consumptions are recommended below in Table 2.1.

Table 2.1 Indicative per capita water demand by mode of service

Mode of services	Per capita demand(l/c/d)		
	Initial(baseline)	10^{th} years from Initial	15^{th} years from initial
On spot schemes	25	25	25
Public tap(PT)	25	30	30
At premise shared (YCS)	35	0	40
At premise private(YCP)	40	45	50
House connection(HC)	50	55	60

(Source MoWE, Rural Water Supply Design Criteria and Guideline)

Criteria for livestock demand: To estimate animal (livestock) water demand, the universally accepted Global Livestock Units or Tropical Livestock Unit(LTU) conversion developed by FAO (FAO, 2003) adapted for sub-Saharan Africa can be used as reference, where 1 TLU in sub-Saharan Africa is equivalent to 1 mature cow of 250Kg with daily water consumption of 50 pcd. With review of various design manuals and tailoring to local context, also optimizing the demand to avoid over sizing, the water consumption per unit of animal type is recommended in Table 2.2

Table 2.2 Recommended livestock water demand

Animal type	Water demand (Liter/head/day)
Camels	35
Cattle (Cow, Oxen)	30
Sheep/ Goats	5
Donkey	15
Horse	30

(Source MoWE, Rural Water Supply Design Criteria and Guideline)

‘In case livestock population is not available, an allowance of not more than 5% of the domestic water demand can be considered but is not encouraged. It is also advised to bear in mind both livestock population and water demand is highly site specific’ (FAO, 2003))

Non-domestic water demand

Priority is given to address domestic water demand in the development of water supply systems. However other socioeconomic activities require water for their day to day activities. These are which are in general called non-domestic water demands. ‘Non domestic water demand includes water required for public and commercial institutions, industries, live stocks (in case circumstance obliges) and others’ (MOWE, 2022). Each of these categories has its specific water requirements which will vary during design period like that for domestic water requirement. In order to estimate total water demand, these have to be assessed, projected and added.

Public /institutional water demand

Institutional water demand refers to the water demand for public facilities or institutions having the following components:

People working/staying there etc such as schools, health institutions, farmers training centers, administrative offices, customers utilizing the institutions and the services provided by the institutions, communal latrines and other public activities like watering public green areas in rural growth poles.

Water consumed at the water supply scheme whenever applicable, unless under special circumstances, water demand for firefighting is not considered in rural after supply. For example if a multi-village scheme supplies for both rural and urban areas estimation of water demand for firefighting is required.

Commercial and industrial demands

Water used for commercial purpose like for instance market places, hotels, restaurants, slaughterhouses etc. is categorized as commercial water demand whereas water required for manufacturing process like leather, textile … or in the actual product manufactured like beverage and agro-processing is termed as industrial water demand. Water demand for commercial and industrial enterprises should be based on the situation prevailing at the time of scheme design, and the expected development plans in the area under consideration.

Table 2.3 Indicative unit consumption for non-domestic water use

Consumers	Unit	Values	Remarks
Day schools	l/head/day	5	With pit latrine
Boarding schools	l/head/day	50	With WC
Health posts/ health cares	l/head/day	10	Out patients only
Health centers	l/head/day	10-50	With no modern facilities
Health centers	l/head/day	20-30	With maternity & pit latrine
Health centers	l/head/day	40- 60	With maternity & WC
District hospitals	l/bed/day	100	
Regional hospitals	l/bed/day	200	
Administrative offices	l/head/day	5-10	
Workshop/shops	l/day/employ	5	
Mosques & Church	l/head/worshipper	5	

Public path	l/visitor	30-40	
Cinema house	l/seat/occasion	4	
Public latrine /with water connection	l/visitor/set	20	
Restaurants	l/seat/day	10	

(Source MoWE, Rural Water Supply Design Criteria and Guideline)

2.2.3 Future water supply scenario

Population estimation

Concept Water supply system is primarily developed to provide potable water for beneficiaries of a certain locality. Hence estimation of both existing and design population of the locality intended to be supplied from an envisaged water supply system is key to estimate water demand. Over estimation of water demand risks investment viability and affordability of operation & maintenance management costs resulting idle investment or wastage of limited resource where as its under estimation leads to service deficiency before the end of design period. Hence, rational and realistic population estimation are both imperative and of paramount importance to be carried out following standard steps. Estimation of population at the end of design period requires utilization of projection methods. The choice of methods to be adopted in each particular case depends on the nature &development potential of the project area, habits of the people, in or out migration, project scope for future expansion etc. Although difficult to calculate, it is worth assessing how migration due to conflict, or land degradation due to climate change might affect the population figures. For many rural areas where agricultural or pastoral land is marginal, it is possible that long term net migration away to urban centres will have an effect on the population figures (Rural Water Supply Design Criteria and Guideline). Though there are various methods for population projection, the three commonly used are highlighted as follows with their equation and graphically on figure 2.1

A) Arithmetic projection method

This projection method gives low value and is generally applicable for;

- Wider areas with scattered settlements and low potential for development or
- Already well settled &established communities and hence no more fast growth expected

B) Geometric projection method

This projection method gives higher value and is suitable for settlements with good potential for development and large scope for vast expansion.

C) Exponential projection method;

In this projection method, very high value is generated and could only be used if in addition to reasons under (B), the baseline population is presumed to be underestimated. Formulas and effect of each projection method on projected population values are explained here under (assuming annual growth rate of 8% to magnify computed factor values). (Source; Rural water supply design criteria and guideline volume –I)

Arithmetic: $P_{f=} P_o (1+ \frac{Gr}{100} * T)$ Where P_f = Future population

Geometric: $P_f = P_o (1+\frac{Gr}{100})^T$ P_o = Current population

Exponential: $P_f = P * e^{Gr/100}$ G_r = Growth rate

T=Time in years

Figure 2 1 Population projection methods

In this projection method, very high value is generated and could only be used if in addition to reasons under (B), the baseline population is presumed to be underestimated

2.3 Hydraulic performance analysis of pipe network

A safe supply of potable water is the basic necessity of human right; therefore water supply systems are the most important public utility. A huge amount of money is invested for providing or upgrading water supply facilities. The major share of capital investment in a water supply system goes to the water conveyance and water distribution network. Nearly 70% to 80% of the cost of a water supply project is used in the distribution system and long distance pipeline like Rural Piped System (RPS)/ Multi Village Schemes (MVS); therefore, using rational methods for designing a water distribution system will result in considerable savings. The water supply infrastructure varies in its complexity from a simple, rural town gravity system to a computerized, remote – controlled, multisource system of a large town/city; however, the aim and objective of all the water systems are to supply safe and reliable water for the cheapest cost. These systems are designed based on least cost and enhanced reliability considerations, possibly meet the 24/7 continuity requirement. Source water GEMS modeling manual.

2.3.1 Types of water distribution system

According to water GEMS modeling manual, the water distribution networks can classified as explained below;

Branched system: Branched system is applied in rural piped system / multi village schemes with the provision of public water points. The branch system can be applied in single or multiple water sources. In this case, it needs to apply modeling to analysis the system capacity and pressure.

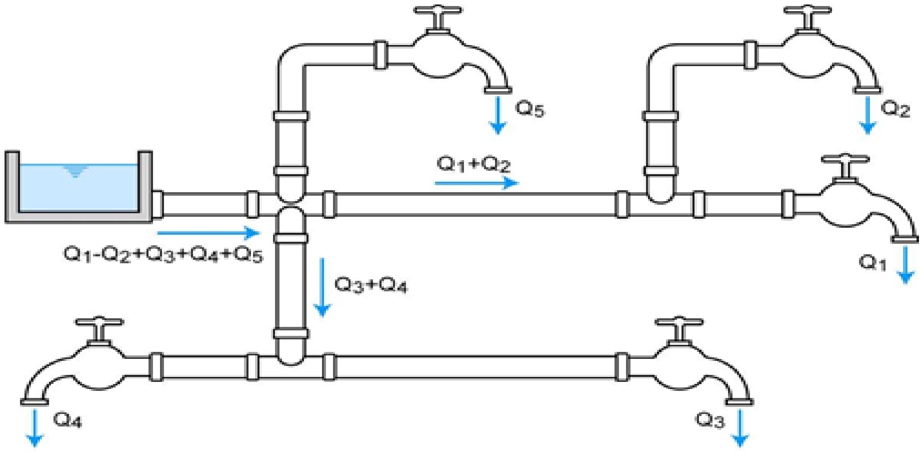


Figure 2 2: Branched networks after network failure

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

With known nodal demands, the flow in all pipes can easily be determined by applying the continuity equation, where Q_n represents the nodal discharge. N is # of nodes, J is connecting pipes. In situations where pipe diameters have to be designed, the maximum allowed hydraulic gradient must be included in the calculation input. The iterative procedure is as follows:

The inputs data are: L, k, Q, ΔH or (S), and T. The result is diameter D.

Assume the initial velocity (usually, v=1m/s),

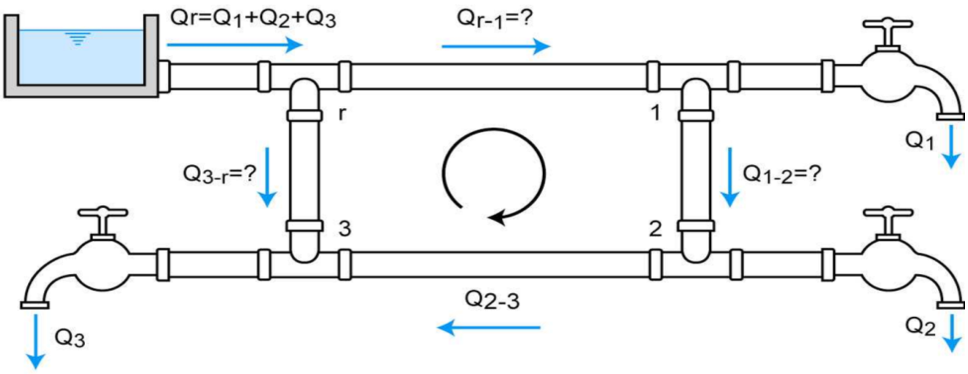
- Calculate the diameter from the velocity/flow relation. D2 = 4Q/ (vл),

- Calculate Re, based on re value, choose the appropriate friction loss equation, and determine the λ-factor. For selected Re- and k/D values, the moody diagram can also be used instead,
- Calculate the velocity.
- If the value of the assumed and determined velocity differs substantially, step 2-5 should be repeated by taking the calculated velocity as the new input.
- After sufficient accuracy has been achieved, the calculated diameter can be rounded up to a first higher commercially available size. This procedure normally requires more iteration. Finally, the pressure in the system will also be determined either by setting the minimum pressure criterion or the head available at the supply point.

Looped networks: The principles of calculation as applied to single pipes are not sufficient in case of looped networks. Instead, a system of equations is required which can be solved by numerical algorithm. This system of equations is based on the analogy with two electricity laws known in physics as Kirchhoff's Laws. Translated to water distribution network, these laws state that:

1) The sum of all ingoing and outgoing flows in each node equals zero ($\Sigma Qi=0$),
2) The sum of all head-losses along pipes that compose a complete loop equals zero ($\Sigma \Delta Hi=0$).

The first law is essentially the mass conservation law, resulting in the continuity equation that must be valid for each node in the system. From the second law, it emerges that the hydraulic grade line along one loop is also continuous, just as the flow in any node is. The number of equations that can be formulated applying this law equals the number of loops.



Yields: (Hr-H1) + (H1-H2) + (H2-H3) + (Hr-H3) = 0

Figure 2 3: Looped networks after network failure

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

2.3.2 Components of water distribution network

The scope for the development of design criteria and guideline development for water supply transmission and distribution system covers the following key components.

Transmission and distribution mains

In a water supply system, transmission of water could be either raw or treated water transmission. Except the difference in the quality of water transported, the purpose of both is to deliver water from a point which can be either source or treatment plant to a destination which could be treatment plant or reservoirs. The distinct feature of a transmission system is that there is no withdrawal in between from both raw and treated transmission lines apart from some exceptions in the latter case. Depending the relative topographical location and length of the transmission line the transportation of water through a transmission system could be gravity flow only or pump pressurized flow only or a combination of both gravity and pumping

(Source Rural Water supply design Criteria and Guideline)

Transmission mains

Transmission mains were consist of components that are convey large amounts of water over great distances, typically between major facilities within the distribution system. In most water supply system, transmission main are mainly used to transport water from treatment plant to service reservoirs/ storage tanks. Whereby, individual customers are usually not served from these mains.

Distribution mains

Distribution mains are an intermediate pipeline used to delivering water from transmission main to customers. The mains are smaller in diameter than transmission mains, and typically follow the general topology and alignment of the town streets. Different fittings such as elbows, tees, reducers, crosses and numerous other accessories are used in the main to connect pipes. While other maintenance and operational appurtenances, such as fire hydrants and valves are also connected directly to the distribution mains. Further, services also called service line were laid and transmit water from the distribution mains to end customers.

Reservoir and storage tanks

In the water distribution system, reservoir and storage tanks are mainly provided in order to meet the fluctuations of water demand and to stabilize pressure within the distribution system. Similarly, these components were reserve water for emergency requirements. Accordingly, the common reservoirs established in the water supply system are circular and/or rectangular type which builds either from concrete or steel materials. And, the recommended locations of such facilities are mainly in elevated area beyond the center of service area (NRC, 2006).

Pump stations

Pumps are used for convey energy to the water in order to boost water at higher elevations. Most pumps used in the water supply systems are centrifugal in nature, and are installed to improve the water distribution, if gravity is insufficient to supply water at an adequate pressure. So in order to control the operational condition of pumps switch-board were provided in the station (NRC, 2006); (Chambers, K., Creasey, J. & Forbes, L., 2004)

Accessory equipment

The accessory equipment in the water distribution pipelines can be classified as fittings, valves (such as; control valves, air release valves, pressure reducing valves, etc), hydrants, drainage facilities, flow meters, and etc. All these accessories have been installed at places were necessary for connecting the network, controlling and management of the system, and for maintenance purposes during failure is occurred (Bhadbhade, N.M. , 2009).

2.3.3 Hydraulic integrity

Hydraulic integrity is the ability of a distribution system to meet all user demands (domestic, commercial, institutional, industrial, firefighting, etc.) while ensuring desirable pressures, velocities and water age in the system. Pressure is the most important indicator of hydraulic integrity. System pressures are at their highest during the night, when flows and energy losses are at a minimum. Conversely, system pressures are at a minimum during peak demand periods when flow rates and energy losses are at a maximum. Design guidelines specify the minimum and maximum pressures in water distribution system to be as indicated below 10 and 70 m. (Source Hydraulic Network Modeling manual using Water GEMS Software)

2.3.3.1 Consequences of a loss of hydraulic integrity

A loss of hydraulic integrity can have severe consequences for service provision in a water distribution system, including the following:

- Inability to satisfy consumer demand due to inadequate system pressures

- Damage to pipes and linings due to negative pressures
- Contamination of the water in the system by external pollutants due to negative pressures
- Depletion of disinfectant residuals due to long retention times of water in the system
- Accumulation of sediments in pipes due to low velocities

2.3.3.2 Causes of a loss of hydraulic integrity

Numerous causes of a loss of hydraulic integrity may be identified. In most cases an interaction between loss of hydraulic, physical and water quality integrity exists. 'In most of the developing regions, the design of water distribution systems is based on the assumption of direct supply, although most of these systems are intermittent systems which result in severe supply, insufficient pressure in the distribution system (pressure losses in several areas in the network), inequitable distribution of the available water and very short duration of supply' (Hussni, Zyoud, 2003). However, the purpose of hydraulic integrity in the water distribution system is to supply water at adequate/acceptable pressure and flow. But, according to (Chambers, K., Creasey, J. & Forbes, L., 2004) ; (Tomas, M.W., Donald, V.C., Dragan, A.S., Walter, G., Stephen, B. & Edmundo, K., 2003); (Marta, G.G.& Rudolf G., 1987); (Hickey, H.E., 2008); (Dighade, R.R., kadu, M.S. & Pande, A.M., 2014) the most common factors for intermittent water supply and loss of hydraulic integrity in the distribution system are;

Low pressure

However, there is pressure loss by the action of friction at the pipe wall and its magnitude also dependent on the water demand, properties of the fluid that is passing through the pipe, the speed at which it is moving, and the internal roughness of the pipe, pipe length, gradient and diameter of the pipe. Such situations may occur where there are: properties on high ground, remote properties at the end of long lengths of pipe, demands that are greater than the design demand, pipes of inadequate capacity (too small diameter), rough pipes (e.g. corroding iron pipes or pipes with a build-up of sediment) and equipment failures such as pumps and valves. In general, the poor pressures tend to be caused by inadequate capacity in a pipe or pump, high elevations, or some combination of the two (Chambers, K., Creasey, J. & Forbes, L., 2004). Therefore, one of the most hydraulic integrity is maintaining adequate water pressure inside the pipe. Hence, the water utilities should be achieving a high degree of hydraulic integrity through a combination of proper system design, operation, and maintenance along with good monitoring.

High pressure during low demand conditions

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

Pump capacity

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

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

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

Demand increase

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

Poor infrastructures

In most of the developing countries it has been observed that pipe network is very old and which is laid many years ago. With aging problem there is considerable reduction in carrying capacity of the pipelines. Although, most of the distribution pipeline were get corroded and leakage were occur, since resulting in loss of water and pressure reduction. Hence, 'All these materials suffer from degradation over time and result in leakage in the network. It is, therefore, Preventive maintenance of distribution system assures and providing conditions for adequate flow through the pipelines. Incidentally, this will prolong the effective life of the pipeline and restore its carrying capacity. Some of the main functions in the management of preventive maintenance of pipelines are assessment, detection and prevention of loss of water from pipelines through leaks, maintaining the capacity of pipelines, cleaning of pipelines and relining' (Dighade, R.R., kadu, M.S. & Pande, A.M., 2014).

Operation and maintenance activities

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

Therefore, lack of attention to the important aspect of operation and maintenance of water supply schemes were leads to deterioration of the useful life of the distribution systems. Further, as per (Dighade, R.R., kadu, M.S. & Pande, A.M., 2014); some of the key issues contributing to the poor operation & maintenance have been identified as follows;

- Lack of funds, operation manuals and real time field information
- Inappropriate system design and poor workmanship,

- Overlapping responsibilities and inadequate training of personnel,
- Inadequate emphasis on preventive maintenance,

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

2.3.4 Basic principles of hydraulic modeling

In line with (Jalal, M.M., 2008); the main reason for modeling a system is to assist designers, managers and planners to explore the governing laws of such systems and to accurately analyze their behavior. Hence, models are employed to resolve problems in system's design and operation. 'Model-based simulation is a method for mathematically approximating the behavior of real water distribution systems. To effectively utilize the capabilities of distribution system simulation software and interpret the results produced, the modeler must understand the mathematical principles involved' (Tomas, et al., 2003).

'In networks of interconnected hydraulic elements, every element is influenced by each of its neighbors; the entire system is interrelated in such a way that the condition of one element must be consistent with the condition of all other elements. These conditions are mainly controlled by two laws' (Tomas, et al., 2003).

Law of conservation of mass

'The principle of conservation of mass dictates that the fluid mass entering any pipe will be equal to the mass leaving the pipe (since fluid is typically neither created nor destroyed in hydraulic systems). In network modeling, all outflows are lumped at the nodes or junctions.' (Tomas, M.W., Donald, V.C., Dragan, A.S., Walter, G., Stephen, B. & Edmundo, K., 2003)

$$\sum_{\mathrm{Pipe}} \mathrm{Qi} - \mathrm{U} = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.2)$$

Where Qi=inflow to node $i-{}^{th}$ pipe (m^3/s)

U=Water used at node (m^3/s)

During extended-period simulations; a term to the accumulation of water at certain nodes are considered, because water can be stored and withdrawn from storage tanks (Tomas, M.W., Donald, V.C., Dragan, A.S., Walter, G., Stephen, B. & Edmundo, K., 2003)

$$\sum_{\mathrm{Pipe}} \mathrm{Qi} - \mathrm{U} - \frac{\mathrm{S}}{\mathrm{dt}} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.3)$$

Where $\frac{S}{dt}$ = Change in storage (m^3/s)

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

Law of conservation of energy

The principle of conservation of energy states that energy can neither be created nor destroyed, but can be transformed from one form to another. Energy must be conserved.

There are numerous forms of energy- mechanical energy, potential energy, heat energy, kinetic energy, sound energy, etc. that it can be transferred to and from.

Bernoulli's equation:

Bernoulli's equation is an energy balance form one point in a hydraulic system to another. It applies to all points on the streamline and thus provides a useful relationship between pressures P, velocity V, and height above a datum, Z. Units of head are in meters, but head is just another way to express energy

$$\frac{P}{\Upsilon} + z + \frac{V^2}{2g} = H \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.4)$$

The Bernoulli equation may be visualized for liquids as vertical distances. The sum of all three terms (or total head) is the distance between the horizontal datum and the Energy Grade Line (EGL) as seen in figure 2.4.

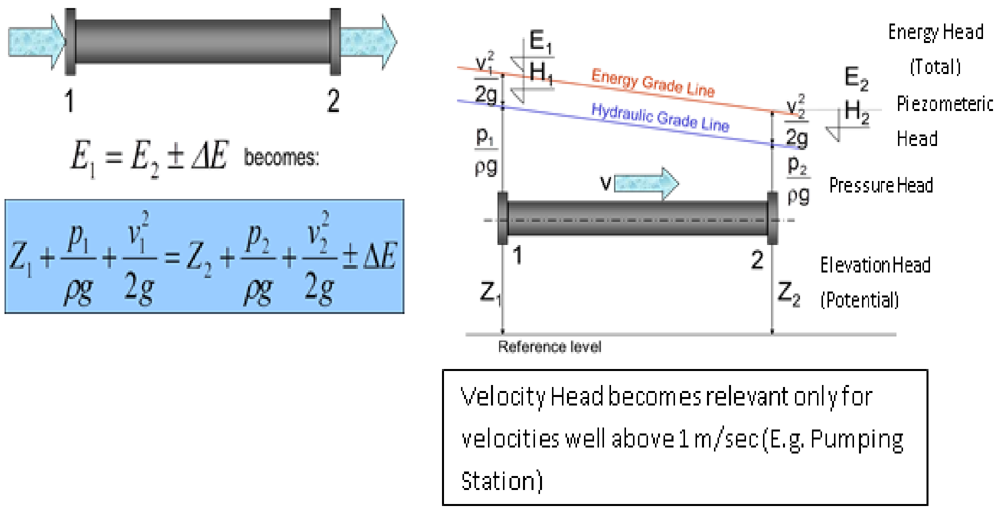


Figure 2 4: Bernoulli’s energy conservation diagram

2.3.5 Water distribution network simulation

‘The term simulation generally refers to the process of imitating the behavior of one system through the functions of another. It can be used to predict system responses to events under a wide range of conditions without disrupting the actual system. Using simulations, problems can be anticipated in proposed or existing systems, and can be evaluated before time, money, and materials are invested in a real-world project’ (Dessalegn G, Fekadu F, 2021); in water distribution networks the most basic type of model simulations are either steady-state or extended-period simulation.

Steady-state simulations: represent a particular view of point in time and are used to determine the operating behavior of a system under static conditions. It computes the hydraulic parameters such as flows, pressures, pump operating characteristics and others by assuming that demands and boundary conditions were not change with respect to time. In general, this type of analysis was used to determining the short-term effect of demand conditions on the system (Dessalegn G, Fekadu F, 2021).

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

Water GEMS: Modeling capabilities

Water Gems provides and allowing modeling practically for any distribution system aspect. Therefore, working with Water Gems used as for decision-support tool for water infrastructures and were help to assess and/or operate (Water GEMS: USER MANUAL)

The hydraulic analysis at a steady-state or an extended-period simulation
Pressure, flow and demands in the system and to see how behaves over time,
The size of pipes, pump and computer system head curves,
Tank, pump and valve behavior in the system
Leakage and water loss from the network,
Calibration the model either manually or use the Darwin Calibrator methods

And, generate fully customizable in graphs, charts and reports form.

2.3.6 Input data for assembling the model

In practice, pipe networks consist not only of pipes, but composed of vary fittings, services, storage tanks and reservoirs, meters, regulating valves, pumps, and electronic and mechanical controls. For modeling purposes, these system elements were organized into the following categories (Water GEMS: User Manual)

Table 2.4 Input parameters and primary purposes of water GEMS tools

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

(Source; Water GEMS: User manual)

2.3.7 Water demand modeling

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

2.3.7.1 Demand modeling approaches

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

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

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

2.3.7.2 Variations in water demand

The per capita demand of a particular town is the average consumption of water for a year. In practice it has been seen that this demand does not remain uniform throughout the year, but it various from season to season, even hour to hour (Venkateswara, P., 2005). Seasonal variation: Water demand varies from season to season. In dry season the water demand is maximum, because the people will use more water for bathing, cooling, lawn watering and street sprinkling. While, demand will becomes minimum in rainy/wet season because less water is used in bathing and there is no lawn watering. Therefore, ‘maximum day water demand is considered to meet water consumption changes with seasons and it used to size source, treatment plant and rising mains. Hence, maximum day demands can be obtained by multiplying the average-day demands to the peaking factor applied to the node' (Venkateswara, P., 2005)

$$Q_{max} = P_f * Q_{avg} \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots (2.5)$$

Where, Q_{max} = Maximum day demand (cfs, m^3/s)

P_f = Peaking factor between maximum day and average day demand

Q_{avg} = Average day demand (cfs, m^3/s)

Daily variation: This variation mainly depends on the general behavior of people, climatic conditions and character of city as industrial, commercial or residential. More water demand is on Sundays and holidays due to more comfortable bathing, washing etc as compared to other working days. Accordingly, 'Average daily water demand is the sum of the domestic, non-domestic and NRW which is used to estimate the maximum day & the peak hour demand' (Venkateswara, P., 2005). It expressed as economic calculations over the projects lifetime.

$$Q_{avg} = \text{Percapita} * \text{ total population of the area} \dots \dots \dots \dots (2.6)$$

Where, Q_{avg} = Average day demand (cfs, l/s)

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

$$Q_{hour} = P_f * Q_{avg} \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots (2.7)$$

Where; Q hour = Maximum day demand (cfs, l/s)

P_f = Peaking factor between maximum day and average day demand

Q_{avg} = Average day demand (cfs, m^3/s)

2.3.7.3 Demand diurnal pattern and multipliers factors

The variations in water usage for water supply systems typically follow a 24-hour cycle. However, in reality, water demand varies over time and for extended period simulation to reflect the dynamics of the real system, these demand fluctuations must be incorporated into the model and it requires both baseline demand data and information on how demands vary over time. These demands can be determined by applying a multiplication factors or a peaking factor. Multiplication/ Peaking factors from average day to maximum day tend to range from 1.2 to 3.0, and factors from average day to peak hour are typically between 3.0 and 6.0. Of course, these values are system-specific, so it must be determined based on the demand characteristics of the system at hand' (Tomas, et al., 2003). Therefore, When more than one demand type is served by a particular junction, the total demand for a junction at any given time is equal to the sum of each baseline demand times with its respective pattern multiplier, and it is used in most software packages to assign a different pattern to the different components of the composite demand as per below (Tomas, et al., 2003).

$$Q_{i,t} = \sum_{j} B_{i,j} P_{i,j,t} \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots . \ldots \ldots . \ldots \ldots \ldots . . \ldots \ldots \ldots \ldots \ldots \ldots \ldots (2.8)$$

Where $Q_{i,t}$ =Total demand at junction i at time t (cfs, m3/s)

$B_{i,j}$ = Baseline demand for demand type j at junction I (cfs, m3/s)

$P_{i,j,t}$ = Pattern multiplier for demand type j at junction i at time t

'In Ethiopian rural context, two peak periods are generally observed, one in the morning and the other one in the afternoon, closer to the evening. The same pattern but a bit relaxed can be assumed to apply for shared yard connections' (Rural water supply design criteria and guideline, 2022)

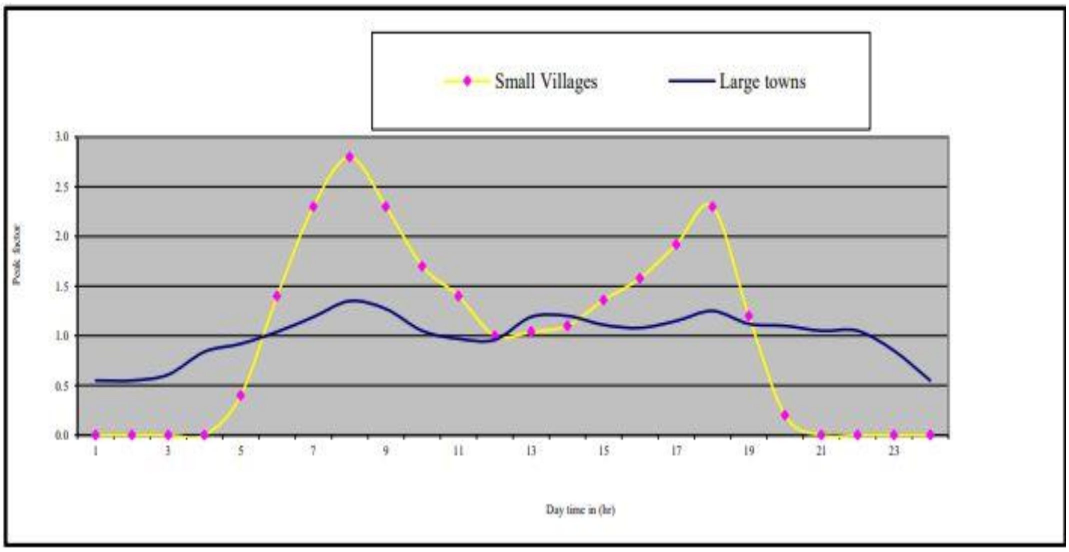


Figure 2 1: Typical hourly consumption patterns

(Source MoWE, Rural water supply design criteria and guideline volume-I)

2.3.8 Model calibration and validation

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

2.3.8.1 Pressure calibration

Collecting pressures data throughout the water distribution system used to indicate the level of service. Pressure readings are done using pressure gauge commonly taken at pump stations, storage tanks, reservoirs, fire hydrants, home faucets, air release and other types of valves. However, different factors can contribute to deviation between model simulation and actual field data. Therefore, 'calibration can be accomplished by adjusting only internal pipe roughness values or estimates of nodal demands until an agreement between observed and computed pressures and flows is obtained. The basis for this claim is that unlike pipe lengths, diameters, and tank levels, which are directly measured, pipe roughness values and nodal demands are typically estimated, and thus have room for adjustment'(Tomas, et al., 2003)

2.3.8.2 Acceptable levels of calibration

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

Pressure criteria

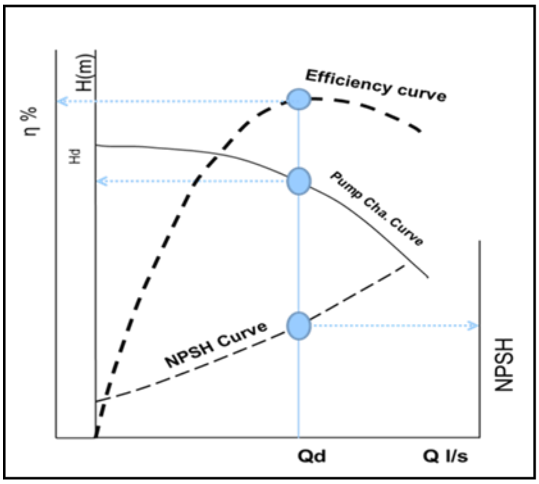
- 85% of field test measurements should be within ± 0.5 m or ± 5 % of the maximum head loss across the system, whichever is greater.
- 95% of field test measurements should be within ± 0.75 m or ± 7.5 % of the maximum head loss across the system, whichever is greater.

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

2.3.9 Pump capacity test

Pump is a device that adds energy to the system in the form of increasing hydraulic grade to water. In water distribution systems, the most frequently type of pump is the centrifugal pump. Accordingly, the performance of these centrifugal pumps is a function of flow rate, and is described by the following four parameters listed as below (Tomas, M.W., Donald, V.C., Dragan, A.S., Walter, G., Stephen, B. & Edmundo, K., 2003);

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



- ❑ Pumps are designed with MDD
- ❑ Manufacturers provide this curve
- ❑ It consists of Head against Discharge, Efficiency, NPSH and Power
- ❑ Determine the Best Efficiency Point
- ❑ System Head curve is calculated by the designer

Figure 2 2: Pump efficiency curve
(Source: Advanced water distribution modeling and management: Haestad Methods)

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

$$E_p = \frac{\text{Water power out}}{\text{Pump power in}} * 100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.9)$$

Where E_p = pump efficiency (%)

Pump power refers to the brake horsepower on the pump shaft, or the amount of power delivered to the pump from the motor. While, water power is the amount of power delivered to the water from the pump and it computed using the following relationship (Tomas, et al., 2003);

$$W_p = C_f * Q = H_p * C \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.10)$$

Where, W_P = Water power, Watts,

Q = flow rate, l/s

H_p = head added at pump, m,

C = Specific weight of water, 9810 N/m3 and

C_f = Units conversion factor, 0.001 for SI

2.4 Water loss and leakage management of water supply

2.4.1 Water loss in distribution network

Water losses occur in all water distribution networks, even new one and it is only the volume that varies. Thereby, the volume of these losses reflects the capacity of water authorities to manage their distribution networks (Dighade, R.R., kadu, M.S. & Pande, A.M., 2014). In general, 'water losses consist of real and apparent losses. And to most water utilities, the level of non-revenue Water (NRW) is a key performance indicator of efficiency. Utility managers should use the water balance to calculate each component and determine where water losses are occurring. By quantifying NRW from the water balance concept, volumes of lost water into system can be calculate and they will then prioritize and implement the required policy changes and operational practices which lead to the proper understood and take the required actions' (Farley, M., Wyeth, G., Ghazali, Z.B., Istandar, A. & Singh, S., 2008). Therefore, the water balance can guides water loss estimation in the distribution system while also indicating the level of accuracy of the non- revenue water calculation.

Table 2.5 Water balance showing NRW components; IWA water loss task force

System input volume	Authorized consumption	Billed authorized consumption	Billed metered consumption	Revenue water
			Billed unmetered consumption	
		Unbilled authorized consumption	Unbilled metered consumption	Non-revenue water
			Unbilled unmetered Consumption	
	Water losses	Apparent losses	Unauthorized consumption	
			Customer meter inaccuracies and data handling error	
		Physical/real loss	Leakage on mains (transmission and distribution)	
			Leakage and overflow from utility storage tank	
			Leakage on service connection up to the customer meter	

(Source: Farley, et al., 2008)

2.4.1.1 Non- Revenue Water (NRW)

According to the above water balance classification, Non-Revenue Water (NRW) is the total amount of water losses in the system from the water treatment plant outlet meter to the customers meter and it consists of real loss and apparent losses. Thus, it is described as the difference of total amount of water production and authorized consumption figure. Unaccounted-for-water also expressed as a percentage and, has generally evaluated as the amount of water produced minus the metered customer use divided by the amount of water produced and multiplied by 100 (EPA, 2010).

$$\text{NRW} = \frac{\text{Water produced} - \text{Metered water used}}{\text{Water produced}} * 100 \ldots\ldots\ldots..(2.11)$$

Where, NRW = Non Revenue Water (%),

2.4.1.2 Physical / real loss

‘Physical losses, sometimes called ‘real losses’, are the annual volumes lost through all types of leaks, bursts, and overflows on mains, service reservoirs and service connections up to the point of customer metering. So, utility managers must be verifying the physical loss assessment of towns or study area water distribution system’ (Farley, M., Wyeth, G., Ghazali, Z.B., Istandar, A. & Singh, S., 2008).

2.4.1.3 Leakage from transmission and distribution mains

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

2.4.1.4 Leakages from reservoirs and storage tanks

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

2.4.1.5 Leakage on service connections up to the customer’s meter

This leakage is more difficult to identify and it covers the greatest volume of physical losses. So that, utility experts can calculate the approximate volume of leakage in service connections by deducting the mains leakage and storage tank leakage from the total volume of physical losses (Farley, M., Wyeth, G., Ghazali, Z.B., Istandar, A. & Singh, S., 2008)

2.4.1.6 Commercial/ apparent loss

Commercial loss also refers to as apparent losses, and it consist of unauthorized consumption, all types of metering inaccuracies and data handling errors. It also includes water that is consumed but not paid by the users (Farley, M., Wyeth, G., Ghazali, Z.B., Istandar, A. & Singh, S., 2008).

In the developing countries, metering inaccuracies (mainly under recorded problem) and illegal users of water within the distribution system is the common problem of water losses. Whereby, they contribute large coverage to apparent losses, so the levels of these losses were one of the significant concerns in developing country water distribution systems (Dighade, R.R., kadu, M.S. & Pande, A.M., 2014). Therefore, 'Apparent losses can amount to a large volume of water than physical losses and often have a greater value, since reducing apparent losses increases revenue, whereas physical losses reduce production costs. For any profitable utility, the water tariff will be higher than the variable production cost and sometimes up to four times higher. Thus, even a small volume of apparent loss will have a large financial impact' (Farley, M., Wyeth, G., Ghazali, Z.B., Istandar, A. & Singh, S., 2008).

2.4.1.7 The infrastructure leakage index (ILI)

Performance indicator for physical loss

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

$$\text{ILI} = \frac{\text{CAPL}}{\text{UARL}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.12)$$

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

$$\text{UARL} = \left(18 * \text{L}_{\text{m}} + 0.8 * \text{N}_{\text{c}} + 25 * \text{L}_{\text{p}}\right) * \text{P} \ldots\ldots\ldots\ldots\ldots\ldots (2.13)$$

Where UARL=Unavoidable annual real loss (liters/day)

L_m = mains length (km);

N_c = number of service connections;

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

P = average pressure (m)

‘The ratio of the CAPL to UARL, or the ILI, is a measure of how well the utility implements the infrastructure management functions. Although a well-managed system can have an ILI of 1.0 (CAPL = UARL), the utility may not necessarily aim for this target, since the ILI is a purely technical performance indicator and does not take economic considerations into account’ (Farley, M., Wyeth, G., Ghazali, Z.B., Istandar, A. & Singh, S., 2008).

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

Table 2.6 Physical loss target matrix; World Bank Institute

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

(Source: Farley, et al., 2008)

- Category- A (Good): Further loss reduction may be uneconomic and careful analysis needed to identify cost-effective improvements.
- Category- B (Potential for marked improvements): Consider pressure management, better active leakage control, and better maintenance.

- Category- C (Poor): Tolerable only if water is plentiful and cheap, and even then intensify NRW reduction efforts.
- Category- D (Bad): The utility is using resources inefficiently and NRW reduction programs are imperative.

2.4.1.8 Non-Revenue Water (NRW) management and controlling methods

Non-Revenue water control program should be flexible and modified to the specific needs and characteristics of town's water supply system. Accordingly, there are three major components to an effective water loss controlling program (EPA, 2010):

Water audit; is an assessment of the distribution, metering, and accounting operations of the water utility and uses accounting principles to determine how much water is being lost and where. This method used to compare and evaluate with consideration of all issues and concerns the public water system faces. Typical steps in water audit include;

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

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

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

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

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

2.4.1.9 Challenges of non-revenue water

Addressing NRW is the responsibility of managers across the water utility, including finance and administration, production, distribution, customer service, and other departments. But, most of developing countries have the problem of infrastructure and establishing operational procedures to begin tackling NRW. Hence, water utilities face greater challenges including (Farley, M., Wyeth, G., Ghazali, Z.B., Istandar, A. & Singh, S., 2008):

- Outdated infrastructure
- Poor operations and maintenance policy, including ineffective record-keeping systems
- Inadequate technical skills and technology
- Greater financial constraints, including unsuitable tariff structure and/or revenue collection policy
- Political, cultural, and social influences
- A higher incidence of commercial losses, particularly illegal connections
- Withdrawing water supply and
- Environmental pollution

2.4.1.10 Causes of water loss

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

2.4.2 Leakage management in water distribution network

2.4.2.1 General

All water supply networks are made of assets, these are the physical components of the system and it includes; pipe, reservoirs, pumps, valves, hydrants, and other components that make up the system (EFC, 2006). Good asset management involves a combination of technical, financial and engineering practices while achieving the least cost and risk over the life cycle, and meeting service standards for customers (WUAM, 2013). In general, urban water distribution networks are one the most valuable parts of the public infrastructure. So that water utilities should be entrusted with the responsibility of managing and expanding them for current and future generations (Alegre, H. & Coelho, S.T., 2013).

2.4.2.2 Infrastructure asset management

'Infrastructure asset management is one of a key topic towards compliance with the performance requirements in water supply and distribution networks. Systems that fully adopt infrastructure asset management principal can achieve many of benefits' (Alegre, H. & Coelho, S.T., 2013). Therefore, the water organizations need to put plans, policies, and strategies for sustainable management of the systems, and they should be responded to the need for;

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

2.4.2.3 Leakage controlling strategy

Many of developing country water utilities face the challenge of controlling their unacceptably high levels of non-revenue water. Accordingly, large volume of this total water loss lead to increasing the cost of delivered water, and money for investment and productivity (WUAM, 2013). Therefore, to reduce these losses the water utility should be considered;

- Identifying the life span of assets,

- Setting maintenance plan and strategy in order to reduce physical failures,
- Financing for the timely replacement and maintenance of existing assets,
- Expand distribution networks to manage with demand growth, and
- Raise their service levels 24 hours a day and 7 days a week, to meet the required outlook

In general, 'to minimize leakage, customer service-level targets are set through consultation and maintenance strategy. Further, the targets are reviewed occasionally or imposed through regulation' (WUAM, 2013)

3 MATERIALS AND METHODS

3.1 Description of study area

3.1.1 Geographical location

Gordama is multi-village area links two kebeles of Altea Wondo Woreda of Sidama regional state. It is geographically located between $6^{0}15'$ and 6^{0} 45´N latitude, and between $38^{0}15'$ and 38^{0} 45´E longitudes. Aleta Wondo Woreda borders with Dale Woreda in the north, Darra Woreda in the south, Bursa Woreda in the west, Chukko Woreda in the east, and Hula Woreda in the south west.

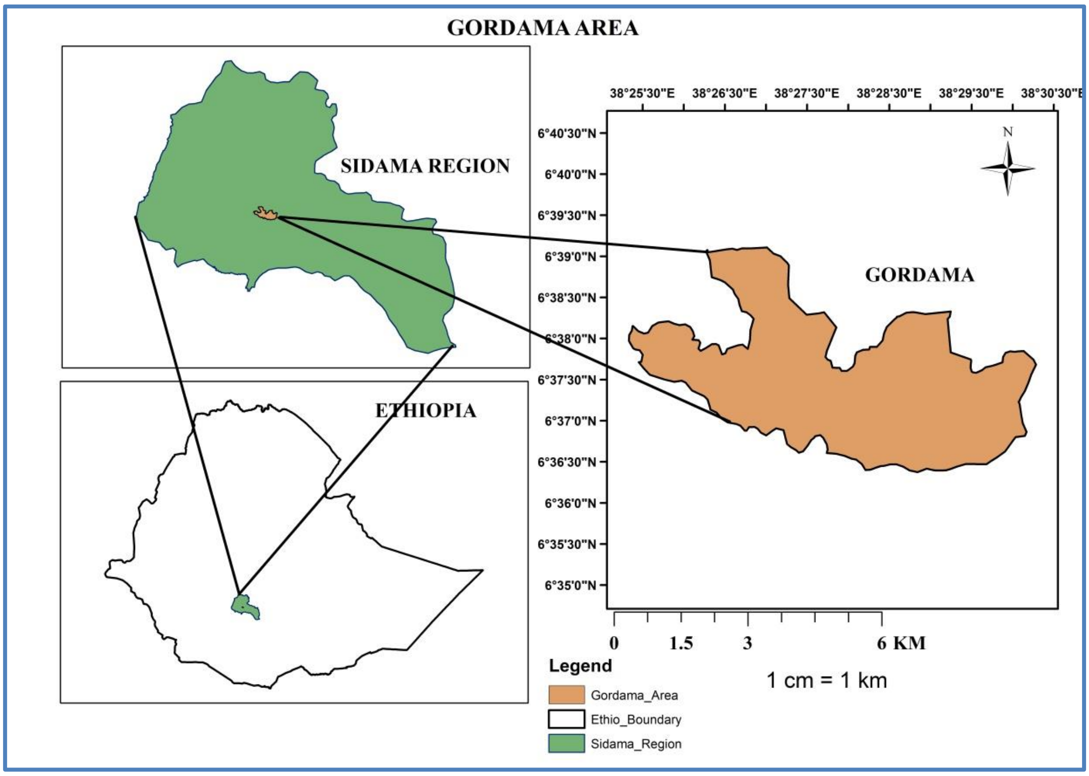


Figure 3.1: Map of study area

3.1.2 Climate

Aleta wondo which is part of the newly founded Sidama Region is located in South Central Plateau at the approximate distance of 340 km for Capital Addis Ababa. Aleta Wondo is characterized by highland weather condition with elevation rages from 1700-2500masl. The pattern of rainfall distribution is bimodal. The short rainy season lasts from mid-November to February whereas the long rainy season is during summer and it extends up to October. The mean annual rainfall ranges between 1200-1600mm.the temperature ranges is 18-21℃. The soil type is majorly of nito soil which is favorable for coffee production. Source (Kerchanshe, 2023)

3.1.3 Socio-economic activity of the Gordama area

Socio economic background

Aleta Wondo Woreda is categorized under the coffee livelihood belt. According to the 2001 SNNPRS livelihood profile Zone described as agro-ecological zone. It falls between 'Dega' (high land) and 'kola' (Tropical)) with the altitude ranging between 1001 and 2,500 above sea level. They produce coffee; supply it to the central Ethiopian market. The main food crops produced are potatoes, maize, sweet potatoes, millet, enset, teff, beans, barely, wheat etc. Cattle, goat, sheep, donkey, horse and mule are the main livestock people used for food and transportation purposes that are reared in the study area. The Woreda has good market option due to its proximity to Dilla, Chukko and Yirgalem. The major economic activities in Aleta Wondo Woreda are trading, mixed access to market (both animals rearing and cultivation of plants), small scale industries and different service sectors. With regard to trading usually cereals, fruits like mango, avocado, papaya, banana, vegetables and mostly coffee are the commodities. The marketing activities in most cases are retailer and whole sales. Most of the residents also live on incomes of food and drinks in their houses. But for some of the inhabitants the economic activities are subsistence agriculture they carry out around Gordama. On the other hand, there are various service sectors which are providing different services such as health, education, transport, hotels, restaurants and shops. Concerning small scale industries, mills, metal and wood works are undertaken in the Gordama etc (Abera, 2013).

Education and health condition

School facilities: According to Sidama Region's Educational office, there are 40 government schools. Of which, three elementary schools are found at Gordama water supply area.

Health facilities: According FY 2023 report of Sidama Region Health Office there are 3 health facilities at Habeja and Gordama kebeles .Two of three facilities are health centers and whereas one is health post. (Source Sidama Region Education Office)

3.1.4 Existing water supply system description

Potential source water; Groundwater is an important source of water supply at Aleta Wondo Woreda. It is s also used to irrigation, industries, municipalities, and rural schemes through the woreda. Ground water occurs in many types of geologic formations known as aquifers. An aquifer is a formation that contains sufficient quantities of saturated permeable material to yield significant quantities of water. Groundwater system includes wells, springs, and infiltration galleries.

Water lifting mechanisms; in the study area, the main water tools gravitational system, hand pump and submersible pumps for springs, shallow well and deep well respectively. Based on spring yield, water is delivered to beneficiaries on spot or conveying to center of services with its own gravitational head in addressing additional water points by connecting pipelines. Hand dug wells were widely adopted for water table available at the depth of ranging from 0 to 7m with a mechanism hand pumps whereas water is discharged from Shallow and deep wells by mechanism of submersible pumps and stored on reservoir before distribution.

Table 3.1 Aleta Wondo Woreda 2015 EC water institutions

S No	Water institution type	Before 2012 E.C	2016 E.C	Total
1	Hand-dug well	38	14	52
2	Shallow well	78	16	94
3	Bore hole	5	0	5
4	Spring source	267	11	278
5	Gravity spring	9	0	9
6	Water points	33	9	42
Total		430	50	480

(Source Aleta Wondo Woreda Water Energy Office)

3.1.5 Components of water supply system of Gordama area

The source water was collected through the borehole of total depth 280m by 2 ½’’galvanized iron in pipe. From the deep-well the raw water is directly pumped to 200m^3 capacity service reservoirs. From the service reservoir after disinfection with chlorine, the water is released into the distribution network. In general the system was connected to five public water points in consideration of future extension

3.1.6 Mode of services

According to the Woreda water service office reports; there are two major modes of services for domestic and institutional water consumers of Gordama community. These are; Public fountains (PF) for and private connection (YCP) for institutions. But, those populations not served from any of these modes of services are categorized as traditional source users (TSU).

3.1.7 Population distribution by mode of services

The percentage of population served by each mode of service is varying with time. This variationis caused because of the changes in living standards, improvement of the service level, changes in building standards and capacity of the water supply service to expand. Base population of According to the CSA 2022 projection table 3.4 below;

Table 3.2 Base population of Aleta Wondo and Town

Woreda/Town	Male	Female	Total
Aleta Wondo Woreda	69943	69642	139585
Aleta Wondo Town	28667	27978	56645

(Source CSA Population Projection 2022, Bofed, Humanitarian Exchange Data)

The overall domestic water supply coverage of Gordama area was indicating 70%. Of this only about 50.0% of the population were house tap connection users. The community of Gordama area was served their water need from both on-spot springs and shared Public fountains found in different villages of the study area covers 40% and 15% respectively. From the total, 25% of the population were obtaining water from their temporary hand dug wells, rope well and roof catchments. The remained 20% of the population were collecting water for their day to day activities from the unprotected surface water (river and streams), and rain water harvesting technique which come during the rainy seasons.

3.2 Materials

3.2.1 Source of data

The source of data was involved both primary and secondary data. For the study, the primary data were obtained from pressure reading, elevation surveying and by made of discussion with water utility staff members to obtain additional relevant information on the subject matter. While, secondary data were collected from different literature reviews, design report, the Gordama water supply service office existing documents and annual reported papers.

3.2.2 Equipment used

GPS instrument was used to collect the required elevation data during pressure reading. Pressure readings were done using pressure gauge which is commonly taken in the selected points of distribution system. 5 liter capacity Jeri can for measuring spring yield

3.2.3 Hydraulic model: WaterGEMs

WaterGEMS is hydraulic modeling software which is used for analysis and design of water distribution network with advanced interoperability, geospatial model building, optimization, and asset management tools. The result obtained was verified by measuring pressures at all junctions whether the flows with their velocities at all pipes are feasible enough not to provide adequate water to the network of the study areas.

3.2.4 Additional software: ArcGIS

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

3.3 Methods

3.3.1 Preliminary data collection

Data collection is the most significant part in research work. In order to accomplish this work, the data were gathered with regard to the necessary input parameters of model simulation, water losses and leakage management trend in the system. The data collection techniques were done byconducting a field visit to Gordama 01 on May 20, 2023. Data were obtained from design report of existing Gordama water supply system, woreda water service office, and field observation. The summarized collected data were presented as below;

3.3.1.1 Population of schools and health facilities:

In Gordama water supply area, there is 4496 total number of students. The figures represent from the total number of students, 1413 from Gordama primary school, 856 from Habeja primary school, 1127 from Habeja-Gordama primary school and 1100 from Habeja high school.

Table 3.3 Gordama area elementary school and student population 2023

Name of school	Number of Student								
	Grade 1	Grade 2	Grade 3	Grade 4	Grade 5	Grade 6	Grade 7	Grade 8	Total
Gordama	177	267	233	188	157	163	125	103	1413
Habeja	170	106	136	98	88	82	90	86	856
Habeja and Gordama	210	210	155	165	100	90	92	105	1127
Total number of students									3396

(Source Sidama Region Education Office)

Table 3.4 Gordama area high school and student population 2023

Name of shool	Number of student				
	Grade 9	Grade 10	Grade 11	Grade 12	Total
Gordama	303	266	329	202	1100
Total number of students					1100

(Source Sidama Region Education Office)

In addition to school facilities, there are three health facilities found at Gordama study area (Gordama 01 health center, Habaja health center and Gordama health post)

3.3.1.2 Assessment of yields of springs

Discharges of functional springs were measured and the average yield for projected years was conducted. Initially the spring's discharges were collected from Aleta Wondo Woreda Water Energy Office (2019).

3.3.1.3 Water supply system of Gordama area

Deep well and pumping station: In the clear water pumping station there was one summered centrifugal type of pump with design capacity of 9 l/s discharge and 350 m head. These pumps were sucked water from the clear water reservoir which is adjacent to the pump station and pumping water to the distribution line and service reservoir at the same time. The pump has a 50 kW of brake horsepower on each pump shaft (pump power). According to Gordama water service office, the pump currently operating in the system were installed before seven years ago and performed without replacing by the new one.

Rising main and distribution pipeline network: The transmission and distribution main line consists of branching system with a total sum length of 20,920 m, and supplying water through public fountain and yard connections by pumping and gravity means. The rising main from the well to well head is 120 m with a diameter 80mm connecting a length of 2015 transmission pressure line to service reservoir. 200 m3 service reservoirs transmit water to main distribution and distribution line of the total length 18.968m.

As observed from the drawn distribution layout; there was one flushing device (wash out valve), one air release, and one pressure reducing device was installed in transmission line at chain age- to connect a low pressure area of the study area. Currently, the PRV was functional. As per information obtained from the study area water service office; the existing distribution pipe line was GI and HDPE pipe type with DN 80 mm to DN 40 mm. The alignment was built on the base of future expansion and connection of interior public water points while; detail description was presented as appendix-N and O,

Reservoirs: There is one circular RC type reservoirs with the capacity of (200 m^3) and internal diameter of 8.0m were used in the area water distribution system, which used as clear water tank (point of disinfection) and service reservoir. Water is pumped simultaneously into the distribution network and service reservoir, thus the part of the study area that is located above the treatment plantis served from the service reservoir and the pumping line. But, the part of the Gordama area that is locatedbelow the treatment plant site is served from the clear water reservoir found at treatment plant.

Power supply units: The associated type of power supply system is diesel fuel for this water supply system. The water utility was served power of 60KW generator which is planted and used ruled by water committee with cost contribution from the community. The water distribution system was operated for 24 hours of its design period.

Elevation data: Setting elevation is one of the significant requirements to simulate the hydraulic characteristics of water in distribution system. Most of elevation data was obtained from the study area water service office which was prepared as the design report of Gordama water supply system (existing document). But, elevation data for expansion area in the study area were served in the field using surveying instrument, global position system (GPS). In appendix-G, all the assigned node elevation data were listed with coordination system.

Table 3.5 Elevation data for clear water pumping and service reservoir site

Description	Coordinates		
	X	Y	Elevation
Deep well	441,511.00	732,719.00	2,079.00
Water pump	441,555.82	732,750.88	1,959.00
Existing service reservoir	443279.00	733605.00	2233.00

(Source Aleta Wondo Woreda Water Energy Office Scheme Report)

3.3.1.4 Roughness coefficients for pipeline

WaterGEMs has an engineering library where different friction factors for different pipe 34 materials are stored. In the Properties Editor for pipes one can select the pipe material. The pipe line had the information regarding the pipe material. By default, WaterGEMs considers that the pipeline is new ductile iron pipe. Generally, pipe made of materials such as steel, PVC and asbestos cement do not tend have as much deposition or corrosion as cast iron pipes (North American Pipe Corporation, 2009; Niquette 1999). This is summarized in table 3.6 and appendix D. Attack on the pipe is defined as the corrosion of the pipe and greater the

‘C’ factor greater is the smoothness and the carrying capacity of the pipe (Haestad Methods, 2003). This table was used to assign the respective friction factors.

Table 3.6: Hazen-Williams roughness coefficients C value

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

(Source: Haestad Methods 2003)

3.3.2 Computation of Gordama water supply units

3.3.2.1 Base year population of Gordama area

Population density of base year was calculated from dividing population of Aleta wondo to shape area of respective Woreda. Population density was obtained by dividing population of Aleta Wondo Woreda of year 2022) by area of Aleta Wondo Woreda

$$\text{Population of Gordama area} = A * P_d \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.1)$$

Where P_d is population density

A is area of study

3.3.2.2 Population projection

According to Humanitarian exchange date or BoFED 2022, Population of Aleta Wondo Woreda was 69,943. Population growth rate for rural area was 2.9.

Exponential projection method was used to determine population of design period. ‘This projection method gives higher value and is suitable for settlements with good potential for development and large scope for vast expansion;

$$P_n = P_o(1 + r)^n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.2)$$

P_o= Base population figure

r = Growth rate and,

n = Number of year

Table 3.7 Rural population growth rate

Year	2021	2022	2027	2032	2037
Growth rate (%)	2.9	2.9	2.9	2.9	2.9

Source (CSA 2007)

3.3.2.3 Base water demand

Domestic demand

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

Step one: Assigning the total population of the Gordama area

Population is the important data to assess water demand in the distribution network. Facts show that there are different population forecasting methods which are used for estimating the current or future population of a given study area, but the results of the methods are vary from one to the otherdue to considering parameters of each method.

Table 3.8 Summarized Gordama area population

Year 2022	Population		
	Male	Female	Total
	69,943	69,642	139,585

(Source Humanitarian Data Exchange)

Step Two: Determination of Gordama study area

Study area of the Gordama area was determined using shape files which provided from the respective woreda's water office by using ArcGIS software. This area determined was used for population density calculations as equation 3.1

Step three: Identification of area and representing population of each node

For this study, Gordama area topographic map was obtained and bought from Ethiopian Mapping Authority, with the scale of 1:50,000 and twenty meter contour interval. In ArcGIS this topographic map was displayed and the study area distribution network map which was drawn in WaterGEMs was exported in to ArcGIS shape file and overlapped it in the topographic map ofthe study area as shown as appendix-A. The Thiessen Polygon creator was used to quickly create polygon layers for use with the LoadBuilder demand allocation module. This created polygon layers was used as service area layers for the proportional distribution by area method of each node. The calculated population at every node was assigned in the network by considering the actual condition of the residents in the Gordama area. Population of Aleta Wondo Woreda ;

$$P_i = P_d * Na_i \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.3)$$

Where P_i is Population node i

Na_i; Node area of node i

Total Area of Aleta wondo woreda is computed using ArcGIS Application is 197.22km2.

Step four: Projecting population of each node

To predict the population of a Gordama area, it is necessary knowing factors affecting the population distribution, size and growth rate. In Ethiopia, the major factors that influences on the changes in population figure are births, death and migration. All these factors are influenced by family planning practice, war, natural disasters, development of the Gordama and the socio-economic activities in and around the study area. Therefore, for this study; based on the historical figures, assumptions considered (available of data) and to be precise, the CSA population estimation method was selected from the different population projection methods. Finally, the population figure of the study area was assessed in (2022 G.C).

Step five: Assigning average day water demand of Gordama area

The water demand for actual household activity is known as domestic water demand. The demand assignment was based on economic, social and climatic factors. Our demand calculation was based on rural water supply design criteria and guideline of 30 l/c/d (MoWE, 2022). The current (2023) and future (2031) average number of person in Gordama area was obtained by projecting population from the report of humanitarian exchange data of year 2022 of Aleta Wondo Woreda. Gordama Water Supply system had been designed at 2016 and it was considered to give service about 15 years. The total number of people in the Gordama area was identified by dividing the total population to the average number of person in the area, and it was estimated at 12195. Therefore, in the opened Microsoft Excel sheet, all the 35 nodal junctions in the system and the number of houses assigned for each node were entered respectively. Then, in the third column the number of assigned houses in each node was converted in to the number of people, by multiplying the average number of person in each house of the area.

$$WD_{\mathrm{i}} = \mathrm{P}_{\mathrm{i}} * \text{Per capita} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.4)$$

Where WD_i =is water demand at junction i

P_i is population at junction i

For assessing the average water demand of the Gordama area, deterministic water demand estimation method was used. Hence, the per capital water consumption of the area was calculated using theannual water consumption recorded data and projected total population figure during year 2022. Therefore using equation below it was assessed.

Summary of water demand of 2031 of Gordama Water Supply Area

Water demand of all nodes was calculated as;

$$\mathrm{WD} = \sum_{\mathrm{i}}^{\mathrm{n}} \text{Poplation of Node} * \frac{30\mathrm{L}}{\mathrm{day}} / \text{Capital} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.4)$$

Where; WD is Water Demand, I = 1, n=31,

Step six: Assigning base water demand in each supply node

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

Base water demand for a supply node population served by that node Total population of the area .In addition to Domestic water demand there was institutional water demands related to Gordama and Habeja Health centers, Gordama Health post and Gordama School

In appendix-D, all the assigned population number, Nodal area and base water demand of each supply node were listed.

Institutional water demand

To assign base water demand for schools, population projection was done for projected period. Hence this population was assigned to respective node created in 3.6.3.1. Base Institutional water demand for schools as computed by multiplying by 5% which is standard water demand (MoWE)

Demand multiplier factors

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

Therefore, When more than one demand type is served by a particular junction, the total demand for a junction at any given time is equal to the sum of each baseline demand times with its respective pattern multiplier, and it is used in most software packages to assign a different pattern to the different components of the composite demand as per below (Tomas, M.W., Donald, V.C., Dragan, A.S., Walter, G., Stephen, B. & Edmundo, K., 2003).

$$Q_{i,t} = \sum_{j} B_{i,j} P_{i,j,t} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (3.5)$$

Where $Q_{i,t}$ =Total demand at junction i at time t (cfs, l/s)

$B_{i,j}$ = Baseline demand for demand type j at junction I (cfs, l/s)

$P_{i,j,t}$ = Pattern multiplier for demand type j at junction i at time t

Table 3.9 Demand conditions with recommended factor values

Demand parameters	Demand factors
Minimum Day Demand	0 to 0.3 of the average day demand
Average Day Demand(ADD)	1.0
Maximum Day Demand(MDD)	1.1 to 1.5 of ADD
Peak Hours Demand(PHD)	1.5 to 2.0 of MDD(>15,000population)
	2.0 to 2.5 of MDD(<15,000 population)

(Source: FDRE, MoWE)

3.3.3 Network simulation

To build and simulate the hydraulic model, Bentley waterGEM V8i connected with ArcGIS system was used. WaterGEMS runs from inside ArcGIS database. New database was created from ArcGIS by use reprocessing to migrate to the new geo-database format. Users can easily save an existing hydraulic model as a geo-database. The water distribution network map was obtained from the study area water service office, which was prepared as blue print drawn map report of distribution network and it was drawn in waterGEMs drawing pane with physical observation. The network simulation was taken extended periods by consideration of hourly demand variation pattern over 24 hour flow duration analysis work.

For this study, the network operational set-up was done by system international; SI unit andthe project liquid were taken water at 20°c. The other model input were taken and carried outas mentioned below;

- Coordinates……………………X-Y
- Setting……………………............pressure

- Tank level………………………elevation
- Drawing Scale………………….Scaled
- Annotation Multiplier…………..adjusted for report visibility

3.3.3.1 Model calibration and validation

The computed parameters of a model and actual field observation are not always has the same value. Therefore, before discussion about the simulated model results, the entire model data quality must be analyzed by calibration and validation technique. Calibration is a process of adjusting the model input data until its results become closely approximate to the measured field data. Whereby, it used to obtain approach, realistic and acceptable results. Therefore, in this study the model data quality analysis was done by comparing and calibrating the computed pressure data with the observed one.

The method of pressure readings was done during July 21, 2023 using pressure gauge commonlytaken both at high and low pressure zone of the selected points in distribution network; such as clear water pump stations, service reservoir, public fountains and different end user taps (like; customers, institution and commercial tap points). These observed pressure data was taken a totalof twenty (20) samples for both peak demand and low demand (night flow) time analysis. All sampling points were selected after the computed model was simulated and knowing the pressure variation area (pressure zone) in the study area (Gordama) water distribution network. The point's data were used for calibration.

Calibration process was performed by adjusting sensitive parameters related with flow; like pipe roughness coefficient and water demand until it was become within the acceptable limit of 85% of field test measurements (it should be within ±0.5m or ±0.5% of the maximum head loss across the system, whichever is greater) and then finally it was validated manually using the correlation coefficient (R^2) method using Microsoft Excel sheet.

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

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

3.3.3.2 Pump efficiency description

The pump efficiency which operates currently in the system was calculated manually after validating the model. For each computing time (a total of 24 hour) value of the water power produced by the pump and which converted in to electric power form (watt) were collected from the model outputs presented in (Appendix-I). Therefore, to calculate these pump efficiency; the maximum value of 28.07 KW water power was obtained from the computed model result and used as the value of water power delivered from the pump. But the value of pump power (brake horsepower of the pump) in the system was obtained from the pump shaft during field observation (July 21, 2023). And based on the observed pump data, the brake horsepower on the pump shaft is 50 kW. Therefore, using these data and equation below the current pump efficiency for Gordama area water supply system was calculated.

$$E_p = \frac{\text{Water Power Out}}{\text{Pump Power In}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.7)$$

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

3.3.4 Water loss assessment and leakage management practice of Gordama

3.3.4.1 Water loss assessment

Unlike the water consumption, the water production of Gordama area was only recorded at the Aleta Wondo water office level. Hence, the water loss analysis in Gordama was assessed at the village level based on the percentage of Non-Revenue Water which obtained from the total production and actual consumption figure.

As per data obtained from the Woreda's Water Office Water service office, the last six year (June, 2018 to June, 2023) water production and consumption (billed water volume) in the system was identified.

Using this data and equation below; the total non-revenue water (NRW) in the system was calculated for each recorded year;

$$\text{Percentage of water loss } (\%) = \frac{\text{unbilled production}}{\text{system production}} * 100 \ldots\ldots (3.8)$$

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

$$\text{UARL (liters per day)} = \left(18 * L_m + 0.8 * N_c + 25 * L_p\right) * P \ldots\ldots\ldots\ldots (3.9)$$

Where, L_m = mains length (km)

N_c = number of service connections;

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

P = average pressure (m)

As per data collected from town water service office, and during the year 2023 the area water system covers;

3.3.4.2 Assessment of leakage management practice

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

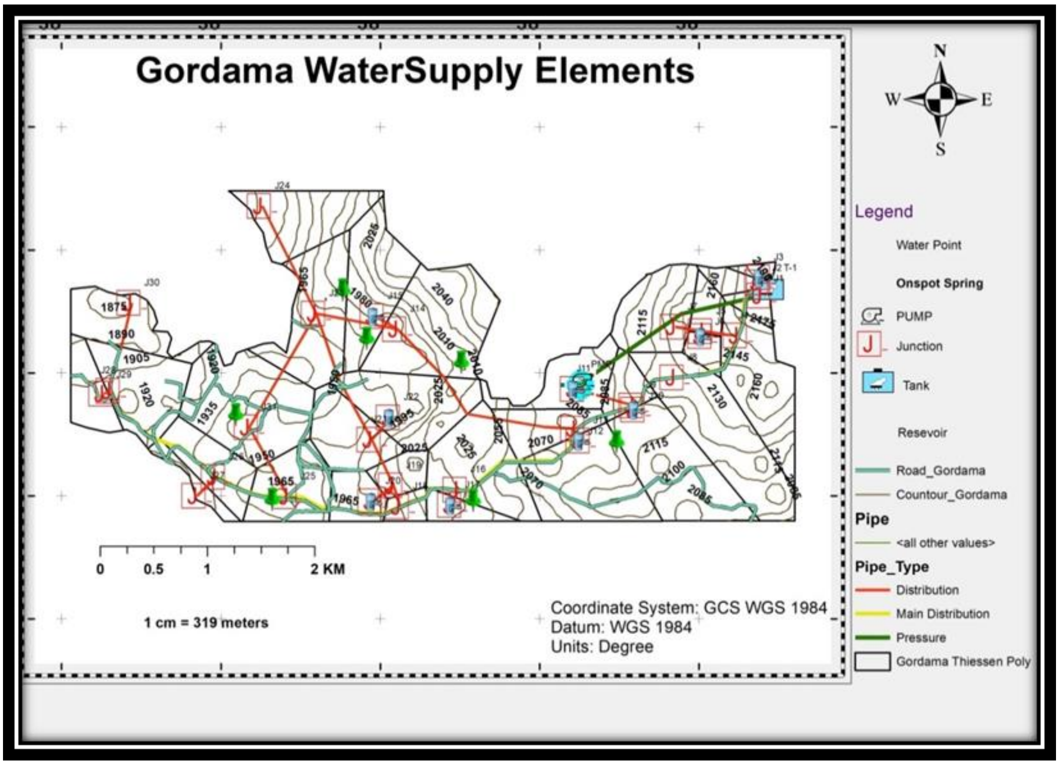


Figure 3.6: Gordama area water supply elements

4. RESULT AND DISCUSSION

4.1 Gordama area water demands and yield assessment of existing sources

4.1.1 Water demands

The result reveals water demands in gordama area were domestic, livestock, and institutional demands. Average demand of year 2023 was (2.85 l/s). The estimated water demand of the Gordama area were used for assessing and sizing system components such as pumping station, reservoirs, and transmission and distribution pipe line.

The water demand of a particular area is proportionally related with the population to be served.

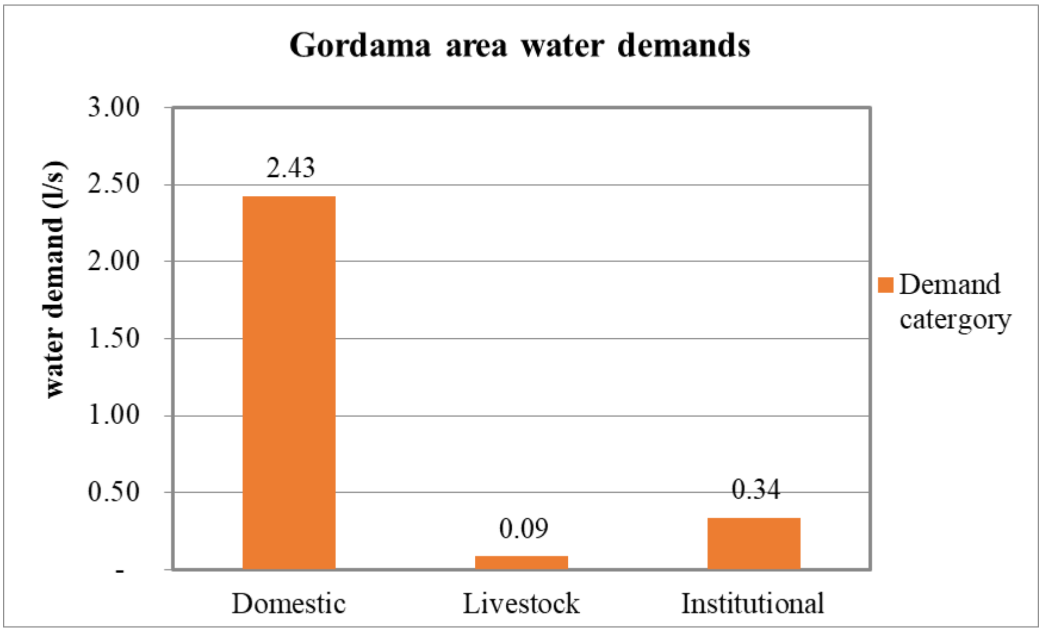


Figure 4.1: Summary of current year water demands of Gordama area

4.1.1.1 Water supply scenario of current and projected years

Base Year Population

Population of Aleta Wondo Woreda of year 2022 was 139,585. From Shape file, area of aleta Wondo Woreda was computed as 197.22 square kilometers. Population density of the woreda was determined as 707.76 people per km square.

Population of Gordama water area Method was calculated by multiplying by density as described on the table 4.1

Table 4.1 Base year population calculated by Thiessen's Polygon

Year	Population of Aleta Wondo Woreda			Area of Aleta Wondo	Average population density	Study area of Gordama	Population of Gordama
	Male	Female	Total	(km2)	P/km2)	(km2)	(P)
2022	69943	69642	139585	197.22	707.7629	13.22	9428

Projection population

Domestic Population Projection: Projected population based, exponential projected method population of Gordama area was computed 9428,9702,10,877 and 12195 for the years 2022, 2023, 2027 and 2031 respectively.

Table 4.2 Population of projected Gordama Area

Description	Growth Rate (%)	Year	Population
Rural Population Growth Rate	2.9	2022 (Base Year)	9,428
	2.9	2023	9,702
	2.9	2027	10,877
	2.9	2031	12,195

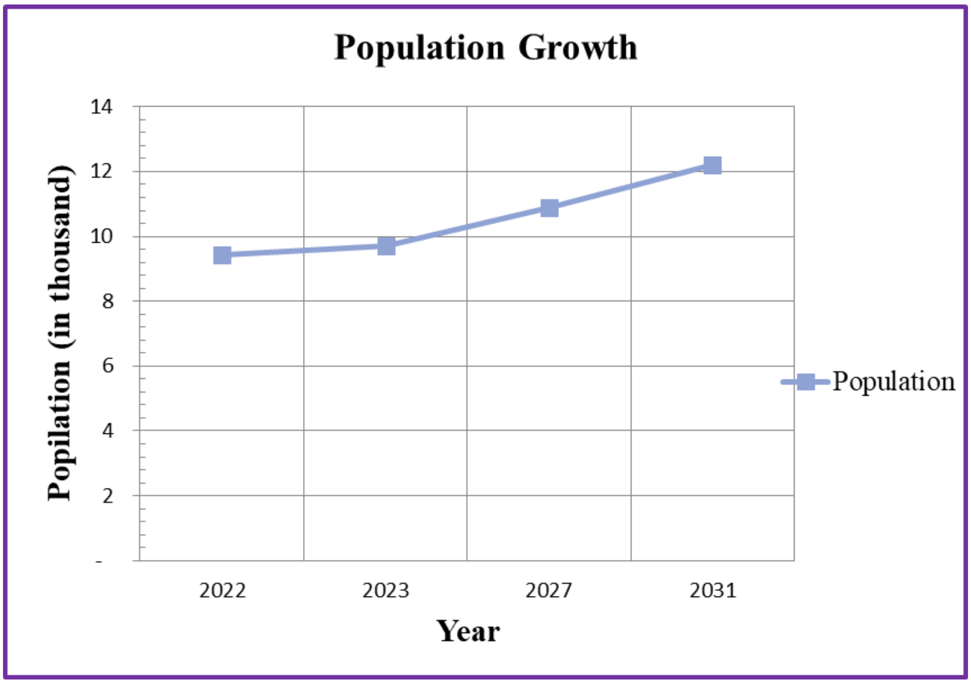


Figure 4.2: Gordama area projected population

Therefore, regard to the above table 4.1; the estimated total population figure of Gordama area was 9428 in year 2022 and 12,195 in 2031. For Simulation of network analysis 12,195 of fiscal year 2031 was adopted

Per capital water consumption

The per-capita water consumption for various demand categories varies depending on the size of the study area and the level of development. In Gordama, because of the growth of the socio-economic activity in both governmental and private sectors, there was the high water demand in the study area. Using the annual water consumption and population figure in (2022), the average per capital consumption of the study area was identified as below. For Gordama water supply with design end year of 2031, 30 (liter/day/person) for domestic water demands was adopted as per Rural Water Supply Design Criteria and Guideline (MOWE, 2022).However, for base year 2023, 25 liters per capital was adopted

Domestic water demand

Domestic water demand of Gordama area for projected year 2031 was calculated by multiplying per capital demand at the end year 2031 with projected population at each node. Total domestic demand of Gordama area for base year (2023) and Design period (2031) was calculated and summarized as: 242,543.31 and 365,841.95 liters per day respectively.

Livestock water demand

Total livestock water demand was computed as multiplying domestic demand by 5 % of the average domestic demand since data of livestock was unable to get from agriculture office. According to Rural Water Supply and Design Criteria (MOWE, 2022), 5% of domestic water demand was considered;

Livestock demand =5%* domestic water demand=5%*365,841.95=18,292.10 Liters per day

Institutional demand

School: In Gordama area there are four schools with a total number of pupils 4,496 at base year 2023 and the projected population of pupil for 2031 was 5,651 respectively. Water demand of primary and high schools for year 2023 and 2031 was 22,480.00 and 28,256.56 liters per day respectively.

Table 4.3 School water demand

Node	Demand/ pupil	Projected population			Demand for projected years(l/s)		
	l/s	Year 2023	Year 2027	Year 2031	Year 2023	Year 2027	Year 2031
J12	5	2513	2817	3159	12565	14087	15794
J16	5	1127	1264	1417	5635	6318	7083
J22	5	856	960	1076	4280	4798	5380
Sum of school water demand					22,480.00	25,203.32	28,256.56

(Source Sidama region education office)

Health facility: According Gordma Helath center assessment, total staff and service done in the center was done. Hence total demand summarized to 6,900.00 for year 2023 and 8673.05liters for year 2031 per day respectively as indicated in the table below

Table 4.4 Gordama and Habeja health centers water demand

Node	Name of health facility	Per capital (l/day)	Year 2023		Year 2027		Year 2031	
			Users per day	Demand per day	Users per day	Demand per day	Users per day	Demand per day
J12	Gordama health center	25	128	3,200.00	144	3,587.66	161	4,022.29
J12	Gordama health post	10	50	500.00	56	560.57	63	628.48
J22	Habeja health center	25	128	3,200.00	144	3,587.66	161	4,022.29
Total sum of health center demand				6,900.00		7,735.90		8,673.05

Therefore, further reviewing work was necessary to fix the recent per capital water consumption of the study area. And as per the revised design report of Gordama area water supply system, obtained from the sum of demands (2031) institutional water demand was obtained by summing up water demand for school and health facilities;

Institutional demand= school water demand +health institutions water demand

=8,673.05+28,256.56= 36,929.62 liters per day

Average water demand

Average per capital water consumption for Gordama area was estimated to the sum of domestic, livestock and institutional (school, health facilities) and deducting 12 hours water supply from springs;

. Total demand of the area was calculated as:

$$Q_{Avg}=Q_{Dom}+Q\ I_{nst}+Q_{Ls}-Q_{sp}$$

Q_{Avg}= 365841.95+36,392.62+18292.10-28296=392,230.67liters per day.

Table 4.5 Summary of current and future water demand pattern

No	Demand Category	Unit	Year		
			2023	2027	2031
1	Domestic demand	l/s	2.18	2.67	3.91
2	Livestock demand	l/s	0.14	0.16	0.21
3	Institutional demand (school)	l/s	0.26	0.29	0.33
4	Institutional demand (health)	l/s	0.08	0.09	0.1
	Total average day demand	l/s	2.66	3.21	4.55

Distribution of demands at nodes

Nodal demands for model computation was distributed by proportional area method which created by Thiessen polygon on ArcGIS

4.1.2 Assessing existing functional schemes at Gordama area

In Gordama (the study area), there are six on-spot springs giving service to the community. The average yield of springs was measured 0.18 liters per second as per field measurement conducted on May 15, 2023. However, the average yield of springs was 0.28 liters per second according to FY 2019 schemes yield measurement report by Aleta Wondo Water Energy Office.

Figure 4.2 Measuring yield of spring at Gordama village

Water supply from existing spring

For base year and design year was calculated 53784 and 28,296. Yields of springs were constantly decreasing.

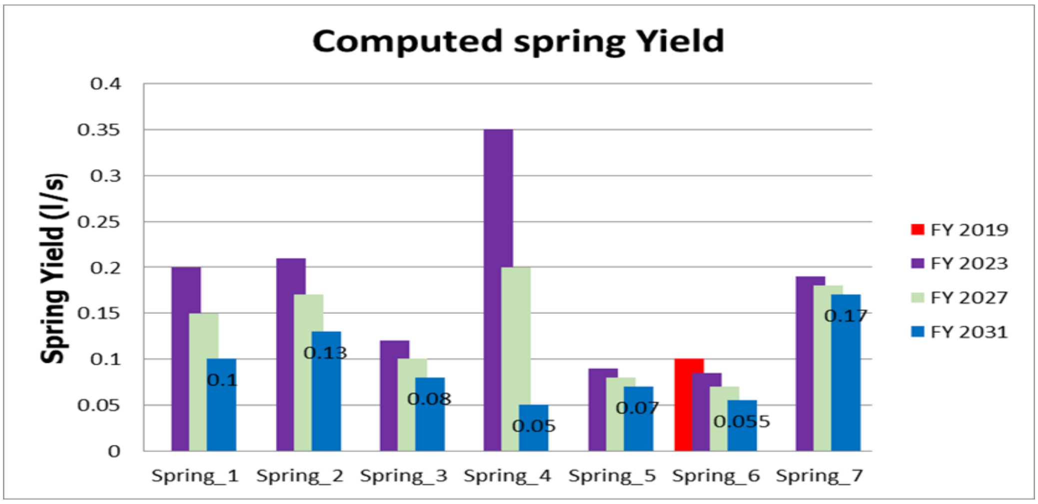


Figure 4.3 Projected yield of water from functional springs

Based on 12 hours service time the average yield from the springs at 3031 was calculated as 53784 liters per day

4.2 Water distribution network analysis

In the modern water supply system, clear water shall be delivered to the service reservoirs directly through the transmission main and which is completely isolated from the distribution system. But, existing Gordama area water supply system which was constructed before 4 years agoand as it was the old system; water is pumped simultaneously into the distribution network and service reservoir. So, the impact of this network configuration and the capacity of distribution system components were described as below.

4.2.1 Existing reservoirs capacity

The capacities of reservoirs in the water supply system were determined using different methods. The most appropriate and economical approach of determining storage volume of reservoir is the 24 hours supply demand simulation mass curves. In order to develop such type of curves, it requires reliable recorded historical data of hourly water demand figures of the study area. But, in the absence of such type of data, to determine the size of reservoirs, it was adopted the commonly practiced in many water supply systems and based on the urban water supply design criteria of the ministry of water resources; it was used for sizing the reservoir volume as one third of the maximum daily demand. Therefore, as per the design criteria of the FDRE; MoWIE, the maximum day factor usually varies between 1.0 and 1.3. Hence, a maximum day factor of 1.3 was adopted for assessing the maximum day water demand and reservoirs capacity for Gordama area community whose population is 12,195 and applied it corresponding to the total average day demand of a particular year 2031;

Maximum day demand = 1.3 * average day demand

$$= 1.3 * 393.12\ m^3\ /day = 511.06\ m^3\ /day$$

Accordingly, the current (2023) required reservoirs volume capacity for water demand of Gordama area was estimated as;

Reservoir capacity = maximum day demand *1/3

$$= 511.06m^3 * 1/3 = 170.35m^{3.}$$

Hence, from the above finding to satisfy the current water demands of Gordama; the service reservoir was sized as a 200 m^3 volume capacity of standard reservoir. This indicate, the existing reservoir capacities was enough l in size comparing with the current water demand of the area, but and it was one of the major factors of the day to day intermittent water distribution in the area. However this the maximum day demand was not discharged to reservoir due to plantation of one pump with capacity with a capacity 12.64l/s . Hence additional water pump is needed and working hour needed to be adjusted from 10 hours operating time to 16 hours

Figure 4.4: Existing service reservoir
(Source: field observation, April 2023)

4.2.2 Pump capacity

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

Pump head curve is one of the necessary input parameters for water distribution modeling and according to Tomas, et al., 2003, is an energy equation which used for solving pipe network problems. Hence, the developed pump head curve during model simulation work was presented as figure 4.2 below. Maximum day demand (MDD) of 14.20 l/s was computed for 10 hours operation time. But well safe yield obtained from Aleta Wondo Water and Energy Office was 9.5 l/s and installed pump capacity was 9 l/s discharge with adopted for pump capacity

Table 4.6 Demand criteria of Gordama water supply

Description	Unit	Year 2023	Year 2031	Remark
Total average day demand	l/day	229824.00	393,120.00	
	m^3/day	229.82	393.12	
	l/s (12hr, distribution, rural context with water point supply)	5.32	9.10	Base
Maximum daily demand coefficient	N/A	1.30	1.30	
Peak hour demand coefficient	N/A	2.00	2.00	
Maximum day demand	m^3/day	298.77	511.06	
	l/s (12hr, distribution)	6.92	11.83	MDD
	l/s(10hr, pumping, rising main)	8.30	14.20	MDD Pump
Peak hour demand	m^3/day	459.65	786.24	
	l/s (12hr, distribution)	10.64	18.20	PHD Dist.

Pump efficiency head curve when time is 2 hours

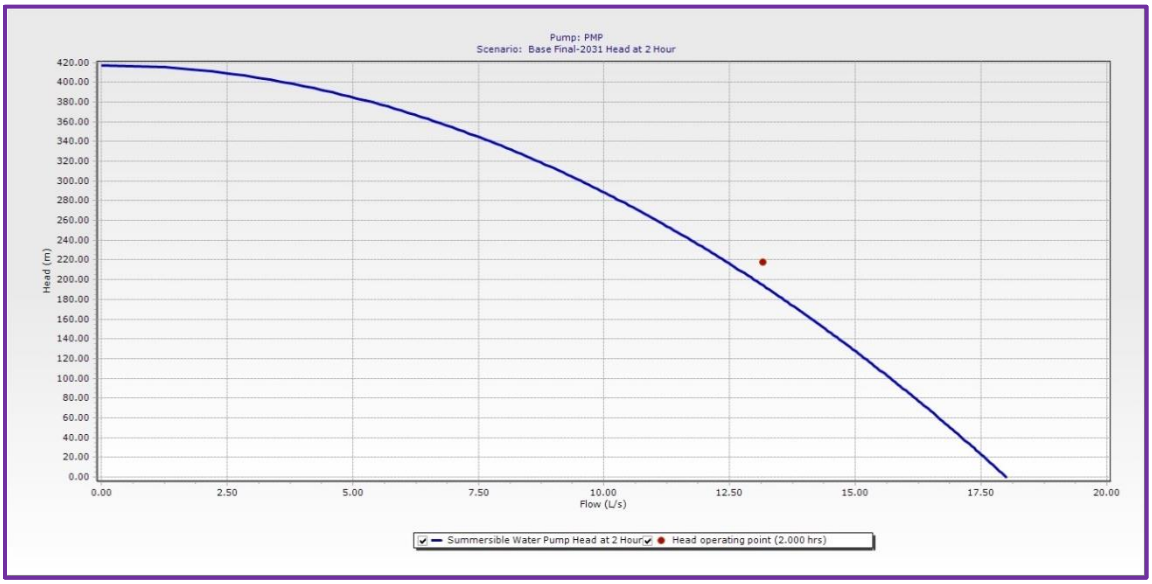


Figure 4.5: Clear water pump head curve

For this study, pump efficiency were conducted in order to determine the pumps capacity in the Gordama water distribution system. According to field observed data and model simulated result which presented in (appendix- G); the pump brake horse power and maximum water power were collected as 50 kW and 28.56 kW, respectively. Therefore, using these finding the efficiency were assessed manually and discussed as pump efficiency was computed 56.13%.

According to the pump characteristic comply with (ISO 9906:2012); most pumps which present and perform in a good condition have an efficiency of 60-80%. While, in Gordama a frequent failure and damaged of pumps due to long service time, the challenges of supplying spare parts and improper repaired after failure; made the pumps perform below the required efficiency. Therefore, the above 56.13% of the pump efficiency was shown that currently those pumps were operating in poor performance and delivering water intermittently.

As per the computed water Gems model outputs (Appendix-I) and information obtained from Aleta Wondo Woreda water service office; those pump performing in the system were operating an average of effective 10 hours in a day. With this the pumps maximum capacity of delivering water to the distribution system was discussed below. By considering 10 hours working time pump performance was obtained by multiplying maximum pump design capacity by effective operating hours and it was computed 612.00 m^3 per day. This finding confirms that the current maximum water demand of the area is 511.06m^3/day, and this indicates that the pumps capacity meets the current water demands of study area Water supply

Figure 4.6: Existing well, pump and generator
(Source: field observation, April 2023)

4.2.3 Transmission main line

It is discussed that the transmission main was not isolated from the distribution network and it gives water to distribution line before entering to service reservoir. Figure 4.6 below shows that, the main cover a total of 2023 m and with the length of this interval, the elevation difference between clear water tank (Deep Well) and service reservoir is 154m.

Figure 4.7: Elevation profile of transmission line

Most of residences found at center of study area, around treatment plant and downstream of treatment plant received water directly from the distribution line before water is reaching to service reservoir. But, as shown in figure 4.6 when length of the pumped water increased, the water pressure in the main decreases. This was shown; the transmission main for Gordama system were a function of the topographic condition of the study area.

Figure 4.8: Pressure profile of transmission line during low demand hour

As per model analysis, maximum water pressure in the transmission main was 329.38m head at pumping station, and this wter head was delivered using small pipe diameter (DN 110mm). Accordingly, the minimum water pressure was recorded as 13.53m head around service reservoir. From these results it was discussed that; small sizes of pipe in the main were lead for a frequent pipe bursting and leakage. While, the minimum value of water pressure indicating that there is intermittent/no water is supplied to the service reservoir. Therefore, most of the residences foundaround and downstream of the service reservoir were not served water from the system.

4.2.4 Distribution main line

Regards the topography of Gordama area, the locations of nodes in the water distribution line is in close proximity to each other. The maximum and minimum water pressure in the distribution system was 286.03.m and 2.2m head around treatment plant and service reservoir, respectively.

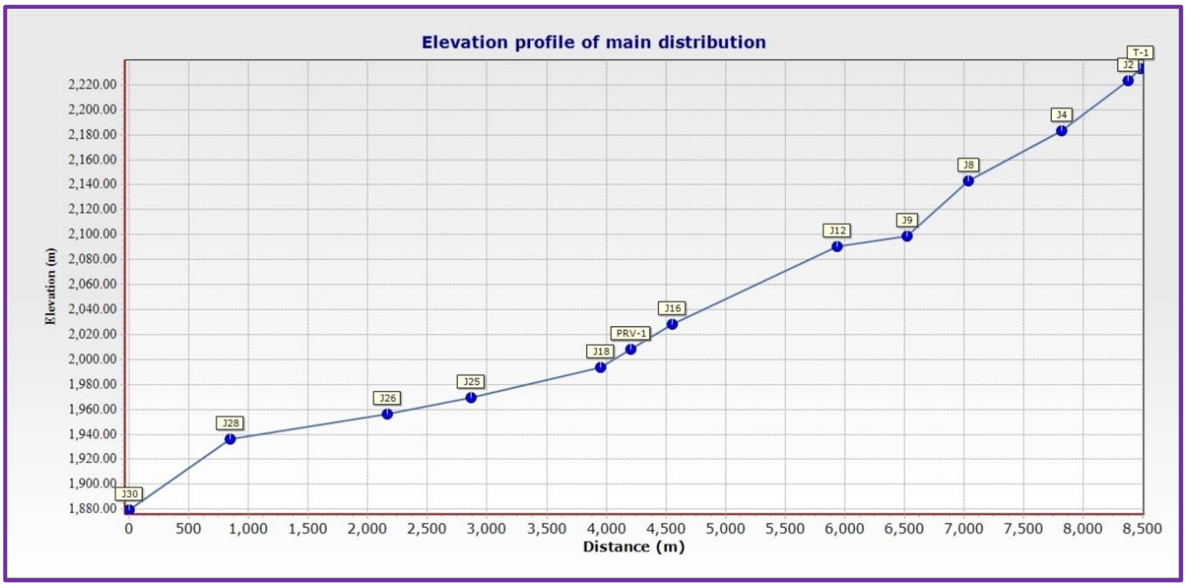


Figure 4.9: Elevation profile of main distribution network

According to the design criteria of the FDRE; MoWIE, the maximum and minimum water pressure in the distribution system is 80m and 15m, respectively. Beside these comparisons; the current Gordama area existing water distribution network was operating out of the recommended limitation. This is because of; water is delivered to the distribution main simultaneously by pumping and gravity means (old system), and the system were served beyond its design life.

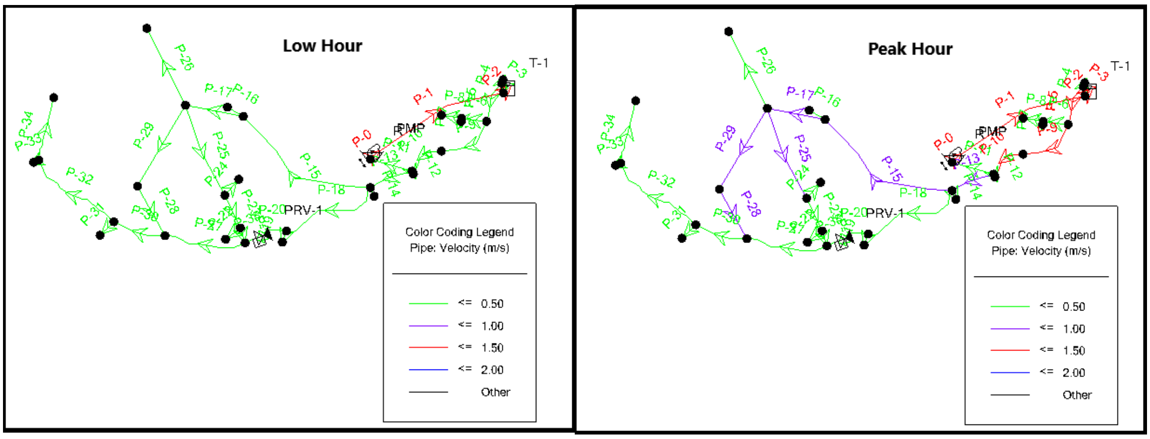


Figure 4.10: Pipe line velocities during peak and low hour demand time

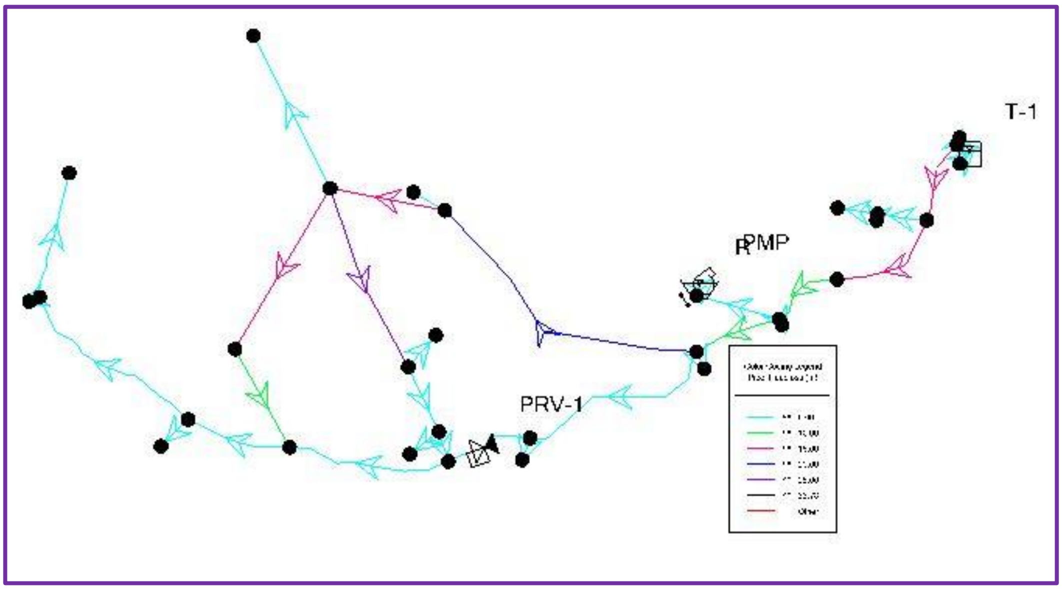


Figure 4.11: Pipe head loss during peak hour demand time

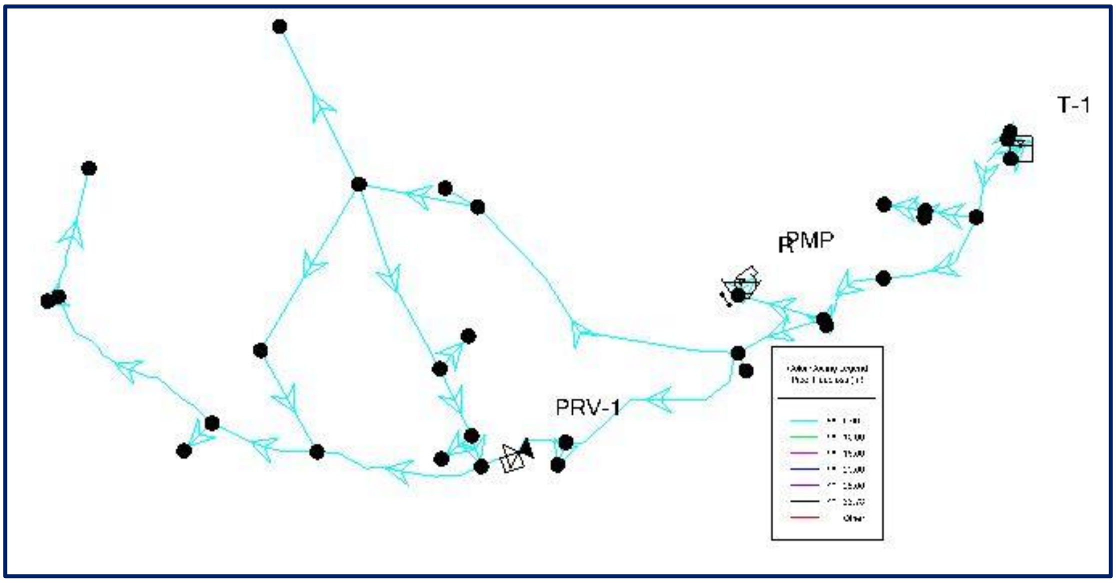


Figure 4.12 Pipe head loss during low hour demand time

4.2.5 Pressure variation in the distribution system and demands

Variation of water pressure in the distribution system is mainly because of hourly fluctuation of water demand. As shown in figure 4.7 and figure 4. Below; the water pressures in Gordama distribution system were a function of this factor. Variation of elevation difference in most partof the study area has also an impact for the rising and reduction of water pressure in the network. Therefore, during peak demand time most part of the network was disconnected from the system and wide residential area of the study area were not getting water (figure 4.7). While, most of the residences were get and collect water at night flow during low demand time (figures 4.8). However, residences found around treatment plant area, downstream of treatment plant and lower part of the study area; were get water continuously (without any intermittent) unless otherwise the system is break at the pumping station.

4.2.6 Negative pressure

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

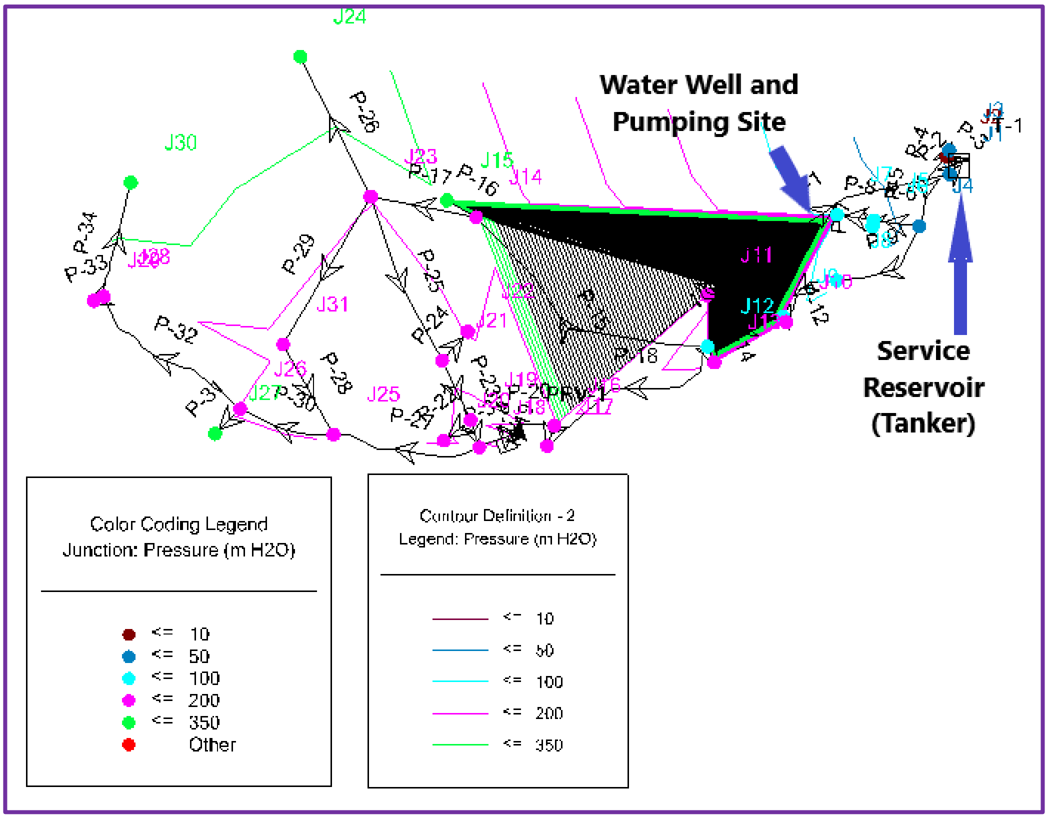


Figure 4.13: water distribution networks during peak demand time

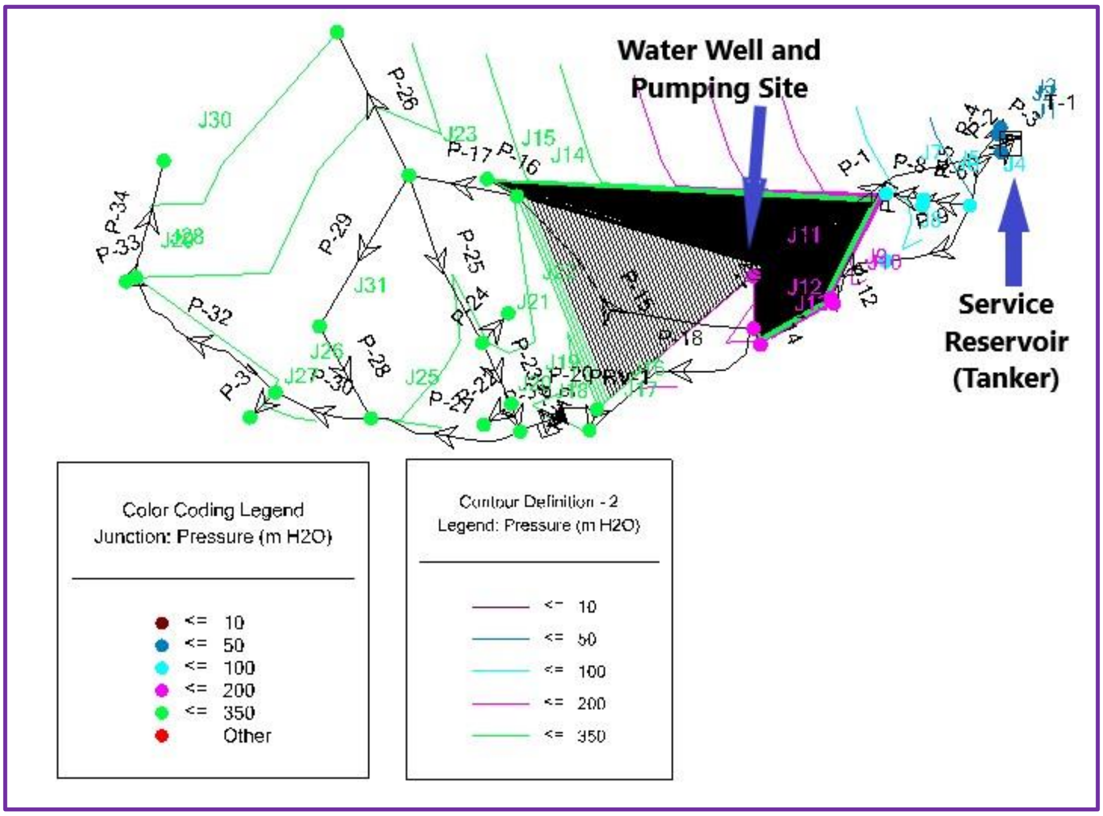


Figure 4.14: Water distribution networks during night flow/low demand time

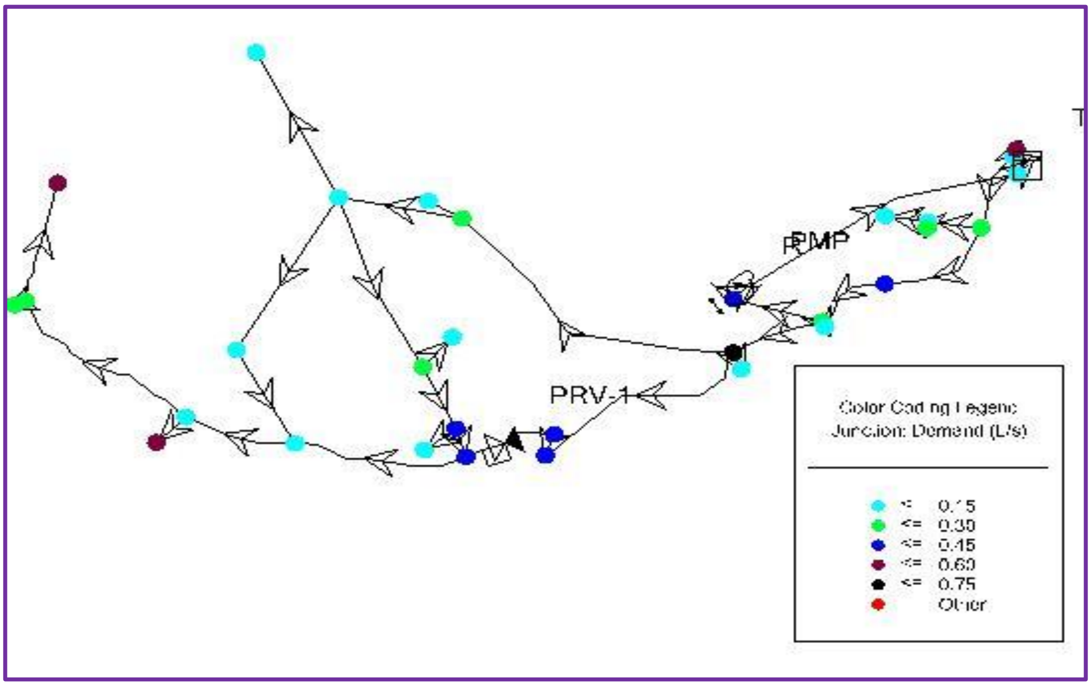


Figure 4.15: Junction water demand during peak demand time

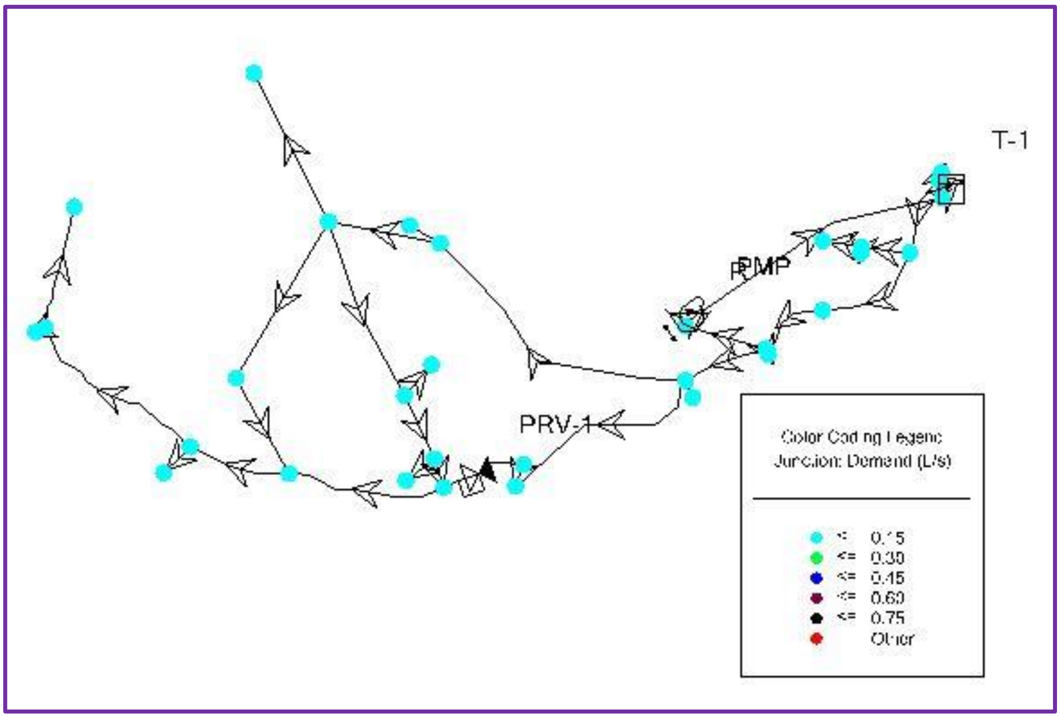


Figure 4.16: Junction water demand during peak demand time

4.2.7 Hydraulic model calibration and validation

In the modern time, water utilities have able to analyze the status of their existing water supply system using hydraulic models. But, for assuring the entered water distribution model inputs data accuracy; the computed model results have been compared with the actual observed field conditions of study area.

As shown in figure 4.10 and 4.11 below; during the comparison of measured pressure value with the simulated one, gaps were recorded up to 18.98 and 19.45 m head at peak and low or night hours respectively and it was out of the pressure standard and limitations suggested by Tomas, et al., 2003. Therefore, the computed pressure value of both scenarios, during peak demand time and low demand time (night flow) were calibrated until the result was approach to the observed pressure value

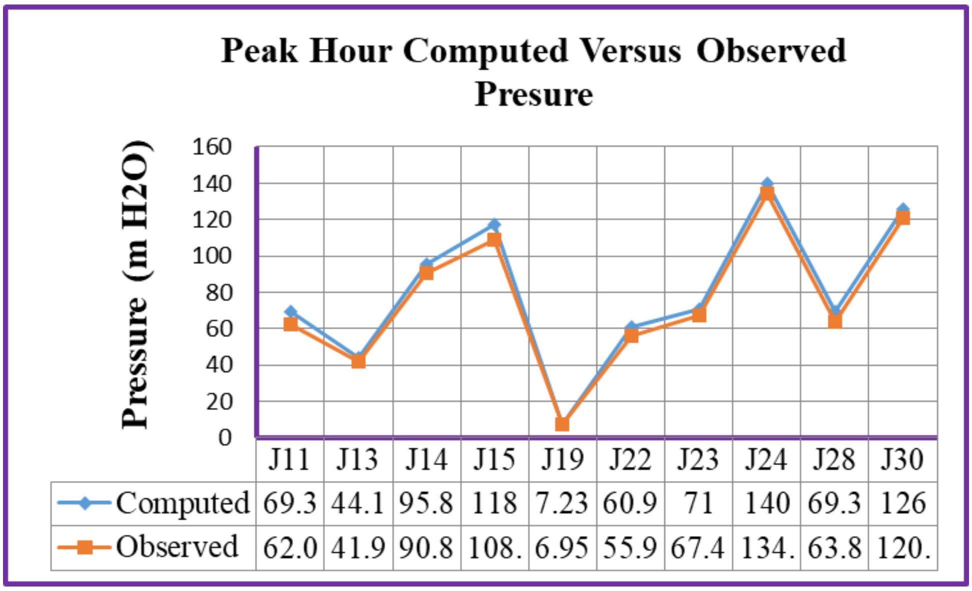


Figure 4.17: Graphical representations of the computed and observed pressure during peak hour

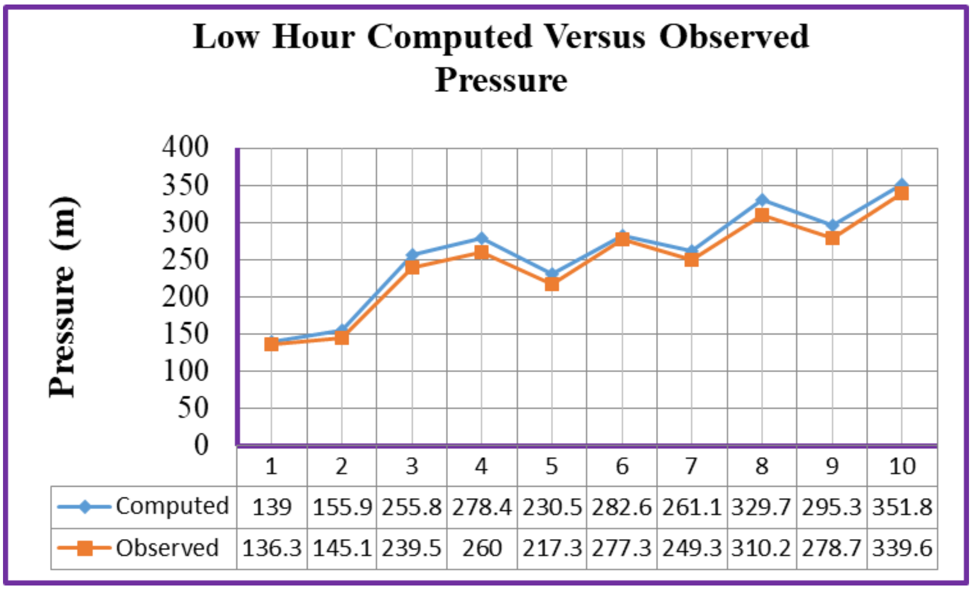


Figure 4.18: Graphical representation of the computed and observed pressure value during night time

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

Therefore, during model calibration; C-factor was used 105 for GI, 110 for HDPE pipe. While, as per discussion with the water utility manager, in Gordama the maximum hour water demand is happen during morning and evening time, when most people use water for bathing, washing and cooking purpose. Accordingly, demand adjustment was undertaken by adopting multiplier factors in reasonable way (a maximum and minimum of 2 and 0.3, respectively) and demand concentration also adjusted based on actual condition of the Gordama area. With regard to these, time series representations of the calibrated pressure head difference were presented as appendix-M

4.2.8 Model validation

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

As shown in figure 4.11 and 4.12; it explain the results of correlation value (R2) for both peak and low demand time was represent as 99.79% and 97.52%, respectively. Thereby, the calibrated pressure value was validated within the recommended standard.

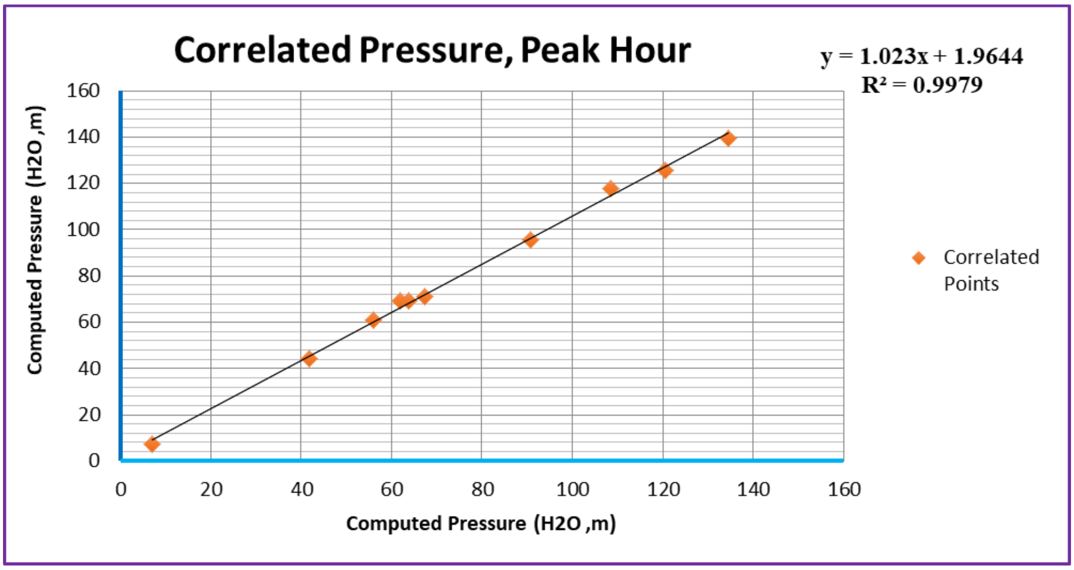


Figure 4.19: Correlated plot during pressure calibration for peak demand time

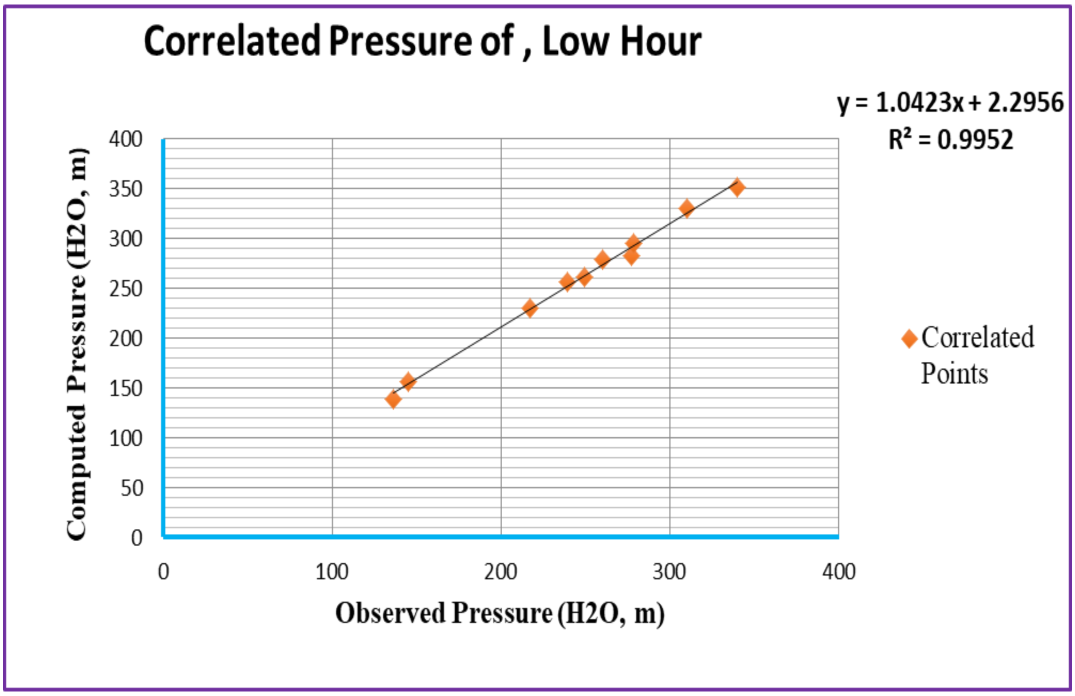


Figure 4.20: Correlated plot during pressure calibration for low demand time

4.3 Water loss analysis and leakage management practices assessment

4.3.1 Water loss analysis

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

4.3.1.1 Percentage of water loss

Non-Revenue Water (NRW) includes water losses in the water distribution system, illegal connections, and improper metering and recording. The amount is expressed as percentage of the totalproduced water from the water supply system. The percentage usually varies and depending on the age of the pipes and complexity of the system. According to the Aleta Wondo water service office; during 2023, total non-revenue Water in Gordama area was estimated as 20%.

Total percentage of water loss (%) $= \frac{\text{unbilled production}}{\text{system production}} * 100$

Table 4.7 Percentage of Non-Revenue Water (NRW) of Aleta Wondo

Production year	Aleta Wondo WS production (m^3/year)	Aleta Wondo WS consumption (m^3/year)	Water loss ($m^{3/}$year)	Water loss (%)
2018	105000	84000	21000	20.00
2019	120000	97200	22800	19.00
2020	125800	104414	21386	17.00
2021	130000	109200	20800	16.00
2022	135000	114750	20250	15.00
2023	144000	115920	28080	19.50

(Source; Aleta Wondo Woreda Water Supply Service , 2023)

As shown in table 4.7 above, due to the constant production/supply of water to the system and accordingly the increases of water consumption in the town was the reason why for reduction of total volume of NRW in recent years. However there was reduction of NRW in recent years, 2023 figures indicates that NRW was increased as a result of age of fittings, and gap of management.

4.3.1.2 Category of water loss

From the above table 4.7, the total Non-Revenue Water during 2023 was recorded as 28,080m^3. These amounts of water loss were categorized as physical/real and apparent loss. Unavoidable Annual Real Loss (UARL) or the minimum achievable annual physical losses was adopted as the total physical loss in the system, and it was analyzed as;

Total mains length of the system = 7.63 km,

Total length of distribution = 11.34km

Number of service connections (registered) = 450 in number, Average pressure was taken from the actual observed pressure value of 24 hour duration, and the average value of 53.84 m was adopted.

While, the total water loss was identified as per number of connection and pipe length at the study area (Gordama) level

$$UARL= (18*7.63+0.8*450+25*11.34)*53.84 \text{ liters/day}= (15{,}344.76 \text{ m}^3\text{/year}.$$

Total apparent losses in the system were determined from the town water balance as;

Apparent Loss = Total NRW– UARL = (28,080– 15,344.76) m^3/year = 12,735.24m^3/year

From the above descriptions, apparent loss was small in volume, and it covers 8.84% of total volume of water losses in Gordama area water distribution system. While, physical losses were also contribute a considerable volume of loss in the system and it covers 10.66% of total NRW

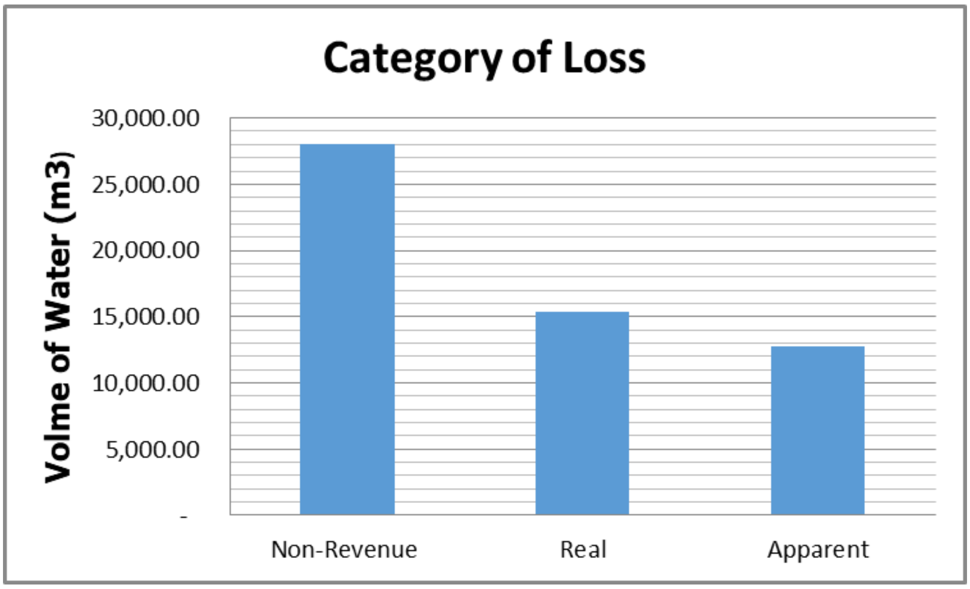


Figure 4.21: Category of water loss, during end of June, 2023

4.3.1.3 Water loss as per number of service connection

One of the appropriate indicators of water loss in the distribution system is describing it as per number of service connection (liters per service connection per day, l/c/d), and it gives more precise figure than NRW as a percentage of inputs volume. Based on the obtained data; the total number of service connections in Gordama was 450 and taken this, the volume of water loss as per connection were analyzed from the total unbilled volume.

Loss per connection = (unbilled volume *1000 l/m^3)/ (total number of connection*365 day)

Loss per connection = (28,080 m^3/year *1000 l/m^3)/ (450*365 day) = 170.96 liters/c/day.

According to (Farley, et al., 2008), performance indicator of physical loss target matrix; Gordama water loss per connection were not found in good condition, which is > 150 l/c/day of loss when the system is pressured in distribution system with average pressure of 30m. However, leakage related with connection contributed as the sources of physical loss in Gordama area water distribution system.

4.3.1.4 Water loss as per pipe length

The other good indicator of water loss in the distribution network is determining loss as per pipe length (liters per kilometer of pipe line per day, l/km/d). According to the Aleta Wondo Water Office, the total length of water distribution line including both main and privet pipe line (from property boundary to customer's meter) were estimated about 18.97 km. Therefore using this, the estimated amount of water loss; as per kilometer of the pipe length of the area was calculated as 4055.43 liters/km/day or 4.06 m^3 /km/day.

Loss per pipe length = (unbilled volume*1000 l/m^3)/ (total pipe length in the town *365 day)

Loss per pipe length = (28080m^3 /year *1000 l/$m^{3)}$/ (18.97*365 day) = 4055.43 liters/km/day

As per revised literature, 'the average condition of water loss as per pipe length is 10,000-18,000 liter/km/day, and bad condition of system is >18,000 liter/km/day.' Therefore, this figure shown that if distribution line is expanded, water loss also increases in the pipe network.

4.3.1.5 Major factors contributing to water loss in Gordama area

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

4.3.1.6 Age and size of pipe

It has been observed that small size and aged pipes were laid in Gordama water distribution network pipe were served without any replaced for the last7 years. Whereby, these pipe materials suffered its quality due to long service time, water carrying capacity and environmental conditions. Therefore, age and pipe size are the main factors for frequent pipe bursting and real losses in the town water distribution network. Accordingly, the water utility were lost 4.06 m^3/l/day of water from the system, and these mainly were occur in the transmission main, and distribution line served around treatment plant and in center of the area

4.3.2 Leakage management practice

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

4.3.2.1 Water utility organization

A strong management and organizational setup is the main reason for good leakage management practice in the water supply services and to the satisfaction of the customers. According to Aleta Wondo Water office, the total numbers of staff members including supporting sections are 6 in number. Of these two are professionals from technique section (one electromechanical, one plumber). Based on the policy and guidelines of the Woreda administration, the town water supply service manager can organize, directs and administers the overall activities of the water service unit. While, regard to the leakage management view, the operation and maintenance section of the utility is the most important section in the water supply service and the head of this section shall be accountable to the manager of the water service office. The main activities of this section are direct, coordinate and control the water supply operations and maintenances work including production, distribution, leakage and laboratory units.

4.3.2.2 Operation and maintenance practice

As per the collected information and field observation, the operation and maintenance culture of Gordama area was mainly based on support from Aleta Wondo Water Office and the main tasks were presented as below

Operation system:

- It includes the operation of the intake, treatment units, pumping stations and the associated transmission mains
- There is no operators dwelling and permanent operators who control the above assets in the pump and generator plant site

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

Maintenance culture:

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

Financial analysis

The main financial source of Gordama water service office was the government budget of Aleta Wondo Woreda. Accordingly, the financial plan and polices in the area water service system was assessed as below;

Water tariff policies

One of the leakage management strategies in the water utilities is the water tariff carried out in the system. The major objective of water tariff is to make financial sustainability and cost recovery, with the consideration of low-income groups. Accordingly, the water tariff structure for Gordama area water system was reviewed and applied based on the regional water, mineral and energy bureau recommended value, and the expected capital benefit of the water utility. As per the town water service office, the tariff structure is adopted as flat and graded rate of 50 cents per Jerican

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The existing water distribution system of Gordama was established for an estimated population of 10,000. But, as compared with the current population figure of 12,196, it was served beyond the design life and low coverage in the Gordama area.

For various demand categories, the current average per capital water consumption of Gordma was found as 25 l/c/d. besides comparing the maximum water demand of the Gordama (511.06 m^3/day), in contrast the size of existing infrastructural components such as clear water reservoir, pumping station and distribution pipes were found small in capacities, and leads to supplying water intermittently. Thereby, water pressure in the distribution network observed that were not performing within the proposed maximum and minimum design criteria set by FDRE, MoWIE. Accordingly, the water distribution network were faced a frequent pipe bursts and failures duringlow demand time and exposed to large volume of water loss especially in high pressure zone areas, while during high demand time mostly residences found in dense population and higher level of the Gordama were not received and/served continuous water from the system.

Although with low water distribution in the town, water loss was analyzed from recorded annual production and consumption figure. Whereby, during 2023 the Gordama water balance indicated that34.64% of treated water was lost as a total Non-Revenue Water. Thus, 13.52% of total losses were within the distribution network and accounted as physical loss, while 21.12% is lost as a result of apparent loss. Further water loss was examined based on the infrastructure leakage index method, and however there is physical losses and contribute considerable volume of water losses in the distribution system. While, apparent losses are more significant and the major sources of water losses in Gordama. In general, aging and size of pipe material, metering inaccuracies (under registration), illegal connections, systematic data handling errors and poor maintenance practices were found as the major sources of water losses in Gordama water distributionnetworks.

Gaps related with financial limitations, lack of professionals' experts, and less attention towards water loss management are the major problems of Gordama area water utility. Whereby, the overall maintenance and leakage controlling practices of the authority were found in poor performance.

In general, it was concluded that the current water distribution network of Gordama was in poor performance and were not conducted adequate water to the various demand categories ofthe town.

5.2 Recommendations

Based on the analyzed findings the following recommendations were mentioned to Gordama area existing water supply system:

- As the current water demand in the study area is much greater of the daily water production of the system, so it is necessary revising the design and rehabilitates the water distribution system by improving the size of reservoirs capacity and replacing the new pumps with the required hydraulic performance.
- Pipe rehabilitation decision should be involved. For the case of Gordama, by considering that most of existing pipe is still an asset for the utility, it is most advisable and recommended installation of a parallel main line with larger diameter than replacement of the old line. While, cleaning and removing deposits from the old pipeline walls should be also advised to improve flow through the pipeline and restoring lost carrying capacity in the mains.
- It was recommended that transmission main should be totally isolated from distribution main, and water should be distributed from the service reservoir by gravity mean.
- To control and protect the pump sets against high pressure conditions and broken pipe situations, the pump station should be provided with new switchboards.
- To control and minimize risks related with variation of pressure, water hammer and back water flow; it was advised installing the necessary valves and accessories in the water distribution system.
- During water loss analysis, it was observed that large amount of water were lost as result of meter under registration. So, it is much advised that; those customer meters were performed in poor status and served for a long period should be replaced by good brand meters. Further, the study area water authority should be regularly examined and tested all customer metering accuracies in accordance with the manufacturer's recommendations.
- Illegal connection and data handling problem is the other challenges of the water utility. So that, the water authority should be provides customer awareness programs and should be encouraged to report illegal connections, and regulations should be in place to penalize the water thieves. While, study area water utility should be improved their data handling system supporting with computerized recording technology.
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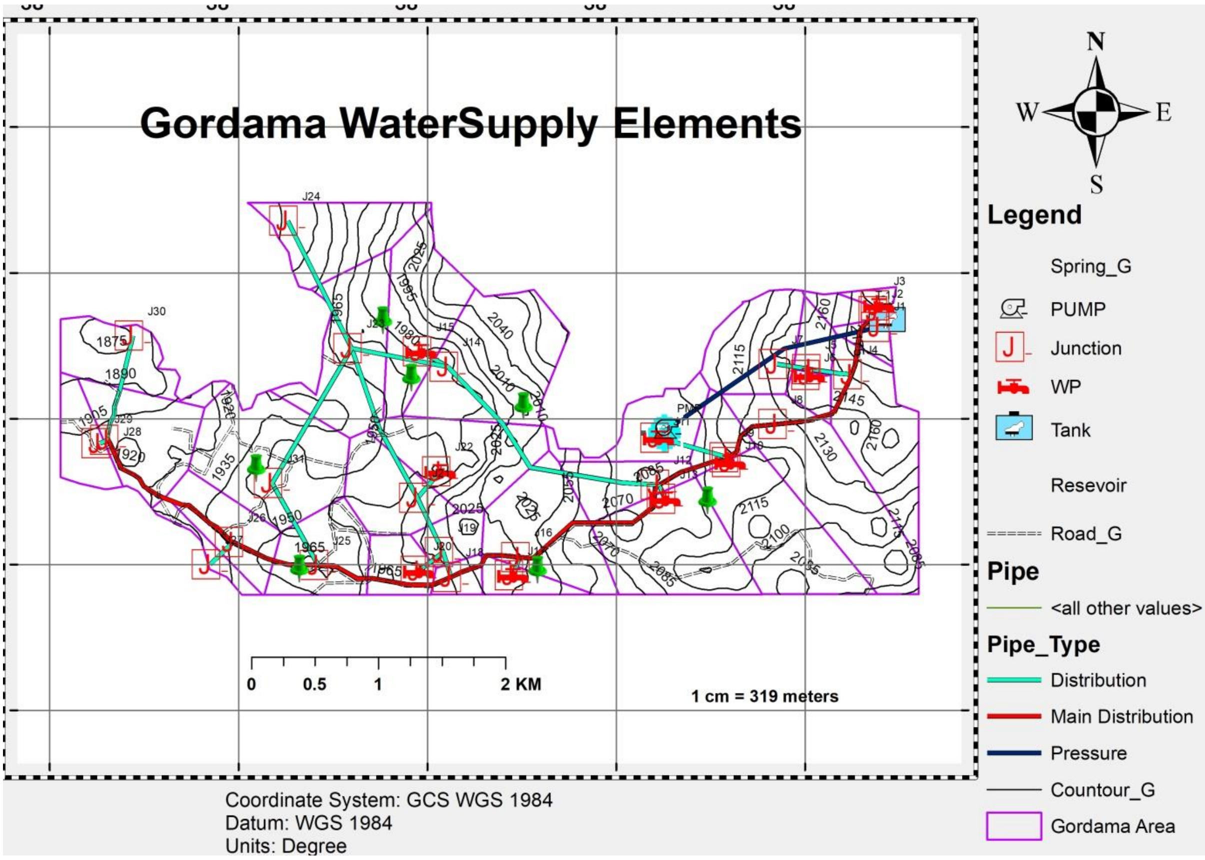
APPENDIX

Appendix A

Map shown the overlapped distribution pipeline map of the Gordama village



Appendix B

Population projection of each School

Node	Name Of School	Projection Year		
		2023	2027	2031
J12	Gordama Primary School	1413	1584	1776
J22	Habeja Primary School	856	960	1076
J16	Habeja and Gordama Primary	1127	1264	1417
J12	Gordama High School	1100	1233	1383
Total		4496	5041	5651

Appendix C

Population projection of health facility

Facility name	Staff or service	Unit	Projected Health Facillity to FY		
			2023	2027	2031
Gordama health center	All Staff	Number/day	57	64	72
	Outpatient	Number/day	65	73	82
	Inpatient	Number/day	1	1	1
	Caregivers	Number/day	5	6	6
Total projected population of Gordama health center			128	144	161
Habeja health center	All Staff	Number/day	57	64	72
	Outpatient	Number/day	65	73	82
	Inpatient	Number/day	1	1	1
	Caregivers	Number/day	5	6	6
Total Projected population Of Habeja health center			128	144	161
Gordama Health Post	Outpatient	Number/day	50	56	63

Appendix D

Base year and future of nodal area and population calculated by Thiessen's Polygon

Node	Nodal area (km^2)	Average day demand(l/s)		
		2023	2027	2031
J1	0.10	0.03	0.03	0.03
J2	0.10	0.03	0.03	0.03
J3	0.94	0.24	0.27	0.30
J4	0.46	0.12	0.13	0.15
J5	0.42	0.03	0.03	0.03
J6	0.22	0.11	0.12	0.14
J7	0.66	0.06	0.06	0.07
J8	0.38	0.17	0.19	0.21
J9	0.26	0.10	0.11	0.12
J10	0.55	0.07	0.07	0.08
J11	0.50	0.21	0.23	0.26
J12	0.37	0.35	0.39	0.44
J13	0.62	0.01	0.05	0.10
J14	0.65	0.09	0.11	0.12
J15	0.44	0.00	0.00	0.04
J16	0.70	0.16	0.20	0.24
J17	0.17	0.17	0.19	0.21
J18	0.79	0.20	0.22	0.25
J19	0.14	0.18	0.20	0.22
J20	0.94	0.04	0.05	0.05
J21	0.54	0.11	0.13	0.14
J22	0.31	0.03	0.04	0.04
J23	0.91	0.00	0.00	0.00
J24	0.10	0.05	0.05	0.06
J25	0.18	0.00	0.00	0.00
J26	0.10	0.04	0.04	0.05
J27	0.80	0.24	0.27	0.30
J28	0.05	0.14	0.16	0.17
J29	0.14	0.08	0.09	0.10
J30	0.09	0.23	0.26	0.29
J31	0.67	0.06	0.08	0.11
Sum		3.31	3.80	4.38

Appendix E

Roughness coefficient, c-factor for different materials

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

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

Galvanized iron smooth and new	120	129	133	134	137	142
Wrought iron smooth and new	129	137	142	145	148	148
Coated steel smooth and new	129	137	142	145	148	148
Uncoated steel smooth and new	134	142	145	147	150	150

Appendix F

Measured yield of existing springs at Gordama area

Node	Spring name	Coordinates			Yield in (l/s)			
		X	Y	Z	Year 2019	Year 2023	Year 2027	Year 2031
J13	Spring_1	441883	732127	2050	0.25	0.2	0.15	0.1
J15	Spring_2	439553	733153	1936	0.25	0.21	0.17	0.13
J16	Spring_3	440537	731648	2088	0.14	0.12	0.1	0.08
J23	Spring_4	439329	733600	1966	0.5	0.35	0.2	0.05
J25	Spring_5	438674	731660	1939	0.1	0.09	0.08	0.07
J14	Spring_6	440430	732928	2029	0.1	0.085	0.07	0.055
J31	Spring_7	438336	732446	1856	0.2	0.19	0.18	0.17

Appendix G

Assigned average day water demands to each supply node for year 2031

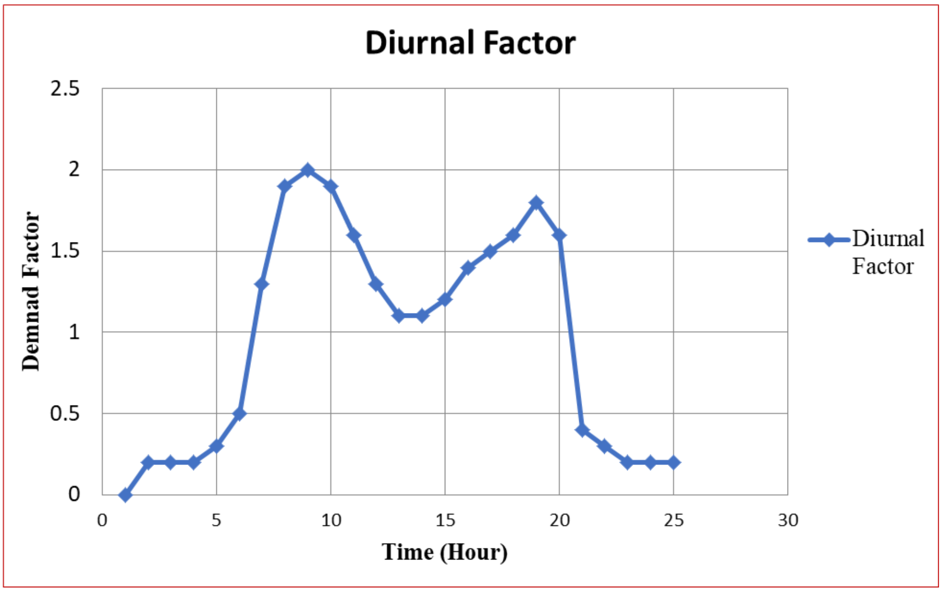
Node	Coordinate			Demand (l/s)			Total average demand (l/s)
	X	Y	Z	Domestic	Livestock	Institutional	
J1	433216.71	73355.00	2224.10	0.0318	0.0016		0.0334
J2	443196.00	733668.00	2223.10	0.0333	0.0017		0.0349
J3	443213.00	733715.00	2221.40	0.2989	0.0149		0.3138
J4	442998.00	733174.00	2183.20	0.1472	0.0074		0.1545
J6	442668.00	733172.00	2155.40	0.1350	0.0068		0.1418
J7	442413.40	733255.79	2146.50	0.0692	0.0035		0.0727
J8	442410.71	732787.19	2143.40	0.2112	0.0106		0.2218
J9	442029.13	732528.05	2098.50	0.1223	0.0061		0.1284
J10	442048.00	732490.00	2094.70	0.0826	0.0041		0.0867
J12	441488.20	732314.43	2090.80	0.1763	0.0088	0.2366	0.4217
J13	441537.00	732202.00	2077.50	0.1080	0.0054		0.1134
J15	439629.00	733358.00	1954.00	0.0513	0.0026		0.0539
J16	440395.57	731748.14	2027.90	0.1575	0.0079	0.0820	0.2473

J17	440344.00	731605.00	2011.10	0.2065	0.0103		0.2168
J21	439595.86	732213.93	1973.20	0.1407	0.0070		0.1477
J19	439797.34	731788.79	2001.00	0.2231	0.0112		0.2342
J20	439606.00	731642.00	1979.30	0.0543	0.0027		0.0571
J18	439859.33	731594.20	1993.90	0.2498	0.0125		0.2623
J26	438151.61	731868.60	1956.40	0.0447	0.0022		0.0469
J27	437973.36	731693.97	1917.70	0.2972	0.0149		0.3121
J28	437177.15	732670.86	1936.10	0.1727	0.0086		0.1814
J29	437108.78	732642.22	1935.00	0.0976	0.0049		0.1025
J30	437370.00	733484.00	1879.50	0.2900	0.0145		0.3045
J23	439083.37	733384.08	1970.90	0.0064	0.0003		0.0068
J24	438581.35	734382.53	1902.20	0.0586	0.0029		0.0615
J5	442675.00	733215.00	2164.00	0.0320	0.0016		0.0336
J11	441486.00	732681.00	2068.40	0.2558	0.0128		0.2686
J25	438817.24	731687.32	1969.10	0.0000	0.0000		0.0000
J22	439777.00	732421.00	1948.90	0.0430	0.0021	0.1088	0.1539

J14	439836.39	733239.05	1976.70	0.0023	0.0001		0.0024
J31	438458.64	732329.85	1963.50	0.1282	0.0064		0.1346
Total				3.9274	0.1439	0.4274	4.4988

Appendix H

Patterns for hourly demand multiplier factors



Appendix I

Time series representation of pressure value; for peak demand time

Item	Sample taken point	Location			Measured time	Computed pressure (m)	Observed pressure (m)	Errors (m)
		X (m)	Y (m)	Z (m)				
1	J11	442413.40	733255.79	2146.50	0.08	69.28	62.00	7.28
2	J13	442675.00	733215.00	2164.00	0.05	44.08	41.90	2.18
3	J14	442998.00	733174.00	2183.20	0.07	95.76	90.85	4.91
4	J15	438458.64	732329.85	1963.50	0.09	117.61	108.63	8.98
5	J19	437177.15	732670.86	1936.10	0.09	7.23	6.95	0.28
6	J22	438581.35	734382.53	1902.20	0.19	60.92	55.98	4.94
7	J23	439083.37	733384.08	1970.90	0.12	71.02	67.40	3.62
8	J24	439777.00	732421.00	1948.90	0.14	139.57	134.58	4.99
9	J28	439797.34	731788.79	2001.00	0.15	69.25	63.85	5.40
10	J30	440395.57	731748.14	2027.90	0.17	125.73	120.50	5.23
R2=0.9979								

Appendix J

Time Series representation of pressure value; for low demand time/night flow

Item	Sample taken point	Location			Measured time	Computed pressure (m)	Observed pressure (m)	Errors (m)
		X (m)	Y (m)	Z (m)				
1	J11	442413.40	733255.79	2146.50	0.26	138.95	136.30	2.65
2	J13	442675.00	733215.00	2164.00	0.29	155.89	145.10	10.79
3	J14	442998.00	733174.00	2183.20	0.30	255.76	239.50	16.26
4	J15	438458.64	732329.85	1963.50	0.31	278.40	260.00	18.40
5	J19	437177.15	732670.86	1936.10	0.33	230.53	217.30	13.23
6	J22	438581.35	734382.53	1902.20	0.22	282.55	277.30	5.25

7	J23	439083.37	733384.08	1970.90	0.48	261.09	249.30	11.79
8	J24	439777.00	732421.00	1948.90	0.49	329.65	310.20	19.45
9	J28	439797.34	731788.79	2001.00	0.50	295.26	278.65	16.61
10	J30	440395.57	731748.14	2027.90	0.51	351.75	339.55	12.20
R2=0.9952								

Appendix K

Pump result: Calculated water power (kW)

Time (Hr.)	Calculated Water Power (kW)	Time (Hr.)	Calculated Water Power (kW)
0:00:00	0:00:00	27.97	13:00:00
1:00:00	1:00:00	28.02	14:00:00
2:00:00	2:00:00	28.07	15:00:00
3:00:00	3:00:00	0.00	16:00:00
4:00:00	4:00:00	0.00	17:00:00
5:00:00	5:00:00	0.00	18:00:00
6:00:00	6:00:00	0.00	19:00:00
7:00:00	7:00:00	0.00	20:00:00
8:00:00	8:00:00	0.00	21:00:00
9:00:00	9:00:00	0.00	22:00:00
10:00:00	10:00:00	0.00	23:00:00
11:00:00	11:00:00	0.00	24:00:000
12:00:00	12:00:00	0.00	

Appendix L

Junctions (nodal) pressure result; for peak demand time

Label	Demand (Calculated) (l/s)	Hydraulic Grade (m)	Pressure (m H2O)
J1	0.06	2,237.65	13.52
J2	0.06	2,228.13	5.02
J3	0.62	2,227.49	6.08
J4	0.30	2,195.57	12.35
J5	0.28	2,194.64	30.58
J6	0.14	2,194.60	39.12
J7	0.44	2,194.02	47.43
J8	0.26	2,160.48	17.04
J9	0.18	2,138.90	40.32
J10	0.84	2,137.93	43.14
J11	0.22	2,137.82	69.28
J12	0.10	2,122.04	31.17
J13	0.50	2,121.67	44.08
J14	0.44	2,072.66	95.76
J15	0.30	2,071.84	117.61
J16	0.46	2,121.18	93.09
J17	0.12	2,121.15	109.83
J18	0.52	2,008.16	14.23
J19	0.10	2,008.24	7.23
J20	0.62	2,007.18	27.82
J21	0.36	2,010.09	36.81

J22	0.20	2,009.94	60.92
J23	0.60	2,042.06	71.02
J24	0.02	2,042.06	139.57
J25	0.12	2,008.21	39.03
J26	0.06	2,006.03	49.53
J27	0.54	2,005.17	87.3
J28	0.00	2,005.49	69.25
J29	0.30	2,005.40	70.26
J30	0.00	2,005.49	125.73
J31	0.26	2,018.61	54.99

Appendix M

Junctions (nodal) pressure result; for night flow/ low demand time

Label	Demand (calculated) (l/s)	Hydraulic grade (m)	Pressure (m H2O)
J1	0.01	2,237.66	13.54
J2	0.01	2,235.20	12.07
J3	0.06	2,235.19	13.76
J4	0.03	2,234.74	51.44
J5	0.03	2,234.73	70.59
J6	0.01	2,234.73	79.17
J7	0.04	2,234.72	88.04
J8	0.03	2,234.25	90.66
J9	0.02	2,233.94	135.17
J10	0.08	2,233.93	138.95

J11	0.02	2,233.93	165.2
J12	0.01	2,233.71	142.62
J13	0.05	2,233.70	155.89
J14	0.04	2,232.97	255.76
J15	0.03	2,232.96	278.4
J16	0.05	2,233.70	205.38
J17	0.01	2,233.70	222.15
J18	0.05	2,231.99	237.62
J19	0.01	2,232.00	230.53
J20	0.06	2,231.98	252.17
J21	0.04	2,232.03	258.3
J22	0.02	2,232.02	282.55
J23	0.06	2,232.51	261.09
J24	0	2,232.51	329.65
J25	0.01	2,232.00	262.37
J26	0.01	2,231.97	275.01
J27	0.05	2,231.95	313.62
J28	0	2,231.96	295.26
J29	0.03	2,231.96	296.36
J30	0	2,231.96	351.75
J31	0.03	2,232.16	268.12

Appendix N

Pipe result; during peak demand time

Label	Length (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (l/s)	Pressure pipe head loss (m)	Head loss gradient (m/m)	Velocity (m/s)
P-0	55	110	GI	105	-12.64	1.51	0.027	1.33
P-1	1,874	110	GI	105	12.64	51.4	0.027	1
P-2	86	110	GI	105	12.58	2.35	0.027	1.32
P-3	105	80	GI	105	8.96	7.17	0.068	1.78
P-4	49	40	HDPE	110	0.62	0.64	0.013	0.49
P-5	551	80	GI	105	8.28	32.56	0.059	1.65
P-6	326	63	GI	105	0.86	0.93	0.003	0.28
P-7	45	40	HDPE	110	-0.14	0.04	0.001	0.11
P-8	265	50	HDPE	110	0.44	0.62	0.002	0.22
P-9	785	80	GI	105	7.12	35.1	0.045	1.42
P-10	517	80	GI	105	6.86	21.57	0.042	1.36
P-11	564	40	HDPE	110	-0.22	1.08	0.002	0.18
P-12	42	40	HDPE	110	0.84	0.97	0.023	0.67
P-13	585	80	GI	105	5.62	16.87	0.029	1.12
P-14	123	50	HDPE	110	0.5	0.36	0.003	0.25
P-15	2,042	75	GI	105	4.31	49.38	0.024	1
P-16	239	40	HDPE	110	0.3	0.81	0.003	0.24
P-17	767	63	GI	105	-3.57	30.59	0.04	1.15
P-18	1,383	80	GI	105	0.71	0.86	0.001	0
P-19	152	50	HDPE	110	0.12	0.03	0	0.06
P-20	601	80	Ductile Iron	105	0.13	0.01	0	0.03

P-21	204	63	GI	105	0.3	0.08	0	0.1
P-22	241	50	HDPE	110	-0.62	1.06	0.004	0.32
P-23	471	63	GI	105	-1.02	1.85	0.004	0.33
P-24	275	50	HDPE	110	0.2	0.15	0.001	0.1
P-25	1,282	50	HDPE	110	-1.58	31.98	0.025	1
P-26	1,118	50	HDPE	110	0.02	0.01	0	0
P-27	1,084	63	GI	105	-0.09	0.05	0	0
P-28	736	50	GI	105	1.11	10.4	0.014	0.57
P-29	1,226	50	HDPE	110	-1.37	23.46	0.019	1
P-30	701	63	GI	105	0.9	2.18	0.003	0.29
P-31	250	50	HDPE	110	-0.54	0.85	0.003	0.28
P-32	1,320	63	GI	105	0.3	0.54	0	0
P-33	74	50	HDPE	110	-0.3	0.09	0.001	0.15
P-34	845	50	GI	105	0	0	0	0
P-35	601	80	Ductile Iron	105	0.13	0.01	0	0.03

Appendix O

Pipe result; during low demand time/night flow

Label	Length (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (l/s)	Pressure pipe head loss (m)	Head loss gradient (m/m)	Velocity (m/s)
P-0	55	110	GI	105	-12.64	1.51	0.027	1.33
P-1	1874	110	GI	105	12.64	51.4	0.027	1.33
P-2	86	110	GI	105	12.58	2.35	0.027	1.32
P-3	105	80	GI	105	8.96	7.17	0.068	1.78
P-4	49	40	HDPE	110	0.62	0.64	0.013	0.49

P-5	551	80	GI	105	8.28	32.56	0.059	1.65
P-6	326	63	GI	105	0.86	0.93	0.003	0.28
P-7	45	40	HDPE	110	-0.14	0.04	0.001	0.11
P-8	265	50	HDPE	110	0.44	0.62	0.002	0.22
P-9	785	80	GI	105	7.12	35.1	0.045	1.42
P-10	517	80	GI	105	6.86	21.57	0.042	1.36
P-11	564	40	HDPE	110	-0.22	1.08	0.002	0.18
P-12	42	40	HDPE	110	0.84	0.97	0.023	0.67
P-13	585	80	GI	105	5.62	16.87	0.029	1.12
P-14	123	50	HDPE	110	0.5	0.36	0.003	0.25
P-15	2042	75	GI	105	4.31	49.38	0.024	0.98
P-16	239	40	HDPE	110	0.3	0.81	0.003	0.24
P-17	767	63	GI	105	-3.57	30.59	0.04	1.15
P-18	1383	80	GI	105	0.71	0.86	0.001	0.14
P-19	152	50	HDPE	110	0.12	0.03	0	0.06
P-20	601	80	GI	105	0.13	0.01	0	0.03
P-21	204	63	GI	105	0.3	0.08	0	0.1
P-22	241	50	HDPE	110	-0.62	1.06	0.004	0.32
P-23	471	63	GI	105	-1.02	1.85	0.004	0.33
P-24	275	50	HDPE	110	0.2	0.15	0.001	0.1
P-25	1282	50	HDPE	110	-1.58	31.98	0.025	0.8
P-26	1118	50	HDPE	110	0.02	0.01	0	0.01
P-27	1084	63	GI	105	-0.09	0.05	0	0.03
P-28	736	50	GI	105	1.11	10.4	0.014	0.57
P-29	1226	50	HDPE	110	-1.37	23.46	0.019	0.7
P-30	701	63	GI	105	0.9	2.18	0.003	0.29

P-31	250	50	HDPE	110	-0.54	0.85	0.003	0.28
P-32	1320	63	GI	105	0.3	0.54	0	0.1
P-33	74	50	HDPE	110	-0.3	0.09	0.001	0.15
P-34	845	50	GI	105	0	0	0	0
P-35	601	80	GI	105	0.13	0.01	0	0.03