Water Supply and Distribution Challenges in Rural areas: The case of Amereba Indato Water Supply System, Oromia, Ethiopia

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A **thesis submitted to the department of Water Resources Engineering**

School of Civil Engineering and Architecture

Office of Graduate Studies

Adama Science and Technology University

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Adama, Ethiopia

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DECLARATION

I hereby declare that this Master Thesis entitled "**Water Supply and Distribution Challenges in Rural areas: The case of Amereba Indato Water Supply System**" is my original work. That is, it has not been submitted for the award of any academic degree, diploma or certificate in any other university. All sources of materials that are used for this thesis have been duly acknowledged through citation

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RECOMMENDATION

I, the advisor of this thesis, hereby certify that I read the revised version of the thesis entitled "**Water Supply and Distribution Challenges in Rural areas: The case of Amereba Indato Water Supply System"** prepared under my guidance by Abebe Tamiru submitted in partial fulfillment of the requirements for the degree of Mater's of Science in Water Resources Engineering (specialize in Water Supply and Environmental Engineering). Therefore, I recommend the submission of revised version of the thesis to the department following the applicable procedures.

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APPROVAL PAGE

I, the advisor of the thesis entitled "**Water Supply and Distribution Challenges in Rural areas: the case of Amereba Indato Water Supply System, Oromia, Ethiopia"** and developed by Abebe Tamiru, hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

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We, the undersigned, members of the Board of Examiners of the thesis by Abebe Tamiru have read and evaluated the thesis entitled "**Water Supply and Distribution Challenges in Rural areas: The case of Amereba Indato Water Supply System**" and examined the candidate during open defense. This is, therefore, to certify that the thesis is accepted for partial fulfillment of the requirement of the degree of Master of Science in Water resource (specialize in Water Supply and Environmental Engineering).

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LIST OF ABBREVIATIONS

ADD	Average Day Demand
ARL	Actual Real Losses
AWWA	American Water works Associations
BH	Borehole
CSA	Central Statistics Agency
DCI	Ductile Cast Iron pipes
EMS	Emergency Management system
GIS	Geographical Information System
GPS	Geographical Positioning System
HG	Hydraulic Grade
IBNET	International Benchmarking Network for water supply
L/C/D	Litre /Capita/Day
MDD	Maximum Day Demand
NRW	Non- revenue water
NE	North East
OWMEB	Oromia Water Mineral and Energy Bureau
MoWE	Ministry of Water and Energy
MAAPL	Minima Achievable Annual physical loss
RPS	Rural pipe system
PHD	Peak Hour Demand
SDG	Sustainable Development Goal
SW	South West
EPS	Extended period simulation
TDD	Total Domestic Demand
UARL	Unavoidable Annual Real Losses
UN	United Nations
UPVC	Un-plasticized Polyvinyl Chloride
WHO	World Health Organization
WSSAI	Water supply System of Amereba Indato

ABSTRACT

The challenges of providing satisfactory water supply to the rapidly growing populations in developing countries are increasing over time. In Ethiopia, ensuring an adequate and reliable water supply in rural areas has become a significant challenge for many water utilities. Amereba Indato rural water supply system was constructed to give service for 162,519 people living in 22 rural kebeles and 3 rural towns in Robe and Tane, Arsi Zone of Oromia Region. However, the scheme is not performing efficiently according to the intended design capacity and 5 kebeles in the distribution radius are not getting water. Hence, the objective of this study was to identify the challenges of Amereba Indato rural water supply system and to propose course of actions to improve system performance and service delivery. Water demand was computed by using population projection and unit rate consumptions considering different mode of service. Hydraulic analysis was conducted employing Water GEMS software to evaluate the pressure and velocity of both transmission and distribution pipe lines. The result shows that 85.32L/s maximum day demand (MDD) is required to address the water supply coverage for the intended design period. With the existing sources spring and borehole was 30L/s and 120L/s capacity can be provided whereas the analysis shows that 36.58L/s capacity is required indicating around the spring sources remain in deficits of 6.58L/s for area supposed to be covered by the spring sources. Hydraulic analysis also shows that 22% of the distribution lines were exposed to high pressure energy (80-181m), whereas, 21.05% of the pipe has velocity less than the recommended 0.6 m/s, which might result in stagnant water formation. The analyzed water losses result indicates that about 41.09% of production is non-revenue water and 32.41% of non -revenue water is estimated as a real lose. The factors associated to water loss were found to be community interest, problem at construction time, illegal connection, and pressure. Thus, it is important to change the more pressurized pipes from the distribution network and transmission pipe by providing pressure reducing valves in the link having high pressure in distribution network. It is also important to give awareness for the community and make a rehabilitation work to cause during construction phase in the area. Finally, it is necessary for the water utility to give attention to water losses reduction policies and strategies to minimize water loss in the rural

Key words: Demand , water supply, distribution network, hydraulic performance, water los

1: INTRODUCTION

1.1. Background

Water is life and especially potable water is essential for life and health. So, access to drinking water, improves overall socio-economic and environmental existence (Gebrehiwot, 2006). According to the United Nations Sustainable Development Goal (SDG) report of 2023, there are still over 2.2 billion people who lack access to safe drinking water. The UNCEF and WHO (2023) also reported that the global coverage of water supplies in rural and urban areas was 73 and 81%, respectively, and that the least fortunate countries to have access to clean water supplies are those in Sub-Saharan Africa, with average coverage of 31 and 53% for rural and urban areas, respectively.

Lack of access to a safe and adequate water supply and health risks associated with water related diseases are major public health problems in many developing countries. Currently more than 700 million people, who mostly live in developing countries, are without access to improved and adequate water (WHO/UNICE,2014).

It is important that sources of water supply be capable of providing service for both the short-term and long-term demands being projected. According to the World Health Organization (WHO), access to drinking water is defined based on the quantity of water used (access to drinking water at least 20 l/c/d within 1 km radius for customers) (Kennedy, 2006). The provision of sufficient potable water for people within reasonable distances from a reliable and acceptable source is essential for people's wellbeing and sustainable economic progress (Yitayh L., 2011). The presence of a safe and reliable source of water is basic requirement for the community to be stable in their life. A fundamental element for good health and human right is access to clean drinking water (WHO, 2011). Additionally, it is difficult to picture a sterile and clean atmosphere without water.

The distribution of rural water supply services is increasingly moving towards piped water projects globally as a result of rising aspirations among rural residents and improving living standards. Transporting water across large distances is the only option when local water sources are insufficient or unsafe to drink, such as when the water is of low quality. Once more, state economies dictate that a single system must serve several settlements or villages. Typical example is the Purna river basin region of India, where the Amravati district is located. Due to the saline aquifer intrusion spanning over area of approximately 5,000 km^2, surface water schemes were developed for the district's community of 156 villages and two towns with piped system for residential use (Hutchings et al., 2020).

Besides to this most rural water supply points are located far away from the sources and water distribution system or transmission lines run long distance to deliver water to the customers. This creates different problem such as leakage /loss in the system, high pressure, inadequate velocity, corrosion of pipe and slipping of valves, which affected the adequacy and reliability of drinking water supply in the system.

Ethiopia, a country known for its diverse topography and varying climatic conditions and it faces significant challenges in providing sustainable and reliable access to clean water for its growing population. As the demand for water continues to rise, particularly in rural areas, the need for efficient extended distribution water supply systems has become crucial. Amareba Indato rural water supply system is one of such very extended distribution water supply systems constructed to feed two districts (Robe and Tane) of the Oromia region. The water supply systems are designed to transport water from distant sources to areas with limited water resources. It was planned to meet vital role in meeting the water demands for milt-villages.

The water supply system comes from boreholes, springs, and a 222.424 km network of pipes laid by various pipe sizes and materials. In 22 rural Kebeles and 3 rural towns, 103 public water points, 29 livestock troughs at central points, and 35 institution taps were constructed (OWWDSE, 2007). Although the system is operational at the moment, the utility has had considerable difficulties in providing the necessary services. Some water points suffered uneven water delivery, which resulted in a line of people waiting to get water. Furthermore, in many parts of the distribution area water is not reaching the user as intended in the design documents. Various reasons can be attributed to these problems. As, there is a lack of comprehensive information regarding the root cause for the aforementioned problems and to suggest practical solutions while contribute for sustainability of the system. Hence, assessing the management practices, examining the systems design to ensure service delivery, and evaluated the system's demand distribution throughout its design period is critical. It had also examined the performance of each basic component in detail.

A variety of approaches were employed to oversee the objectives set in accordance with internationally and nationally recognized scientific approaches. To ascertain whether the source's capacity can guarantee the necessary supply throughout the design period, the demand of the beneficiaries was assessed. Throughout the pipe layout, the allowable pressure and velocity were determined, which govern the pipe nominal pressure and diameter size.

Therefore, this research investigated the Water Supply and Distribution Challenges in Rural areas of Amereba Indato Water Supply System, based on the objectives set in accordance with internationally and nationally recognized scientific approaches by used WaterGEMS models, causes and impacts of the identified challenges. The outcomes of this research may contribute to evidence-based decision-making and support the development of effective strategies for long-term water supply planning and management for long distance ruralwater supply system.

1.2. Statement of the Problem

Water is one of the most vital yet increasingly scarce natural resources. Despite its critical importance, many people worldwide still lack access to safe and adequate water supply services, which negatively impacts various aspects of their lives (Yibeltal, 2011). Effective water distribution involves ensuring that water reaches consumers in the right quality, quantity, and pressure (Bibhabasu, 2010). When there is inadequate access to water, it leads to a multitude of issues, affecting social well-being, economic development, health, education, and human dignity.

According (MoWIE, 2021), 19% of the 155,482 rural water supply schemes in Ethiopia were found to be non-functional during a recent inventory, amounting to 29,204 non-functional schemes. Of these, 27,476 were on-spot schemes, accounting for 17.67% of non-functionality, while rural piped water supply systems contributed to 1.1% of the total non-functional schemes (1,728 in number). Based on the information obtained from the study areas, the system has been facing a serious water supply deficit, unequal distribution of water among consumers, significant water losses along major sections of the pipeline, some sections experiencing dry pipes, and five kebeles entirely lacking access to water. Additionally, operational challenges, such as the inability to run all five boreholes simultaneously due to frequent pipe bursts, have further hindered water supply. Construction of portable water projects in rural regions is the first step toward expanding access to drinkable water and improving the health of people living there. However doing so by itself could not achieve all of the goals set.

Therefore, this research aims to assess the challenges faced by the Amereba Indato water supply system, with the goal of identifying practical solutions to improve unequal water distribution among consumers, significant pipe leakage along distribution and resolve technical issues. By identifying critical areas for improvement, the study will help develop sustainable strategies to enhance the water supply in this region, ensuring more equitable and reliable access for the population.

1.3. Objectives of the study

1.3.1 General objective

The main objective of this study to challenges faced of rural water supply system in Amereba Indato and to propose the of action to improve system performance and service delivery.

1.3.2 Specific objectives

- To assess the actual demand of the water supply area with in design period of the project
- To assess the hydraulic performance of distribution system by using WaterGEMS.
- To analyze water loss in system.
- To identify and assess the major factor contributing to water loss in the water supply system.

1.4. Research Questions

- What is the total demand of the water supply at the current and end the design period?
- What is the hydraulic performance of the distribution system in the Amareba Indato?
- What is the extent of water loss in the water supply of Amereba Indato?
- What are the major factors to water losses in the existing water supply of the Amareba Indato?

1.5. Significance of the Study

This study holds considerable significance as it assesses the current status of the Amareba Indato rural water supply system by utilizing hydraulic modeling through WaterGEMS software. By comparing the existing system with the modeled ideal, this research identified gaps in the water distribution system and proposes tailored solutions to improve its performance. Specifically, the study has focused on addressing critical issues such as water scarcity, high pipe leakage within factors of contribution to loss in distribution system.

The insights gained from this research will be instrumental in enhancing the sustainability and operational efficiency of the Amareba Indato water supply system. This, in turn, will dependent on it, directly contributing to their health and well-being. Moreover, the data generated by this study may serve as a valuable resource for government bodies or agencies involved in future expansions or upgrades of the system. It will also provide a foundation for further research, helping optimize the system's performance and fostering long-term solutions for similar water supply challenges.

1.6. Scope of the Study

The research evaluated the Water Supply and Distribution Challenges in Rural areas of Amereba Indato Water Supply System by examining its key components against standard engineering evaluation criteria. These criteria included demand, distribution system within pipe size and material, source capacity and pressure at junction points. Additionally, the study identified factors contributing to water loss and leakage within the system. The construction methods of the distribution line, with a focus on the materials used along the pipeline routes, were also examined, alongside major causes of water loss. Discussions with water utility staff provided critical data on the system's operation, construction history, and maintenance records. The research focused on technical aspects and did not include water quality analysis or financial management of the utility.

1.7. Thesis organization

The main body of the Thesis is classified in to five chapters and started with an introduction which includes background of the study, statement of the problem, objectives of the study, research questions, delimitation and limitation of the study and Thesis organization. Literature review is the second chapter which presents the comprehensive development of the problem background. In the third chapter Materials and methods of studies used to materialize the research were included. Result and discussion are the fourth chapter where the analysis of data presented indicated using the proposed methodologies and materials in different forms. The fifth and final part of the study was conclusion and recommendations which the summary and reflections given based on the result of the thesis.

2: LITERATURE REVIEW

2.1. Water Supply

In developed countries, water services are generally reliable, with utility customers expecting high-quality service due to significant investments in infrastructure. The design, maintenance, and operation of these water systems require a deep understanding of flow dynamics in complex systems and the associated energy losses. Pipelines are particularly advantageous solutions, as they directly connect water sources to consumption sites and offer efficient, cost-effective pressurized water transmission (Miao et al., 2017).

However, many developing countries in Africa and Asia, including Ethiopia, struggle with access to clean and safe drinking water. Despite ongoing efforts, these countries face severe water crises, with many inhabitants lacking sufficient potable water (Mekuriaw, 2019). The development of rural water supply schemes is often prohibitively expensive for low-income nations, given their limited resources (Lockwood, 2002; Biswas, 2005). In some cases, large-scale water projects fail, leaving one in four rural water facilities either broken down or poorly functioning. In certain countries, the construction of new water systems cannot keep pace with the breakdown of older ones (Kleemeier, 2000). The challenge of providing reliable potable water is exacerbated by rapid population growth, particularly in countries like Ethiopia. Public utilities frequently fail to supply adequate water and sanitation services, and water must often be transported over long distances via pipelines to reach consumers.

In rural areas, many people travel long distances for hours to fetch impure water for daily use (Chala, 2011). Households are the primary consumers of drinking water, and for rural water systems to be sustainable and reliable, appropriate technology and effective operational methods are required.

Without active community involvement, even the availability of spare parts and repair technicians may not guarantee the sustainability of water systems (Arouna and Dabbert, 2008). Community participation enhances sustainability by allowing local populations to prioritize their needs, choose appropriate facilities and technology, contribute financially to capital costs, provide labor for construction, manage operations, and handle system maintenance and repairs (Harvey and Reed, 2004).

Improving access to potable water in Africa is a priority for governments and stakeholders alike. Ethiopia has launched numerous small- and large-scale water supply schemes, including long-distance water supply projects in rural areas. The Amareba Indato water supply system is one such initiative, designed to address water access challenges in the rural area of Robe. Water supply systems are categorized as large, based on population size, customer service requirements, and living standards (MoWE, 2021). Sustainability is achievable when appropriate operational and organizational structures are established and maintained.

2.2. Water Demand

Water demand refers to the total quantity of water needed for various uses by all groups of consumers, assuming there are no limiting factors. It is a crucial first step and a fundamental aspect in the design of water supply projects. While it is often equated with water consumption, the two terms are not technically synonymous. In many developing nations, theoretical water demand is significantly higher than actual water consumption (Zewdu, 2014).

I). Population Forecasting and Demand Analysis

Population forecasting involves estimating both the current and future population of the area to be served by the water supply system. The key factors considered in population forecasting include the base population, growth rate, and the project's design period. Estimating the population at the end of the design period requires using projection methods, which depend on the development potential of the project area, migration trends, population habits, and the scope for future expansion. Once the projected population is determined, the water supply demand is calculated based on the population forecast and the water consumption rate of the community.

A) Population forecasting

Every planned water supply project has a usable life span during which the recipients receive sufficient services. Therefore, the first step in doing a demand analysis is to project the population. Although there are several approaches to population projection, the geometrical rise, exponential, incremental, and arithmetical increase or arithmetical mean methods are frequently employed for population forecasting. Geometrical increase approaches were applied in this study area.

Geometrical Increase Method

This method is adopted for young and developing towns, where the rate of growth of population is proportional to the population at present. Therefore, it is assumed that the percentage increase in population is constant. It is similar to compound interest calculations. This projection method gives higher value and is suitable for settlements with good potential for development and large scope for vast expansion.

$$Pn = Po\,[1 + (r/100)]^{n} \text{--} \quad (2.3)$$

Where, *Po* - last known population; *Pn* - population (predicted) after '*n*' number of decades;n - Number of decades between *Po* and *Pn* and *r* – Population growth rate = (increase in population/initial population) * 100 (%).

B) Water demand projection

The demand for water supplies rises quickly over time due to factors such as population growth, better living standards, industrial expansion, and climatic change (Wang *et al*., 2016). Thus, provided demand projections are made appropriately, the water supply system that has been designed can support this rising demand.

Different guide lines, design criteria's and immediate transformation plans are developed for town and rural water supply to use for water supply system designing. Demand projection depending on the consumption rate and mode of services are among for which the criteria's is set. MoWE has provided design guide lines in 2006 for town and in 2022 for rural water supply design.

i. Mode of water service for Rural

The most widely used modes of service are the public tap, shared and private yard connections, and house connections. Table 2.2 illustrates the indicative projected mode of service for rural water supply (MoWE, 2022), which takes into consideration the likelihood of the project area's potential for development.

Table 2.1: Forecasted Mode of water servi**ce** (Sources: MoWE, 2021)

Likelihood of Project area`s potential for development	Baseline (%)				End of Design period (%)			
	PF	**YCS**	**YCP**	**HC**	**PF**	**YCS**	**YCP**	**HC**
High	70	20	30	10	50	30	15	15
Medium	80	20			65	25	10	
Low	95	5			80	20		

ii. Per capita water consumption

The amount of water used per person varies depending on a number of factors, including the mode of service, water use culture, level of service, community size, standard of living, quantity and quality of water, water tariffs, and long and seasonal term climate conditions. Rural water supply per capita water consumption is taken as 25 l/c/d, which is too large when compared to MoWE design criteria (Rangaiah et al., 2021). MoWE guidelines and design requirements were also issued in 2022, along with a per capita production that took the previously mentioned elements into account.

Table 2.2: Per capita water consumption pattern over the design period for rural community (Sources: MoWE, 2022)

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

C) **Demand categories:**

Domestic water demand

Domestic water demand refers to the water used in households for activities such as cooking, drinking, bathing, flushing, and laundry (WHO, 2000). This demand varies based on factors such as lifestyle, economic status of the community, climate, type of service provided, cost, and accessibility of the service. Livestock water demand is also included in domestic water demand unless it is used for commercial purposes. Typically, domestic water usage constitutes the largest portion of total demand in most water supply systems (Keshavarzi et al., 2006). Equation 2.5 demonstrates how domestic water demand is determined by population size, service types, and per capita water consumption.

Domestic demand (DD) =P x PCD X PSM-- *(2.5)*

Where; *P*= population in number; *PCD*= Per capita demand in *L/C/D* and *PSM*= Percentage of service mode in *%*.

Non-Domestic Water Demand

Non-domestic water demand refers to the water required for businesses, public institutions, commercial organizations, industries, and livestock. Each category has distinct water needs, which fluctuate throughout the design phase of the water system. To accurately estimate total water demand, these non-domestic needs must be assessed, projected, and added to the domestic water demand. One method for estimating non-domestic water demand is by collecting baseline data on the number of institutions in the project area, as well as the commercial demands of businesses such as markets, hotels, and restaurants. For livestock, when exact population data is unavailable, an allowance of up to 5% of the domestic water demand may be considered, as per the rural water supply guidelines (MoWE, 2022).

Non-Revenue Water (NRW)

Non-Revenue Water (NRW) refers to water that is produced and incurs costs but is not billed due to factors such as leaks, overflows, theft, unauthorized free provision, and internal utility uses. NRW is calculated as the ratio of water lost through unbilled authorized consumption and apparent and real losses to the total water supply. According to the International Benchmarking Network for Water and Sanitation Utilities (IBNET), which tracks data from over 900 utilities in 44 developing countries, the average NRW percentage in these countries is around 35%a significant figure that affects both water demand and operational management (Kanchana et al., 2018).

If NRW is not accounted for, the intended source capacity may prove insufficient to meet consumer needs. The Ministry of Water and Energy's (MoWE., 2021) design guidelines for rural water supply outline specific NRW allowances for various types of water schemes, including on-spot systems and rural piped systems (RPS), depending on the distance, with specific classifications provided in Table 2.5 of the guidelines.

Table 2.3: NRW classified for rural water supply based on scheme type

Water supply system size	Planned	NRW at start	NRW at end (%)
On spot schemes	Up to 500	5	5
RPS with pipe less than 15km	Up to 10000	18	15
RPS with pipe greater than 15km	> 10000	20	15

The percentage of NRW can be also easily determined using water production and consumption data as indicated in the equation 2.6.

$$NRW\ (\%) = ((YWP - YWC) / YWP)*100 \text{ -- } (2.6)$$

Where: *NRW* = non-revenue water (in % of production); *YWP* = Yearly Water Production (m^3 /year); *YWC* = Yearly Water Consumption (m^3/year).

C). Water supply demand variations

The average water demand changes on hourly, daily or weekly bases due to various factors among which particular occasion such as holidays, season (Bergel et al., 2017), price of water consumption (Koledo, 2016) and climate (Chang et al., 2014).

$$ADWD = DWD + NDWD\ (PWD+IWD+CWD+LWD) + NRW \text{-------------------------------} (2.7)$$

Where: *ADWD*-Average day water demand, *DD* –Domestic water demand, *NDWD*-Non-domestic water demand, *PWD*-Public water demand, *IWD*-Institutional water demand, *CWD*-Commercial water demand, *NRW*-Non –revenue water and *LWD*-Livestock water demand.

Maximum day demand: One of these demand circumstances, known as maximum day demand (MDD), is the greatest amount of water needed in a single day over the course of a year. The reality that different regions have widely differing climates and that there is year-round variation in water use. Peak seasonal parameters are anticipated to be impacted by climate change, with greater temperature differences leading to higher water demands during the warmest times of the year. Which is defined as the quotient of maximum day demand over average day demand, will be applied in this case.

MDD = MDF X ADD--- *(2.8)*

Where: *MDD*-Maximum day demand, *MDF* –Maximum day factor and *ADD*- Average day demand.

Peak hour demand: Water demand also varies greatly from hour to hour during the day. This is termed as Peak hour demand and represents the highest hourly demand in a single day. The hourly variation in domestic water demand during the day is much greater and practically, two peak periods can be observed: One in the morning and one late in the afternoon. The peak hour demand can again be taken into account with the use of hourly peak factor (PHF), which is expressed as the quotient of Peak Hour Demand over a Day of maxim Demand, often expressed as peak of the peak expressed as quotient Peak hour demand over Maximum day demand.

PHD= PHF X MDD-- *(2.9)*

Where: *PHD* - Peak hour demand ,*PHF* – Peak hour factor,*MDD- Maximum Day Demand*

The peak hourly demand is usually estimated by adding allowances to ADD or MDD based on population size.

Table 2.4: MDD and PHD demand factors according to 2021 rural design manual (Source: MoWE, 2021)

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(for Population>15,000)
	2.0 to 2.5 of MDD(for Population<15,000)

For town water supply the study guide line produced by (MoWE, 2006) fixes the hourly variation based on the population size.

Table 2.5: Peak hour demand factors used for water consumption (Sources: MoWR, 2021)

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

2.3. Hydraulic Analysis

Hydraulic modeling has been described by numerous scholars as a crucial tool for understanding and managing water distribution systems. According to (Thomas M. Walski, Donald V. Chase, and others .,2003), hydraulic modeling involves the creation, validation, and execution of network models for distribution systems, providing valuable insights into their operational processes. They argue that water distribution modeling is vital for the effective design and management of water distribution systems, ensuring that they can deliver water safely, efficiently, and reliably to communities in the future. The key governing principles of fluid flow in pipe networks include the conservation of mass, conservation of energy, and the friction-induced head loss within pipes. These principles are fundamental to understanding and optimizing the flow within water distribution systems.

A. **Conservation of Mass:**

Whatever the flow channel, the energy difference between two sites in a network must remain the same in order to comply with the conservation of energy principle (Walski et al., 2003). Conservation of mass states that, for a steady state system, the flow into and out of the system must be the same (Lansey, K, 1991).

$$\sum Q_{in} - \sum Q_{out} = Q_{demand} \quad \text{(2-10)}$$

$$\Sigma Q_{in}\, \Delta t = \Sigma Q_{out}\, \Delta t + \Delta V_S \quad \text{(2-11)}$$

Where: ΣQ in: flows in pipes entering the node or the total flow into the node. ΣQ out: flows in pipes exiting the node or the total demand at the node. Q demand: the user demand at that location. ΔV_S: is the change in the storage. Δt: is the change in time.

B. Conservation of Energy:

Regardless of the flow path, the energy differential between two nodes in a network must be the same in order to adhere to the conservation of energy principle (Walski et al., 2003). Bernoulli's equation is the name given to the energy equation. For hydraulic analysis, this principle can be represented in terms of a head as follows:

$$\frac{P1}{\gamma} + z1 + \frac{V1\ 2}{2g} + HG = \frac{P2}{\gamma} + z2 + \frac{V2\ 2}{2g} + HL \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(2-12)}$$

Where: P: is the pressure (Ib/ft^2 or N/m^2) , γ: is the specific weight of the fluid (Ib/ft^3 or N/m^3), z: is the elevation at the centroid (ft or m) ,V: is the fluid velocity (ft/s or m/ s) , g: is gravitational acceleration (ft/s^2 or m/ s^2) ,HG: is the head gain, such as from a pump (ft or m) and HL: is the combined head loss (ft or m).

C. Friction loss in pipes:- Friction head-loss has a major impact on flow across pipeline networks. The Darcy-Welsbach, Hazen-Williams, and Manning equations are the three empirical equations that are most often employed. Pipe velocity, length, diameter, and roughness are all related to head or friction loss in pipes by these three equations.

The Hazen Williams equations, which are expressed as follows, are the ones that modeling software uses the most frequently to calculate pipe-friction head-loss.

hl = 10.7 L D4.87 *(Q/C) 1.85 -- (2.13)

Where: hl= head loss due to friction (m); L= Length of pipes (m); D= Pipe Diameter (m); Q= Pipe flow rate (m3 /sec) and C= Hazen-Williams Coefficient (Dimensionless).

D. Hydraulic Modeling

According to Thomas M. W. Alski, Donald V. Chase, and others (2003), water distribution modeling serves various purposes, including long-term master planning, fire safety studies, water quality assessments, energy management, system design, and daily operations such as operator training, emergency response, and troubleshooting. Several hydraulic model types are available for analyzing and assessing water supply distribution.

EPANET: Developed by the National Risk Management Research Laboratory, this computerized simulation model helps water utilities understand the movement and changes of treated water within their distribution systems, ensuring compliance with regulations and customer satisfaction. As noted by Abu-Mahfouz, Hamam, Page, and Djouani (2016), EPANET simulates hydraulic behavior and water quality over extended periods in pressurized pipe networks. It is open-source software that predicts dynamic hydraulic and water quality behaviors in drinking water distribution systems.

Bentley WaterCAD: Manual procedures for hydraulic analysis of large water networks are now impractical due to their time-consuming nature and susceptibility to errors. Since the mid-1960s, computer models have been available for analyzing and designing water distribution systems (Ormsbee and Chase, 1988). The significance of modeling and analyzing water distribution networks is paramount, as its wider application among water utility companies has led to the development of various software solutions with enhanced capabilities for water supply management. Consequently, Bentley's OpenFlows WaterCAD has emerged as a powerful water distribution network modeling tool.

he application provides unlimited undo and redo functions, a merging tool for nearby nodes, automatic element labeling, aerial views, dynamic zooming, and multiple layer support for graphical editing (Bentley, 2014; Izinyon & Anyata, 2009). Its hydraulic modeling capabilities include steady-state and extended period simulations, and it supports various data formats, including traditional shapefiles, DXF files, spreadsheets, databases, and Open Database connections. It also supports Oracle Spatial and GIS-ID properties for maintaining record associations, graphical SCADA elements, customer meter elements, and automatic demand allocation from geospatial data. To accurately represent the water distribution network, WaterCAD can extract elevation data from DEM, TIN, and shapefiles, as well as CAD drawings and surfaces (Bentley, 2014).

H2ONET: H2ONET and H2OMAP (2015) are commercial software integrated with GIS, designed for the design, analysis, and optimization of various types of water distribution networks. They are particularly useful for leakage detection, fire-flow and hydrant analysis, and cost optimization. A standout feature of this software is its programmed and automated online SCADA interface. For simple water distribution systems with limited pressure zones, these programs yield similar results when addressing comparable problems. Therefore, model evaluation often hinges on comparing the presentation differences of model results—both graphical and textual—as well as the additional capabilities and features each package provides.

Cybernet Software: Developed and distributed by Haestad Methods, Cybernet offers hydraulic modeling features similar to WaterCAD, but it operates within the AutoCAD environment for Windows. Essentially, Cybernet integrates the functionalities of WaterCAD, allowing users access to all features of AutoCAD Release which enhances graphical compatibility with other organizational drawings. This environment not only facilitates modeling results but also provides greater graphical flexibility for producing high-quality graphical outputs.

WaterGEMS: WaterGEMS is a comprehensive and user-friendly water distribution modeling application that can be integrated with ArcGIS, AutoCAD, and MicroStation, or used as a stand-alone application. Its ability to operate across different platforms gives WaterGEMS an advantage over OpenFlows WaterCAD. Building on the foundation of WaterCAD, WaterGEMS allows users to seamlessly upgrade to a more advanced modeling environment. This application can pull data from a GIS database, run various scenarios, and push data back to the GIS database.

WaterGEMS V8i serves as a multi-platform hydraulic and water quality modeling solution for water distribution systems, featuring advanced interoperability, geospatial model-building, optimization, and asset management tools. It provides an intuitive environment for engineers to analyze, design, and optimize water distribution systems, addressing tasks ranging from fire flow and constituent concentration analyses to energy consumption and capital cost management. Additionally, WaterGEMS V8i effectively manages water system data, time-series hydraulic results, current and future scenarios, and other core infrastructure data within the same GIS environment (Mehta et al., 2017).

Overall, WaterGEMS software has been selected for this study based on various criteria. The unified, shared environment offered by WaterGEMS' connection edition enhances output, teamwork, and project performance, making it possible to model nearly every aspect of the distribution system. By using WaterGEMS, assistance is provided in assessing and operating water infrastructures through a decision-support tool (WaterGEMS, 2018).

The hydraulic analysis capabilities include Simulation of system pressure, flow, and demands, and their behavior over time, assessment of the sizes of pipe, pump, and computer system head curves, evaluation of tank, pump, and valve behavior within the system, monitoring water loss and leakage from the network. Model calibration can be conducted manually or with the help of Darwin Calibrator techniques, and the software produces fully configurable graphs, charts, and reports. E. Model

Calibration and Validation

Model simulation tools may contain uncertainties from various sources, leading to deviations between their outputs and real measured data. This necessitates adjusting several model-setting parameters, a process known as model calibration. Calibration continues until the model accurately reflects the actual system. Validation, on the other hand, involves comparing the calibrated model's behavior to that of the real system. The uncertainty in results from water distribution network modeling arises from multiple factors, which (Hutton et al., 2014) classify into three categories: structural uncertainty, measurement uncertainty, and parameter uncertainty.

Typically, changes in pipe roughness become more apparent during steady-state calibration, while demand distribution changes are more evident in extended period calibration. Thus, one effective calibration strategy involves using pressure and flow test results from the physical network to refine pipe roughness parameter values, followed by utilizing telemetry data on flow, pressure, and water levels to further adjust the demand distribution (Ormsbee & Lingireddy, 1997).

$R\ 2 = \sum (x-\overline{x})\ (y-\overline{y})\ /\ \sqrt{\sum (x-\overline{x})\ 2 \sum (y-\overline{y})\ 2}$ --- (2.14)

Where: R2=correlation coefficient; X and y are the simulated and field measurement pressure values respectively and $\bar{x}$ and $\bar{y}$ are the average value of simulated and field measurement pressure values respectively. Coefficient of determination (R2) describes the degree of linearity between simulated and measured data. R2 range between 0 and 1, which describes the proportion of the variance in the measured data and is explained by the model, higher values indicate less error variance. Typically, $R^2 > 0.5$ is considered acceptable (Santhi et al., 2001).

In general for this study of water supply and distribution challenges in rural areas: The case of Amereba Indato Water Supply System it is crucial to project out the specific objectives stated and follows the WaterGEMS18 procedure's or steps to model water distribution systems and analyze overall objective based on the output of the results of the model.

2.4. Water loss Analysis

Leakage represents one component of total water loss within a network, encompassing physical losses from pipes, joints, and fittings, as well as overflow from service reservoirs (WHO, 2001). The following are the fundamental principles of water balance components, as defined by (Lambert 2003) and (Jose Sardinha. 2017). System Input Volume is refers to the annual volume of water entering the distribution system.

Unbilled Authorized Consumption: This includes the annual volume of water that is either metered or unmetered but is actually consumed by clients, the supplier, or those who are implicitly or explicitly authorized to use water. This category covers social commitments and legitimate uses by the fire service. Additionally, it incorporates the volume of water that is exported and existing leaks at client service connections.

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

Real Losses (Physical Losses): These consist of leakage from all parts of the system and overflow at storage tanks, often resulting from inadequate operations and maintenance, insufficient active leakage control, and poor infrastructure quality.

Apparent losses (commercial losses): caused by inaccurate metering, data handling errors and illegal water tapping.

(NWR) =Water losses + unbilled authorized consumption …………………………….. (2.15)

Water losses =Real losses + Apparent losses…………………………………………… (2.16)

Unaccounted-for water (%) = $\frac{(\text{Production} - \text{metered use})\text{x } 100\%}{(\text{Production})}$ ……...………………..….(2.17)

Approaches of water losses determining

There are two primary methods for estimating water loss: the top-down annual water balance method and the bottom-up method, which is based on minimum night flow analysis. The bottom-up approach often fails to yield accurate assessments of real losses during the night because consumers may continuously open taps to fill storage tanks in intermittent supply systems. As a result, top-down water audits are generally recommended in developing countries (AL-Washali et al., 2016).

According to (Lambert 2000), the actual volume of water lost from a distribution system varies between utilities and is influenced by local factors such as topography, the length of mains, the number of connections, service standards, and the effectiveness of system operation and maintenance. Unauthorized connections can affect water quality, flow, and pressure, while inadequate or missing backflow prevention can lead to contamination. (Dighade et al. 2014) suggest that when analyzing current water loss in the distribution pipe network, it is appropriate to relate the percentage of losses to both the distribution system and the total pipeline length.

Unavoidable Annual Real Losses (UARL)

Unavoidable Annual Real Losses (UARL) represents the minimum level of leakage that any water distribution system can achieve. For systems with customer meters located close to property boundaries, the UARL is determined by the density of connections per kilometer of mains (Farley & Costanza, 2010). The Unavoidable Annual Real Loss, also referred to as the Minimum Achievable Annual Physical Losses (MAAPL), is defined by the International Water Association (IWA). Its formula has been formulated for practical application using pre-determined pressure (Farley & Costanza, 2010).

$$UARL(liter/day) = (18*L_m+0.8*N_C+25+L_p)*P \quad (2\text{-}18)$$

Where UARL= Unavoidable annual real loss L_m = mains length (km) N_c= number of service connection L_p= total length of private pipe, property boundary to customer meter (km) P= average pressure (m)

2.5. Major factors to water loss

Many factors contribute to water loss in distribution systems, including aging infrastructure, poor construction quality, excessive pressure, corrosion inside and outside pipelines, water meter inaccuracies, and inadequate guidelines for spring water supply, low community awareness regarding sources and pipelines, and insufficient operational and maintenance procedures.

Absence of Water Auditing System: A water audit employs accounting principles to quantify the amount and location of water loss within a distribution system, as well as the associated costs due to leaks, theft, metering inaccuracies, and unauthorized withdrawals. This process helps identify problem areas and vulnerabilities while enhancing understanding of how water transforms once it leaves the source (Vermersch and Carteado et al. 2016).

Pipe Network Upgrades: The aim of network upgrades is to provide superior infrastructure management, which allows utility practitioners to reduce losses, improve network performance, and efficiently assess pressure and flow profiles, especially in locations were soil erosion is a problem.

Community Behavior: Water loss is not only a technical issue but also a socio-cultural one that necessitates changes in community attitudes and behaviors concerning water usage. Alongside technical measures, initiatives have been implemented to address illegal connections through walkthrough surveys, aiming to legitimize and integrate unauthorized connections into the network.

Pressure Management System: Implementing a pressure management system is a key strategy for significantly reducing water loss in distribution networks. Two commonly used components in water distribution systems are variable speed pumps (VSPs) and pressure-reducing valves (PRVs). Pressure management has become recognized as essential for optimizing the management of water supply and distribution systems, offering substantial potential to enhance efficiency, lower costs, and mitigate the risks associated with water scarcity (Koop & van Leeuwen, 2015)

3: MATERIALS AND METHOD

3.1. Description of the study area

The Amareba Indato Water rural Supply System is located 225 kilometers from Addis Ababa in the Arsi zone, Robe, and Tane districts of the Oromia Region. The target areas are situated to the southeast of the district offices of Robe. Geographically, the project area lies between the Wabe and Hulul Robe gorges, east of Chilalo Mountain, and is bounded by 821739m to 881280m North and 555414m to 631106m East. The system is currently serving a total of 162,518 peoplem in town 17697 and rural kebele 144,821.The system was built for 25 rural kebeles.

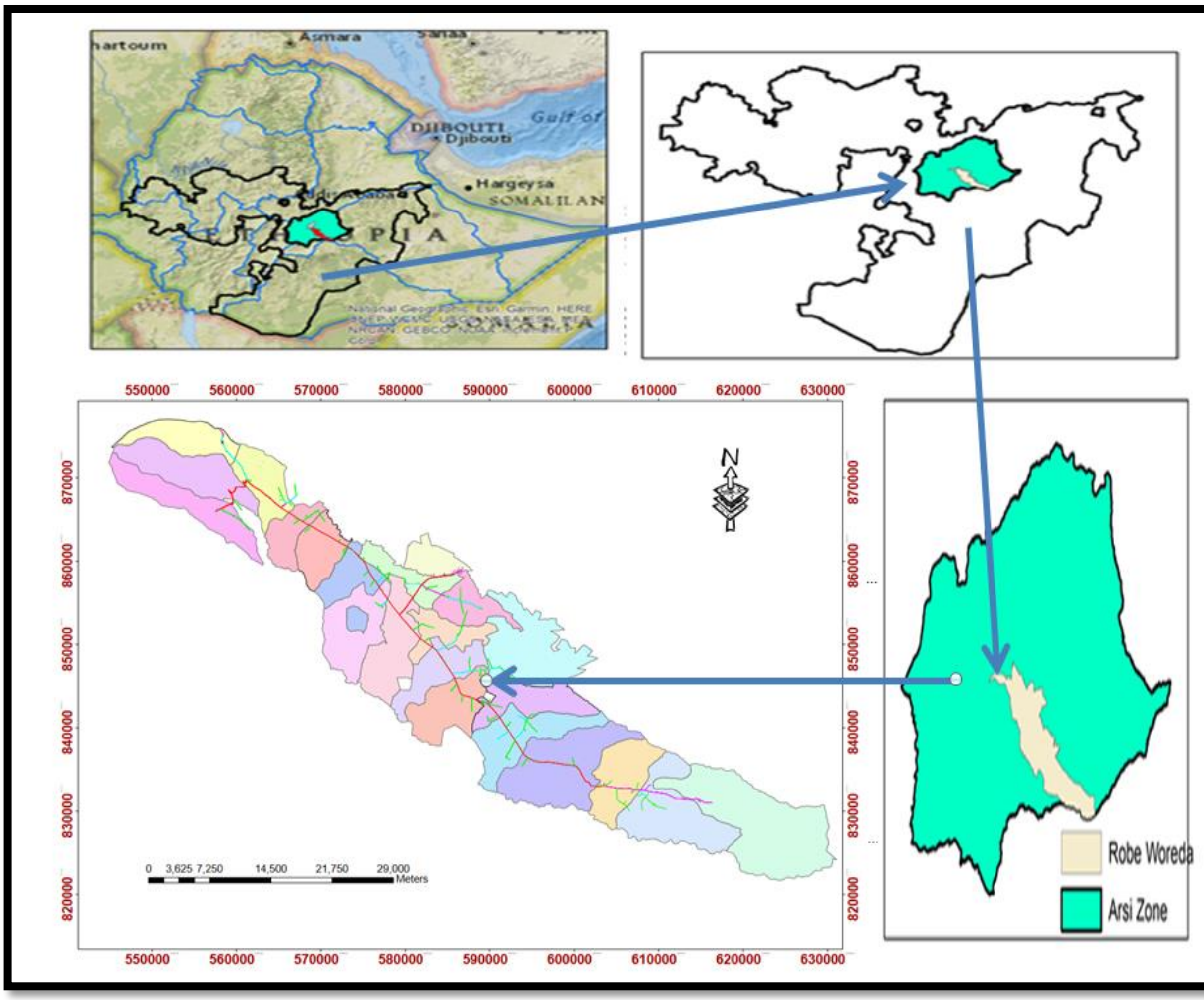


Figure 3.1: Location of Amareba Indato Water Supply Project

3.1.2. Topography and Drainage

The landscape of the areas of study was gently or smoothly descending to the east and southeast. Significant elevation changes occur in the NE, SW, and S regions of the region. In the middle part of the target area, the terrain is plain with little variation in height, particularly around the Robe-Habe area. The research area is separated by water; most of the catchment area and its environs are high, ranging from 4050m in the Galema Mountain range to 1400m along the Wabe River.

3.1.3 Climate of the Project Area

According to data from the Woreda agricultural office, the study area was primarily classed as warm temperate, with a small fraction falling into the temperate climatic category. The mean annual temperature is between 14 and 17 °C, and the mean annual rainfall ranges from 24000 - 3000 mm.

3.1.4. Existing Water Supply System

The water supply system involves 103 public water points spread,29 cattle trough, 35 institutions tap and 222.424km of different types and sizes of pipe line. The existing source water supply comes from two (2) springs and five bore holes. The two springs of Amereba Indato water supply system supposed to serve 11 kebeles with the total population of 77,413 as 2024 prediction. The distribution network of the spring sources covers a total length of 80.9km.On the other hand, the total population connected to the five boreholes are 85,106 as of 2024 prediction. It was designed to cover 14 kebeles and a total network length of 141.52 km. Table 3.1, show the location and discharge capacities of the existing water sources of Amereba Indato rural water supply system.

The two springs and the five boreholes supply 30 l/s and 110 l/s, each, for the water supply sources. In addition, the system includes three 500m^3 service reservoirs, one 300m^3 service reservoir, two 100m^3 service reservoirs, one 75m^3 service reservoir, two 10m elevated 100m^3, three 20m elevated 250m^3 balancing reservoirs, one 100m^3 wet well, and auxiliary buildings. The system was administered by utility established at Robe woreda level within all office structure.

Table 3.1: Locations and discharge capacities of existing source

Name of Source	Location			Discharge(l/se)
	Easting	Northing	Elevation	
Amereba Spring	557582	866008	2554	24
Jawi-sire Spring	557884	875353	2826	6
Misranje Temama BH1	583409	858479	2393	22
Misranje Temama BH2	584642	858579	2385	22
Misranje Temama BH3	585870	858932	2375	22
Misranje Temama BH4	586329	859188	2376	22
Misranje Temama BH5	586769	859291	2372	22
Total				140

3.2. Materials used

Different types of materials were used for the collection, analysis and data collected from different sources and directly from project sites. The materials used are shown in Table 3. 2.

Table 3.2: Materials/Tools used for data collection and analysis

S/No	Material/Tools type	Purpose
1	GPS	Used to Collect location data at site level
2	Pressure gauges	Used to Collect pressure readings at water points
3	Meters	Used to measure length wherever required
4	Digital Camera	Used to take pictures /photos during data collection
5	WaterGEMS software	Used for modeling and to conduct hydraulic analysis of the distribution system
6	Arc GIS 10.4	Used to prepare location Maps and shape files to use as integral part in Water GEMS for hydraulic analysis
7	AutoCAD 2018	Used to prepare drawings of the system components
8	Excel spreadsheet	Used to organize elevation data, nodal base water demand requirement of distribution network simulation and for manual pressure validation

3.3. Data Collection methods

The source of data collected for this study involved both in primary and secondary data.

A) Primary Data Collection Methods

Primary data were collected from a sample community using various methods, including in-person interviews, field measurements and questionnaire assessments regarding water supply during instances of water loss. Key informant interviews were conducted with individuals specifically chosen from the community and the water supply utility. Questionnaire-based data were gathered from community members, technical staff, and utility management.

Through key informant interviews, information was obtained about the causes of water loss, technical challenges, and other efforts to address issues related to utility services. Measurements of pressure at nodal water locations were made in order to calibrate the water model instruments. Pressure gauge fixed at institutional taps and public fountains to measure the pressure and adequate samples of the existing nodal data were taken for hydraulic performance studies, using calibrated pressure gauges. Field surveys were conducted, and data were recorded using GPS, water meters, along with photographs of relevant sites and infrastructure.

Qualitative data were collected in the sort of interviews and questionaries' involved participants from the Amereba Indato water supply, user groups, and professionals such as woreda experts and artisans engaged in project implementation. The sampling approach consisted of selecting 45% of the kebeles served by the water supply and 5% of beneficiary at house hold level. The kebeles which water not get is not included in these sample sizing, those are one rural town and four rural kebeles. Depending on the two source (Springs and borehole) 9 kebele is selected from 20 kebeles for served by the system. Those kebeles Masarnje Oda, Jawi Sire,Korefta Lamlem,Gado Lamen , Sadiqa Qarsa, Abo Ali Sadiqa 01,Jena Babuqo and Sabro Cafe. The beneficiary kebele selected was the total number of households (3393) and the level of precision (5% is taken). Thus, the sample size was 358. The sample size was determined using Yamane (1967):

$$n = \frac{N}{(1+Ne^2)} \text{ -- } (3.1)$$

Where: n = sample size; N = total number of households; and e = margin of error (expressed as a decimal)

$$n = \frac{\mathbf{3393}}{(\mathbf{1+3393}\boldsymbol{x}(\mathbf{0.05})^{\mathbf{2}})} = 358$$

B) Secondary data collection

Secondary data was gathered from records kept in the administrative offices of the Robe district, the Amareba Indato water utility, the Oromia water and energy bureau, survey reports, census reports, topographic maps, system maps and published documents.

3.4. Data Analysis Methods

3.4.1. Water Supply Demand

The basic elements used for the analysis of demands are forecasted population, unit rate consumption, demand category and mode of the service adopted. For population projection different methods adopted as per the beneficiary under consideration which includes the socio-economic factors in to consideration. According to (MoWE,2022) guide line rural of Water Resource different alternative population projection method mentioned but geometric progression method was applied in this study case because government offices used it taking in to account high reproduction trend of the community under consideration. Utility has identified the current number of users in each beneficiary village and hence it was used as base population for projection by using Geometrical increase method.

The per capita consumption is taken from design manuals for rural areas and rural towns, respectively. Regarding mode of service medium level distribution proposed based on the likelihood of project area`s potential for development as described in rural manuals. Once domestic demand determined, using equation 2.5 in chapter two, non-domestic demands were computed from it in percentage.

$$\textit{Domestic demand (DD)} = P \times PCD \times PSM \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. \quad (3.2)$$

Where; *P*= population in number; *PCD*= Per capita demand in *L/C/D* and *PSM*= Percentage of service mode in *%*.

Total day water demand was eventually derived by adding up all of the domestic and non-domestic demands.

Demand variation: Due to different factors, the demand can vary over the hours of the days, weeks, monthly bases and seasons. Therefore, allowances were made to average day water demand to address for any water consumption going beyond the average demand. According to design criteria published (MoWE, 2021) for rural water supply the daily and hourly factors recommended depending on guidelines and mode of service adopted.

Table 3.3: Maximum day and peak hour factors in terms of average day demand

Demand Parameters	**Factor of average demand**	
	minimum	**Maximum**
Maximum day demand	1.1	1.5
Peak hour demand	1.5	2.5

3.Water Source

A water source capacity needs to be sized for the total water demand including unaccounted-for water. Peak seasonal demand, taking into account the peak day factor, was determine the magnitude of the source capacity.

Network Pipe

Maximum daily demand and Peak hour demand consideration in the system of the transmission and distribution was analysis. The flow rate, maximum design pressure, transient pressure, and external pressure serve as the foundation for the design. Based on these the existing network pipe the size and type of pipe was chosen. Model simulation of the pipe network was conducted using existing pipe line data (length between nodal point's, pipe diameters, pipe material, fitting along the line), allocating demand and inserting nodal elevations to WaterGEMS software. According to MoWE design criteria, the performance of the pipe network checked based on the analysis result where pressure head and velocity expected to fall in the range 10m-80m and 0.6-2.6m/s, respectively. With regard to velocity the result was checked for minimum and maximum in the network. Several factors were considered at analyses depending on pipe material. Pipes of material show in Table 3.4 within the diameter of pipe

Table 3. 4: Type of Pipe and length in Amereba Indato water supply system

Types of pipe	PIN(mm)	Length of pipe(M)	Coverage in system(%)
uPVC DN 400	10	220037	98.92
Ductile Iron DN 200	10	2261	1.02
Galvanized steel pipes DN100		126	0.06
Total		**222424**	100

(source: Utility water supply office)

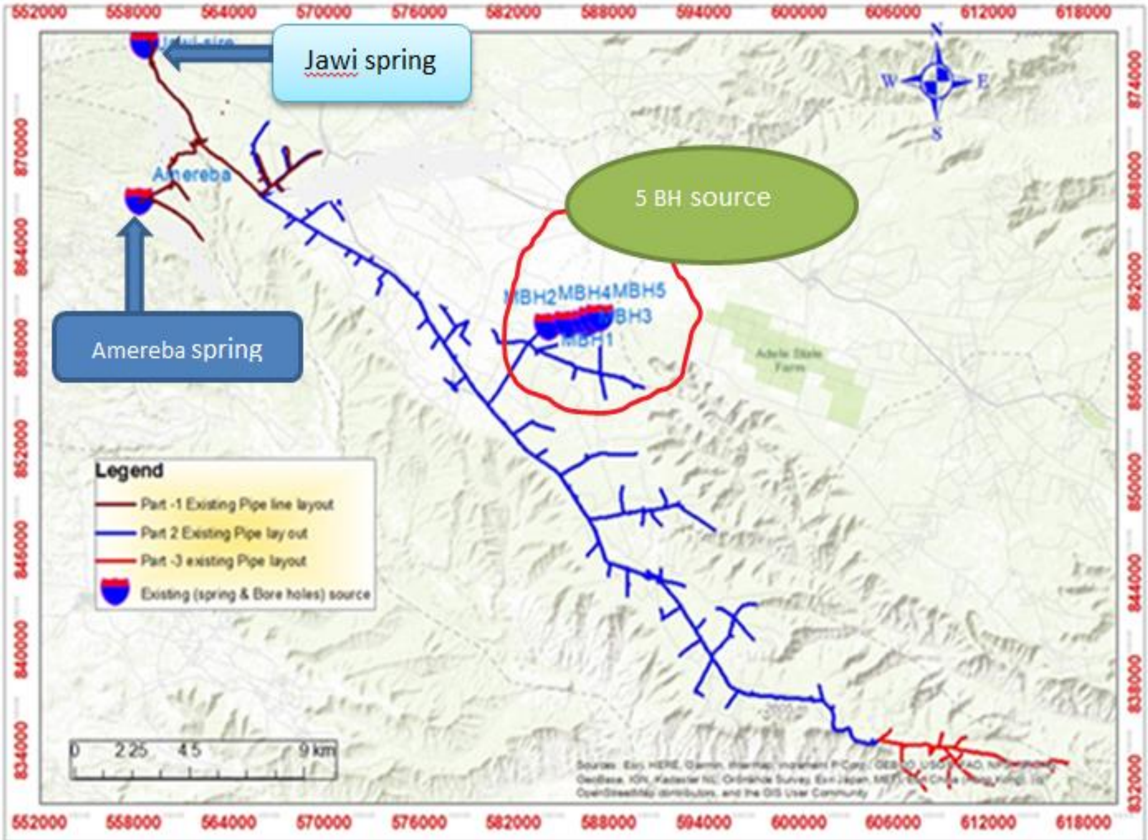


Figure 3.2: Existing Water supply of Amereba Indato pipe network

Electro mechanical equipment:-used in the transfer of water from source to the required reservoir stations. Amereba Indato water supply system has five pumping stations. It is well known that well data is very important to analysis the water supply system in electro mechanical parts. All water well data is received from borehole data report and from field survey to use as input data for analysis of electromechanical parts. Head loss calculation for boreholes input data and analysis summary. The WaterGems was used to check the capability of the equipment's with the input data of existing pumps (head, discharge and pump efficiency), topographic and pipe data. From hydraulic analysis of actual pump out put , its duty point or working point shifted from manufacturers pump curve set and this lead to a significant decrease in the pump's overall efficiency and The existing pump and electro mechanical was in at good condition and also used up to designed period. Existing Pumps and power plants in five Pumping stations show in Table 3.5 and 3.6 within the Existing head and efficiency.

Table 3.5: Existing Pumps and power plants in five Pumping stations

Bore Hole stations Reservoirs	Location		well head Elevation (m)	Available Discharge (L/se)	Depth (m)	DWL (m)	Pump Position	Casing Diameter	Storage Tank Elevation
	Easing	Northing							
Misrange 1	583412.4	858566.4	2365	22	197	62.37	163	8"	2,433.9
Misrange 2	584648.2	858670.2	2365	22	199	63.72	166	8"	2,433.9
Misrange 3	585783.7	858714.5	2360	22	196	65.84	166	8"	2,433.9
Misrange 4	586235.9	858955.8	2360	22	200	68.48	169	8"	2,433.9
Misrange 5	586660.7	859056.2	2353	22	209	65.48	166	8"	2,433.9

Table 3.6 : Existing head and pump efficiency

S. N0.	Boreholes ID	Discharges (L/S)	Head (m)	Efficiency of Pump (%)	Efficiency of Motor (%)	Power of Motor (KW)
1	Misrange 1	22	159	80	95	45
2	Misrange 2	21.95	164	76	93	45
3	Misrange 3	22	174	75	90	45
4	Misrange 4	22	181	75	90	45
5	Misrange 5	22	186	77	92	45

3.4.2. Assessment of Hydraulic performance

Hydraulic Modeling Tools

The hydraulic modeling program, WaterGEMS, was used to gain a comprehensive understanding of the water distribution network's operation and to analyze pressure, velocity, and head loss. The simulation also aimed to evaluate and enhance the current water distribution system. Base demands were allocated on the population size of each village, as detailed in Annex-1. For systems with reservoirs or tanks, essential data like tank elevation, reservoir shape, and location were incorporated into the model. Similarly, pump data such as flow rate, head, and efficiency were included to simulate the system.

Steady-state analysis, on the other hand, is a static simulation that models the system at a specific point in time, assuming that all the system's parameters (e.g., demands, tank levels, pump operations) are constant. This type of analysis is useful for; Evaluating the system's, hydraulic conditions, such as pressures and flows, under a specific set of conditions, sizing pipes, pumps, and other components, analyzing the impact of changes in the system, such as the addition of new demands or the expansion of the network.

For rural villages therefore it is better to use steady state for it can help in the evaluation of the system and do not need multiplier factors over the times. For static pressure evaluation in distribution line zero flow condition adopted and for maximum flow condition a factor of two used to multiply the base or average demands.

A) Building a Data Model

The model was developed to simulate the network's normal operating conditions. The first step involved compiling the necessary data for the model, including physical attributes, demand distribution, network layout, and other relevant information. The modeling process consisted of gathering input data, creating the model in WaterGEMS, entering data (e.g., elevations, XY coordinates, base demand, pump, tank, and pipe specifications), testing the model, conducting hydraulic simulations, and analyzing any issues. The following data were entered into the nodes:

- Pipes: Diameters, lengths, material type, and friction coefficients.
- Tanks: Base, minimum, and maximum elevations, along with tank diameters.

- Pumps: The pump curve, which defines pump operation.
- Reservoirs: Elevations for system analysis.
- Elevations and Base Demand: For distribution system analysis.

C) Model Calibration and Validation

Model Calibration:

The hydraulic performance evaluation was conducted using WaterGEMS software. The model is considered calibrated when its output closely matches actual field measurements. To achieve this, the pipe roughness coefficient was adjusted until the difference between the field-tested data and the model's projected values was within an acceptable margin. According to Tomas M. et al. (2003), this acceptable range is ≥85%. Accurate model calibration requires careful selection of sample size and measurement locations.

1. Sample Size: The sample data collected in any water supply network is related to its links and nodes. The water consumption nodes of Amereba Indato water supply project are water points of thirteen faucets where pressure data was collected. According to AWWA (2005) the sample size for pressure at nodes >=2%. Water points served as nodes for the gauges used to measure pressure in this study.

2. Sample Location: Walski et al., (2003), recommends that it is better to conduct random sampling where the point have adequate length from boundary nodes in the networks and have high pressure and flow rate. Since the majority of the pipe network is a branching system and the water points are between 0.2 and 2 km apart from the main distribution line, inclusive point data were taken to ensure that representative and confidential data was gathered.

Model Validation:

Once calibration was complete, the model's simulated results were compared with field measurement data using the correlation coefficient (R^2). Values for R^2 range from 0 to 1, with $R^2 > 0.5$ being acceptable. The closer the R^2 value is to 1, the stronger the correlation between the model and actual data by used equation 2.14.

$$R2 = \sum (x-\bar{x})(y-\bar{y}) / \sqrt{\sum (x-\bar{x})2 \sum (y-\bar{y})2} \quad \text{……………………………….(3.3).}$$

3.4.3. Water Loss Analysis

The analysis of water loss in the Amereba Indato rural water supply system was conducted by evaluating several key metrics, including the percentage of non-revenue water (NRW), water loss per unit length of pipe, and water loss per connection, among others. This assessment focused on ensuring that water reaches the desired level without encountering dry pipes.

Percentage of Non-Revenue Water Losses:

Annual water production and consumption (billed water volume) for the system from July 2022 to June 2023 were determined using data collected from the utility's water service office. Water loss and unaccounted for water percent were used calculate the overall percentage of water loss in the system based on the information.

During the field visit, it was found that the utility had no documented data on average leak flow, the number of reported bursts, or average leak duration. Consequently, the physical water losses in the mains were assessed using the available data, taking into account the system’s minimum attainable annual physical losses, also known as unavoidable annual real losses (Farley et al., 2008).

$$UARL\ (liters/day) = (18\ Lm + 0.8 * Nc + 25 * Lp) * P \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.4)$$

Where, Lm is mains length (km), Nc is number of service connections, Lp is total length

of private pipe, property boundary to customer meter (km), P is average pressure (m)

According to information gathered from the Amereba Indato Long distance water service office, the system encompasses the following in 2024:

- Total length of pipe line 222.424km and 192.816km was analysis but 29.608km dry pipe only.
- From 183 tolal number of connections 167 was analysis and 16 dry.
- Average pressure was taken from the actual observed pressure and the average value of 65.1m was adopted. This average pressure gained from competed model results.

However, based on the quantity of connections and pipe length at water system flow throughout the pipe, the overall water loss was determined and represented as a percentage (NRW).

3.4.4 Major Factors Contributing to Water Loss

The key factors contributing to water loss were evaluated from management and technical and communities' perspectives, focusing on construction and planning, operation and maintenance practices, cost drivers, policies, and strategies. To investigate the causes of leakage, field visits were conducted. During these visits, interviews were held with system users and questionnaires were conducted with user and utility water supply professionals to gain insights into common issues affecting the system.

3.5. Conceptual Framework

In order to create a more sustainable water supply system, this study primarily focuses on two main areas: evaluating the overall changes of the water supply system under consideration and identification of interventions through system solving problem.

Gathering data such as Population, Pipe nodal and link, , sample survey data questionnaire and interviews was the primary step to organize the research works. Using these data, hydraulic simulation of water supply system and capacity determination of components conducted to identify gaps based on standard set of parameters, relative capacity compared to existing components and against standards.

Finally the research was winded up with potential recommendation that proposed to bridge system problems identified. A synopsis of the entire study protocol is provided in the framework as show in Figure 3.3.

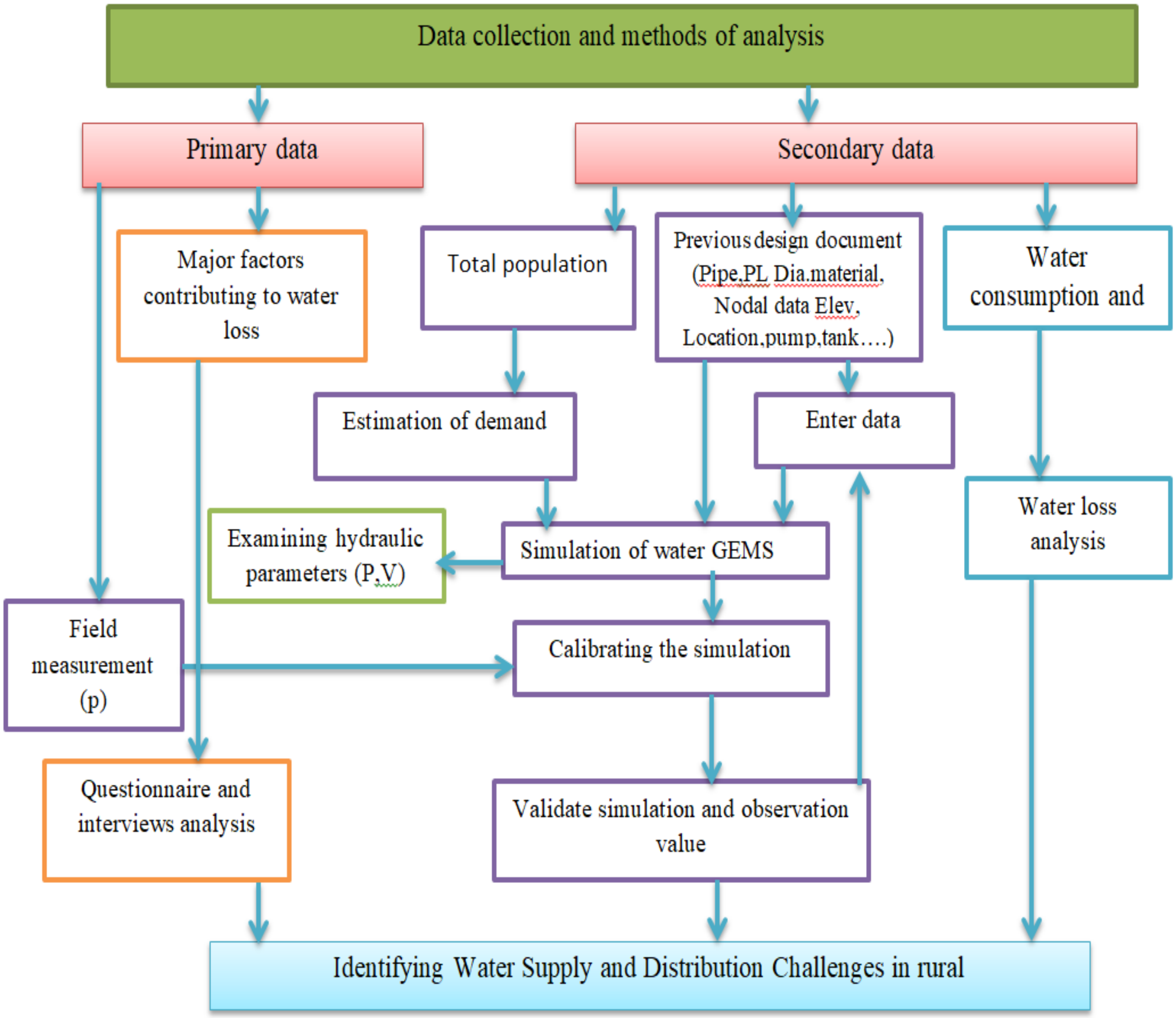


Figure 3.3: Conceptual Framework

4: RESULT AND DISCUSSIONS

4.1. Water Demand Analysis

The main inputs into the water demand analysis process are the demand categories and the time period for which the demand forecast is expected. Previous studies showed that the project design time of 20 years was defined, and this served as the basis for determining the system's active usable period. Demand analysis was done using the projected beneficiary population, average consumption rates, by mode of services proportion, and variations in demand.

4.1.1. Population projection

Geometric progression was applied to determine the projected population over the design period. Since the design period extends to 2036 and the growth rate data was only set up to 2030 in the 1994 CSA report, linear extrapolation was utilized to fix the remaining growth rate. Population projection was made using different growth rate suggested for urban and rural households of the region as shown in the equation below.

$$Pn = Po\,[1 + (r/100)]^{n} \quad \text{………………………………………………………….4.1}$$

Table 4.1: Town and Rural Population Medium Variant Growth Rate for Oromia Region

Year	**Growth rate**		**Population**		**Total population**
	Urban(%)	**Rural(%)**	**Urban**	**Rural**	
2023	3,51	1.68	17,097	142,623	159,721
2024-	3.51	1.68	17697	144,821	162,519
2029	3.35	1.41	20867	154,469	175,336
2034	3.19	1.16	24415	162,939	187,354
2036	3.03	0.93	25917	162,482	188,399

Based on the Table 4.1, the population of Amereba Indato rural water supply system projected population is 162,519(2024) and 188,399 (2036)

4.1.2. Determination of Water demand

According to equation 4.1, projected population and per capita consumption rate are the elements used to calculate domestic demands. The per capita consumption rate can be either determined by adopting from design manual or calculated from the currently observed consumption records. The detail utility water production and billed amount and the consumption rate computed from this data was shown in Table 4.2.

Table 4.2: Calculated per capita water consumption based on utility water utilization

Average Water production (m3/day) (2023-2024) (a)	**Average Consumption (m3/day) (b)**	**Average Water loss (m3/day) (a-b)**	**Consumption rate (L/c/day)**
1024744	603663.00	421081.00	35.54

The rural design manual of 2021 indicated that per capita consumption rate falls in the range between 25l/c/d and 60l/c/d for public tap and house connection mode

Hence using the forecasted population and per capita consumption rate for the design period, domestic demand of the population within the reach of system computed using equation 3.2. Non domestic demand is the categories of demand and the design manual recommends depending on gathering data of the existing institutions, commercial centers, industries and livestock by adopting percentage of the domestic demands.

Non-revenue water for all demand categories also considered as it has considerable effect in water supply system demands. Design manual for both rural and town recommends as percentage of the total day water demand and follows a decreasing approach taking in to consideration the gradual improvements of operation and maintenance skills of staffs in the management sectors. It is not possible to properly fulfill the demands for which the system units were designed by computing all of these demands alone. Hourly and daily allowances of the average demand must be taken into account in order to guarantee sufficient sources and system unit capacities. The design handbook suggests an average demand of 1.1 to 1.5 for daily maximum demand, and an allowance for peak hour consumption based on population size.

$$ADWD = DWD + NDWD\ (PWD + IWD + CWD + LWD) + NRW \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (4.2)$$

Where: *ADWD*-Average day water demand, *DD* –Domestic water demand, *NDWD*-Non-domestic water demand, *PWD*-Public water demand, *IWD*-Institutional water demand, *CWD*-Commercial water demand, *NRW*-Non –revenue water and *LWD*-Livestock water demand.

Maximum day demand: One of these demand circumstances, known as maximum day demand (MDD), is the greatest amount of water needed in a single day over the course of a year.

$$MDD = MDF\ X\ ADD \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots. (4.3)$$

Where: *MDD*-Maximum day demand, *MDF* –Maximum day factor and *ADD*-Average day demand.

Table 4.3: Summary of water demand analysis for design period of 2024-2036

Demand variation		**Design Period**		
		2024	**2030**	**2036**
Average day demand	m3/d	4416.0	5587.5	6701.8
	l/s	51.11	64.67	77.57
Maximum day factor		1.1	1.1	1.1
Maximum day demand	m3/d	4857.6	6146.3	7372.0
	l/s	56.22	71.14	85.32
Peak hour factor		1.5	1.5	1.5
Peak hour demand	m3/d	6624.0	8381.3	10052.7
	l/s	76.7	97.0	116.4

The average day demand computed for the entire design period was 77.57 L/s and when converting to per capita consumption rate it becomes 35.57L/c/d. According to GTP II for rural community 25L/c/d proposed at 1km radius and for towns with population less than 20000, 40L/c/d fixed (Temesgen, 2019). Hence the analyzed demand accommodates the requirement of both rural and town according to the GTP II set design criteria.

Table 4.4: Detail demand analysis of Amareba Indato long-distance water supply system**.**

Description	Unit	Design Period			Design Period			Design Period		
		2024	2030	2036	2024	2030	2036	2024	2030	2036
		Rural Town			Rural Villages			Total		
Population										
Population growth rate	%	3.51	3.19	3.03	1.68	1.16	0.93			
Population projection	No.	17697	21533	25917	144821	156124	162482	162519	177657	188399
Coverage by service type										
HC	%	0	0	0	0	0				
YCP	%	10	21	0	0	5	10			
YCS	%	5	15	20	0	20	25			
PF	%	85	58	80	100	65	65			
Population served by										
HC	No	0	0	0	0	0	0			
YCP	No	1770	4522	0	0	7806	16248			
YCS	No	885	3230	5183	0	31225	40621			
PF	No	15043	12489	20733	144821	101481	105614			
PerCapita Demand										
HC	L/cap/d	50	60	70	50	55	60			
YCP	L/cap/d	25	27.5	30	40	45	50			
YCS	L/cap/d	30	35	40	35	40	40			
PF	L/cap/d	20	22.5	25	25	30	30			
Socio economic factor		1	1	1	1	1	1			
Climate factor		0.8	0.8	0.8	0.8	0.8	0.8			
Total Domestic Demand	M3/d	297.32	414.72	580.53	2896.43	3715.76	4484.51	3193.75	4130.48	5065.04
	l/s	3.44	4.80	6.72	33.52	43.01	51.90	36.96	47.81	58.62
Public demand (6%)	m3/d	17.84	24.88	34.83	173.79	222.95	269.07	191.62	247.83	303.90
	l/s	0.21	0.29	0.40	2.01	2.58	3.11	2.22	2.87	3.52
Livestock water demand 5%	%				5.00	5.00	5.00			
	m3/d				144.82	185.79	224.23			
	l/sec				1.68	2.15	2.60			
Industrial demand (3%)	m3/d	8.92	12.44	17.42	86.89	111.47	134.54	95.81	123.91	151.95
	l/s	0.10	0.14	0.20	1.01	1.29	1.56	1.11	1.43	1.76
Total non-domestic water demand	m3/d	26.76	37.32	52.25	405.50	520.21	627.83	432.26	557.53	680.08
	l/s	0.31	0.43	0.60	4.69	6.02	7.27	5.00	6.45	7.87
Total demand (domestic +Non -domestic)	m3/d	324.08	452.05	632.78	3301.93	4235.96	5112.34	3626.00	4688.01	5745.12
	l/s	3.75	5.23	7.32	38.22	49.03	59.17	41.97	54.26	66.49
Non-Revenue water	%	40%	35%	30%	20%	18%	15%			
	m3/d	129.63	158.22	189.83	660.39	741.29	766.85	790.02	899.51	956.69
	l/s	1.50	1.83	2.20	7.64	8.58	8.88	9.14	10.41	11.07
Average day demand Including Loss	m3/d	453.71	610.26	822.61	3962.32	4977.25	5879.20	4416.02	5587.52	6701.81
	l/s	5.25	7.06	9.52	45.86	57.61	68.05	51.11	64.67	77.57

Based on the population, categories of demand, and mode of service in the area surrounding each node, the base or average water demand was calculated and assigned. Specific base demand allocated for node at public water point, cattle trough and institutional water collection centers.

To analyze Ameraba Indato rural water supply demand, the user communities categorized into two parts depending on sources. Part, one was Jewi and Amereba springs which supposed to serve 11 kebele of 89,993 population. The total length of the distribution line these two springs cover is 80.974 km with discharge capacity of 6 from Jawi sire spring and 24 L/s from Amereba springs with the total 30 L/S discharge capacities. The second part includes those beneficiaries getting water from 5 borehole supposed serve 14 kebele of 98,406 population within length 141.450 km with total discharge of 110L/s and located in Meseranje Oda well filed. On this system the line was connected within spring source and borehole source line but the system was give the water depending on geographical siting.

To check the sufficient capacity of the aforementioned sources against the computed demands, it is important to evaluate the supply distribution was framed as shown in the table 4.5.

Table 4.5: Supply distribution against the computed demand

From Spring source				
Jewi spring demand	**Ameraba spring demand**	**Total Existing demand**	**Total computed demand from both spring**	**Gaps of demand at part one**
MDD	MDD	MMD	MDD	
6 L/s	24	30 L/S	36.58L/S	6.58L/S

Where; Maximum Day Demand (MDD).

It is quite clear that, maximum day demand is used to fix the source capacity. However, in this water supply system case part of the communities collecting water directly from gravity main conveying spring and reservoir. The existing source of the two springs mentioned gives only 30l/s whereas the required source capacity is 36.58l/s and from this analysis the springs remain in deficit of 6.58l/s as show in the Table 4.4.

Regarding Borehole sources, the total yield summed up to 110l/s but the maximum day demand calculated is only 53.97l/s and this indicated that an excess of 56.03l/s observed.

4.2. Water Distribution Network Analysis

4.2.1. Model set up and demand allocation

Model set up was conducted based on the existing project reach, Pipe line laying is following the construction of public water points, cattle trough and institutional taken as end users at pipe network in long distance of Ameraba water supply. Demand allocation was carried out as per the population seize of each village located in two towns and twenty-two rural kebeles as shown in annex-1 and annex-2. The average per capital consumption at end design period was calculated 77.57L/c/s and each village the existing zone consumption share was accordingly computed as shown.

4.2.2. Model calibration and Validation

Sample size and Location

Ameraba Indato long distance water supply system is a branched system currently feeding the user communities through water points and institution tap, hence the pressures data were easily collected using the necessary tools and equipment's. The distribution line network sizes are in the range of 50-400mm and it was not adequate to take data of junction points on this line due to the absence of equivalent size pressure gauges. Pressure readings were measured using a pressure gauge on May 13–19, 2024, at both high- and low-pressure zones of the chosen distribution network points at end user taps (water point and institutional water collection centers). During the peak and average demand analyses in various pipe size networks, thirteen and six samples, respectively, were obtained for the calibration and validation data sets. See photo taken while collecting data under Annex-3. After simulating the computed model, all sampling points were chosen. See observed data in Table 4.6 and 4.7 and photos under while collecting observed data at water points

Table 4.6: Simulated and observed value of pressure at the sample node

No	Name of Sample taken	Field test location			Computed pressure (m)	Observed Pressure (m)	Error
		X	Y	Z			
1	SK.Kersa WP-1	593004.25	838071.17	2393.91	27.41	26.61	0.81
2	SK.Kersa WP-2	591508.32	839243.27	2392	88.77	87.	1.65
3	ST.Sedika WP-	588406.75	842446.84	2400	23.15	24.17	-1.02
4	MO.Mese Adamitu WP-1	584,784.60	856,411.28	2,397.09	31.74	30.14	1.6
5	Go.Goro Jena WP-3	583,280.49	851,208.33	2,372.21	51.53	53.1	-1.57
6	Chefe WP-2	561,332.61	869,756.15	2,512.19	51.68	51.31	0.37
7	Jawwi WP-3	558,883.73	873,198.47	2,704.22	55.8	56.88	-1.08
8	Ay.Akiya WP-4	566,774.53	864,504.53	2,414.58	85.84	84.07	1.77
9	Hu.Hulul WP-1	577,546.02	856,817.86	2,390.00	95.99	97.35	-1.36
10	Azazara WP-2	560,668.87	866,140.42	2,416.07	136.11	136.18	-0.07
11	Korobta Lamlem WP-2	559407.78	865606.99	2414.12	155.31	157.14	-1.83
12	MO. Church end Tap	576570.37	859830.21	2411.56	8.05	7.09	0.96
13	STo.Sedika School Tap 2	590,051.29	843,556.45	2,406.00	19.18	20.35	-1.17

Table 4.7: Distribution line filed level data at water points for validation

No	Name of Sample taken	Field test location			Computed pressure(m)	Observed Pressure in m	Error
		X	Y	Z			
1	Jawwi WP-2	558326.98	874042.03	2,756.35	1.168	0.92	0.248
2	GL.Gado Leman WP-2	586296.8	846554.17	2,392.53	33.23	34	-0.77
3	Chefe WP-3	561669.56	869582.31	2,508.73	53.7	52	1.7
4	Ay.Akiya WP-6	569,680.89	866,253.19	2,413.86	82.55	81.22	1.33
5	Ab.Aboli WP-2 (1)	568,087.50	863,500.62	2,411.85	84.92	85	-0.08
6	Hu.Jena Achool Tap	577,318.96	854,892.63	2,363.65	120.07	118.37	1.7
7	GL.Gado Lema WP-1	585,207.18	848,428.92	2,394.95	28.52	27.8	0.72
8	Chefe HS Tap	561,767.82	869,760.59	2,512.98	51.06	53.01	-1.95
9	Jawwi WP-1	558,449.32	874,950.14	2,766.91	59	61.23	-2.23

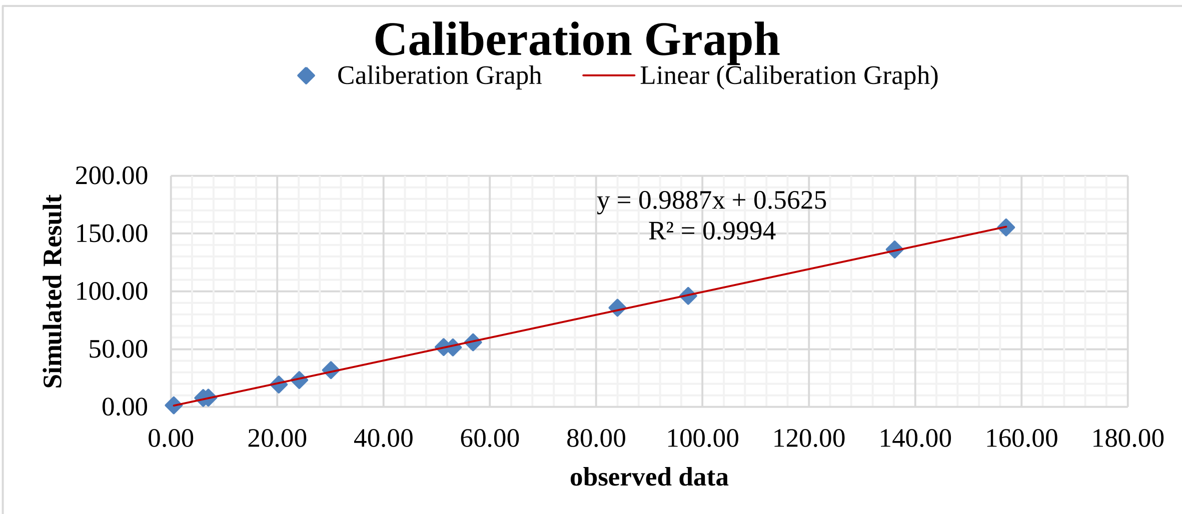
Model calibration

Ormsbee and Lingireddy (1997) state that calibration procedures can involve modifying system requirements, throttling valves, smoothing or roughening pipes, altering pump operating conditions, and modifying other model parameters that influence simulation outcomes. In this study, the model parameters were calibrated in a steady state to align with the pressures derived from field observations. While the extended period calibration is more sensitive to variations in the distribution of demands, the steady state calibration will often be more sensitive to changes in pipe roughness. The WaterGEMS software's has package of Darwin Calibrator module that can be used to fine-tune the pipe roughness parameters so that the simulated and observed values reasonably match. The preliminary pipe roughness value estimation, as displayed in Table 4.8 derived from the rural water supply design manual published in 2021which mainly considers pipe material type and service age in to consideration. The pressure filed data were collected and shown in Table 4.6 encoded in to software and after consecutive iteration the result having coefficient of determination R^2 = 0.999 obtained and according to AWWA (2012), the value of R^2 >0.5 is considered acceptable for model performance. After model calibration the revised C value was used in the pipe line performance of the hydraulic analysis.

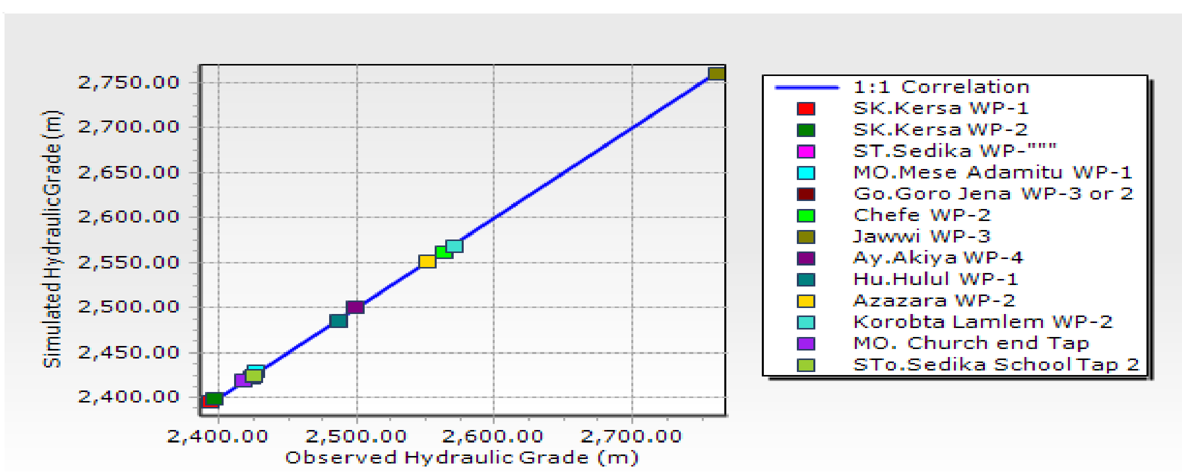
Table 4.8: Revised C value during calibration

Pipe Material	Original C value	Revised C value
uPVC	150	150
DCI	130	104
GS	120	72

After model simulation the correlation graph of the simulated result versus filed data collected was depicted as shown in Figure 4.1 below.



(a)



(b)

Figure 4.1: Model calibration using Darwin model calibrator package in WaterGEMS software

The other model calibration performance criteria is Root mean Square error (RMSE) and according to ATSDR (2000) explained that RMSE of pressure difference of ± 15.2kPa (± 1.52m) to a maximum difference of ± 50.3kPa (± 5.03m) characterizes a good performance set and in this study case RMSE= 1.47

Model validation

After calibrating the model, the corrected C values for all pipes were used in the model simulation and the result for other water points were checked if well correlated with the measured filed data. For this purpose, the data of nine water points were collected as shown in Table 4.7 and the correlation coefficient in this case is R^2= 0.987 shows the model was well calibrated to undertake simulations for further works such as pipe network performance evaluation.

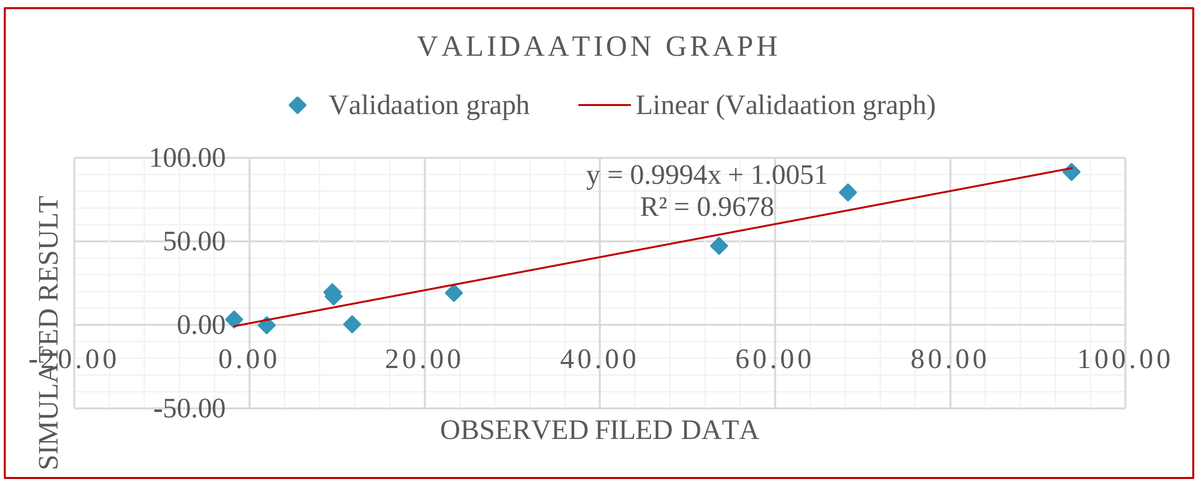


Figure 4.2: model calibration using Darwin model validation

I. Transmission Pipe line performance evaluation

Amereba Indato rural water supply system has three pipe line sections through which the water transferred to reservoirs and injection to pipe stations. The project was designed as one phase project for 20 years 2016-2036 service period according to review document. Hence the performance evaluation was conducted for the entire design period. The constructed pipe were uPVC, DCI and GIS pipes with different PN to resist the static and pump induced pressure. High pressure and velocities are generated due to variation in topography and pipe diameter and the pipe performance was checked for these parameters against the standards and the result was indicated in Table 4.9.

Based on the analysis results show as the pressure created by high topographic set up around spring source area (particularly in the Jawi sand Amereba springs) and pump head against geodetic differences and head loss due to pipeline friction (BH1), is much greater than the nominal pressure values for DCI pipes and uPVC pipes installed in some sections of existing pipes. The overall results show that the transmission pipeline is unable to deliver water to the entire system; mainly due to low nominal pressure of the existing uPVC PN 10 pipes to comply with high pressure occurred in this section. Hence adequate mechanisms used to reduce these pressure need to be devised to ensure the sustainability of the system to offer the required services for the proposed useful period.

Table 4.9: Transmission Pipe line nodal pressure

Pressure(m)	Node(Number)	Percentage (%)
>80	53	33.8
80-10	104	66.2
<10	0	0
Total	157	100

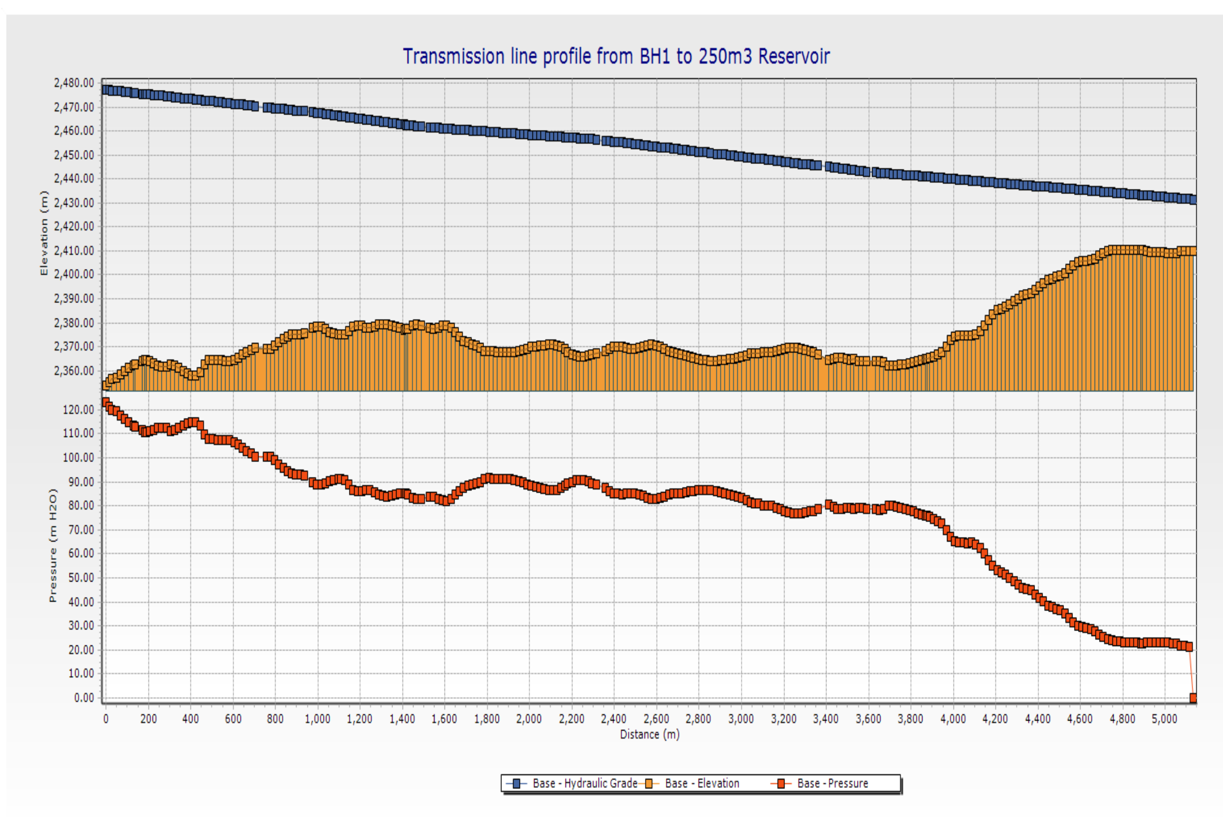


Figure 4.3: Transmission line profile from 5 BH collector to 300 m3 Indato Reservoir

Transmission lines of long distance Amereba Indato waters supply system from spring are indicated on the following figure.

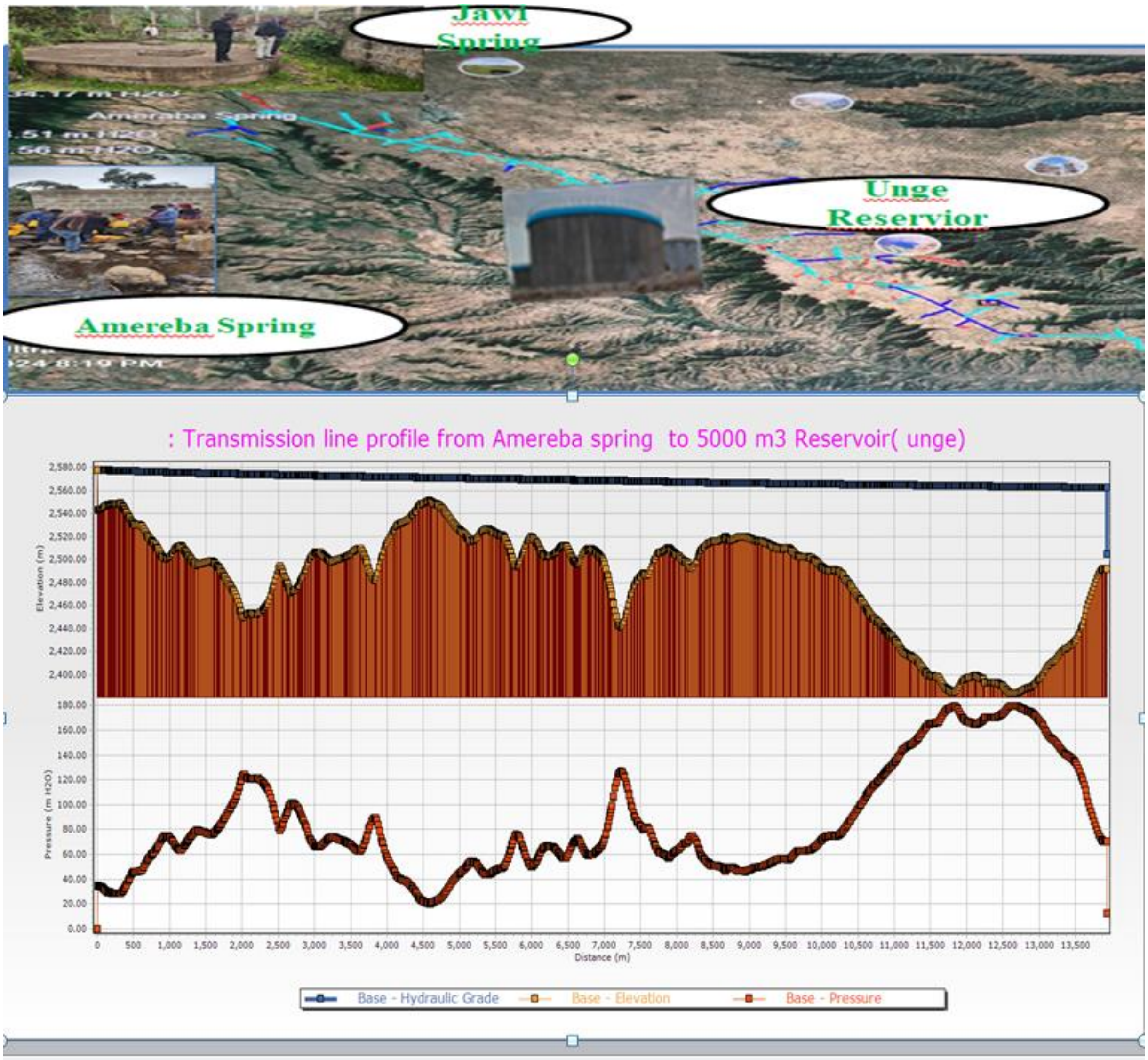


Figure 4.4: Transmission line profile from Amereba Spring to 500 m3 Ungagged Reservoir

ii) Distribution Line performance

In the entire network of existing distribution system, the pipeline size ranges between 40mm – 200mm and the material types are uPVC. According to the analysis conducted using hydraulic modeling many Nodes fail to attain the recommended residual pressure of 10-80m as per the 2021 rural design criteria as shown in the table below.

Table 4.10: Pressure of Distribution line for Amereba Indato water supply

Range of pressure	Node	Percentage (%)	Comment
>80	48	22	It needs pipe change and pressure reducing valves
10-80	126	58	Safe
<10	44	20	It needs technical solution taken in recommendation
Total	218	100	

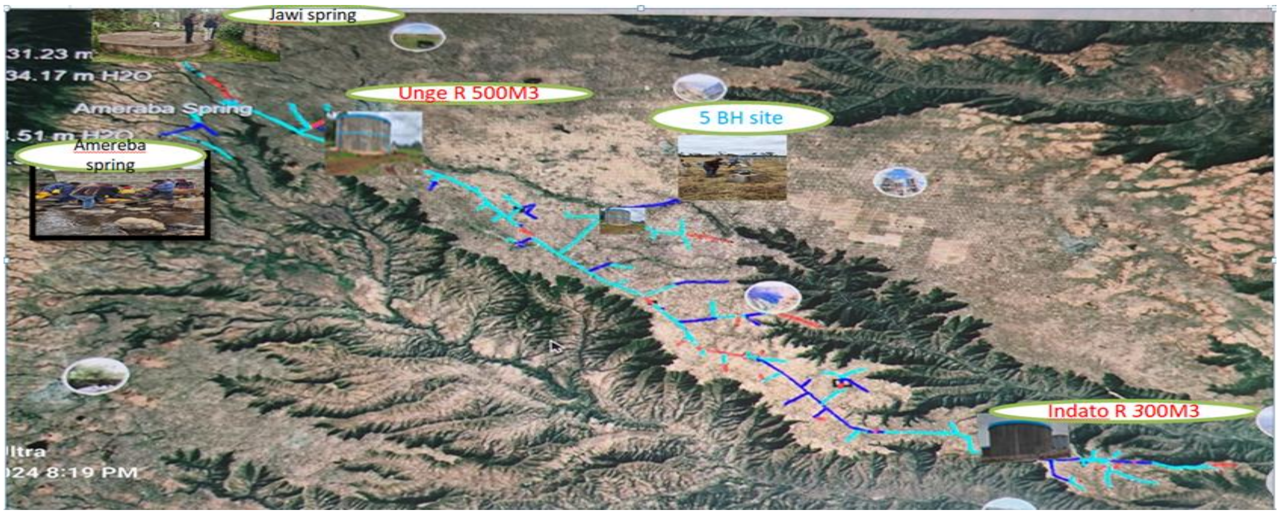


Figure 4.5 Pipe Lay out of the distribution Line from 5 BH collecter to 300 m3 Indato Reservoir

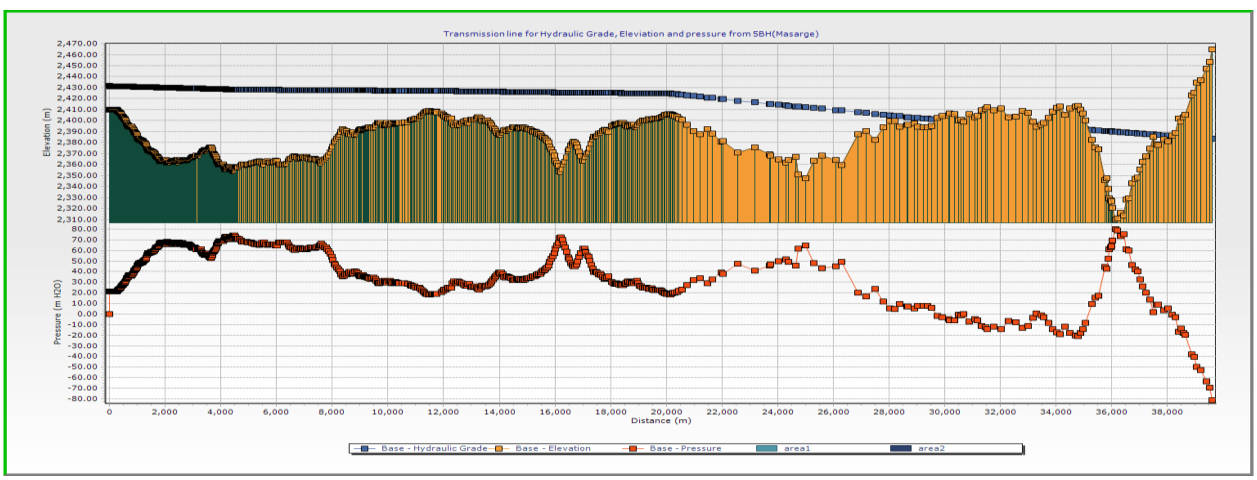


Figure 4. 6:Distribution line profile from 5 BH collecter to 300 m3 Indato Reservoir

The water distribution line's nodes are situated close to one another in relation to Amereba Indato water supply system topography. The distribution system's maximum and minimum water pressures are 80 m and 10 m, respectively, according to the design manual of MoWIE (2022). Hydraulic analysis conducted shows that, 42% of Amereba Indato water supply system distribution network nodes was functioning outside the recommended boundaries regarding the required pressure in the pipe line. Minimum pressure existing around reservoir and this is the usually expected situations which do not have any effect on the system as no nodal consumption point in the area. Still however there are nodes located at far distance from reservoir with pressure ranging between 10m to - 80m due to topography and pipe diameter to accommodate the required amount of water and this needs redesign of the system to propose adequate mechanism that can increase pressure to the required amount. This may include changing of pipe size or recommendation of booster station. But for the pressure greater than the maximum permitted value (80m) it needs alternative solutions and in this particular case the nodes accounting 22% above and 20% under the recommended boundaries of distributed over pipe line sections identified to fall with in static pressure of 80m to 181m which shows the pressure is above allowable limit.

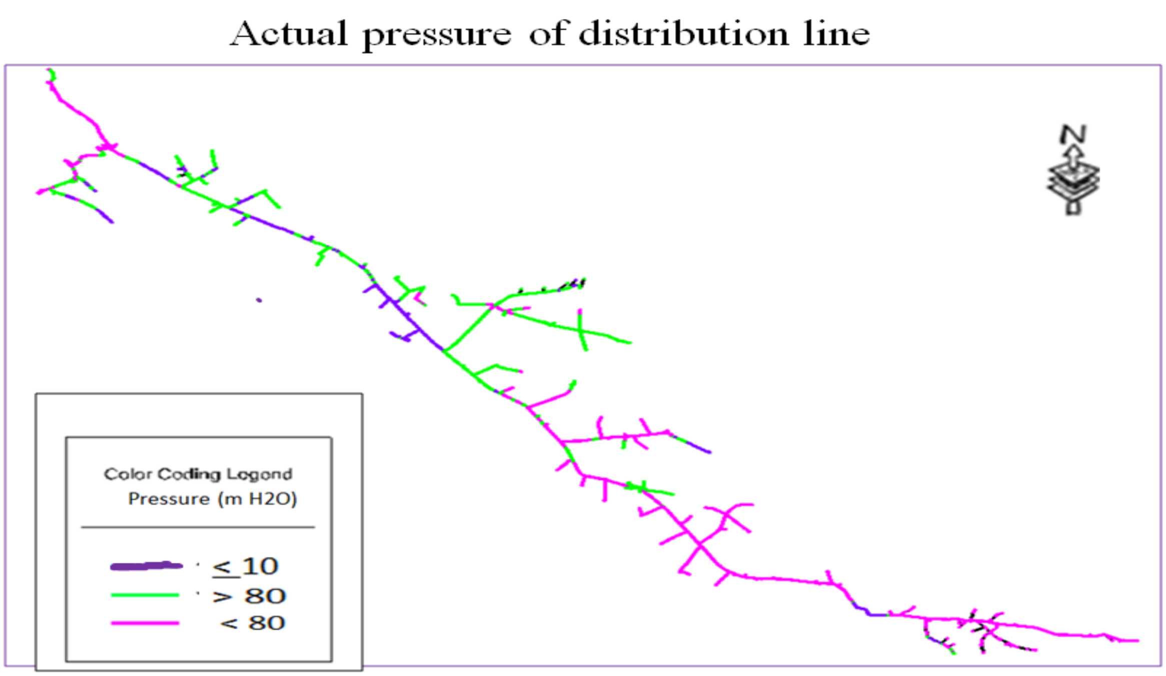


Figure 4. 7: Pressure variation in the distribution system at average day demand

The performance of the distribution line were also evaluated for maximum flow condition where link velocity and node pressure are the dependable parameters. From continuity equation principles, flow and velocity have direct relations and hence at maximum peak hour consumption maximum velocity created. However, in the case of pressure due to increasing of pipe line friction, the pressure at nodal points was getting decreasing in the direction of flow.

From the existing 103 water points,29 cattle trough and 35 institution tap taken as end nodes of the branched system 9 water points have found negative pressure (-8.68 to -81.47m) due small pipe size which resulted in substantial friction head losses and many other nodes close to reservoirs and break pressure tanks also have pressure less than 10m. The detail was attached in annex-5 . The velocity of the links in peak hour demand ranges between 0.03-2.83 m/s and the detail was summarized in Table 4.13

Table 4. 11: summary of the velocities distribution line

Velocity Range in m/s	Pipe line covered in Km	%
$\leq$ 0.6	19.584	21.05
0.6-2.5	66.667	71.63
>2.5	6.812	7.32

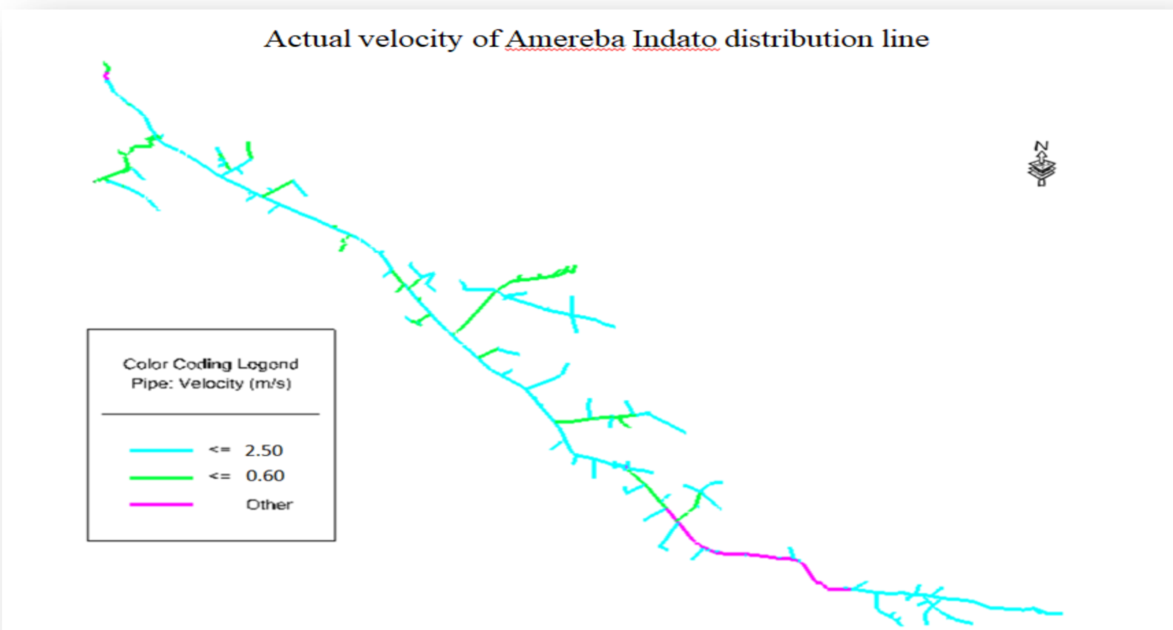


Figure 4. 8: Actual velocity Long Distance of Amereba Indato Distribution

According to rural water supply design manual of 2022, the velocity in branched distribution system has to get in the range of 0.6 to 2.5m/s and from the result of the hydraulic analysis it was observed that 21.05% of the pipes fail to attain the minimum velocity required. Therefore this will bring about the deposition of silts along the pipe line and hence become potential health risks on end users. The maximum velocities determined are within the permissible limit and problems like pipe abrasion or pipe wearing not expected to happen. See Annex-6 for more detail information.

The maximum pressure in the main is considered not to exceed 80m to limit leakage and stress on pipes (Mosissa, 2008). According to (Totsuka et al., 2004), those consumers further away from supply points always collect less water than those nearer to the source due to pressure losses in the network, which increase as far as the source. Pressure was increased as elevation decreased and vice versa. Households located at a higher elevation and close to the reservoir site have access to water at low water pressure (Mekonnen, 2014).

4.3.Water losses analysis

Due to its complexity, analyzing water loss and leakage needs detailed data. In developing countries, data are generally few, and long–distance water supply Ameraba Indato position is not an exception.

A major challenge of utilities is the high amount of water lost through long distance pipe networks. It becomes difficult to meet demand if a significant amount of provided water is lost, which creates difficulties in maintaining the system's water tariffs at an acceptable level. Thus, the following assessment and discussion of Ameraba Indato water supply system water loss resulted.

4.3.1. Percentage of Water Loss

Water losses in the water distribution system, unauthorized connections, and inaccurate metering and recording are all considered non-revenue water. The proportion of the total water produced by the water supply system is used to express the amount. The yearly Non-Revenue Water in Ameraba Indato long-distance water supply for the year 2019 was estimated using the formula Equation 2.14.

Table 4.12: Percentage of Non-Revenue Water

Water meter reading (2022-2023)	Production from Jawi spring (m3)	Production from Amerab a spring (m3)	sum of 5 borhole productio n(m3)	Total production from two source(m3)	Billed Comsum ption (m3)	Non –Revenue	
						m3	%
Jul-23	15552	62208	4345	82105	43614	38491	46.88
Aug-23	15552	62208	7821	85581	47493	38088	44.51
Sep-23	15552	62208	8612	86372	46231	40141	46.47
Oct-23	15552	62208	8192	85952	43527	42425	49.36
Nov-23	15552	62208	5432	83192	48723	34469	41.43
Dec-23	15552	62208	9652	87412	56341	31071	35.55
Jan-24	15552	62208	5643	83403	53832	29571	35.46
Feb-24	15552	62208	6723	84483	53271	31212	36.94
Mar-24	15552	62208	7845	85605	54521	31084	36.31
Apr-24	15552	62208	10232	87992	55243	32749	37.22
May-24	15552	62208	10671	88431	51396	37035	41.88
Jun-24	15552	62208	6456	84216	49471	34745	41.26
Total	**186624**	**746496**	**91624**	**1024744**	**603663**	**421081**	**41.09**

(Source: Amereba Indato water supply system)

As shown in Table 4.6, the total water loss, or non-revenue water (NRW), is increasing monthly in relation to water production. The average annual NRW for the system is 41.09%. According to the Ministry of Water, Irrigation, and Energy (MoWIE, 2021), an NRW of 20% at the project startup and 15% by the end of the design period is considered acceptable for water supply systems that serve more than 10,000 beneficiaries and span over 15 km. Therefore, the current percentage of water loss in the long-distance water supply system of the Amereba Indato distribution network is significantly high and requires strategic management interventions.

4.3.2. Water Loss as Per Number of Service Connection

Based on the collected data, the total number of service connections in the Amereba Indato water supply system was 167. Using this information, the volume of water loss per connection was analyzed from the total unbilled water volume. This metric is a more accurate indicator of water loss within the distribution system compared to NRW expressed as a percentage of input volume, as it provides a clearer and more precise understanding of the losses on a per-connection basis.

Loss per connection = $\left(\frac{\text{water losses} * 1000\ \text{l/m3}}{\text{total number of connection} * 365\text{day}}\right)$ … … … … … …… ….(4.4)

Loss per connection = (421081m*1000l/m3)/(147*365 day)=6908L/con/day

According to José Sardinha's 2017 guidelines, water loss in a distribution system is classified as "good" if it is less than 250 liters per connection per day, "average" if it falls between 250 and 450 liters, and "bad" if it exceeds 450 liters per connection per day. Based on this guideline, the performance indicator for physical water loss in the long-distance Amereba Indato water supply system is in poor condition, with a loss of 6,908 liters per connection per day, far exceeding the "bad" threshold of 450 liters per connection per day.

4.3.3. Water Loss as Per Pipe Length

Another key measure of water loss in a distribution network is water loss per pipe length, expressed in liters per kilometer of pipeline per day (l/km/d). The total length of the Amereba Indato water distribution network, including public water point pipelines, main lines, and institutional lines, is 222.424 kilometers. However, only 192.816 kilometers were analyzed, as 29.608 kilometers were found to be dry. The following formula was used to calculate the estimated water loss per kilometer for the long-distance Amereba Indato water supply system.

System within Loss per pipe = $\left(\frac{\text{Unbilled volume} * 1000\ \text{l/m3}}{\text{total number of pipe length} * 365\ \text{day}}\right)$ … … … ….(4.5)

Loss per connection= (421081m*1000l/m3)/(192.816*365 day)= 5983.15 liters/km/day

According to updated guidelines by José Sardinha (2017), water loss in distribution systems is classified as being in "good condition" if the loss per length of pipe is less than 10,000 liters per kilometer per day. Water loss between 10,000 and 18,000 liters per kilometer per day is considered "average," and losses exceeding 18,000 liters per kilometer per day indicate a "poor condition." Based on these guidelines, the water loss per pipe length in the Amereba

Indato long-distance water supply system falls within the "good condition" range.4.2.2 Real and apparent loss analysis

The total loss, often known as non-revenue, is the sum of apparent and real loss. However, due to full leakage data, unavailability of leakage detection instruments from all system components, and overflows storage tanks, the study was unable to calculate the whole real loss; instead, unavoidable real loss was calculated using the IWA expression.

Unavoidable Average Real Losses (UARL): The following equation should be used as a basis for calculating the UARL in liters/service connection/day. Equation mentioned above was utilized to assess and compute the overall physical loss in the system, which was determined to be the unavoidable annual real loss (UARL) or the minimum achievable annual physical losses.

UARL (liters/day) = (18 Lm + 0.8 * Nc + 25 * Lp) * P ………………………………. (4.6)

Where, Lm = mains length (km); Nc = number of service connections; Lp = total length of

Public fountain and institution pipe, and P = average pressure (m).

Accordingly, Amereba Indato long distance water supply UARL is calculated as

UARL (liters/day) = (18*173.214 + 0.8 *167 + 25 *19.602) * 65.1

= 243571.78 liters/day or UARL =88903.69M3/Year

Total apparent losses plus avoidable real loss in the system was determined from the Amereba Indato water balance as

Apparent loss plus avoidable real loss = Total water loss– UARL ……………………… (4.7)

(421081-88903.69) M^3/year = 332177.31 M^3/year

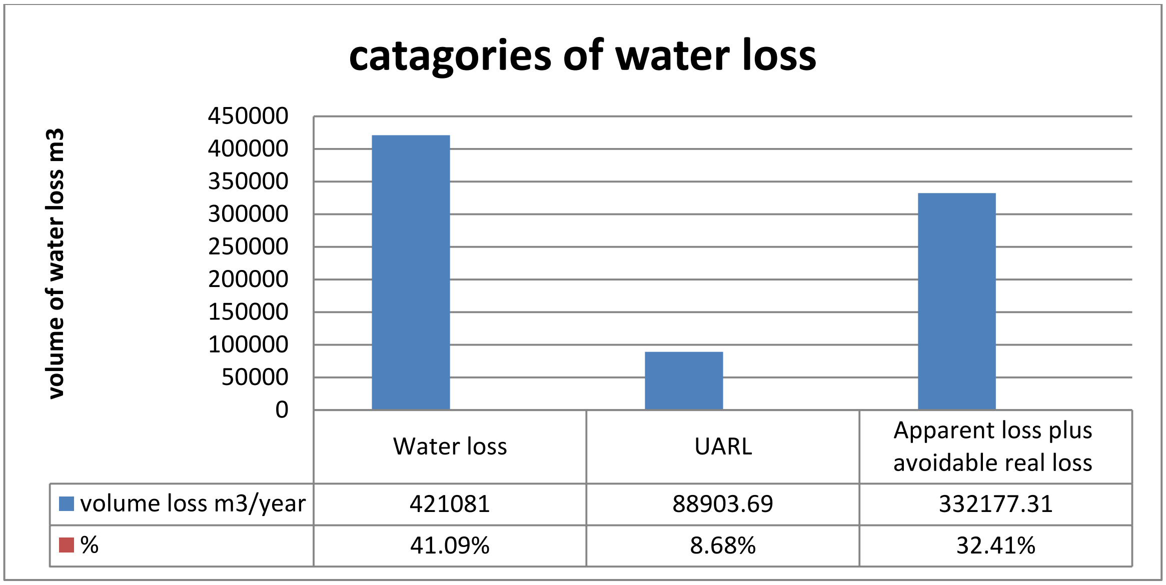


Figure 4. 9: Category of water loss

According to the water loss analysis shown in figure 10 the apparent plus avoidable real loss only accounts for 32.41% of the total loss calculated using production and consumption data, with the unavoidable real loss covering 8.68% of the total. This ratio shows that a huge amount of water loss is in the research region. This indicates that the long distance of Amereba Indao water supply system distribution infrastructure is particularly vulnerable to leaks and bursts.

4.4. Major Factors Contributing to Water Loss

The high rate of water loss in the pipe networks can be attributed to several factors. In the long-distance Amereba Indato water supply system, the main reasons for water loss include issues related to quality management systems, inadequate materials for maintenance, infrastructure challenges, and local cultural and social influences, as well as financial constraints. This section aims to review operational practices within the network to analyze these causes.

To gain a deeper understanding of the major factors contributing to water loss, focus group discussions, questionnaires, interviews, and limited observation techniques were employed to gather information from utility staff. In this study, all team leaders and management members were included as informants.

Table 4.13: Maintenance Performance of Amereba Indato **(**Source: AIWSS, Annual Report 2023)

Sn	Descriptions	Unit	Plan	Achievement Actual	%
1	Water line inventor	Time	4	1	25
2	Distribution lines simple maintenance	Km	32	26	81.25
3	Medium water lines maintenance	Km	41	19	46.3415
4	Heavy Maintenances of water line	Km	10	2	20
5	Prevent maintenance of all line	Km	38	6	15.7895
6	Up grading and Rehabilitation of water line	Km	0.5	0	0
7	Water meter maintenance	No	16	14	87.5
	Replacement water mater	No	20	0	0
8	Maintenance of EM	No	23	16	69.5652
9	Maintenance of civil work	No	67	45	67.1642

This report showed as prevent maintenance, up grading and rehabilitation of water line not done and the achievement on maintenance is total not good, In case of this the water loss is related within prevent maintenance and after the pipe broken maintenance highy.

The following primary causes of water losses are listed in order of expected magnitude based on information collected from staff and users.

a) Costumer usage and Behavior: This issue has been a regular occurrence in the source area, particularly in Jawi Spring and Amereba. Principal Water lines in some areas have been broken frequently due to local interests. An interview was conducted with community members near the source to learn that they don't require water through a pipe line and not need to pay for it; instead, they require it for irrigation purposes. In this instance, people have frequently broken the pipe and used water for irrigation proposes since 49 % only need for drinking , 26% requires for irrigation and 25% for both purposes. See detail questionaries' used for assessment under annex -4.

b) Construction Problem: During the construction phase, several issues came up, The installation of pipes above ground, pipes in rivers without support, height of anchor blocks and a galley crossing structure is not fitting within place needs. Interference of local administration in contract management and administration where the locally established unskilled small micro-enterprises forced to take the sub contract from contractor was also the major factor to the contribution of poor construction quality. Information was gathered through field observation, staff interviews, and user interviews. It has several problems during construction, including poor design for the anchor block and improper placement of the gale cross. The pipe laying in river flow also difficulty to maintenance and some leaks cannot be visually seen or felt, especially at the blow of river flow and other structures and underground pipes. Measuring system leakage at the field level is a very challenging task


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Figure 4. 10: Amereba Indato Pipe laying without any support of pipe line

C) Illegal Connections: Data collected from utility records and field observations indicate that illegal connections along the pipeline route are a significant issue. Many community members use water without charge, primarily for cattle and gardening, accounting for 13% of the water loss identified in this study.

Due to the extensive distance of the Amereba Indato water supply network, numerous illegal users affect the level of service provided to legitimate customers. The system’s long distribution route, combined with its challenging geographical location and lack of access roads for maintenance, has hindered the AIWSS utility’s ability to determine the exact number of illegal connections. Water loss from these unauthorized connections primarily occurs around common ponds used for cattle and irrigation. Although this study did not assess the extent of water losses due to illegal connections due to data limitations, it is crucial for the water utility to manage these connections effectively, as they significantly contribute to water loss and can impact the system's financial sustainability.

D) Environmental Factors: The Amereba Indato water supply system distributes water over long distances across varying climates and soil types, which affects the pipelines. Many sections of the pipeline are exposed to sunlight due to soil erosion; specifically, 16 km of pipeline has suffered erosion, leading to damage from both animals and humans. Additionally, during the wet season, strong floods from the surrounding highlands can destroy the transmission DCI pipe that carries water from Amereba Spring across the steep gorge rivers of Enemor and Hulul.

Figure 4. 11*: Amereba Indato water supply of pipe line affected by Soil erosion*

f) Pressure Management: The topography of the Amereba Indato water supply system is varied, resulting in continuous water flow to the lower sections, while the higher areas do not always receive water consistently. This inconsistency is exacerbated by the lack of zonal metering and the absence of control valves within the system. There are no clear pressure management practices established by the utility, leading to water loss in areas of high pressure and through small-diameter pipes.

g) Speed and Quality of Repairs (Repair Downtime) The speed and quality of maintenance work are significantly influenced by the availability of qualified technicians and engineers, as well as the materials, tools, equipment, and procedures required for repairs. In the Amereba

Indato water supply system, the capacity of technicians is limited, with no engineers specializing in water-related issues. Additionally, there is a lack of logistics, tools, and equipment necessary to address water leaks promptly and effectively. The information available for managing losses is also insufficient, with limited engagement from the user community and other stakeholders.

Leakage Management practice of long distance of AIWSS

Water loss is often over looked by water utilities due to their lack of upkeep resources. It was noted that the long distance of Amereba Indato Water supply Service lacked a sufficient budget, appropriate tools, accessories, and strong policies to effectively address leaks. Due to a discussion with the water supplier, they didn't implement any management procedures before the break down and they only fixed the system after getting the report of the pipe burst. These have a major impact on the local water distribution system's physical losses.

The key to effective leakage management practices in water supply services and customer satisfaction is a robust management and organizational framework. One strategy to lower water loss in the distribution system is to use good leak management practices. The Amereba Indato Water Supply Service Office's leakage management structures were evaluated and discussed as below.

The Amereba Indato water service office reports that there are 59 employees overall, including those in supporting departments. Seven professionals, two plumbers, one electromechanical, one electrician, and three operator from the technique division. The town water supply service manager is responsible for organizing, directing, and overseeing the entire operations of the water service unit in accordance with the policies and norms set forth by the regional administration. From a leakage management perspective, the utility's operation and maintenance section is the most important part of the water supply service, and the head of this department will report to the manager of the water service office. This section's primary responsibilities are to oversee, manage, and coordinate the water supply.

Management practice:- Based on the data collected and field observations, the long-distance operation and maintenance culture of the Amereba Indato water distribution system has made managing water loss extremely challenging.

Operational System: The utility's borehole pumping stations and associated transmission mains exemplify its operational systems. However, these assets are not consistently managed by qualified operators. The employees present are primarily responsible for inspections and lack the necessary training to operate the system effectively. Most operational oversight is provided by utility guards, as there is no permanent operational residence at the site.

Maintenance Culture: Maintenance of system components occurs reactively—only when damage or failure arises rather than on a scheduled basis. There has been improper and infrequent maintenance of installed accessories, including main valves. The network also lacks operational check valves to control flow effectively. When failures occur, the utility typically addresses the issue by shutting off the pumps at the borehole or, in the case of springs, by releasing water during operational hours.

A significant issue contributing to water loss, as discussed in this research, is the absence of a suitable operating manual to address technological and operational challenges. Many operational and technical issues remain unresolved due to reliance on traditional practices. Furthermore, effective pressure regulation is hindered by the area's topographic characteristics.

The utility also lacks an adequate information system to communicate with customers and the general public, which could assist in reporting leaks, line breaks, or illicit activities involving water lines. Additionally, there is no appropriate database system that provides details on assets such as pipes and fittings, including their types, lengths, ages, origins, and registration zones within the distribution and transmission networks. Collaborative efforts between the utility and other government offices to reduce water loss have not been effectively implemented.

5: CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusion

The study evaluated the challenges of rural water supply and distribution in the Amereba Indato water supply system. It focused on the hydraulic efficiency of the distribution network, analyzed water loss, and identified the factors contributing to this loss, as well as the ability to meet water demand to update the current state of water supply and identify the challenges of distribution line in rural area.

This system has significant gap between water supply and demand in the Amereba Indato rural water supply system, which is served by spring sources. The community in the spring zone requires a supply of 36.58 L/s, while the current discharge capacity of the springs is only 30 L/s. This results in a gap of 6.58 L/s for the upper area (Zone 1). Consequently, communities at higher elevations frequently experience insufficient water supply, necessitating the development of additional sources near the springs to meet their needs.

The borehole sources in the system yield a total of 110 L/s, while the highest daily demand was need only 53.97 L/s, resulting in an excess of 56.03 L/s. However, to effectively address the capacity gaps in the upper areas, additional infrastructure, such as booster pumps, electromechanical systems, and reservoirs is essential for transporting water from the boreholes in to areas served by the springs (zone 2).

The existing transmission line needs to resist high pressure to minimize water loss and ensure sufficient supply. The maximum head in pressure line is attributed to the steep topography of the pipeline routes and diameter of pipes, which needs an equivalent PN 20 pipes laid to mitigate potential pressure effects, as part of the hydraulic analysis result ranges between except for the aforementioned uPVC pipe segments. While the calculated velocities fall within recommended ranges, exceptions exist where uPVC pipes at the outlets of BH1, BH2, BH3, BH4, and BH5 exceed allowable velocity limits by 0.3 m/s.

With regard to distribution line, analysis was conducted for minimum and maximum flow condition to check pipe performance with respect to pressure and velocity respectively. Maximum pressure of 80-183m occurred in the pipe line by 22 % node and minimum pressure of -81-10 occrred 20% of the distribution line. The maintenance history of the utility proofs the result facts where pipes are broken in the mentioned area frequently. Minimum velocity less than 0.6m/s occurred in 21.05% of the pipe length and this has happened mostly in large pipe segment of the system. Changing this huge quantity of pipe is not practical and hence it is better to install valves that used to flush where ever silts stored in the sagging points of the pipe due to this low velocity.

The primary challenges identified in the distribution system include small pipe diameters, low PN ratings, low velocities, excessive water pressure (maximum of 181 m), and negative water pressure. According to rural water supply guide line of 2022 published by MoWE, the system pressure operating outside the permissible range of 10-80m. Consequently, numerous pipe breaks and failures occur during low-demand periods, resulting in significant water loss, especially in high-pressure zones. In instances of negative pressure, parts of the system do not supply water, leaving 29,608 m of pipeline dry.

Water loss analysis, utilizing annual water balance figures for production and consumption in the Amereba Indato long-distance water supply system, reveals that 41.09% of water is classified as total Non-Revenue Water. This indicates poor leakage management practices within the utility, which has not still prioritized to solve the significant water loss reduction.

The main causes of water loss in system of distribution networks were identified as high water pressure, incorrect pipe installation, pipe material size, metering errors, illegal connections, systematic data handling errors, customer usage trends, environmental factors, an overloaded pipeline system, and poor maintenance practices.

Overall, the results of this study showed that the Amerebe Inadto rural water supply's current water distribution system performed poorly technically, as evidenced by low water production rates in the spring zone, high levels of non-revenue water, inability to serve communities at high elevations, and velocity and pressure that were outside of the permitted ranges.

5.2. Recommendation

The following recommendations were made for Long distance of Amereba Indato current water supply system based on the analysis of the findings:

- The existing water supply taken from source using two spring needs additional source to balance demand and supply at zone-1 which served by spring.
- The currently existing gravity system proposed to reach Indeto Kebele failed and the system remain dry due to inadequate head(low pressure). Therefore it is advisable to use booster pumps equipped with the required structures such as wet wells and pumping stations that need to be located in Sadiqa Kebele where the existing distribution line currently supplies adequate amount of water.
- The hydraulic analysis of the distribution line shows that, considerable segment of the pipe line fall under excessive pressure ranging between 80-181m.This will therefore subject to loss of water and increase maintenance costs. However the utility can easily control the pressure proposing new PRV and break pressure tanks as required, especially for pressure prone areas discussed in the result section.
- The distribution line system does not have adequate amount of necessary valves. Gate valves should be installed at the recommended standard distance between the branch junctions to be used during maintenance operations and to save water loss in the event of a pipe break. Although there are a number of failure points on the pipe profiles that favor silt deposition, there is no flush valve on the pipe lines which needs to be installed. An additional air release valve is also required at crest pipe profile points especially in areas where the distance between water points is large.
- To secure the life time of system and decrease the possible water loss holistic awareness creating among the user communities must be conducted. Different interests such as using water for other services like irrigation is one challenges around the source and this should be solved either convincing the communities through local by laws or looking for alternative compensations.
- During the assessment it was identified that, some structures were not properly constructed among which anchor blocks, gulley crossing and shallow depth of pipes that easily eroded and exposed to surface observed. Hence these structures and pipes have eventually

damaged by weather and animals. Therefore early replacement of the mentioned damaged structures and pipes must be conducted to avoid further deterioration of scheme.

- To solve the long down time of systems without service , it is advisable to construct access roads, supply the required logistics ,repairing spares and materials, and hire skilled human resource at different sections of system operation specially plumbers, electro-mechanical personnel etc.

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Annex-1: Demand Allocation for each Kebele

NO	Description	Design period												
		2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036
		Rural												
	Growth Rate	1.68	1.41	1.41	1.41	1.41	1.41	1.16	1.16	1.16	1.16	1.16	0.93	0.93
1	Akiya Ayifila	4920	4989	5059	5131	5203	5276	5338	5400	5462	5526	5590	5642	5694
2	Abi Ali	5128	5200	5273	5348	5423	5500	5563	5628	5693	5759	5826	5880	5935
3	Sabro Café	11782	11782	11782	11782	11782	11782	11782	11782	11782	11782	11782	11782	11782
4	Jawi Sire	9022	9149	9278	9409	9542	9677	9789	9902	10017	10133	10251	10346	10443
5	Sadiqa Qarsa	9418	9551	9686	9822	9961	10101	10219	10337	10457	10578	10701	10801	10901
6	Sadiqa Burqa	3436	3485	3534	3584	3634	3685	3728	3771	3815	3859	3904	3940	3977
7	Sadiqa Atucha	7018	7117	7217	7319	7422	7527	7614	7703	7792	7882	7974	8048	5970
8	Sadiiqa Tokuchuma	3679	3731	3784	3837	3891	3946	3992	4038	4085	4133	4181	4219	3130
9	Gado Lameen	6275	6364	6453	6544	6637	6730	6808	6887	6967	7048	7130	7196	7263
10	Goro Jena	7085	7185	7286	7389	7493	7599	7687	7776	7866	7958	8050	8125	8200
11	Adamtuu Lamaan	6269	6358	6447	6538	6630	6724	6802	6881	6960	7041	7123	7189	7256
12	Jena Hulul	11800	11966	12135	12306	12480	12656	12803	12951	13101	13253	13407	13532	13658
13	Jena Gadamsaa	7147	7248	7350	7454	7559	7666	7755	7844	7935	8028	8121	8196	8272
14	Jena Baburqoo	7918	8030	8143	8258	8375	8493	8591	8691	8792	8894	8997	9080	9165
15	Masaranje Oda	8117	8232	8348	8465	8585	8706	8807	8909	9012	9117	9223	9308	9395
16	Qako Adajabo	5701	5782	5863	5946	6030	6115	6186	6258	6330	6404	6478	6538	6599
17	Indato Walta'i	5335	5410	5486	5563	5642	5721	5788	5855	5923	5992	6061	6117	6174
18	Kalisa Dambal	3208	3253	3299	3346	3393	3441	3481	3521	3562	3603	3645	3679	3713
19	Marari Sibe	8319	8437	8556	8676	8799	8923	9026	9131	9237	9344	9452	9540	9629
20	Korefta Lamlen	3330	3377	3425	3473	3522	3572	3613	3655	3698	3740	3784	3819	3855
21	Wadago Misa	6351	6441	6532	6624	6717	6812	6891	6971	7052	7134	7216	7284	7351
22	Azezera Baredu	3561	3611	3662	3713	3766	3819	3863	3908	3953	3999	4046	4083	4121
	Total for rural	144821	146697	148600	150529	152485	154469	156124	157799	159492	161206	162939	164345	162482
NO	Description	Design period												
		2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036
		Urban												
	Growth Rate	**3.51**	**3.35**	**3.35**	**3.35**	**3.35**	**3.35**	**3.19**	**3.19**	**3.19**	**3.19**	**3.19**	**3.03**	**3.03**
1	Sadiqa 01	6860	7090	7327	7572	7826	8088	8346	8613	8887	9171	9463	9750	10046
2	Jena 01	7325	7571	7824	8087	8357	8637	8913	9197	9491	9793	10106	10412	10727
3	Indato 01	3512	3630	3752	3877	4007	4141	4273	4410	4550	4696	4845	4992	5143
	total	17697	18290	18903	19536	20191	20867	21533	22220	22928	23660	24415	25154	25917
	Total rural and urban	162519	164988	167503	170065	172676	175336	177657	180018	182421	184866	187354	189499	188399

Annex-2 Pupolation used from 2 two spring source and 5 BH

NO	Kebele	Design period		
		2024	2030	2036
1	Jena 01	7325	8913	10727
2	Akiya Ayifila	4920	5338	5694
3	Abi Ali	5128	5563	5935
4	Sabro Café	11782	11782	11782
5	Jawi Sire	9022	9789	10443
6	Gado Lameen	6275	6808	7263
7	Jena Hulul	11800	12803	13658
8	Jena Baburqoo	7918	8591	9165
9	Korefta Lamlen	3330	3613	3855
10	Wadago Misa	6351	6891	7351
11	Azezera Baredu	3561	3863	4121
	Total	77413	83954	89993

Population used 5 BH source

NO	Kebele	Design period		
		2024	2030	2036
1	Sadiqa 01	6860	8346	10046
2	Indato 01	3512	4273	5143
3	Sadiqa Qarsa	9418	10219	10901
4	Sadiqa Burqa	3436	3728	3977
5	Sadiqa Atucha	7018	7614	5970
6	Sadiiqa Tokuchuma	3679	3992	3130
7	Goro Jena	7085	7687	8200
8	Adamtuu Lamaan	6269	6802	7256
9	Jena Gadamsaa	7147	7755	8272
10	Masaranje Oda	8117	8807	9395
11	Qako Adajabo	5701	6186	6599
12	Indato Walta'i	5335	5788	6174
13	Kalisa Dambal	3208	3481	3713
14	Marari Sibe	8319	9026	9629
	Total	85106	93703	98406

Annex-3: Photos taken while collecting Observed data of pressure at selected location using pressure gauge

Annex-4: Sample data collected from utility and user community to evaluation major factor (Summary of respondents interview)

Summary of Interview Questionaries' user community(357)

S/No	Interview Questionaries’	Response	No of respondent	%
1	Do you Want to pay for water?	Yes	173	48.46
		No	184	51.54
2	For what purpose you have used water from this scheme?	For drinking	175	49
		For irrigation	89	25
		For both	93	26
3	Did the broken pipe will be repaired on time for you?	Good	47	13%
		medium	107	30%
		No	203	57%
4	Have you given information when water line is broken to concerned body?	Yes	96	27
		NO	261	73
5	Did you have participated in this project (planning-construction) in one way or the other?	No	340	95.2
		Yes	17	4.8
6	Have you seen someone punished for breaking water line?	Yes	0	0
		no	357	100
Summary of Interview Questionaries’ for utility management(12 person including technic team)				
1	Do you have spare part to major maintenance?	Yes	0	0
		No	12	100
2	Do you have spare part to minor maintenance?	Yes	9	75
		No	3	25
3	How fast can your group react to situations of water loss or leaks that are reported	Good		
		Medium	8	66
		Low	4	34
4	Do you identify common causes of water loss in your network system	Yes	12	100
		no	0	0
5	Do you have preventive maintenance is done irregularly	yes	0	0
		NO	12	100
6	Do you have utility coordination with other government office to reduce loss	Yes	0	0
		No	12	100
7	Do you have get information about leakage happened from community on time	Good	0	0
		Medium	7	58.3
		Low	5	41.7
8	Do you think holding the long distance of system is affecting maintenance or to reduce the water loss	Yes	12	100
		No	0	0

Annex-5 Assessment of the Water Distribution Network of Amereba indato Nodal hydraulic output at minimum flow condition for existing

Label	Elevation (m)	Demand (L/s)	Hydraulic Grade (m)	Pressure (m H2O)	X (m)	Y (m)
J-62	2,508.34	0	2,425.22	-82.95	593,092.27	840,079.31
J-61	2,507.17	0	2,425.25	-81.75	593,080.39	840,064.32
J-59	2,506.45	0	2,425.29	-80.99	593,054.43	840,050.34
J-60	2,506.27	0	2,425.28	-80.83	593,066.11	840,055.39
J-16	2,546.52	0	2,492.89	-53.52	565,706.63	866,057.23
J-14	2,545.30	0	2,492.97	-52.22	565,585.29	866,126.71
1773`	2,548.64	0.924	2,517.44	-31.14	559,107.85	868,472.95
J-332	2,417.55	0	2,386.37	-31.12	596,080.91	840,345.20
SK.Lgo WP-1	2,416.23	1.219	2,385.06	-31.11	596,164.29	840,258.38
1774`	2,548.34	0.924	2,517.18	-31.1	559,129.26	868,505.18
JG.Jenan WP-1 (Mesaranje Oda WP-1)	2,401.32	1.541	2,371.80	-29.46	578,403.55	857,502.58
J-3419	2,417.69	0	2,388.29	-29.34	595,952.45	840,468.36
1651`	2,546.02	0	2,527.39	-18.59	557,672.04	866,039.95
J-64	2,510.15	0	2,491.57	-18.55	567,640.38	864,897.09
1645`	2,545.30	0	2,526.78	-18.48	557,732.58	866,032.97
Korobta Lamlem CT-2	2,545.19	0.51	2,526.76	-18.39	557,735.94	866,031.29
1646`	2,545.76	0	2,527.41	-18.32	557,668.31	866,037.01
Korobta Lamlem CT-1	2,545.24	0.061	2,527.00	-18.21	557,761.90	866,040.89
1633`	2,544.76	0	2,526.97	-17.76	557,713.56	866,026.30
J-4969	2,407.26	0	2,389.66	-17.56	578,022.13	858,303.65
1630`	2,544.88	0	2,527.42	-17.42	557,663.73	866,030.00
J-4626	2,414.00	0	2,396.59	-17.38	595,454.50	841,036.95
1629`	2,544.85	0	2,527.45	-17.37	557,655.82	866,030.50
Korobta Lamlem WP-1	2,544.15	0.718	2,526.75	-17.36	557,735.54	866,020.68
1627`	2,535.07	0	2,517.72	-17.31	559,220.81	868,319.56
J-1431	2,412.19	0	2,395.06	-17.09	595,547.32	840,932.14
2091`	2,706.92	0	2,698.59	-8.32	558,830.18	873,213.93
J-2324	2,412.92	0	2,404.66	-8.24	594,738.28	841,702.17
J-4058	2,412.10	0	2,403.84	-8.24	594,467.62	841,872.87
Korobta Mosque Tap	2,535.42	0.072	2,527.42	-7.99	557,669.26	865,955.61

Jawwi WP-3	2,704.22	1.168	2,697.60	-6.61	558,883.73	873,198.47
SB.Lgo WP-2	2,407.73	0.556	2,402.09	-5.62	593,832.04	842,097.26
Jawwi WP-2	2,756.35	1.168	2,752.99	-3.35	558,326.98	874,042.03
JEBS-139	2,409.22	0	2,405.89	-3.32	594,879.68	841,539.03
JEBS-124	2,411.61	0	2,408.32	-3.29	594,821.41	841,248.03
MO.WP-New	2,403.27	0.657	2,400.66	-2.61	578,324.27	858,891.17
1955`	2,628.92	0	2,626.31	-2.6	559,994.71	872,071.18
1394`	2,514.49	0	2,512.60	-1.89	561,158.98	869,528.98
Jawwi Mosque Tap-2	2,699.47	0.361	2,697.60	-1.86	558,923.14	873,165.76
1940`	2,619.04	0	2,618.00	-1.03	560,145.77	871,928.18
Jawwi WP-4	2,639.74	1.168	2,638.71	-1.02	559,766.53	872,308.14
1929`	2,613.71	0	2,612.73	-0.97	560,231.42	871,830.89
J-2	2,493.44	0	2,493.32	-0.11	565,164.29	866,471.73
1305`	2,516.65	0.924	2,516.54	-0.11	559,529.59	868,197.62
MS.Qako Adejebo CT	2,456.64	0.075	2,458.49	1.85	603,874.30	833,939.61
SA.Atucha Werabu WP	2,398.00	0.834	2,400.36	2.36	590,548.65	846,685.92
1281`	2,508.70	0	2,512.71	4.01	561,183.49	869,428.14
Chefe WP-1	2,578.34	0.941	2,582.37	4.02	560,692.39	871,234.50
*URTJ-8	2,488.00	0	2,493.20	5.19	565,276.00	866,483.00
J-825	2,395.36	0	2,400.56	5.19	596,174.15	842,334.81
SB.Lgo CT-1	2,395.18	0.062	2,400.40	5.21	596,226.05	842,347.18
MO. Church end Tap	2,411.56	0.649	2,419.61	8.04	576,570.37	859,830.21
1902`	2,591.41	0	2,599.48	8.05	560,447.56	871,581.25
J-3085	2,397.29	0	2,405.86	8.55	590,284.74	846,946.84
J-2603	2,409.50	0	2,418.09	8.57	590,812.02	847,975.94
1169`	2,506.27	0	2,514.86	8.57	560,614.28	868,913.50
Chefe HS Tap	2,512.98	0.111	2,521.57	8.57	561,767.82	869,760.59
J-6657	2,408.69	0	2,420.07	11.35	576,671.48	859,742.34
J-3022	2,406.38	0	2,417.98	11.58	590,120.76	847,636.92
J-3977	2,397.51	0	2,409.11	11.58	590,127.75	847,100.92
SB.Lgo Mosque WP-3	2,388.69	0.556	2,400.30	11.58	596,260.10	842,357.99
SA.Near WP	2,407.13	0.834	2,418.74	11.58	590,864.27	847,798.89
J-6656	2,408.54	0	2,420.15	11.59	576,686.38	859,723.52
J-3280	2,409.62	0	2,421.23	11.59	591,877.81	843,157.09
J-3279	2,409.64	0	2,421.28	11.62	591,858.70	843,163.01
J-53	2,411.00	0	2,422.66	11.63	589,558.14	843,915.56

J-1727	2,405.78	0	2,417.46	11.66	589,414.02	847,690.34
GL.Atucha St. Gorgis Church Tap	2,405.72	0.274	2,417.42	11.68	589,384.83	847,731.73
1807`	2,547.39	0	2,559.44	12.03	560,904.14	870,647.93
1808`	2,547.49	0	2,559.59	12.07	560,902.69	870,651.00
SB.Kersa WP-4 (Sedika Burka)	2,386.73	0.556	2,399.58	12.83	590,396.23	841,407.24
J-1147	2,404.61	0	2,417.47	12.83	593,882.85	846,446.58
J-2899	2,405.72	0	2,418.71	12.96	591,019.68	847,812.55
SB.Sedika Nur WP-2	2,407.74	0.556	2,420.78	13.01	592,043.40	843,092.17
J-2627	2,403.56	0	2,417.52	13.93	593,734.00	846,527.23
KD Mane Kebele WP-3	2,408.27	0.692	2,422.24	13.94	614,418.95	832,545.39
1769`	2,539.45	0	2,553.76	14.28	560,963.61	870,503.07
J-2073	2,401.15	0	2,415.47	14.29	586,683.49	856,795.26
J-4641	2,407.17	0	2,421.49	14.29	591,782.28	843,186.67
J-5036	2,403.27	0	2,417.59	14.29	593,518.28	846,672.30
1800`	2,810.32	0	2,824.65	14.3	558,286.49	875,530.50
J-422	2,408.00	0	2,422.34	14.31	591,458.04	843,261.75
KD Mane Kebele CT-2	2,407.92	0.058	2,422.27	14.32	614,428.50	832,564.60
1304`	2,507.17	0	2,522.09	14.9	561,513.47	869,656.34
Chefe Clinic Tap	2,534.86	0.111	2,549.79	14.9	561,024.99	870,371.07
J-1022	2,400.97	0	2,415.90	14.91	588,038.70	848,046.20
J-1323	2,402.51	0	2,417.55	15.01	590,823.23	848,190.32
J-1971	2,384.67	0	2,399.71	15.01	590,453.02	841,442.21
EW Waltaei CT-3	2,423.41	0.07	2,438.47	15.03	610,467.80	833,210.99
J-1322	2,402.53	0	2,417.60	15.04	590,835.90	848,174.80
J-6650	2,405.55	0	2,420.64	15.06	576,781.01	859,610.74
J-2776	2,402.42	0	2,417.51	15.06	589,313.62	847,555.23
J-2890	2,407.49	0	2,422.59	15.07	591,363.56	843,289.27
J-3044	2,402.39	0	2,417.51	15.08	593,766.78	846,504.30
SA.Atucha Mosque Tap	2,402.28	0.075	2,417.47	15.15	593,932.21	846,464.47
J-6452	2,384.54	0	2,399.72	15.16	590,458.23	841,445.44
KD jarsa GUDA WP-1	2,406.28	0.692	2,421.69	15.38	614,658.67	832,483.40

J-3047	2,402.00	0	2,417.43	15.4	588,994.13	847,528.50
J-1831	2,400.64	0	2,416.20	15.53	588,064.43	847,908.58
J-3222	2,406.00	0	2,421.56	15.53	577,727.33	858,628.74
990`	2,509.29	0	2,524.88	15.55	558,257.65	866,248.61
QA WP-2	2,428.64	0.737	2,444.24	15.57	607,467.98	833,251.49
J-3849	2,409.66	0	2,425.27	15.58	577,275.59	858,910.19
J-3066	2,401.36	0	2,417.50	16.11	589,253.80	847,550.52
J-3972	2,404.61	0	2,420.75	16.11	576,809.28	859,586.57
1582`	2,786.55	0	2,802.69	16.11	558,318.57	874,704.46
SA.Atucha Mosque	2,402.00	0.075	2,418.15	16.12	591,837.18	847,756.14
*URTJ-16	2,477.00	0	2,493.15	16.12	565,351.00	866,550.00
J-820	2,402.00	0	2,418.15	16.12	591,824.07	847,742.67
J-2984	2,401.89	0	2,418.09	16.16	592,018.37	847,589.61
SA.Atucha CT-2	2,402.00	0.094	2,418.20	16.17	591,877.13	847,803.17
J-245	2,402.00	0	2,418.20	16.17	591,860.13	847,812.62
SK.Sedika Kersa WP-1	2,396.53	1.219	2,413.12	16.55	594,883.26	840,412.38
J-1652	2,400.94	0	2,417.53	16.56	589,442.21	847,418.03
J-5205	2,383.11	0	2,399.72	16.57	590,397.77	841,555.44
941`	2,502.46	0.924	2,519.07	16.57	559,578.23	867,558.40
J-605	2,401.33	0	2,418.17	16.81	591,798.69	847,767.22
J-2680	2,400.57	0	2,417.42	16.81	588,925.72	847,520.11
SA.Atucha Sokole Wp	2,398.85	0.834	2,415.71	16.83	590,489.63	848,884.47
J-3203	2,406.00	0	2,422.86	16.83	577,690.63	858,612.85
J-3963	2,384.72	0	2,401.72	16.96	595,822.48	842,200.13
Azazara WP-1	2,495.82	1.276	2,512.82	16.97	559,752.89	867,203.24
J-4186	2,394.79	0	2,412.15	17.33	593,218.29	837,969.28
J-2716	2,404.89	0	2,422.46	17.53	590,278.16	843,431.57
J-3964	2,384.09	0	2,401.68	17.55	595,833.58	842,204.99
J-4790	2,400.61	0	2,418.21	17.57	591,740.94	847,844.26
J-2896	2,405.12	0	2,422.75	17.59	591,303.68	843,299.82
SK.Kersa WP-1	2,393.91	1.219	2,411.58	17.63	593,004.25	838,071.17
J-1064	2,382.04	0	2,399.72	17.64	590,389.36	841,582.30
J-1209	2,394.42	0	2,412.10	17.65	593,270.19	837,939.17
J-4961	2,400.79	0	2,418.48	17.66	591,271.25	847,877.45
J-656	2,399.71	0	2,417.41	17.66	590,791.42	848,241.38
J-692	2,394.17	0	2,412.18	17.98	593,182.90	837,987.82

EW WP-4	2,410.26	0.791	2,428.28	17.98	611,555.74	831,191.06
J-2675	2,399.37	0	2,417.39	17.98	588,826.03	847,512.21
J-5318	2,398.05	0	2,416.09	18	586,682.53	856,715.27
J-676	2,407.68	0	2,425.72	18	577,321.42	858,843.59
J-1793	2,399.24	0	2,417.28	18	588,288.04	847,440.38
J-466	2,409.98	0	2,428.03	18.02	577,530.47	858,516.80
J-2090	2,406.21	0	2,424.27	18.02	577,176.39	859,062.28
J-5266	2,399.22	0	2,417.28	18.03	588,307.86	847,443.04
J-2879	2,409.62	0	2,427.70	18.04	577,544.11	858,557.87
AT.Adamitu School Tap	2,406.31	0.125	2,424.39	18.05	589,452.41	854,924.83
J-1697	2,403.06	0	2,421.15	18.06	576,831.37	859,521.30
J-5154	2,399.56	0	2,418.05	18.45	592,141.03	847,525.50
J-1785	2,398.80	0	2,417.30	18.46	588,446.62	847,461.70
STo.Sedika Sheep Market WP-	2,401.66	1.404	2,420.16	18.46	590,293.26	843,293.16
J-1790	2,398.59	0	2,417.27	18.64	588,228.58	847,432.38
J-2738	2,408.32	0	2,427.00	18.64	577,464.44	858,656.02
J-1321	2,393.32	0	2,412.01	18.65	593,372.16	837,875.91
EW Waltaei HS -2	2,418.00	0.212	2,436.71	18.67	610,261.47	831,516.11
J-6489	2,398.58	0	2,417.30	18.68	588,473.65	847,465.12
SB.Sedika Existing WP-1 (Teklye Church)	2,406.15	0.15	2,424.88	18.69	590,534.19	844,088.67
JEBS-95	2,394.50	0	2,413.22	18.69	594,681.28	840,667.18
J-1732	2,399.05	0	2,418.22	19.13	591,665.46	847,870.79
J-1693	2,398.61	0	2,418.01	19.36	592,244.53	847,464.78
J-2892	2,403.50	0	2,422.91	19.36	591,241.03	843,304.82
QA EM WP-1	2,430.00	0.737	2,449.42	19.38	607,491.04	833,441.67
J-47	2,404.77	0	2,424.19	19.38	590,330.25	843,537.05
J-2922	2,380.29	0	2,399.72	19.39	590,369.51	841,657.05
J-2195	2,397.56	0	2,417.14	19.54	588,126.83	847,475.15
SK.Kersa WP-2	2,392.00	1.219	2,411.58	19.54	591,508.32	839,243.27
STo.School (New)	2,404.00	0.079	2,423.60	19.56	589,755.21	843,529.22
917`	2,492.37	0	2,512.92	20.51	561,074.94	869,432.87
SA.Atucha Kole WP-1	2,396.70	0.834	2,417.24	20.51	591,640.28	848,070.98

J-739	2,404.48	0	2,425.15	20.62	590,021.98	843,625.56
2133`	2,732.51	0	2,753.18	20.63	558,418.73	874,153.83
EW Mane Kebele WP-1	2,420.59	0.791	2,441.27	20.64	609,800.48	834,090.72
J-1617	2,403.26	0	2,424.00	20.7	589,893.82	843,738.87
AT. Church Tap (New)	2,403.93	0.125	2,424.68	20.71	588,121.30	855,572.62
J-891	2,397.54	0	2,418.30	20.71	591,485.31	847,911.79
ST.Sedika WP-7	2,395.85	0.35	2,424.78	28.87	588,399.97	844,159.96
J-717	2,396.00	0	2,424.93	28.87	588,983.96	843,901.65
761`	2,485.21	0	2,514.49	29.22	560,938.68	868,912.34
SK.Kersa WP-3	2,389.15	1.219	2,418.44	29.23	593,047.15	841,417.32
1637`	2,791.94	0	2,821.70	29.69	558,416.74	875,270.25
EW Mane Kebele WP-2	2,402.52	0.791	2,432.27	29.69	611,528.42	832,977.03
J-575	2,398.00	0	2,427.78	29.72	595,316.18	836,738.74
J-5112	2,393.24	0	2,423.21	29.9	588,420.35	842,734.50
GL.Sedika Tokuchuma WP-2	2,394.00	1.478	2,423.97	29.91	586,398.62	846,388.56
J-3525	2,386.11	0	2,416.11	29.94	590,551.97	848,728.86
J-3462	2,384.21	0	2,415.34	31.07	591,864.85	839,530.88
ST.Sedika Tokuchuma WP-3	2,392.14	0.35	2,423.28	31.08	585,854.29	844,859.89
QA WP-1	2,424.02	0.737	2,455.17	31.09	605,081.99	834,544.87
J-719	2,393.76	0	2,424.94	31.11	589,083.02	843,870.44
EW FE CT-1	2,402.00	0.07	2,433.52	31.45	607,173.05	831,616.86
Jat-104	2,393.06	0	2,424.58	31.46	587,975.65	847,370.74
J-4821	2,392.00	0	2,423.74	31.67	588,422.92	843,214.41
MO.Mese Adamitu WP	2,397.09	0.657	2,428.83	31.68	584,784.60	856,411.28
J-4781	2,395.55	0	2,427.30	31.69	585,633.19	855,997.37
J-2457	2,393.04	0	2,424.80	31.69	587,893.73	855,524.49
J-629	2,392.72	0	2,424.59	31.8	586,281.79	846,548.20
GL.Gado Leman WP-2	2,392.53	1.478	2,424.39	31.8	586,296.80	846,554.17
J-1662	2,398.61	0	2,430.49	31.82	581,383.58	857,659.28
J-1867	2,385.57	0	2,417.45	31.82	589,356.55	846,928.62

1601`	2,789.41	0	2,821.30	31.83	558,413.53	875,227.99
JmoR-30	2,399.46	0	2,431.36	31.83	581,806.71	857,135.96
J-2695	2,384.13	0	2,417.42	33.23	589,331.01	846,730.36
Go.Goro Jena WP-2	2,370.00	0.764	2,403.32	33.25	586,769.09	851,027.00
624`	2,348.90	0	2,385.11	36.14	560,353.87	865,018.23
MO.Masaranje Oda WP-4	2,390.06	0.657	2,426.33	36.2	579,862.41	858,338.20
J-1396	2,400.39	0	2,479.94	79.38	575,648.95	858,946.61
J-6688	2,364.00	0	2,443.55	79.39	583,395.27	858,521.71
J-276	2,401.66	0	2,481.22	79.4	575,761.86	859,061.22
Ay.Akiya CT	2,412.00	0.78	2,491.58	79.42	567,121.62	865,254.38
J-99	2,412.00	0	2,491.63	79.47	567,114.16	865,259.44
414`	2,442.32	0	2,522.02	79.54	558,875.79	865,885.97
J-5240	2,412.00	0	2,491.73	79.57	567,101.33	865,244.10
Hu.Hulul WP-	2,390.00	2.541	2,469.74	79.58	577,546.02	856,817.86
J-1397	2,400.03	0	2,479.78	79.59	575,634.60	858,932.68
JMBH-184	2,362.20	0	2,441.99	79.63	583,250.38	858,438.97
JB.Barbuqo WP-2	2,402.65	1.024	2,483.13	80.32	573,886.08	861,283.75
J-1796	2,400.71	0	2,483.36	82.48	573,840.21	861,431.71
315`	2,423.81	0	2,506.46	82.48	560,244.95	866,652.06
J-957	2,403.65	0	2,486.35	82.53	569,343.88	865,988.76
Hu.Hulul WP	2,364.48	2.541	2,447.19	82.54	577,595.43	854,698.04
J-6642	2,400.79	0	2,486.69	85.73	571,207.07	862,312.55
J-3736	2,404.30	0	2,490.23	85.76	568,220.74	865,160.96
JB.Barbuqo WP-5	2,390.87	1.024	2,476.91	85.87	576,779.74	857,718.23
Ay.Akiya WP	2,400.33	0.636	2,488.61	88.11	568,126.18	865,539.19
J-856	2,400.98	0	2,489.59	88.43	568,644.26	864,065.82
J-347	2,386.79	0	2,475.45	88.48	577,685.95	856,302.86
AB.Aboli Mosque	2,397.94	0.112	2,486.65	88.54	571,166.51	862,223.36
226`	2,285.51	0	2,374.23	88.54	561,125.32	864,227.27
J-431	2,400.69	0	2,489.43	88.56	568,486.29	865,369.19
J-295	2,387.63	0	2,479.06	91.24	576,283.86	858,255.73
Ab. Aboli WP-6 (new)	2,393.73	0.829	2,485.20	91.28	570,568.63	862,783.07
JMBH-94	2,367.80	0	2,459.29	91.31	585,076.86	858,776.54
127`	2,279.48	0	2,378.31	98.63	560,885.49	864,570.06
JB.Barbuqo	2,374.59	1.024	2,473.44	98.66	576,020.01	857,270.71

WP-6						
J-2912	2,362.67	0	2,461.66	98.79	577,361.10	854,873.73
Hu.Hulul CT-1	2,362.87	0.429	2,461.86	98.79	576,819.98	855,437.01
248`	2,421.53	0	2,520.52	98.8	559,237.93	865,707.50
J-3590	2,362.91	0	2,462.00	98.89	576,888.65	855,381.72
157`	2,397.07	0	2,496.75	99.48	564,475.04	867,121.59
J-370	2,374.99	0	2,474.70	99.51	578,322.97	855,415.49
Hu.Jena Achool	2,363.65	0.257	2,463.36	99.51	577,330.05	854,897.91
J-3591	2,362.26	0	2,461.97	99.51	576,874.73	855,393.34
J-369	2,374.95	0	2,474.70	99.55	578,294.34	855,456.58
39`	2,267.54	0	2,367.91	100.17	561,524.48	863,718.18
J-3367	2,365.02	1.182	2,465.43	100.21	577,626.67	855,138.05
JMBH-40	2,369.26	0	2,469.72	100.26	586,110.67	859,100.15
J-3585	2,366.00	0	2,467.30	101.1	577,768.86	855,248.41
J-4131	2,372.71	0	2,474.01	101.1	578,153.67	855,581.68
Hu.Hulul WP-2	2,360.14	2.541	2,461.45	101.1	577,151.04	855,179.08
J-1308	2,361.09	0	2,462.42	101.12	577,104.68	855,233.91
Hu.Hulul School Tap	2,360.74	0.257	2,462.66	101.71	577,266.48	855,210.89
JMBH-37	2,368.67	0	2,470.61	101.74	586,182.37	859,157.67
J-6682	2,362.75	0	2,464.71	101.75	585,708.96	858,741.62
J-6564	2,361.56	0	2,463.56	101.8	577,375.26	854,937.15
J-2980	2,371.27	1.182	2,473.32	101.85	578,122.59	855,556.51
154`	2,398.17	0	2,500.24	101.86	563,789.00	867,733.69
24`	2,265.22	0	2,367.46	102.04	561,556.87	863,684.58
Hu.Hulul Mosque Tap	2,361.14	0.257	2,463.40	102.05	577,601.05	854,820.07
J-1539	2,369.69	0	2,471.96	102.07	578,044.03	855,494.63
J-135	2,361.12	0	2,463.41	102.08	577,589.59	854,822.00
Korobta Lamlem WP-4	2,265.15	0.718	2,367.44	102.09	561,558.40	863,682.64
J-451	2,385.59	0	2,501.35	155.52	563,542.45	867,902.81
JG.Jena WP-New	2,358.37	1.001	2,474.26	167.66	579,216.07	854,215.66
J-833	2,310.83	0	2,445.80	174.7	601,478.40	834,535.82
J-1451	2,309.73	0	2,445.53	181.52	601,441.33	834,588.30

Annex-6 Link result of hydraulic analysis

Label	Length (Scaled) (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)
P-3	57	289.3	PVC	150	32.725	0.5
P-2	30	289.3	PVC	150	32.725	0.5
P-285	15	289.3	PVC	150	32.725	0.5
P-286	21	289.3	PVC	150	32.725	0.5
AUP-766	20	200	PVC	150	29.485	0.94
AUP-767	20	200	PVC	150	29.485	0.94
AUP-768	19	200	PVC	150	29.485	0.94
AUP-770	20	200	PVC	150	29.485	0.94
AUP-775	20	200	PVC	150	29.485	0.94
AUP-779	20	200	PVC	150	29.485	0.94
P-8	50	289.3	PVC	150	28.369	0.43
P-35	50	289.3	PVC	150	28.369	0.43
P-43	53	289.3	PVC	150	28.369	0.43
P-41	50	289.3	PVC	150	28.369	0.43
P-34	50	289.3	PVC	150	28.369	0.43
P-20	50	289.3	PVC	150	28.369	0.43
P-36	52	289.3	PVC	150	28.369	0.43
P-14	50	289.3	PVC	150	28.369	0.43
P-9	50	289.3	PVC	150	28.369	0.43
P-21	50	289.3	PVC	150	28.369	0.43
P-32	50	289.3	PVC	150	28.369	0.43
P-18	50	289.3	PVC	150	28.369	0.43
P-46	56	289.3	PVC	150	28.369	0.43
P-49	50	289.3	PVC	150	28.369	0.43
P-259	35	289.3	PVC	150	28.369	0.43
P-260	50	289.3	PVC	130	28.369	0.43
P-261	53	289.3	PVC	130	28.369	0.43
P-262	50	289.3	PVC	130	28.369	0.43
P-263	50	289.3	PVC	130	28.369	0.43
P-264	50	289.3	PVC	130	28.369	0.43
P-265	50	289.3	PVC	130	28.369	0.43
P-266	50	289.3	PVC	130	28.369	0.43
P-267	50	289.3	PVC	130	28.369	0.43
P-269	50	289.3	PVC	130	28.369	0.43
P-287	30	289.3	PVC	150	28.369	0.43
P-288	50	289.3	PVC	150	28.369	0.43

P-289	50	289.3	PVC	150	28.369	0.43
P-290	53	289.3	PVC	150	28.369	0.43
P-291	90	289.3	PVC	150	28.369	0.43
P-292	50	289.3	PVC	150	28.369	0.43
P-293	50	289.3	PVC	150	28.369	0.43
P-294	50	289.3	PVC	150	28.369	0.43
P-54	50	289.3	PVC	150	28.369	0.43
P-51	50	289.3	PVC	150	27.349	0.42
P-57	50	289.3	PVC	150	27.349	0.42
P-52	49	289.3	PVC	150	27.349	0.42
P-55	58	289.3	PVC	150	27.349	0.42
P-56	50	289.3	PVC	150	27.349	0.42
P-53	50	289.3	PVC	150	27.349	0.42
P-50	33	289.3	PVC	150	27.349	0.42
P-58	13	289.3	PVC	150	27.349	0.42
P-62	41	289.3	PVC	150	27.349	0.42
P-65	50	289.3	PVC	150	26.713	0.41
P-60	50	289.3	PVC	150	26.713	0.41
P-64	50	289.3	PVC	150	26.713	0.41
P-61	50	289.3	PVC	150	26.713	0.41
P-63	45	289.3	PVC	150	26.713	0.41
P-67	50	289.3	PVC	150	26.713	0.41
P-87	50	289.3	PVC	150	26.713	0.41
P-70	52	289.3	PVC	150	24.805	0.38
P-79	50	289.3	PVC	150	24.805	0.38
P-88	50	289.3	PVC	150	24.805	0.38
P-82	43	289.3	PVC	150	24.805	0.38
P-71	50	289.3	PVC	150	24.805	0.38
P-91	50	289.3	PVC	150	24.805	0.38
P-92	50	289.3	PVC	150	24.805	0.38
P-243	82	289.3	PVC	130	24.805	0.38
P-244	100	289.3	PVC	130	24.805	0.38
P-245	100	289.3	PVC	130	24.805	0.38
P-246	150	289.3	PVC	130	24.805	0.38
P-247	175	289.3	PVC	130	24.805	0.38
P-296	101	289.3	PVC	130	24.805	0.38
P-297	89	289.3	PVC	150	24.805	0.38
AUP-4	9	200	DCI	104	24.7	0.79
AUP-5	12	200	DCI	104	24.7	0.79
AUP-6	8	200	DCI	104	24.7	0.79

AUP-8	12	200	DCI	104	24.7	0.79
AUP-9	8	200	DCI	104	24.7	0.79
AUP-1(1)	13	200	DCI	104	24.7	0.79
MBH3SP -2	11	100	GS	72	24.219	3.08
P-159(2)	6	100	GS	72	24.219	3.08
JwP-10	19	150	PVC	150	24.219	1.37
JwP-8	12	150	GIS	72	23.981	1.36
JwP-9	20	150	GIS	72	23.981	1.36
P-133	11	150	GIS	120	23.981	1.36
P-126	50	229.6	PVC	150	23.976	0.58
P-110	50	229.6	PVC	150	23.976	0.58
P-102	50	229.6	PVC	150	23.976	0.58
P-122	50	229.6	PVC	150	23.976	0.58
P-127	54	229.6	PVC	150	23.976	0.58
P-97	50	229.6	PVC	150	23.976	0.58
P-137	50	229.6	PVC	150	23.976	0.58
P-105	50	229.6	PVC	150	23.976	0.58
P-99	50	229.6	PVC	150	23.976	0.58
P-107	50	229.6	PVC	150	23.976	0.58
P-114	50	229.6	PVC	150	23.976	0.58
P-104	50	229.6	PVC	150	23.976	0.58
P-117	49	229.6	PVC	150	23.976	0.58
P-135	50	229.6	PVC	150	23.976	0.58
P-98	50	229.6	PVC	150	23.976	0.58
P-100	50	229.6	PVC	150	23.976	0.58
P-124	50	229.6	PVC	150	23.976	0.58
P-134	97	229.6	PVC	150	23.976	0.58
P-132	60	229.6	PVC	150	23.976	0.58
P-131	50	229.6	PVC	150	23.976	0.58
P-118	50	229.6	PVC	150	23.976	0.58
P-115	38	229.6	PVC	150	23.976	0.58
P-94	50	229.6	PVC	150	23.976	0.58
P-111	50	229.6	PVC	150	23.976	0.58
P-128	50	229.6	PVC	150	23.976	0.58
P-138	50	229.6	PVC	150	23.976	0.58
P-96	50	229.6	PVC	150	23.976	0.58
P-112	50	229.6	PVC	150	23.976	0.58
P-130	50	229.6	PVC	150	23.976	0.58
P-125	50	229.6	PVC	150	23.976	0.58

P-109	50	229.6	PVC	150	23.976	0.58
AUP-125	12	200	DCI	104	22.496	0.72
AUP-126	14	200	DCI	104	22.496	0.72
AUP-127	0	200	DCI	104	22.496	0.72
AUP-128	6	200	DCI	104	22.496	0.72
AUP-129	11	200	PVC	150	22.496	0.72
AUP-130	9	200	PVC	150	22.496	0.72
AUP-131	4	200	PVC	150	22.496	0.72
AUP-132	16	200	PVC	150	22.496	0.72
AUP-133	8	200	PVC	150	22.496	0.72
AUP-134	12	200	PVC	150	22.496	0.72
AUP-135	19	200	PVC	150	22.496	0.72
AUP-136	1	200	PVC	150	22.496	0.72
AUP-137	12	200	PVC	150	22.496	0.72
AUP-138	7	200	PVC	150	22.496	0.72
AUP-139	1	200	PVC	150	22.496	0.72
AUP-140	20	200	PVC	150	22.496	0.72
AUP-141	6	200	PVC	150	22.496	0.72
AUP-142	14	200	PVC	150	22.496	0.72
AUP-143	20	200	PVC	150	22.496	0.72
AUP-144	20	200	PVC	150	22.496	0.72
AUP-145	18	200	PVC	150	22.496	0.72
AUP-146	2	200	PVC	150	22.496	0.72
AUP-147	20	200	PVC	150	22.496	0.72
AUP-148	1	200	PVC	150	22.496	0.72
AUP-149	19	200	PVC	150	22.496	0.72
AUP-150	20	200	PVC	150	22.496	0.72
AUP-151	20	200	PVC	150	22.496	0.72
AUP-152	0	200	PVC	150	22.496	0.72
AUP-153	20	200	PVC	150	22.496	0.72
AUP-154	18	200	PVC	150	22.496	0.72
AUP-155	2	200	PVC	150	22.496	0.72
AUP-156	20	200	PVC	150	22.496	0.72
AUP-157	3	200	PVC	150	22.496	0.72
AUP-158	17	200	PVC	150	22.496	0.72
AUP-159	12	200	PVC	150	22.496	0.72
AUP-160	8	200	PVC	150	22.496	0.72
AUP-161	20	200	PVC	150	22.496	0.72
AUP-162	9	200	PVC	150	22.496	0.72
AUP-163	11	200	PVC	150	22.496	0.72

AUP-164	11	200	PVC	150	22.496	0.72
AUP-165	9	200	PVC	150	22.496	0.72
AUP-166	20	200	PVC	150	22.496	0.72
AUP-167	12	200	PVC	150	22.496	0.72
AUP-168	8	200	PVC	150	22.496	0.72
AUP-169	20	200	PVC	150	22.496	0.72
AUP-170	11	200	PVC	150	22.496	0.72
AUP-171	9	200	PVC	150	22.496	0.72
AUP-172	20	200	PVC	150	22.496	0.72
AUP-173	20	200	PVC	150	22.496	0.72
AUP-174	20	200	PVC	150	22.496	0.72
AUP-175	17	200	PVC	150	22.496	0.72
AUP-176	3	200	PVC	150	22.496	0.72
AUP-177	20	200	PVC	150	22.496	0.72
AUP-178	9	200	PVC	150	22.496	0.72
AUP-179	11	200	PVC	150	22.496	0.72
AUP-180	20	200	PVC	150	22.496	0.72
AUP-181	0	200	PVC	150	22.496	0.72
AUP-182	20	200	PVC	150	22.496	0.72
AUP-183	3	200	PVC	150	22.496	0.72
AUP-184	17	200	PVC	150	22.496	0.72
AUP-185	19	200	PVC	150	22.496	0.72
AUP-186	1	200	PVC	150	22.496	0.72
AUP-187	18	200	PVC	150	22.496	0.72
AUP-188	2	200	PVC	150	22.496	0.72
AUP-189	20	200	PVC	150	22.496	0.72
AUP-190	14	200	PVC	150	22.496	0.72
AUP-191	6	200	PVC	150	22.496	0.72
AUP-192	20	200	PVC	150	22.496	0.72
AUP-193	20	200	PVC	150	22.496	0.72
AUP-194	5	200	PVC	150	22.496	0.72
AUP-195	15	200	PVC	150	22.496	0.72
AUP-196	20	200	PVC	150	22.496	0.72
AUP-197	7	200	PVC	150	22.496	0.72
AUP-198	13	200	PVC	150	22.496	0.72
AUP-199	14	200	PVC	150	22.496	0.72
AUP-200	6	200	PVC	150	22.496	0.72
AUP-201	20	200	PVC	150	22.496	0.72
AUP-202	16	200	PVC	150	22.496	0.72
AUP-203	4	200	PVC	150	22.496	0.72

AUP-204	20	200	PVC	150	22.496	0.72
AUP-205	10	200	PVC	150	22.496	0.72
AUP-206	10	200	PVC	150	22.496	0.72
AUP-207	20	200	PVC	150	22.496	0.72
AUP-208	8	200	PVC	150	22.496	0.72
AUP-209	12	200	PVC	150	22.496	0.72
AUP-210	20	200	PVC	150	22.496	0.72
AUP-211	17	200	PVC	150	22.496	0.72
AUP-212	3	200	PVC	150	22.496	0.72
AUP-213	20	200	PVC	150	22.496	0.72
AUP-214	8	200	PVC	150	22.496	0.72
AUP-215	12	200	PVC	150	22.496	0.72
AUP-216	10	200	PVC	150	22.496	0.72
AUP-217	10	200	PVC	150	22.496	0.72
AUP-218	20	200	PVC	150	22.496	0.72
AUP-219	1	200	PVC	150	22.496	0.72
AUP-220	19	200	PVC	150	22.496	0.72
AUP-221	20	200	PVC	150	22.496	0.72
AUP-222	6	200	PVC	150	22.496	0.72
AUP-223	7	200	PVC	150	22.496	0.72
AUP-224	7	200	PVC	150	22.496	0.72
AUP-225	18	200	PVC	150	22.496	0.72
AUP-226	2	200	PVC	150	22.496	0.72
AUP-227	20	200	PVC	150	22.496	0.72
AUP-228	20	200	PVC	150	22.496	0.72
AUP-229	5	200	PVC	150	22.496	0.72
AUP-230	15	200	PVC	150	22.496	0.72
AUP-231	2	200	PVC	150	22.496	0.72
AUP-232	18	200	PVC	150	22.496	0.72
AUP-233	14	200	PVC	150	22.496	0.72
AUP-234	6	200	PVC	150	22.496	0.72
AUP-235	20	200	PVC	150	22.496	0.72
AUP-236	3	200	PVC	150	22.496	0.72
AUP-237	17	200	PVC	150	22.496	0.72
AUP-238	15	200	PVC	150	22.496	0.72
AUP-239	6	200	PVC	150	22.496	0.72
AUP-240	19	200	PVC	150	22.496	0.72
AUP-241	10	200	PVC	150	22.496	0.72
AUP-242	10	200	PVC	150	22.496	0.72
AUP-243	20	200	PVC	150	22.496	0.72

AUP-244	1	200	PVC	150	22.496	0.72
AUP-245	19	200	PVC	150	22.496	0.72
AUP-246	6	200	PVC	150	22.496	0.72
AUP-247	14	200	PVC	150	22.496	0.72
AUP-248	15	200	PVC	150	22.496	0.72
AUP-249	5	200	PVC	150	22.496	0.72
AUP-250	20	200	PVC	150	22.496	0.72
AUP-251	2	200	PVC	150	22.496	0.72
AUP-252	8	200	PVC	150	22.496	0.72
AUP-253	10	200	PVC	150	22.496	0.72
AUP-254	6	200	PVC	150	22.496	0.72
AUP-255	16	200	PVC	150	22.496	0.72
AUP-256	11	200	PVC	150	22.496	0.72
AUP-257	7	200	PVC	150	22.496	0.72
AUP-258	12	200	PVC	150	22.496	0.72
AUP-259	8	200	PVC	150	22.496	0.72
AUP-260	20	200	PVC	150	22.496	0.72
AUP-261	11	200	PVC	150	22.496	0.72
AUP-262	9	200	PVC	150	22.496	0.72
AUP-263	20	200	PVC	150	22.496	0.72
AUP-264	4	200	PVC	150	22.496	0.72
P-192	16	200	PVC	150	22.496	0.72
P-181	52	229.6	PVC	150	22.206	0.54
P-183	48	229.6	PVC	150	22.206	0.54
P-175	51	229.6	PVC	150	22.206	0.54
P-180	45	229.6	PVC	150	22.206	0.54
P-188	58	229.6	PVC	150	22.206	0.54
P-189	50	229.6	PVC	150	22.206	0.54
P-184	50	229.6	PVC	150	22.206	0.54
P-182	50	229.6	PVC	150	22.206	0.54
P-193	50	229.6	PVC	150	22.206	0.54
P-191	50	229.6	PVC	150	22.206	0.54
P-173	43	229.6	PVC	150	22.206	0.54
P-185	51	229.6	PVC	150	22.206	0.54
P-177	55	229.6	PVC	150	22.206	0.54
P-171	48	229.6	PVC	150	22.206	0.54
MBH2SP-1	7	229.6	PVC	150	22.206	0.54
MBH2SP-2	8	100	GS	120	21.959	2.8

P-157(1)	6	100	GS	120	21.959	2.8
P-157(2)	12	150	PVC	150	21.959	1.24
AUP-622	8	150	PVC	150	21.959	1.24
AUP-623	16	200	DCI	104	15.326	0.49
AUP-624	4	200	DCI	104	15.326	0.49
AUP-625	20	200	DCI	104	15.326	0.49
AUP-626	13	200	DCI	104	15.326	0.49
AUP-627	7	200	DCI	104	15.326	0.49
AUP-628	20	200	DCI	104	15.326	0.49
AUP-629	14	200	DCI	104	15.326	0.49
AUP-630	6	200	DCI	104	15.326	0.49
AUP-631	20	200	DCI	104	15.326	0.49
AUP-632	15	200	DCI	104	15.326	0.49
AUP-633	5	200	DCI	104	15.326	0.49
P-739	8	50	PVC	150	1.276	0.65
P-740	50	191.3	PVC	150	-6.512	0.23
P-741	49	191.3	PVC	150	-6.512	0.23
P-742	44	191.3	PVC	150	-6.512	0.23
P-743	50	191.3	PVC	150	-6.512	0.23
P-744	51	191.3	PVC	150	-6.512	0.23
P-745	50	191.3	PVC	150	-6.512	0.23
P-746	49	191.3	PVC	150	-6.512	0.23
P-747	50	191.3	PVC	150	-6.512	0.23
P-729.5	19	191.3	PVC	150	-6.512	0.23