THE WATER SUPPLY AND DEMAND ANALYSIS IN THE CASE OF BONOSHA TOWN SHASHOGO WOREDA HADIYA ZONE SNNPR

MSC THESIS

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HAWASA UNIVERSITY HAWASSA, ETHIOPIA

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THE WATER SUPPLY AND DEMAND ANALYSIS IN THE CASE OF BONOSHA TOWN SHASHOGO WOREDA HADIYA ZONE

SNNPR

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ADVISORS' APPROVAL SHEET

(Submission Sheet-1)

This is to certify that the thesis entitled "**THE WATER SUPPLY AND DEMAND ANALYSIS IN THE CASE OF BONOSHA TOWN SHASHOGO WOREDA, HADIYA ZONE SNNPR**" submitted in partial fulfillment of the requirements for the degree of **Masters** of Science in **Water Resource Engineering and Management**, the Graduate Program of the Department/**Faculty of Biosystems and Water Resources Engineering**, and has been carried out by **Timotiwos Tufa** , under my/our supervision. Therefore, I/we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

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We, the undersigned, members of the Board of Examiners of the final open defense by **Timotiwos Tufa** have read and evaluated his/her thesis entitled "**THE WATER SUPPLY AND DEMAND ANALYSIS IN THE CASE OF BONOSHA TOWN SHASHOGO WOREDA, HADIYA ZONE SNNPR**", and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree of **Master of Science in Water Resource Engineering and Management.**

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Final approval and acceptance of the thesis is contingent upon the submission of the final copy of the thesis to the School of Graduate Studies (SGS) through the Department/School Graduate committee (DGC/SGC) of the candidate's department.

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Declaration

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

Name: Timotiwos Tufa

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This MSc Specialty or equivalent thesis has been submitted for examination with my approval as thesis adviser

Nama: Mihiret Dananto (PHD)

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Place and Date of Submission: ________________________________

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LIST OF ABBREVIATIONS

BTWSSE	Bonosha Town Water Supply and Sewerage Enterprise
C	Hazen-Williams coefficient
T	Tank
D	Diameter
DI	Ductile iron
DCI	Ductile cast iron
Fig	Figure
GI	Galvanized iron
Km	kilometer
L	Length
l/s	liter per second
Lpcd	Liter per capital per day
m	meter
m/s	meter per second
mm	millimeter
Q	discharge
UFW	Unaccounted for water
V	velocity
MDD	Maximum day demand
PHD	Peak hour Demand
ADD	Average day demand
LHD	Low hour demand
J	Junction
P	Pipe
R	Reservoir
T	Tank
PRV	Pressure reduce valve
MDG	Millennium Development Goal
NRW	Non Revenue Water
O&M	Operation and Maintenance
MWR	Ministry of Water Resource

ABSTRACT

The main objectives of this study was to analyze water supply and demand of Bonosha town water distribution system. The supply part was assessed by collecting data of water source yield in town, which is located north of the town. The town has single water source, which has 4 l/s yield. To evaluate the demand of the town population, number of schools, churches, mosques, health institution, government office and restaurants are assessed. To analyze the existing water distribution system, the EPANET model was developed.The model used to identify the level of velocity through pipe and the zone of high and low pressure. The result of research shows that the non-domestic and domestic water supply coverage of the Bonosha town is 30%. This implies that there is high gap between demand and supply. In addition, from three years obtain data of production and consumption the average loss become 20.18%. Furthermore, the model analysis result shows that there significant problem of pressure and velocity in the system. The majority of junctions have negative pressure. This implies that user are not getting sufficient water. Moreover, the problem of Bonosha town water is insufficient yield, design problem and increasing population. The pump failure is one of major challenge in town.

Key Words: EPANET Hydraulic, Water supply and demand

CHAPTER ONE

1. Introduction

1.1. Back Ground

Access to safe drinking water supplies and sanitation services in Ethiopia are among the lowest in Sub-Saharan Africa. Access to safe potable water for urban areas was 91.5 per cent, while the access to potable water in rural area is about 68.5%1 (within 1.5 km) in the year 2010. However, increasing the number of people with access to safe water supply, sanitation and hygiene has proven to be a tremendous challenge throughout the developing world. Despite huge investments over the years in the water and sanitation sector in the country millions of rural and urban poor communities, still remain without adequate water supply and lack improved sanitation services. Although numerous schemes have been planned and implemented in Ethiopia, only a proportion of these schemes continue to provide water to the communities that they were intended to serve. The failure in service may have been caused by a multitude of reasons including poor design and technology selection, insufficient maintenance, inadequate community planning or participation and many others. Sefiu.A .et.al(2010).

Due to its unreliability and non-sustainable nature, the existing service level in different parts of Ethiopia is lesser than the required levels. The poor water supply and sanitation facilities, along with other infrastructure services, bear a high level of impact on national and regional development, inclusive of both urban and rural communities. The Ethiopian government‟s efforts on implementing a water sector development program are based on the national resource management policy. The national water resource sector strategy is a part of the water sector development program. The government receives a credit or grant for the urban water supply and sanitation (UWSS) component of the project. At the regional level, the Water Resource Bureau is engaged in planning, distributing, operating, maintenance, monitoring, and evaluating urban water supply and sanitation projects. Initially, there were scattered efforts in Water Supply and Sanitation to improve the provision of clean and adequate water supply and sanitation facilities, yet their coverage remains low and inadequate. AMCoW(2015).

The Southern Nations, Nationality and Peoples Regional Government (SNNPRG) is located in the south-western part of Ethiopia. It has an area of about 118,000 square kilo meter with a population of about 13million and Over 91% of the population lives in rural areas. BoFED. (2015).

1.2. Statement of the Problem

Bonosha town water supply distribution system is responsible for supplying water for the town.There is only one water source which has potential of 4 l/sec that is serving the town. Usually this source faces pump defect due to power fluctuation. As the result of pump defect, the drinking water problem aggravated. The water distribution problem include low service coverage and irregular mode of water distribution system characterized by frequent cut-off and technical incompetence with recent two years. The consequence of poor water supply service in terms of health and financial costly, especially to the low-income households could be enormous and could affect national economic development. The shortage of water distribution observed especially during rainy season. The problem could be attributed to limited or scare water sources, inadequate hydraulic capacity and aging water distribution infrastructure coupled due to increasing water demand.

The pump defect aggravated the town water supply problem and community enforced to buy 20-liter jar by 10 birr up to 15 birr. Some poor family are obliged to drink Bilate river which is located beside the town . A systematic investigation will help to to establish the position of Bonosha water supply distribution in terms of its technical, financial and management capacity to develop the mechanisms and improving the overall capacity of the utility to deliver better service. However, in the study area there are no study conducted to my knowledge on water supply and demand analysis and performance evaluation of water supply distribution system. Thus, this study will identify the possible technical problems with regard to demand supply scenario, hydraulic performance, water loss and there by recommend possible solution.

1.3. Objective of the study

1.3.1. General objective

The main objective of the study was to assess Water supply and demand scenario in Bonosha town.

1.3.2. Specific objectives

The specific objective of this research includes:

- To quantify the gap maginitude between current and future supply and demand in Bonosha town
- To analyze Hydraulic performance of the water distribution system
- To evaluate the institutional arrangement and management capacity of the utility
- To recommend the corrective measure which ensure the sustainability of the town water supply system.

1.4. Research Question

1. What is remedial measure to satisfy demand?
2. What is the performance of Bonosha town water distribution sytem ?
3. How much water is lost compared to produced water in town ?

2. Literature review

2.1. Introduction

The developing cities like cities in Africa, south Amrica and Asia have great difficulty in both technical and financial capacity to develop and expand water supply projects. One of the difficulties among the other is imbalance between demand and supply, losses of water by various ways in all level of the distribution system and poor management of the scheme. As a result the distribution of water among the available resource becomes unsatisfactory. Because of the poor management, the existing infrastructure asset increases the level of losses in the water supply and results demand gap. As this research deals with imbalance between demand and supply, bursting and leakage problem of pipe system and hydraulic modeling issues related will be reviewed in this chapter.

2.2. Water demand and Coverage

2.2.1. General

"The MDG drinking water target, to halve the proportion of the population without sustainable access to safe drinking water (an increase in coverage from 76% to 88%) between 1990 and 2015, was met in 2010. Between 1990 and 2012, 2.3 billion people gained access to an improved drinking water source, raising global coverage to 89% in 2012. In a further 35 countries, 26 of which are in sub Saharan Africa, coverage of improved drinking water supply was between 50% and 75%. 1.6 billion gained access to a piped supply on premises, and 700 million gained access to an improved supply, which could range from a public tap to a hand pump, protected dug well or protected spring. Within Southern Asia, India increased access for 534 million people, and within Eastern Asia, China increased access for 488 million people, greatly contributing to both regional and global increases in coverage.197,060 people with access to improved safe sanitation in temporary locations (UNICEF Annual Report, 2020)

2.2.2. Urban Water Supply coverage

All sources confirm that water supply coverage in Ethiopia is on a strong upward trajectory. According to official government data, water supply coverage has risen in 1990 from (11 percent rural, 70 percent urban) to in 2009 (62 percent rural, 89 percent urban).As Figure 1 shows, based on the official government data, Ethiopia has already met the MDG target of 60 percent. Estimates of current coverage from the international Joint Monitoring Program (JMP) are significantly more cautious, due to a range of factors . Nevertheless, the JMP data still portray a remarkable increase in coverage of over 1 million people per year (1990–2008).AMCoW(2015)

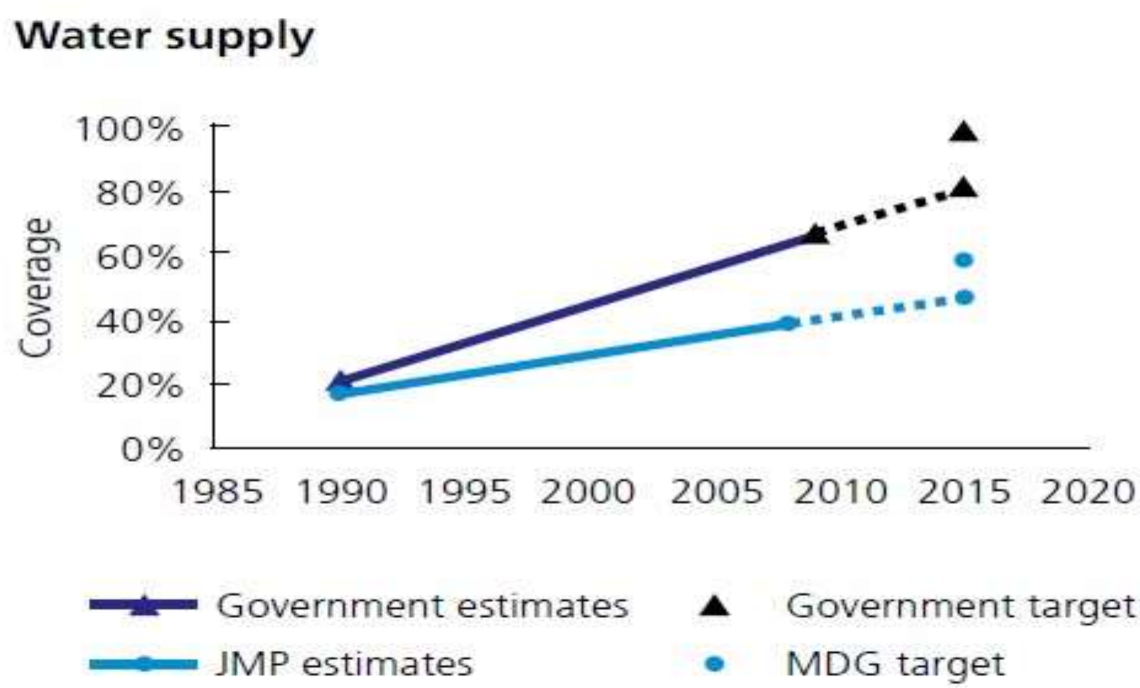

Figure 1. Ethiopia Water supply and sanitation coverage(source : Gov & JMP Report of 2015)

A municipal water supply system has the objective of providing an adequate and reliable water supply to meet the following demands:

- Residential occupancy water consumption;
- Commercial occupancy water consumption;
- Industrial occupancy consumption;
- Municipal and educational building use; etc.

- Needed Fire Flows (NFFs) that are available from a planned location of fire hydrants throughout the municipality; and
- Water for special community needs that include parks and recreation, street cleaning, decorative water fountains, sale of water to contractors through metered water from fire hydrants, etc. Harry E.(2008)

The problem of inadequate access to water and sanitation exists in both rural and urban areas, the problem is particularly pressing in cities. With internal migration and the "urbanization of poverty," cities are where an increasing proportion of the poor live. In the last three decades, growth in urban populations in developing countries exceeded that of rural areas three times more. USI (2012)

The water, sanitation and hygiene infrastructure of many cities is therefore stressed beyond current capacity and infrastructure investments have not kept pace with rapid and unplanned urbanization. There may be smaller-scale, off-grid, innovative supply solutions. The solutions necessarily require clever innovations in design of contracts, pricing policies, and market development. USI identifies three key barriers:

Insufficient supply: Building water and sanitation infrastructure is costly and may involve numerous technical, bureaucratic, and legal constraints–particularly in the developing world. There may be smaller-scale, off-grid, innovative supply solutions like solar pump, but realizing those solutions requires clever innovations in design of contracts, pricing policies, and market development.

Insufficient demand: Even in places where water and sanitation network exist, due to the limited demand to some services, it is technically feasible to connect to it. Some reasons and factors which could be seen as the cause of the limited demand can be also mentioned. These are lack of willingness-to-pay, different people‟s demand may be inter-linked, and the presence of transient or migrant populations may not be available as potentially dedicated customers.

Institutional constraints: if regional and local levels of government are not involved to facilitate implementation in the local context, centralized supply solutions may not be sustainable or even possible at all. In addition, coordination problems can arise when the sanitation or water infrastructure is shared and must be jointly maintained.

2.2.3. Municipal Water System Demands

Demand is an informed expression of desire for a particular service, measured by the contribution people are willing and able to make to receive this service. The contribution does not have to be monetary, but should be: Perceived by the potential user as affordable Sufficient to empower the user as a consumer with associated rights and responsibilities and Related to the cost of the associated option, in order to facilitate the achievement of cost recovery objectives. Paul et.al.(2001)

The demand for water supplied by a municipal water system has two driving components. consumer consumption: the amount of water per day that is used by all of the taps on the water mains to supply single-family homes, multiple-family residences of all types, health care facilities, schools at all levels of education, commercial enterprises, industrial complexes, and adjunct uses (street cleaning; water fountains; watering public parks and recreation including swimming pools; and the sale of water to contractors for building roads, structures, etc.) and An adequate and reliable water supply for fire protection. Consumer consumption: Consumer consumption is assessed by determining the amount of water that actually is used by consumers, based on three levels of usage as follows: Average daily consumption (ADC).The total amount of water used in a day by the population and does not consider usage by different classes of occupancy including commerce and industry. AWWA reports this figure varies considerably by State and region.

1) **Maximum Daily Consumption (MDC)**: This value represents the single day within a year-long period on which the consumption rate was the highest.

2) **Instantaneous flow demand**: There are generally two peak periods in the day when consumption is greatest: between 7 a.m. to 9 a.m. and between 5 p.m. to 7 p.m.

Fire flow demand: At any time, the municipal water supply system should be able to deliver needed fire flows to representative fire risks throughout the municipality from properly located fire hydrants. An adequate amount of water is essential to confining, controlling, and extinguishing hostile fires in structures. The actual amount of water needed differs throughout a municipality, based on different building and occupant conditions. Therefore, water damage for structural fire protection must be determined at a number of different locations throughout a given municipality or fire protection district. These locations are selected by the Insurance Services Office, Inc. (ISO), to represent typical fire risks, including residential, commercial, institutional, and industrial properties for insurance rating purposes. Harry E.(2008).

2.3 Hydraulic modeling

In hydraulic simulation modeling a distribution network is considered to be one in which all elements are connected to each other, every element is influenced by its neighbors, and each element is consistent with the condition of all other elements. These conditions are mainly controlled by two laws: Law of Conservation of Mass and Law of Conservation of Energy. “Thus the total mass of water entering the system should be equal to the total mass of water leaving the system, and the sum of the flows at any given node should be equal to zero. The principle of conservation of energy is mainly dictated by the Bernoulli’s equation, which states that the difference in the energy between any two points should be the same regardless of the path taken” (Haestad Methods, 2003).

2.4. Water Loss and Leakage

2.4.1. General

In hydraulic simulation modeling a distribution network is considered to be one in which all elements are connected to each other, every element is influenced by its neighbors, and each element is consistent with the condition of all other elements. These conditions are mainly controlled by two laws: Law of Conservation of Mass and Law of Conservation of Energy. “Thus the total mass of water entering the system should be equal to the total mass of water leaving the system, and the sum of the flows at any given node should be equal to zero. The principle of conservation of energy is mainly dictated by the Bernoulli’s equation, which states that the difference in the energy between any two points should be the same regardless of the path taken” (Haestad Methods, 2003).

Before proceeding to discuss about causes, consequences and other aspects of leakage, it will be good to examine the types of leaks that are commonly encountered and their category of occurrence (where they can be located) in a distribution network of water supply system. Leaks can be categorized in different ways like physical and administrative losses. Physical losses can be caused by leaks which may occur in any part of the system like transmission pipes, service reservoirs, pumps, distribution networks, and house connections. Whereas administrative losses can be related to illegal connections, faulty (under-registering) or broken meters, inaccurate billing, etc. Allan.L(2003).

Table 1. The IWA 'best practice' standard water balance

<table>
<tr><td rowspan="9">system input Volume (corrected for known errors)</td><td>Authorized consuption</td><td>Billed Authorized consumption</td><td>Billed metered consumption (including water exported)</td><td rowspan="3">Revenue water</td></tr>
<tr><td rowspan="8">Water loss</td><td>Unbilled Authorized consumption</td><td>Billed unmetered consumption</td></tr>
<tr><td>Apparent Loss</td><td>Unbilled metered consumption</td></tr>
<tr><td rowspan="6">Real Losses</td><td>Unbilled unmetered consumption</td><td rowspan="6">Non-Revenue water (NRW)</td></tr>
<tr><td>Unauthorized consumption</td></tr>
<tr><td>Customer metering inaccuracy</td></tr>
<tr><td>Leakage on transmission and or distribution mains</td></tr>
<tr><td>leakage and overflows at utility`s storage Tanks</td></tr>
<tr><td>Leakage on service connections up to point of customer metering</td></tr>
</table>

Source. IWA WATER LOSS TASK FORCE (Water 21 - Article No 2)

2.4.2. Occurrence of Leaks

Generally, leaks can be divided into five main categories depending where they commonly occur or may be located. The five major categories are:

Water main leaks typically range from 0.1 l/s to over 70 l/s. Leaks due to corrosion of water mains usually start through small holes but can grow to very large leaks. Mostly Causes on excessive pressure, poor workmanship, settlement of trenches overloading, and improper materials, and temperature stresses are the major ones.

Service Line leaks typically range from as low as 0.03 l/s to over 1 l/s. The causes for service line leaks are the same as for the mains.

Customer Meter Box leaks (near or within meter boxes of customers) range from 0.1 l/s to 0.7 l/s. Leaks may be caused by loose nuts on the meter, broken or damaged couplings and broken or damaged meters.

Customer Connection Line leaks (customer side of the line) range from 0.1 l/s to 1.0 l/s; holes or breaks in customer connection lines and shutoff valves may cause these leaks.

Valves and Appurtenance leaks in distribution system typically range from 0.07 l/s to as high as 30 l/s; loose connection and broken valves are common causes for this type of leak. Valve leaks in a water supply system may be observed in isolating valves, pressure reducing valves, drains and air-release valves. (Water Aid Ethiopia,2010).

2.4.3. Non-Revenue Water

2.4.3.1. General

The waste of resources resulting from high NRW levels in developing countries is considerable. The total cost to water utilities caused by NRW worldwide can be conservatively estimated at $141 billion per year, with a third of it occurring in the developing world. In developing countries, about 45 million cubic meters are lost daily through water leakage in the distribution network which is enough to serve nearly 200 million people. Similarly, close to 30 million cubic meters are delivered every day to customers, but are not invoiced because of pilferage, employees‟ corruption, and poor metering. All this directly affects the capacity of utilities in developing countries to become financially viable and fund necessary expansions of service, especially for the poor**.** (World Bank 2006).

Table 2. The International Non-Revenue Water Assessment Matrix

Non-Revenue water in lit/con/day when the system is pressurised at average pressure						
NRW management performance category		10	20	30	40	50
High income country	A1		<50	<65	<75	<85
	A2		50-100	65-125	75-150	85-175
	B		100-200	125-250	150-300	175-350
	C		200-350	250-450	300-550	350-650
	D		>350	>450	>550	>650
	A1	<55	<80	<105	<130	<155
Low and middle income country	A2	55-110	80-160	105-210	130-260	155-310
	B	110-220	160-320	210-420	260-520	310-620
	C	220-400	320-600	420-800	520-100	620-1200
	D	>400	>600	>800	>1000	>1200

Source: R.Leimberger . Recommendation for initial NRW assessment .IWA water loss 2010. sao Paulo, Brazil. June 2010

This matrix is based on an extreme simplification. Commercial loss allowances are based on an assumed average billed consumption per connection of 1,000 liters per day. This means that in systems with substantially higher average consumption, the values might be difficult to achieve, especially in the “A” categories.

Category A1: World-class NRW management performance; the potential for further NRW reductions is small, unless there is still potential for pressure reductions or the accuracy improvement of large customer meters.

Category A2: Further NRW reduction may be uneconomic, unless there are water shortages or very high water tariffs; a detailed water audit is required to identify costeffective improvements.

Category B: Potential for marked improvements; establish a water balance to quantify the components of NRW; consider pressure management, better active leakage control practices, and better network maintenance; improve customer meter management, review meter reading, data handling and billing processed, and identify improvement potentials.

Category C: Poor NRW record; tolerable only if water is plentiful and cheap; even then, analyze level and causes of NRW and intensify NRW reduction efforts.

Category D: Highly inefficient; a comprehensive NRW reduction program is imperative and high priority. Rudolf F.et.al(2010)

Not all the unaccounted-for water is physically lost. Either the utilities master meter or some of the customers‟ meters may be inaccurate. In cases of inaccurate customers‟ meters, the water is "lost" as far as accounting and billing is concerned, any meter inaccuracies make it difficult to assess the amount of water actually lost through leaks. Therefore it is important for a utility to reduce metering errors. Customer meters usually tend to under register. Sometimes customer meters do not work at all. Meters that under register or which do not function properly may account for significant water and revenue losses. An ongoing customer meter testing and replacement program is essential to alleviate this problem. The acceptable level of unaccounted-for water differs from community to community. A 15-percent loss has generally been considered acceptable for large water utilities. It is probably profitable to control any loss above 10 percent. Most authorities recommend implementing a water loss control program where losses exceed 20 percent or more.(Bill kingdom et.al(2006).

A case study was conducted by Metaferia Consulting Engineers for Water Aid Ethiopia in 2010 in seven towns which reveals the extent of leakage of each town. Summary of the study result is shown in table below.

Table 3. Level of Non-Revenue water in towns of Ethiopia

S/N	Town	Level of leakage
1	Welkite	43%
2	Butajira	22%
3	Burayo	23%
4	Hossaena	51%
5	Miza-Aman	61%
6	Sebeta	21%

Source. Case Study Report of Water Aid Ethiopia (2010)

2.4.3.2. The Importance of Registered Services in UFW Assessment Water utilities need some means of accurately measuring the water that is delivered to the consumer through the distribution network. If the meters are of the recording type, valuable information regarding hourly rates of consumption will be available. Metering of delivered water consists of placing a recording meter in the line leading from the water main to the area served. The area could be a district or zone. Consumers are then billed for the water they use based on reading of water meters installed at their yards. The alternative to these methods is charging the consumers on the basis of flat rates, which has no relation to the actual consumption or to the amount wasted. Billing by metering is advantageous. Pumping and treatment cost money and wasting of water means a greater cost to the utility, which in turn will be distributed among the customers. If deliveries are unmetered, the careful consumers are made to bear the burdens imposed on them by the careless and wasteful ones. It is almost impossible to construct a good system of water charges without the use of water meters and hence the necessity for the use of it to collect water charges based on actual consumption**.** James et. al.(2005).

2.3.4. Leakage Detection

Old or poorly constructed pipelines, inadequate corrosion protection, poorly maintained valves and mechanical damage are some of the factors contributing to leakage. Leak detection has historically assumed that all, if not most, leaks rise to the surface and are visible. In fact, many leaks continue below the surface for long periods of time and remain undetected. With an aggressive leak detection program, water systems can search for and reduce previously undetected leaks. Water lost after treatment and pressurization, but before delivered for the intended use, is water, money and energy wasted. Accurate location and repair of leaking water pipes in a supply system greatly reduces these losses. Once a leak is detected, the water utility must take corrective action to minimize water losses in the water distribution system.

Advances in technology and expertise should make it possible to reduce losses and unaccounted for water to less than 10 percent. Every industrial and commercial water system facility should implement cost effective water loss control measures that will minimize distribution system water losses. Water systems with pressurized distribution systems should promote water auditing, leak detection, and leak repair as a means to reduce operating costs and conserve water. The water audit can be used on systems with customer meters, while leak detection and repair can be used on any pressurized water system. EPD, (2007).

2.3.5. Pressure and Leakage

The efficiency of a distribution system can be judged on the basis of the pressure available in the system for a specific rate of flow. Pressures should be great enough to adequately meet consumer needs. At the same time they should not be excessive and as pressure increases, leakage increases and money is then spent to transport and process a product that is wasted.

Utilities are always expected to provide adequate and safe drinking water with sufficient pressure at all delivery points. Pressures at consumers yard connections have to be as close to the minimum level as possible, though this is supposed to be determined by local authorities. The minimum pressure at a customer‟s connection is in the order of 15 meters while the maximum is as high as 40 meters; pressure range in excess of 60 to 70 meters will damage house installations such as boilers, float valves, taps, gaskets inside the fittings, etc. Pressure variation in a distribution network is caused, amongst others, by changes of demand of the users. Frequent starts and stops of pumps, closure and opening of control valves that induce water hammer are also some of the causes to be mentioned for pipe breakage and water loss.

The pressure will be low during daytime with increase of demand and high during night hours when the demand is low. There is an excess pressure build-up in a network when demand drops especially during the night. Obviously, there is the need to cut down this unduly excessive pressure in order to avoid the bursting of pipes or reduce the amount of leakage. Variations of pressure may cause frequent pipe bursts or damages. Studies shows on the effect of high pressure on losses, particularly leakage from pipes, have indicated that leakage is almost proportional to the service pressure.

The following methods can be envisaged to achieve the cutting down of the excessive pressure:

Adjust the speed of pumps to maintain reasonably constant pressures in the distribution network for areas supplied directly by pumps.

Install a pressure-reducing valve (PRV) or

Divide the system into pressure zones.(Water Aid Ethiopia,2010)

2.4. Design and operation of piped networks

2.4.1. Introduction

In design and operation of piped networks the purpose of a system of pipes is to supply water at adequate pressure and flow. However, pressure is lost by the action of friction at the pipe wall. The pressure loss is also dependent on the water demand, pipe length, gradient and diameter. Several established empirical equations describe the pressure flow Relationship and these have been incorporated into network modeling software packages to facilitate their solution and use. When designing a piped system, the aim is to ensure that there is sufficient pressure at the point of supply to provide an adequate flow to the consumer .Pressure at any point in the system should be maintained within a range whereby the maximum pressure avoids pipe bursts and the minimum ensures that water is supplied at adequate flow rates for all expected demands. Hydraulic models are used to identify where, when and how negative and high pressures may occur and help to identify Preventative measures such as system controlling methods and for selection of pipe materials according to the pressure. Kay Chambers et,el(2004).

2.4.2. Model skeletanization

As Elsheikh (2013**)** reviewed that, in order to skeletonize the model to an acceptable degree, the study should follow the listed conditions such as: (1) At least 50% of total pipe length in the distribution system (2) At least 75% of the pipe volume in the distribution system (3) All 300 mm diameter and larger pipes; (4) All 200 mm and larger pipes that connect pressure zones, influence zones from different sources, storage facilities, major demand areas, pumps and control valves (5) All 150 mm and larger pipes that connect remote areas of a distribution system to the main portion of the system; (6) All storage facilities with controls or settings applied to govern the open/closed status of the facility that reflects standard operations; and (7) All active pump stations with realistic controls or settings applied to govern their on/off status that reflects standard operations.

Developing the hydraulic model for network follow the following steps.(1) Creating a pipe network from the water system GIS files using Water GEMS (Bentley Systems) and ArcGIS (Environmental Systems Research Institute); (2) Spatially allocating customer demands and pipe leakage to pipe network nodes; (3) Assigning elevations to pipe network nodes; (4) Incorporating boundary condition elements (e.g., pumps, ground storage reservoirs); and (5) Performing calibration and a quality control review of the resulting model.

2.4.3. Calibration

As Elsheikh (2013) reviewed the discussion and finding of Walski(1983) and ATSDR (2000). described 7 steps process for model calibration, including: identifying model use, determining parameter estimates, collection of calibration data, evaluation of model results, macro calibration, sensitivity analysis, and micro calibration. Pipe grouping for roughness calibration was based on pipe type, age, and size. The optimization approach for calibration was compared to the analytical and simulation approaches based on skeletonizing of the system by removing pipes.

Walski [1983] suggested that pressure-measuring devices should be located near points of high demand, near the perimeter of the skeletonized network, and generally distant from water sources. ATSDR(200) illustrated that an average pressure difference of ± 15.2 kPa (± 1.51 m) with a maximum difference of ± 50.3 kPa (± 5.03 m) represents a "Good" data set and an average pressure difference of ± 29.6 kPa (± 2.96 m) with a maximum difference of ± 97.9 kPa (± 9.79 m) represents a "Poor" data set.

2.5. Institutional Assessment on Water Supply System

Institutional assessment of water supply system will help to identify poor operation and maintenance situation of relevant functions like defective design, ineffective supervision, insufficient training, lack of and absence of clarity of roles which consequently the water supply components fail to operate at optimum efficiency. WHO (2006).

The water schemes assessment made by Amy.et.al.(2010) from nine zone of Amhar Regional State from observed 32 water supply schemes in the study area they found that it is only 32% of the schemes are functional whereas the remaining 68% is either completely non-functional (14%) or functional with disrepair (54%) under the current management arrangement. However, if those schemes functioning with some disrepair are not properly maintained, they will stop functioning within a short period of time. Consequently, non-functional schemes in the study area will rise to 59%. Among those functioning with some technical breakdowns, damage of the faucets and valves are the major disrepair followed by leakage from pipes and poor construction of the scheme‟s components.

The sustainability of water supply facilities mainly depends on a timely and regular maintenance and operation of the system MoWR (2002). However, in most developing countries, including Ethiopia, it has been found out that operation and maintenance (O&M) of water supply facilities is in a poor state of condition and the sustainability of the scheme is at stake. Regarding this, MoWR (2002) identified the following underlying problems.

In appropriate tariff setting without emphasis on full cost recovery; Lack of clear guidelines for urban tariff setting including issues related to fairness, and financial sustainability; Inappropriate or lack of institutional incentives for urban WS to achieve financial viability and improved operational performance; Poor technical and financial capacity among the urban service providers that leads to high levels of unaccounted for Water (UFW); and Poor or nonexistent consumer services and grievance handling system that leads to a lack of willing to pay user charges.

3. Methods and materials

3.1. Description of the study area

3.1.1. General

The study area is located in southern nation and nationality regional state of Ethiopia and the south west of Addis Ababa, the capital of Ethiopia in between 7 and 8^0 Latitude North and between 37 and 39^0 Longitude East of and at a distance of 266 km south west of Addis Ababa, 121 km to the North of Regional Capital Hawassa, and 54 km East of the zonal capital, Hossana town. The total population of the town 25490 with annual growth rate of 4.5% .

The town has started getting water supply system in 2004.From the period of started up to 20019 the water source used by the town is deep well.

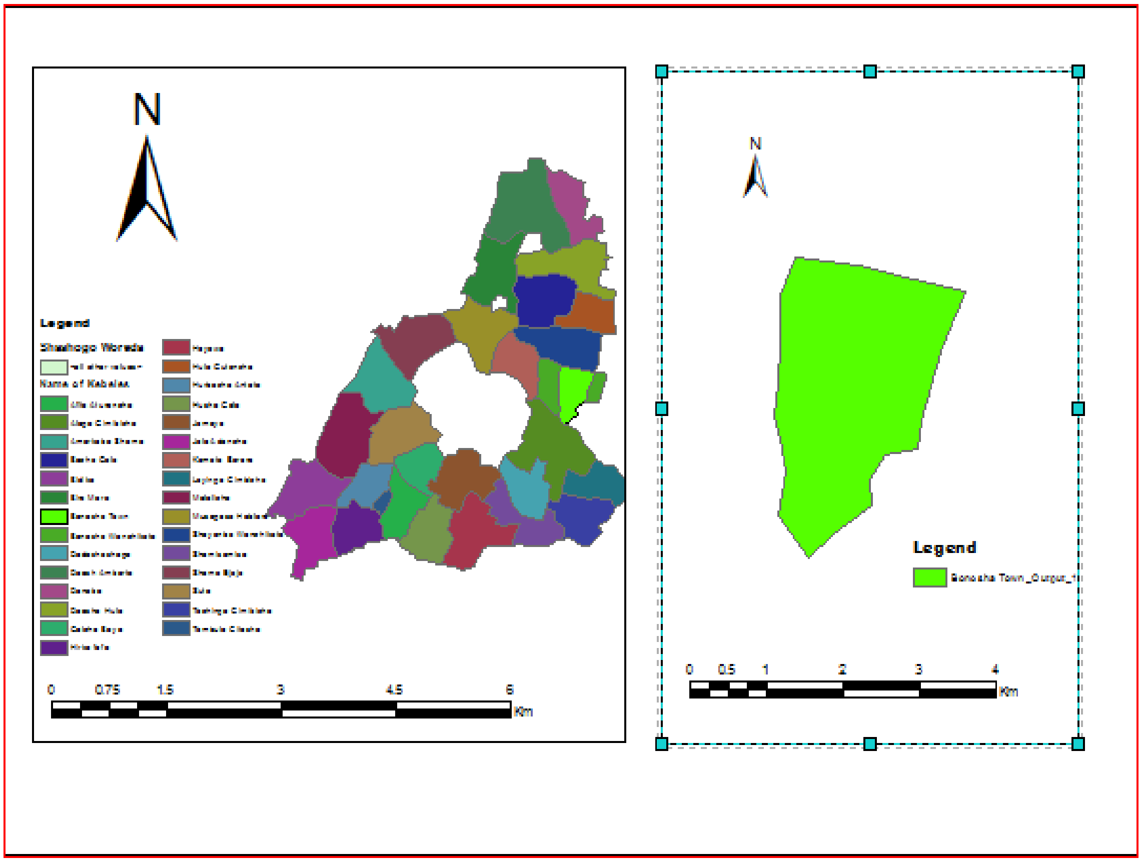

Figure 2. Location Map of the Study Area

3.1.3.Existing Water Supply System

The source of water supply for the town is ground water, which was constructed in 2004 as an emergency water supply for the town with discharge of 5l/s and flows for 24 hrs. There is also one more solar pumped water supply scheme which was constructed by German government. However this scheme is not functional due to technical problem. The water utility of the town can be responsible for the connection of the service lines with the distribution main and installation of the water meter. The private properties were allowed to be connected only with (1”), $1^{1/2}$” and smaller (Bonosha town water supply utility, 2013) report.

Each customer has to buy his own water meter, register at the town water utility and apply for installation by the water worker staff. According to the water the information from water department each month about 40 to 50 water meter become faulty due to solid particles which stop the turbine of flow the flow meter. The main supplier of water meter is near to the technique college which has been working 80%. The water meter are installed either inside the houses or at outside walls of the house.

3.2. Materials and Methods

The research methodology applied in this study was on different literature review based on articles published in academic journals, reports, conference proceedings and text books on methods and tools applied to evaluate water supply coverage and water loss and its management for water supply distribution system. The first step of this study was evaluation the water coverage of the town. Evaluating the water supply coverage focus on the volume of consumption and level of water connection.

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

1. Domestic demand
2. Public demand
3. Commercial & industrial demand
4. Fire demand
5. System loss
6. Animal water demand
7. Institutional water demand

Water supply coverage of the town was first evaluated before analyzing the water loss using the percentage of mode of service with current customer to determine the level of connection or per -capita demand of the town .Total water production and total billed data are basic instrument to evaluating the water supply coverage and losses of the town distribution system.

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

The water demand was calculated by using population data of town.

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

Table 4 Institutional and Commercial Demand

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

Source: Design Criteria Guide Line of MoWR (2006)

Table 5.Population Vs MDDF and PHF

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

Source: Design Manual of MoWR (2006)

3.2.1. Population Forecasting

From various methods used to forecast the population, the method used by the Central statistic Authority selected for this study.

$Pn = P \quad (\mathbf{1} + \frac{r}{100})^t$ Where: - Pn = population at n years

t = number of years

r = growth rate

P = population at present

The total churches, mosques and restaurant demand in town are assessed to evaluate the total demand of the town. The loss can be evaluated by using data of water production and consumption which is collected by Bonosha water supply and sewerage enterprise (BWSSE) for the purpose of fee collection . The next step would be analyzing hydraulics performance of Bonosha town water distribution system. EPANET hydraulic model software was selected to analyze. To run this model data are collected from Bonosha town water distribution system. By using GPS each nodes elevation, x and y coordinate are collected. Demand on each node, pipe materials, pipe diameter, types of valve, pump specification, flow head and other required by model are collected and used to analysis.. Finally, from analyzed data problems identified and remedial measures are suggested.

To evaluate scheme performance coverage of water supply connection, per capita supply of water, extent of metering of water connections, extent of non revenue water, cost recovery of water supply services and efficiency in collection of water supplied related charges indicators are assessed .

3.3. Selection of sample study area

Since the study did not covared all the study area it needed sampleing mechanism. The major factors considered during the selection of sample areas are:-

- The zone of high problem found
- Topography
- Data availability
- Suitability to identify the problem

3.4. Water Supply Coverage

The Bonosha town water supply coverage evaluated based on the average per capital consumption by using yearly domestic consumption with the current population and level of connection per family by using the total domestic connection as per average family size. The water coverage has been evaluated by considering urban domestic as well as Non-domestic user by using Design guide line of MOW(2006).

3.4.1. Demand Projection

The detail design of the Bonosha Bore hole has been made first in year 2004 with discharge of 4l/s.

- Design horizon –year 2004 up to year 2029
- Design population is taken as 15000
- Per capita water demand is taken as 20 l/day. 60 for public tap users, 20 l/c/day for neighborhood tap shared, 30 l/c/day for yard connections and 70 l/c/day for house connections in the town and 7 l/c/day for the rural areas.
- Population growth rate taken as 4.5%
- Public demand taken as about 36% of the domestic demand
- Unaccounted for water taken as 30% of the maximum day demand
- Total maximum day demand i.e 350 m3/day or 4 l/s
- The maximum day demand assuming 24 hour pumping is 6 l/s.
- Average per capita water demand calculated as 20 l/c/day.

3.5. Water Loss

In order to identify the total loss of water in the study area, the total volume of water supplied to the network distribution system was compared with the actual water consumption. In this case, yearly aggregated production and consumption data obtained from BWSE for three years from 2017 up to 2019 and used to compute NRW. Moreover, the apparent and real loss computed by using IWA water loss task force developed equation by providing input data of length of mains pipe line, number of service connection, total length of service connection from the edge of the street to customer meters.

Three year production and consumption data of the study area used to compute the total loss as shown in table.10 below by using this expression.

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

3.6. Data collection

The Bonosha water supply system have no organized data specially for old distribution network, hence all relevant primary data of main and distribution pipe line network has been collected . the datas are collected by using GPS and skiled experts who knows the junctions. Collected data are the elevation of major junction, x and y cordinate of junction, pipe materials, diametre of pipe, valve types and where they are connected, pump data, the height and diametre of tank, pump capacity ond other demanded data by EPANET hydraulic model. The secondary data of gravity main line used to compare with the surveyed data ,forecasted population ,Consumption and production data and level of connection dates are collected from concerned governmental institutions.

3.5.1.Collected Primary data

- By using GPS surveying instrument elevation data of the 6 km surveyed (1km main Pressure Pipe line up to 100m3 balancing reservoir. ----------see Appendix 1

- ❖ The type of pipe materials, fittings and valves has been collected from the site in order to identify whether the system have material quality problem or not. ---Appedix 4
- ❖ Pressure and discharge test measurement taken at different time and in different location to perform calibration and validation.
- ❖ Discussion with the officials and experts in the concerned government bureau to acquire organizational information on WSS.

3.5.2. Secondary data

- ❖ Total number of population taken from Bonosha town municipality which 25490
- ❖ Per capital demand of the town collected from Bonosha Town Water Supply and Sewerage Enterprise.
- ❖ The amount of customer data as per level of connection is collected which used to analysis the coverage collected from Bonosha Town Water Supply and Sewerage Enterprise
- ❖ The boundary of the water supply system taken from Bonosha Town Water Supply and Sewerage Enterprise
- ❖ Three years Water production and Consumption data used to analysis the Water loss collected from Bonosha Town Water Supply and Sewerage Enterprise
- ❖ Personnel, Income and expenditure data of the BTWSSE Bonosha Town Water Supply and Sewerage Enterprise collected from Bonosha Town Water Supply and Sewerage Enterprise

3.6. Hydraulic Model Analysis

3.6.1. Modeling of water supply system

By using surveyed data of pipe line elevation, Size of pipe, material type data of the pipe line and other relevant parameters the water supplies network developed by using EPANET software.

The hydraulic analysis has been done under the following conditions.

- ❖ Standard pressure
- ❖ Standard velocity

.

3.6.2. Data Organization

Depend on the guide line of EPANET the collected primary and secondary data organized on excel sheet. Elevation, Tank label, pump data, demand, length, pipe size and type of materials are the input of EPANET and velocity at pipe and pressure at every junction was the output.

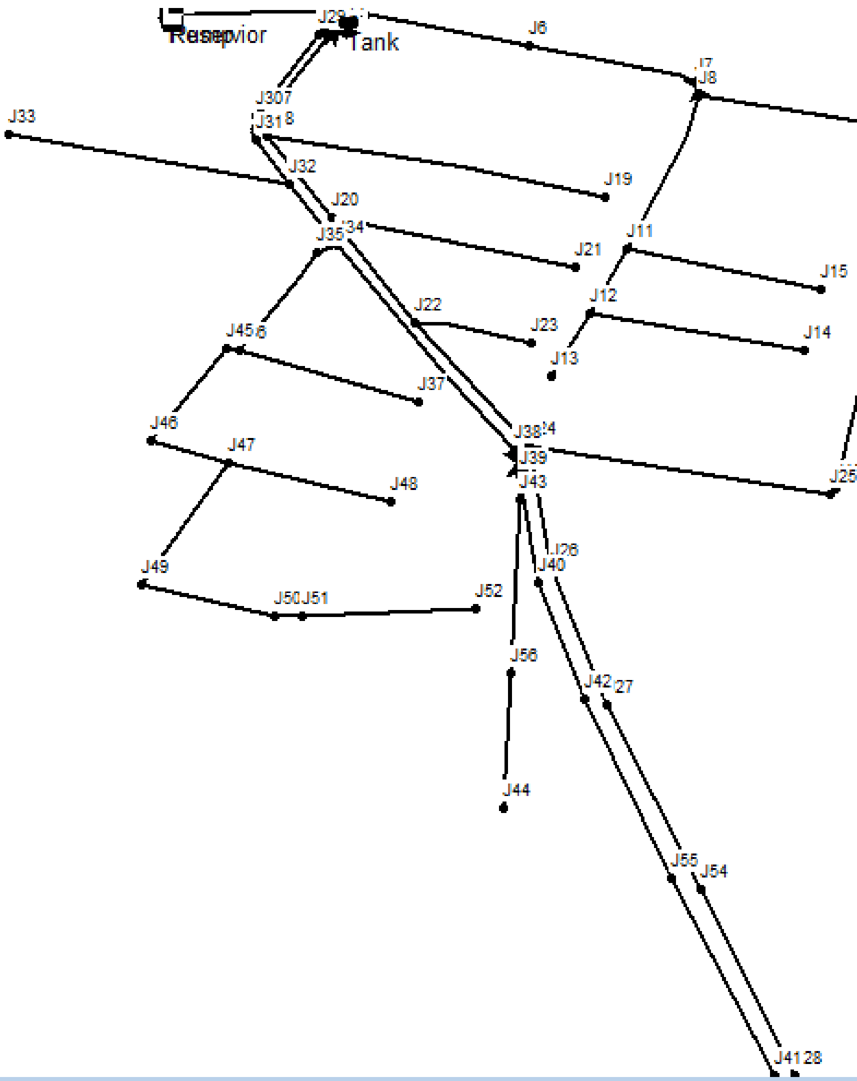

Figure 3. Bonosha Town Water Distribution Layout

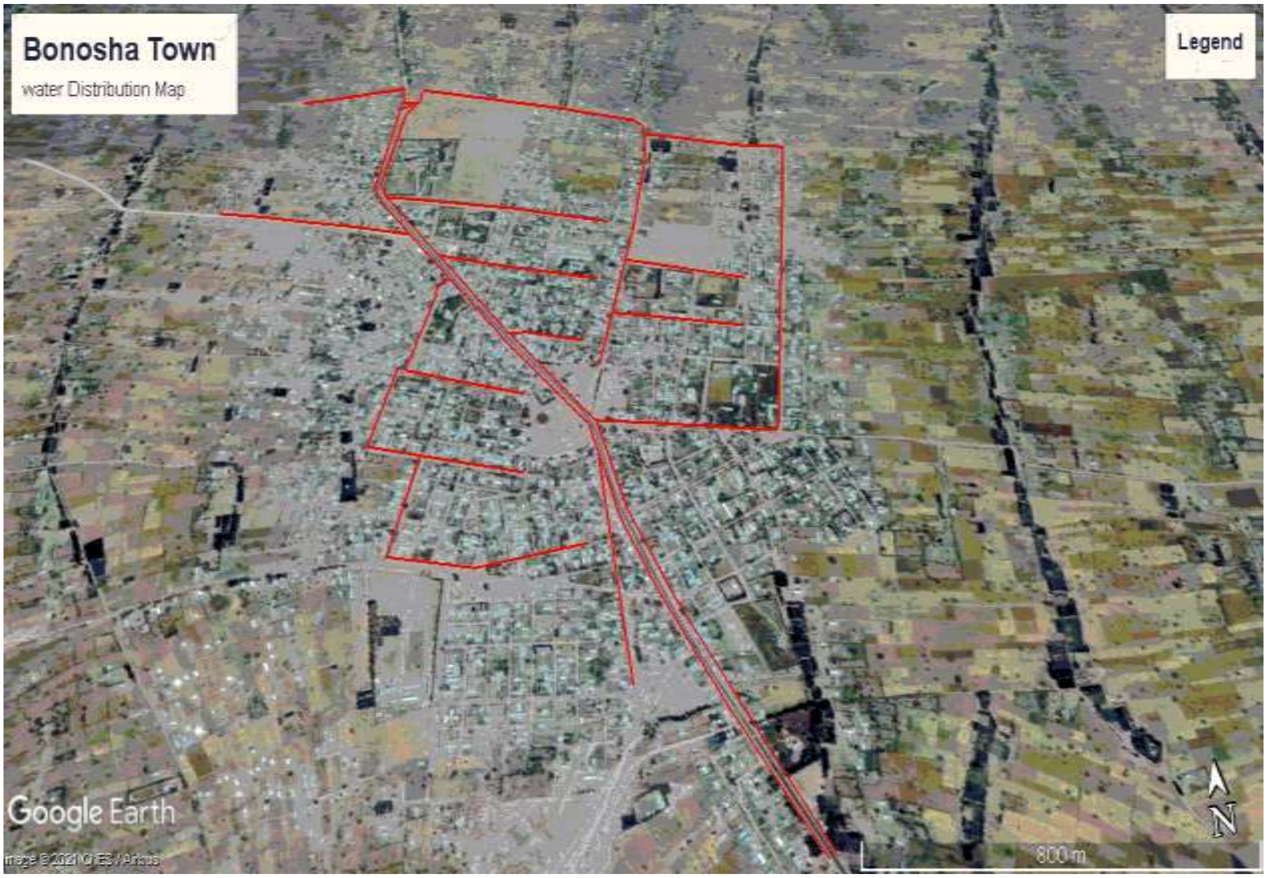

Figure 4. Major junction of study area

3.6.3. Model Calibration and Validation

Once a water distribution model has been developed, it must be calibrated so that it accurately represents the actual working real life water distribution network under a variety of condition. This involves making minor adjustment to the input data then the model accurately simulated the pressure rate in the system. Pressures are measured throughout the water distribution system using pressure gage instrument to use the data for model calibration. The presure measuring gage instribument used for this research was from Bonosha Town Water Supply and Sewerage Enterprise

The computed pressure and measured field pressure will not exactly match, for every node contained within the network system is 85% of field test should be with +- 0.5m the computed or simulated pressure by EPANET software at that nod.

3.6.3.1. Calibration and Validation Statistics

There are many ways to judge on the performance of model calibration, the calibration statistics used in this study was by calculating the squared relative difference between observed and simulated pressure for each test. The results and the observation data were entered to an excel sheet and the value of squared error was calculated for every test then the mean square error and standard deviation calculated from Excel sheet: the lower values of these parameters, the higher is the accuracy of the calibration process.

3.6.3.2. Calibration and Validation data

Devices used for taking field measurements were the static pressure gauges and flow measurement device by Ultra-Sonic waves or water meter to take three types of data as follows: (1) Steady state measurements; (2) Fire flow tests; and (3) Extended period measurements.

The hydraulic model was calibrated by adjusting Hazen-Williams coefficients (C) according to these three types of field measurements data:

3.6.3.3.Pressure and Discharge Test Measurement

By taking the above criteria in to consideration the main pipe line between balancing reservoirs to services reservoir and on selected point of distribution system the level of pressure and discharge measurement has been made at 10 selected points by using pressure gage and water mater instrument.

- The distribution pipe system of the town is old and has no suitable net-work. Hence, difficult to model the whole existing network by using primary survey data, therefore, only 6 km main and secondary pipe line data surveyed and used for net-work model building.

4. Result and Discussion

4.1. Water Demand and Supply Coverage

4.1.1. Water Demanded Coverage Analysis

. To identify the gap between demand and supply three ways are followed, first by using annual domestic consumption identifying the level of per capital demand per person, second computing the per capital demand by level of connection and third by following the design criteria calculating the overall demand coverage.

4.1.2. Urban Domestic Water supply Coverage

To evaluate the amount of water consumption, the annual water consumption is converted to average daily per capita consumption using the population data of the town and the number of domestic connection per family has been also used to analyze the level of connection.

4.1.2.1. Average Daily Per Capita Consumption

The volume of water consumed for domestic purpose has been distributed for all beneficiaries of the town to analyze the distribution of the water supply coverage.

The average daily per capital consumption of the town was computed using this expression

$$\text{Per Capital Consumption(l/person/day)} = \frac{\text{Annual Consumption(m3)} * 1000 \text{ l/m3}}{\text{Population number} * 365 \text{ day}}$$

The annual domestic consumption amount in 2020 is 178582 .5 m3 and the current total population of the town from three sub cities was estimated as 25490 , therefore by using the above expression the average daily per capita consumption became 19.2 l/per capita/day.

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

4.1.2.2. Level of Connection per Family

Level of water connection per family is one mechanism to evaluate the level of water coverage. The total number of connection or water meter with in the town are about 2539 are for domestic users, according to the census of the 1994,average family size of 5.5 is used for calculating the average number of connection per family using the following expression.

$$\text{Connection per family} = \frac{\text{Total number of connection}}{(\text{number of population of the town/Average family size})}$$

The level of water connection as per the above expression became 0.548; this implies that the current connection coverage is only 54.8%.

4.1.3. Analysis of overall Water demand Coverage

4.1.3.1. General

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

Some of the factors that affect water demand are:-

1. **Climatic condition: -** Water consumption during winter is more than summer. During winter no rain and more water used for all activities and also consumption of drinking and bath increased.
2. **Size of the town: -** Generally, the demand of water per head will be more on big city than that in small city. In big cities lot of water is required for maintaining clean and health environments while in small towns more or less small.
3. **Culture of people: -** High class community uses more water due to their better standard of living and high economic status. Middle class people uses water at average rate and for poor people a single water tap may be sufficient for several families.
4. **Industries: -** more water is used in highly industrial city

5. **Cost of water: -** If cost of water is high, the water demand will be less .Hence the rate at which water is supplied to consumer may affect the rate of demand.

6. **Quality of water: -** A water work system having good facility and portable water supply will be more popular with consumers.

7. **Pressure in the distribution system: -** There would be of great importance in the case of localities having number of two or three storied buildings. Adequate pressure would mean an uninterrupted and constant supply of water.

8. **System of supply: -** The system of water may be continuous or intermittent. In continuous system water is supplied all 24 hours .while in the case of intermittent system water is supplied for hours of the day only results in some reduction in the consumption. This may be due to decrease in loss and other waste of full use.

9. **Method of charging: -** In a town where meters are used less quantity of water will be used than in towns without meters in their system. A metered supply ensures minimum of waste as the consumer because of they know there is payment as per consumption.

4.1.3.2. Domestic water demand

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

A. Demand Computation by mode of service

The following are the most common model of domestic service in Ethiopia and are used in Bonosha town.

1. House connected tap users (HTU)
2. Yard Connection shared
3. Public tap uses (PTU)

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

Table 6. Primary water source of sample house hold

Connection Type	% of Population By mode of Service	Number of population per service(2016)
Privet house connection	50	12745
Yard Connection shared	40	10196
Public Tap users	10	2549
Total		25490

Source.From Bonosha town water supply and sewerage enterprise

C. Average Demand by mode of service

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

Table 7 Domestic water demand for different categories of consumer

S/N	Connection Type	Stage 1	Stage 2
1	House Connection	50 l/c/day	70 l/c/day
2	Yard Connection	25 l/c/day	30 l/c/day
3	Yard Connection shared	30 l/c/day	40 l/c/day
4	Public Tap User(Rural)	20 l/c/day	25 l/c/day

Source .Design criteria of MoWR(2006)

Table 8 Analysis of Average Day Demand

S/N	Connection Type	Number of population per service(2019)	Per Capital Demand(l/c/day)	Average Demand(l/c/day)	Average Demand(m3 /day)
1	Privet house connection	12745	50	637250	637.25
2	Privet yard shared	10196	30	305880	305.88
3	Public Tape users	2549	25	63725	63.725
	Total				1006.9

4.1.3.3. Non-Domestic Demand

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

Non – domestic demands are:-

1. Institutional demand
2. Commercial demand

3 .Industrial demand

4 .Fire demand

A. Institutional demand

❖ **Educational demand**

The water demand for educational institutes is classified in to

i. Day schools **ii.**

i. Day Schools

There are two elementary schools with 2670 students, and one senior high school with 2341 students a total 5011 students attending day school according to the Bonosha town administration office education desk and the schools themselves.

Demand =5011*5 l/Stu/day=**25.1m3.**

- **Health institutes**

In 2020, there was one health center and eight private clinics with a total of about 70 beds. The demand by taking average per capita demand per bed of 60lit/capita/bed became

Demand =70 bed *60lit/capita/day=**4.2m3/day**

❖ **Public and governmental offices**: -

According to CSA about 51% of the population of Bonosha town is economically active and 7% of the population of Bonosha is employed in public or government offices. And their consumption is recommended as 5 l/c/d.

=25490*7%*5l/c/d=**11.5m3/day**

B. Commercial demand:-

The water demand of the hotels, bars and restaurants estimated as by taking the consumption in 2020 is **44m3/day.**

C. Churches and Mosques

There are 9 churches and 3 Mosques in Bonosha town. The current (2020) water demand for mosques and churches is 27 m3/day.I.e.18 m3/d for churches and 9m3/d for mosques.

D. Fire Demand

Fire generally breaks out at commercial centers, stores etc. Big cities which have valuable properties require large quantity of water for firefighting. Fire demand is the quantity of water needed to extinguish fire which depends upon population, centers of buildings density of buildings and their resistance to fire. The quantity of water for fighting is small compared to the annual average consumption of water which will not be more than the amount of water distributed during the max day water demand. For this study firefighting purpose 10% added on the reservoir Size.

Table 9. Summary of Non-Domestic Demand

S/N	Institutions	Demand (M3/Day)
1	Day School	25.1
2	Health Institution	4.2
3	Public and Government office	11.5
4	Commercial demand	44
	Total Sum	84.8

Table 10. Summary of Domestic and Non-Domestic Demand

S/N	Item	Demand (M3/Day)
1	Domestic	1007.9
2	Non-Domestic	84.8
	Sub Total	1091.7
3	30% loss(for scheme having 10 year service)	327.51
	Total Sum	**1419.21**

4.1.4. Demand Variations

4.1.4.1. Seasonal Peak

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

4.1.4.2. Peak Day Factor

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

4.1.4.3. Peak Hour Factor

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

- **Calculation MDD and PHD**

Average Day Demand (ADD) = 1419.21 m3/Day

Maximum Day Demand= ADD *MDF = 1419.21 *1.25 = 1774.01m3/day = 20.53/sec

Peak Hour Demand (PHD) =ADD * PHF =1419.21 *1.9 =2488.62m3/day= 31.21 l/sec

4.1.5. Reservoirs

4.1.5.1. General

Operational reservoirs should be provided to command a distribution system, located at elevations providing the required pressure for water flow within the system. They should have sufficient storage to cover the difference between hourly peak demand and actual supply from the source, firefighting demands if to be allowed for, and for a limited emergency volume in case of power breakdown, repairs or O&M activities.

4.1.5.2. Types of Reservoirs

The two main types of reservoir are the ground level type (GLR) and elevated water tank type (EWT). Whenever the local topographical conditions permit, ground level reservoirs are preferable. Ground level reservoirs will be usually be of solid block masonry or reinforced concrete, cylindrical or rectangular but under special circumstances may be of glass reinforced plastic (GRP). Elevated water tanks will be cylindrical or conical in reinforced concrete (M0WR 2006).The study area has one ground concrete reservoir which is functional

4.1.5.3. Reservoir Location

A reservoir location should maintain the desired pressure range in the supply network. Possible future extension of the storage capacity should be taken into consideration when selecting a site.

4.1.5.4. Reservoir Equipment's

Reservoirs should be provided with inlet, outlet, drainpipe, overflow pipe, water level indicator, manhole Ladder, ventilation pipe, lightening conductor.

4.1.5.5. Total Storage Requirement

In order to provide for security of supplies above the need for balancing purposes it is recommended that the minimum total reservoir storage capacity be in the range of 30% to 50% of the average daily demand (M0WR 2006).

The study area has one 100m3 reservoir To check this capacity enough or not the total average day demand as per computed above is 1419.21 m3/day and the level of existing storage computed as:-

= 0.3*1419.21 to 0.5 *1419.21

= 425.76 m3/day to 709.60m3/day

Their for, the storage capacity of existing reservoir has been not on the recommended renege and so far the scheme is not safe regarding to storage capacity .

4.2. Water Loss Analysis

4.2.1. Non-Revenue Water

Non-revenue water is the difference between the volumes of water put into a water distribution System and the volume that is billed to customers. NRW comprises three components: physical (or real) losses, commercial (or apparent) losses, and unbilled authorized consumption.

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

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

A. Total Water loss expressed

Table 11. Computed Total Water Loss

Year	Production(m3/Year)	Consumption(m3/Year)	Loss(M3/Year)	Loss (%)
2017	194794	155971.6	38822.4	19.93%
2018	176840	140446.33	36393.67	20.58%
2019	157680	126396.26	31283.71	19.84%

Because of pump problems the water production decreases from time time.

B. Water loss expressed as per number of connection

The total number of connection in the study area is 2539,the water loss per connection computed by using this expression

From three years computed total annual water loss the last year value 31283.71 m3/Year used .

Water loss =31283.71*1000/(2539*365)= 33.75l/connection/day

C. Water loss expressed as per length of pipes

Expressing water loss as per kilo meter of main pipe is one way to indicate the loss. The total length of pipes of size 63mm to 90mm is 8km and these total pipe length used to express the water loss.

Table 12. Summary of pipe length by age category

S/N	Age Category	Pipe Length(m)	% from total	Remark
1	10 to 20 years	8140	100	Distribution

Water loss = Annual loss/(Length in Km *365)

=31283.71/(11.14km *365) = 7.69m3/km/day

4.2.2. Water demand projection

Table 13. Growth rate for Urban population

Year	Urban Growth rate %
1995-2000	4.3
2000-2005	4.1
2005-2010	4.06
2010-2015	3.88
2015-2020	3.69

2020-2025	3.51
2025-2030	3.35

Source. CSA's Country Level Population Growth Rates

Sample calculation for Town

$$\boldsymbol{P_t = p_o(1 + \frac{r}{100})^t}$$

Table 14. Forecasted Population

Years	Population
2024	30288
2029	35712
2034	42438

Table 15. Demand Projection

Description	Unit	Year		
		2024	2029	2034
Population	No.	30288	35712	42108
Domestic demand (urban)				
Proportion of Population Served by		1	1	1
Private house connection		0.95	0.96	0.97
private yard shared		0.03	0.03	0.02
Public Tap user		0.02	0.01	0.01
Population using different service level				
Private house connection	No.	28774	34283	40845

private yard shared	No.	909	1071	842
Public Tap Users		303	357	421
Per capita Water Demand by Categories				
Private house connection	l/c/day	60	65	70
Private yard shared	l/c/day	40	41	42
Water Demand by Categories				
Private house connection	m3/day	1726	2228	2859
private yard shared	m3/day	36	44	35
Public Tap User	m3/day	8	9	10
Non Domestic Water Demand				
Average day demand	m3/day	1770	2281	2904
Losses and Leakage (25-30%)	m3/day	531	684	871
Average day demand including loss(ADD)	m3/day	2301	2965	3775
Total Maximum Day demand(ADD *1.2)	m3/day	2761	3558	4530
Total Maximum Day demand	l/sec	31.95	41.18	52.43
Peak Hour Demand(1.8 *ADD)	m3/day	4142	5337	6435
Peak Hour Demand	l/sec	47.94	61.77	74.47
Storage Capacity (33% MDD)	M3/day	911	1174	1495

From the water demand analysis as shown in the table above to satisfy maximum day demand until end of design period needs a discharge of 41.18 l /s and to satisfy the peak hour demand needs 61.77 l/sec. This means, the pressure main pipe should have a capacity to deliver 41.187l/s and the pump should have a capacity to lift 61.77 l /sec. In addition the main distribution pipe should have a capacity to deliver 61.77 l/sec.

4.3. Hydraulic Model Analysis

4.3.1. Model skeletonization

In order to skeletonize the model to an acceptable degree, Elsheikh et al,(2014) reviewed the study adopted by USEPA(2005) and conditions used as follows:

- ❖ At least 50% of total pipe length in the distribution system (as shown in Table 16 below) for current study reaching 53%.
- ❖ At least 75% of the pipe volume in the distribution system (as shown in Table 16 below) for current study reaching 88.64%).
- ❖ All 63,80 and 90 mm diameter and larger pipes, storage facilities, major demand areas, pumps, and control valves are part of the study.

Table 16. Length, volume and Diameter of Surveyed Pipe material

S/N	Length of pipe(L=KM)	Diameter(mm)	Diameter(m)	V=Pi*D2/4*L
1	0.4	90	0.09	2.543
2	5	80	0.08	25.12
3	0.6	63	0.063	1.98
Sum	6			29.643

4.3.2 Pressure Analysis

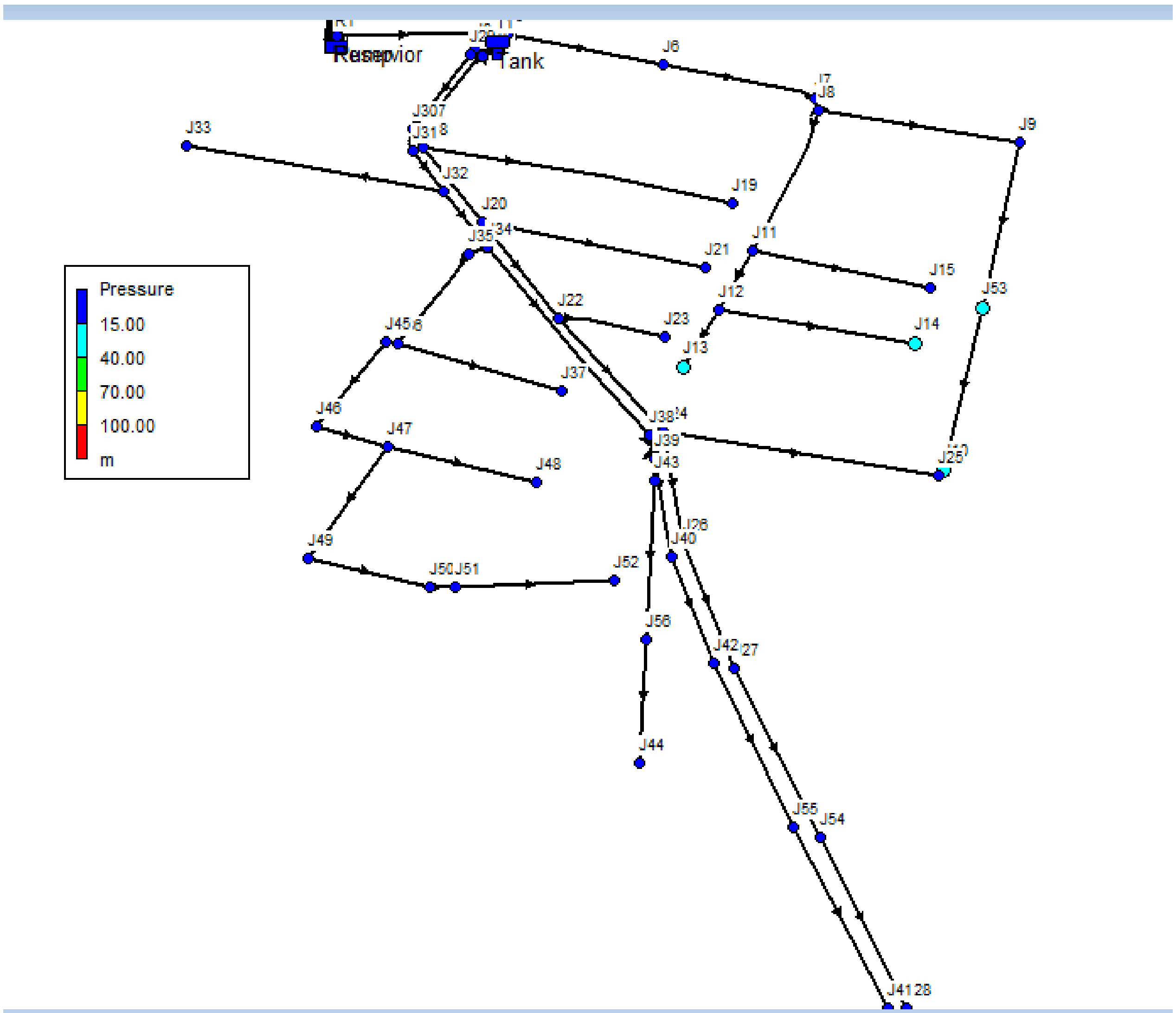

Figure 5. Model out of Pressure Analysis

The pressure of the town water distribution system is far below as it indicated on figure 5 above. The blue color indicates there is low pressure in the system. As the design criteria of MoWR (2006) , which is in between 15 and 70 but the study area is below the standard. Only Junction 10, 13, 14 and 53 can meet the standard. This is because of poor design that tried to manage the system by gravity. However, the study area is very flat which is difficult to manage the system by gravity system. So it is good to install some small pump within the distribution system.

4.3.3. Analysis and discussion of the model out put

According to the imported data to model the pipe system network established and hydraulic analysis made in all junction. As the network system built as per different demand pattern, the level of pressure and velocity compered between four Scenarios.

❖ **At Maximum Day Demand**

Maximum Pressure =17.48m, at J-10

Minimum Pressure = -2494742m, at J-4

Maximum Velocity =0.38m/s at p-1 Pipe diameter 90 mm

Minimum Velocity = 0.00m/s, at P-47 Pipe diameter 63

4.3.4 Velocity Analysis

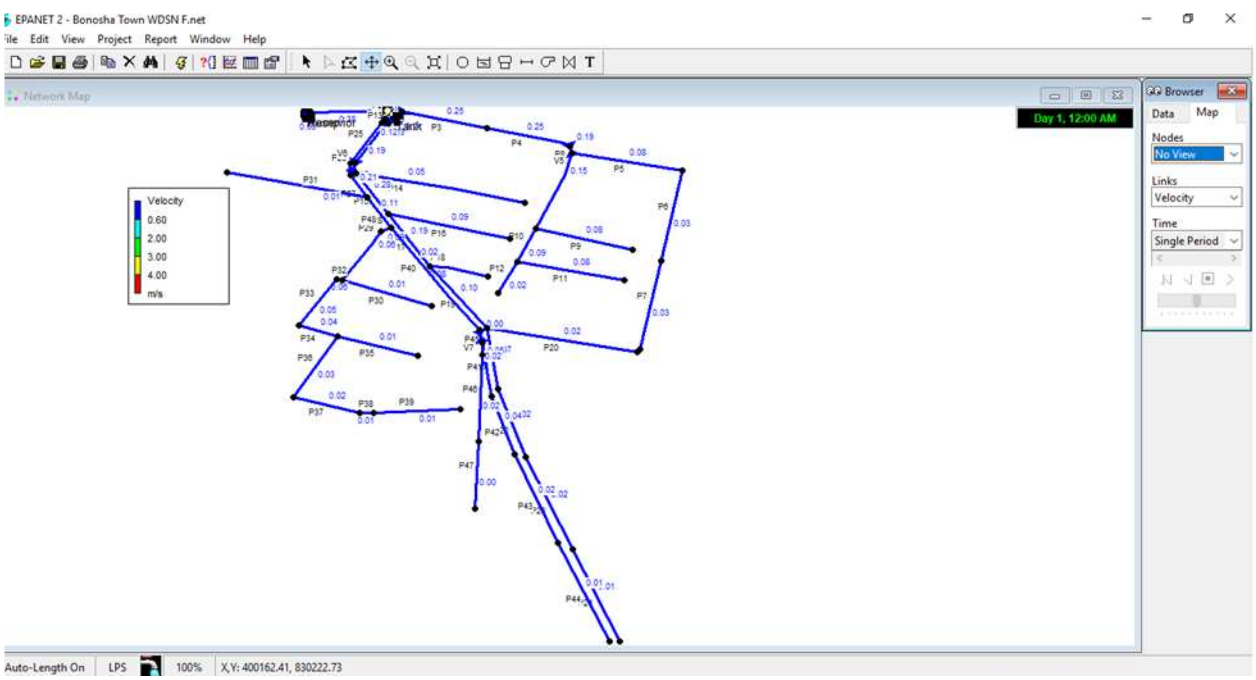

Figure 6. Model out put of Velocity

Velocity of Bonosha town water distribution system is very low as it indicated on above figure 6. 0.6 up to 2 m/s is normal standard in Ethiopia as Design criteria of MoWR(2006). But the result shows us there is very low velocity in pipe which is inefficient for users. In figure 6 above the blue color indicates the area of low velocity. This the result of inefficient water in source and pump problem. The availability of water in system is very low and needs development of additional water source for the town.

Population Vs MDDF and PHDF

Table 17 Population Vs MDDF and PHDF

Population	MDF	PHF
0 – 20,000	1.3	2
20,0010 - 50,000	1.25	1.9
50,001 and above	1.2	1.8

Source: Design Manual of MWR(2006 ❖ Adjustment due to socio-economic factors

Socio-economic factors determine the degree of development towns.

Group A towns enjoying high living standards and with very high potential for development factor of 1.10

Group B towns having a very high potential for development but now lower living standards factor of 1.05

Group C towns under normal Ethiopian conditions a factor of 1.00. MoWR(2006)

However, low demand is occur at night time and to fix low demand factor needs night flow data of the system. But, Bonosha town water supply system has no night flow data. Their for, for this research by considering the level of the town, population size and climate condition Arbaminch town 24 hours demand patter data has been used. Hence, the low demand factor is 0.25.

Their for, by using those factors and computed base demand for each junction peak hour ,low hour and maximum day demands calculated for all junctions and by using the model the level of Pressures analyzed in four scenario (ADD,MDD,PHD,LHD).

❖ **Here Sample Calculation made at J-3**

J-3 located near tank, which supplies water for 170 users from the main line ..

Base Demand = Population * Per Capita Demand per Day (20 l/day)

= 170 * 20 l/day=3400 l/day=0.04l/sec

MDD=0.04 l/s *1.25 =0.05 l/s

PHD =0.05 l/s* 2 = 0.1 l/s

LHD= 0.05 l/s *0.25= 0.0125 l/s

- J-10 Sample calculation

J-10 is located east of tank, which supplies water for 1086 users

1086 * 20l/day =21720 =0.25l/s

PHD =0.25 l/s* 2 = 0.5 l/s

LHD= 0.25 l/s *0.25= 0.0625 l/s

4.3.5. Model Calibration and Validation

Calibration is an iterative procedure of parameter evaluation and adjustment by comparing simulated and observed values

The EPANET Model was calibrated by adjusting sensitive parameter such as Hazan Williams Coefficient. As the model gives automatically C value of DCI Pipe 120 and for uPVC pipe 150 since, the system is old and should have roughness coefficient of pipe less than the model value. There for, this standard value of Hazen-Williams (C-value) used to adjust the model Value until closed to the measured value.

Table 18. C value for pipe materials

S/N	Type of Material	C value for New Pipe
1	uPVC	130
2	GI	120
3	PVC	150

Source. Design Criteria Manual of MoWR (2006)

Twenty observed pressure data were collected at main and distributing networks as shown in the table below for calibration and validation.

Model validation is in reality an extension of the calibration process. It is used to assure that the calibrated model property assesses all the variables and conditions, which can affect model results, and demonstrate the ability to predict field observation different data set. The hydraulic model calibration parameters that are typically set and adjusted pipe roughness factors. The change in this parameter affects head losses and pressure. The result shows that when the Hazan-Williams roughness coefficient increases the value of the pressure increases and head losses decreases.

4.3.5.1. Model Performance Evaluation criteria

There are many ways to judge on the performance of model calibration. The evaluation was made by calculating the squared relative difference between observed and simulated pressure for each test. The evaluation criteria used was statically method Using correlation coefficient (R^2).

$$R2 = \frac{\text{Sum(X-X mean)(Y-Y mean)}}{\text{(SQUR(Sum(X-X mean)}^2\text{) x Sum(Y-Y mean)}^2}$$

Where R^2 is Correlation Coefficient, X and Y are measured and simulated values, Xmean and Ymean are average value of measured and simulated data respectively.

4.3.5.2. Pressure Calibration and validation

ATSDR(200) illustrated that an average pressure difference of ± 15.2 kPa (± 1.51 m) with a maximum difference of ± 50.3 kPa (± 5.03 m) represents a "Good" data set and an average pressure difference of ± 29.6 kPa (± 2.96 m) with a maximum difference of ± 97.9 kPa (± 9.79 m) represents a "Poor" data set.

Table 19. Data Arrangement for pressure Calibration and Time series with pressure networks

S/N	Sample Location pints	Measured Pressure(m)	Computed Pressure	Difference	Measured Time	Sample Location		
						X(m)	y(m)	Elevation
1	J-5	1.4	1.5	-0.1	5:00	399882.87	830385.75	1971
2	J-6	5.47	5.49	-0.02	5:30	400057.89	830368.96	1963
3	J-7	8.46	8.48	-0.02	5:55	400039.13	830319.75	1966
4	J-8	1	0.00	1	6:00	399872.73	830355.86	1958
5	J-9	5.1	5	0.1	6:15	399971.15	830318.69	1954
6	J-10	17.60	17.48	-0.01	6:30	399879.81	830381.71	1954
7	J-11	11.47	11.48	-0.01	5:15	399964.78	830266.64	1960
8	J-12	13.49	13.48	0.1	5:30	399937.16	830275.14	1958
9	J-13	15.49	15.48	0.1	5:45	399865.65	830256.38	1957
10	J-14	15.46	15.48	-0.02	4:45	399914.86	830208.23	1957

The calibration of pressures were done in statically method and Figure 7 shows that the statistical correlation plot of observed versus computed pressure during calibration process .The result shows that R2=99.98%.this implies that the computed pressure are within the acceptable limit.

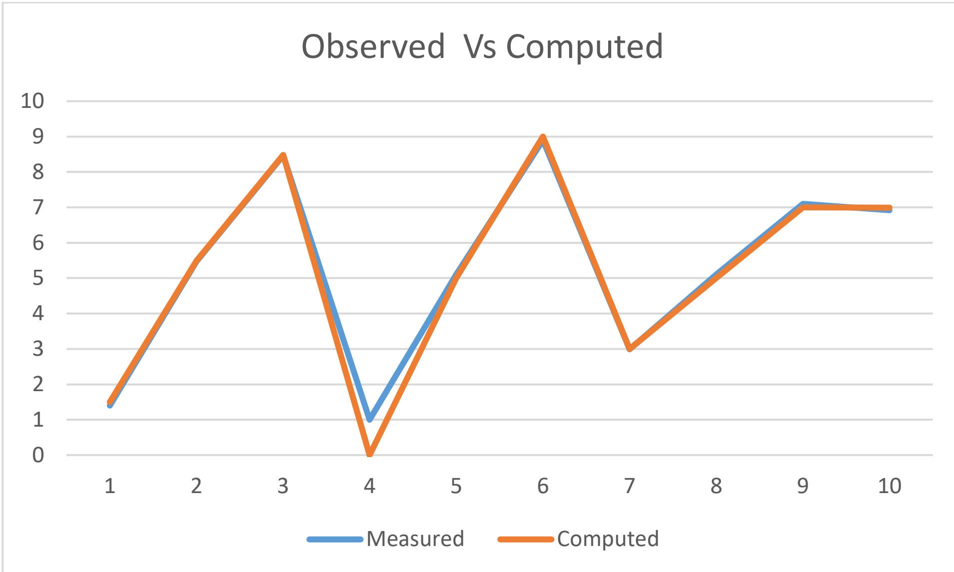

Figure 7. Correlated Plot of Observed Vs Computed Pressure during Calibration

The Validation of pressures were done in statically method and Figure 7 shows that the statistical correlation plot of observed versus computed pressure during Validation process .the result shows that R^2=99.98%.this implies that the computed pressure are within the acceptable limit.

4.4.Water Supply Scheme Management

4.4.1. Sector Organization

4.4.1.1. Key Sector Institutions

Ethiopia‟s decentralized local government structure consists of federal level ministries, nine regional administrations, 2 administrative cities (Addis Ababa and Dire Dawa), and about 550 weredas (district) level administrations. Each wereda consists of a number of Kebeles (villages), which is the lowest level of government at the community level.

Over the past 50 years, the institutional structure of the water sector had undergone significant changes. The Water Resources Department was established in 1956 under the Ministry of Public Works, the Water Resources Commission in 1982 followed by Ministry of Natural Resources in

1992. Ministry of Water Resources (MOWR) was established in 1995, the Water Works Construction Enterprise, the Water Well Drilling Enterprise and Water Works Design and Supervision Enterprise were also created to undertake water resources development works.
In line with the Government"s on-going decentralization strategy, the responsibility for both urban and rural water supply was transferred to the regional governments in 1995 while the Addis Ababa Water Supply and Sewerage Authority (AAWSSA) remained an autonomous public Authority. Presently, the main institutional players in the water sector are described below.

4.4.1.2. Role of the Sector Institutions

The Ministry of Water Resources (MOWR): The MOWR at federal government level is mainly entrusted with the policy formulation, guidance and review, implementation, operation and regulatory work. It also has the responsibility of building the capacity of Regional Governments regarding water resource development, and preparation of plans for the proper utilization of water resources. The ministry also coordinates donor funded projects among the financiers and the implementing regions.

The Regional Water, Mine and Energy Bureaus(RWMEB): Each of the nine regions and the Dire Dawa city administration has a Regional Water Bureau and Addis Ababa Water and Sewerage Authority. The Regional Water Bureaus are in charge of all water resources development activities within their regions. They are responsible for the strategic planning, organizing, coordinating, supervising the overall development and management of water resources projects. RWMEBs are now responsible for approving the zonal water, mine and energy department programmes as well as consolidating M&E reports of the Zones for transmittal to MOWR.

The Zonal Water, Mine and Energy Department (ZWMED): are in charge of all water resources development activities within their zone. They are responsible for the strategic planning, organizing, coordinating, supervising the overall development and management of water resources projects. ZWMEDs are now responsible for approving the woreda water, mine and energy office programmes as well as consolidating M&E reports of the weredas for transmittal to RWMEB.

The Wereda Water Office (WWO): are in charge of water supply development activities for rural areas within their weredas including financial and procurement management and monitoring and evaluation and for contracting with consultants and local service providers. Moreover the office will have a role in initiating, facilitating and providing motivation for community management of rural water services, the application of cost recovery principles, and in monitoring and evaluation. The office will be assisted by the regions, zone and wereda support groups (to build capacity in preparing wereda level Water Supply and Sanitation Programmes.

Town Water Boards (TWBs): Town Water Boards will be responsible for planning and managing their town water supply systems. The Water Boards will enter into performance agreements or contracts with utility operators to handle routine operations and maintenance and improve efficiency and expand their system over time. Board members are in most cases representatives of government organizations from municipality, water bureau, health, education and representatives of the user community.

The Town Water Supply Enterprises: The town water supply enterprises are concerned mainly with the scheme expansion and rehabilitation, operation and maintenance, financial and human resources management of the water supply scheme in the urban areas. The enterprises, depending on the size of the town, are autonomous in terms of town water supply development, operation and maintenance. The enterprises are managed by boards and the executive body of the enterprise leads the activities of the utility and its management.

4.4.2. Management and Personnel of Bonosha Town Water Supply Enterprise

The town water supply is managed by a manager accountable to the Board of Directors chaired by the Mayor of the town. The board meets once in semiannual . The purpose of the meeting is to evaluate the level of service and assurance of remedial solutions taken to alleviate the current shortage of water in the town.

The General Manager(GM) is the responsible person of the town water supply and sewerage enterprise and under GM there are three units such as administration unit, finance unit, and technical unit.

The service has currently 21 permanent and 9 contract employed staffs. The head of technical staff, finance staff and administration are accountable to the general manager. The scheme have only eleven personnel for operation and maintenance and unable to address all area to solve the current problems. Moreover, Bonosha water supply enterprise has no enough committed technical as well as administration staff. In addition to skill gap of the export the enterprise is not well equipped. When pump stack or bursting happened on main line technical invited from zone or region, through this the community suffers for days and weeks by searching water from far away.

4.4.3. Customer Service and Fee Collection

The Bonosha town water supply system have a total 2539 Water Connection, Due to urban expansion and development a lot of privet as well as commercial customers looking for connection. However, due to water production shortage, technical and management problems water distribution can not made for all user by the same time.

Regarding water fee collection all private, commercial and government sector connection, all the users pay in monthly bases.

4.4.4 Financial Management

Bonosha town water supply and sewerage enterprise have analysis of three years income and expenditure of the water supply system from 2017 to 2019. Income of enterprise is from water sales, technical service, from sales of plumbing materials and other different sources and expenditures are staff salary and cost of operation and maintenance. Hence, the result shows that the difference between yearly income and expenditure in 2017 and 2019 became negative.

However, following the water policy and its strategy the regional governments have also issued their respective proclamations in which they have made urban water supply services to operate as autonomous entities on full cost recovery base and its management by independent water board where urban populations are over 15,000. Therefore, the town water supply enterprise has responsibility to recover 1457546.10birr/year but for last three years the scheme unable to cover the cost and works only to cover the salary of the staff.

Table 20. Income and expenditure of Bonosha town water supply enterprise

S/N	Description	Unit	Year		
			2017	2018	2019
1	Income	Birr	657976.90	689541.20	700532.65
2	expenditure	Birr	669643.80	694329.8	710125.44
3	Difference	Birr	- 11666.90	-4788.6	-9592.79
4	Annual expected cost recovery amount	Birr	1469213	1623178.80	1976534.90
5	Saving after cost recovery	Birr	- 1457546.1	- 1618390.20	-1966942.11

Source. Bonosha town water supply and sewerage enterprise report document

Therefore, the management of Bonosha town water supply totally need improvement to maximize the overall performance of the utility. Locally there are a number of Water Supply Services that operate efficiently with the principle of cost recovery and full autonomy. Visiting such institutions would be helpful to improve the performance of the board, management and staffs. Timely upgrading and acquaintance of staffs with innovative ideas, new technologies with clear purpose and change is a tool for improvement.

5. Conclusion and Recommendation

5.1. Conclusion

- As the result of population growth, technical and management problem the water supply problem remained series problem. The water supply of adequate quantity and acceptable quality is one of the basic needs of human beings, but the provision of potable water for Bonosha town is inefficient.

❖ The existing sources of potable water are small spring found 1 km away from the town with good potential. However, due to availability of high problem in Production, specially pump problem as well as distribution pipe system and poor operation and maintenance unable to use this potential.

❖ Moreover, the state of water supply in the town in terms of coverage both in spatial and Population, reliability, accessibility, and sustainability is not at the required standard. The rate of meter connection is low and the distribution system is inefficient. The major Constraints of distribution systems identified are low density of pipeline networks, and their unfair distribution, inadequate pressure in the pipe, low pumping capacity and no well-established distribution network.

❖ The research has confirmed that there is high gap between demand and supply. This gap is due to not only low water production, but also town expansion. Moreover, a lot of unplanned connection along the gravity main line, poor water management and under estimation of the demand during the construction of the water system has contributed enormously to the shortage of potable water in the town.. In addition, low pressure along the gravity main pipe line which is result poor design.

❖ From the water supply analysis as per level of average per capital consumption of the town found to be 19.2 l/day. This average per capital consumption is lower while compared with the minimum requirement of per capita demand which is 20l/day. .

❖ The other way followed to evaluate the town water coverage was the level of connection per family. The average level of connection in house or yard is about 54.8% which is relatively good coverage compared to per capita demand.

❖ The other issue that has been addressed was analysis of water coverage as per domestic and non-domestic demand. The result shows as the total average demand is 1419.21 m3/day. However, the amount of water production in 2020 shows as 157680 m3/year or 432m3/day, which have only 30.5% coverage of the demand. Therefore, the result shows there is high gap between demand and supply.

- On the other hand the analysis shows as the maximum day demand is 1774.01m3/day (20.53l/s) and peak hour demand is 2488.62 m3/day (31.21 l/sec).Therefore, to satisfy the demand the pressure main pipe line and the capacity of the pump should have ability to meet the maximum day demand 20.53 l /sec. However, the existing pump has a capacity to lift 12 l/s, which is 58.45% of the maximum day demand. This show as the need of more pumps.

5.2. Recommendation

- Currently existing pump has a defect and is not giving intended service. It was maintained previously by electromechanical team from SNNPR Water bureau and made slight modifications on internal motor parts. As water supply has been cutoff due to pump problem the situation is badly affecting the community, peoples are forced to fetch unprotected water from the Bilate river which is found near to the Bonosha town. However, as observed and made assessment after maintenance, the problem was not solved and it continued with very low performance and cannot pump the required amount of water to reservoir. To serve the increasing water demand of the Bonosha town population, the immediate solution is to replace with new pump which can lift more than 20 l/s.
- During assessment, the majority of population could not get water as they are far from major junction of town. The town is in need of pipe extension. As the result of this the residents forced to buy water from different water point. To solve the problem the municipality has to extend the pipes to rest area.
- The water system has in sufficient pressure and velocity which can not serve the resident in town. So the remedial measure is to redesign the system.
- The town has only one source of water, which is 1 km away from town. This water has been serving the town and unable to serve the current demand of the town. Thus the Bonosha municipality has to look for the alternative source as the town is expanding in alarming rate.
- There is no well-trained and inspired staff who can serve the residents. Hence the institution has to give consecutive capacity building training for its staff. Very simple technical problem stay weeks and the residents suffer as the result of it. Working on staff commitment is also vital.
- The town has only one tank which can hold 100 m^3 water. This tank is unable to meet the daily demand of the town. In order to solve the problem the municipality has build one extra tank.

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Appendix 1. Surveyed Pipe Line Data for Junction

Label	X (m)	Y (m)	Elevation (m)
J-1	0399127	831211	1960
J-2	0399475	831052	1971
J-3	0392424	830780	1970
J-4	0399531	830615	1970
J-5	0399577	830533	1970
J-6	0399683	0830344	1966
J-7	0399820	0830118	1963
J-8	0399956	0829928	1963
J-9	0399993	0829905	1958
J-10	0400000	0829656	1954

J-11	0400028	0829659	1960
J-12	0400040	0829520	1958
J-13	0400147	0829218	1956
J-14	0399528	0831109	1956
J-15	0399771	0831188	1957
J-16	0399885	0831114	1969
J-17	0399914	0831151	1966
J-18	0399950	0831129	1966
J-19	0400180	0801048	1961
J-20	0400224	0831042	1962
J-21	0400235	0831037	1959
J-22	0400251	0830936	1958
J-23	0400187	0830656	1957
J-24	0400085	0830357	1954
J-25	0400046	0830244	1954
J-26	0399996	0830075	1952
J-27	0399989	0829910	1947
J-28	0400039	0829983	1949
J-29	0400153	0829872	1969
J-30	0400253	0829847	1966
J-31	0400405	0829812	1965
J-32	0400400	0829799	1963
J-33	0400460	0829791	1958

J-34	0399509	0831188	1962
J-35	0399456	0831151	1962
J-36	0399439	08311049	1958
J-37	0399437	0831036	1955
J-38	0399402	0830911	1954
J-39	0399388	0830846	1954
J-40	0399404	0830782	1954
J-41	0399631	0830375	1949
J-42	0399692	0830281	1948
J-43	0399771	0830145	1954
J-44	0399814	0830093	1946
J-45	0399948	0829807	1957
J-46	0399836	0830095	1957
J-47	0399786	0830027	1956
J-48	0399702	0830051	1951
J-49	0399581	0830088	1954
J-50	0399450	0830134	1952
J-51	0399418	0830040	1952
J-52	0399369	0829923	1951
J-53	0399750	0829783	1956
J-54	400237	0829734	1949
J-55	400210	0829742	1948
J-56	400090	0829903	1951

Appendix 2. Pump Data

Label	X (m)	Y (m)	Discharge(l/s)	Head(M)
Pump 1	399722	830314.18	12	250

Appendix 3. Tank Data

Label	X (m)	Y (m)	Base Elevation (m)	Minimum Water Label	Maximam Water label
T-1	399889.30	830386.08	1970	1	3

Appendix 4. Pipe Data

Leble	Start node	End Node	Material	Diameter(mm)	Length(m)
p-1	source	PMP	Galvanized Iron	90	45.63
p-2	J-2	J-3	Galvanized Iron	90	3.89
p-3	J-5	J -6	Galvanized Iron	80	98.80
P-4	J-	J-8	Galvanized Iron	80	63.59
P-5	J-8	J-9	Galvanized Iron	80	105.88
P-6	J-9	J-10	Galvanized Iron	80	81.10

P-7	J-7	J-11	Galvanized Iron	80	48.70
P-8	J-11	J-13	Galvanized Iron	80	21.64
P-9	J-13	J-14	Galvanized Iron	63	59.98
P-10	J-13	J-15	Galvanized Iron	80	19.15
P-11	J-16	J-17	Galvanized Iron	80	26.83
P-12	J-17	J-18	Galvanized Iron	80	7.77
P-13	J-18	J-19	Galvanized Iron	80	101.23
P-14	J-18	J-20	Galvanized Iron	80	32.34
P-15	J-20	J-21	Galvanized Iron	80	72.76
P-16	J-20	J-22	Galvanized Iron	80	44.13
P-17	J-122	J-23	Galvanized Iron	63	31.35
P-18	J-22	J-24	Galvanized Iron	80	50
P-19	J-J-24	J-25	Galvanized Iron	80	36.63

P-20	J-27	J-28	Galvanized Iron	80	27.35
P-21	J-28	J-29	Galvanized Iron	80	9.50
P-22	J-29	J-47	Galvanized Iron	80	15.79
P-23	J-47	J-30	Galvanized Iron	80	26.04
P-24	J-30	J-31	Galvanized Iron	80	81.19
P-25	J-32	J-33	Galvanized Iron	80	10.08
P-26	J-33	J-34	Galvanized Iron	80	23.15
P-27	J-34	J-35	Galvanized Iron	80	160.87
P-28	J-33	J-36	Galvanized Iron	80	86.73
P-29	J-47	J-48	Galvanized Iron	80	85.97
P-30	J-30	J-37	Galvanized Iron	80	5.23
P-31	J-37	J-38	Galvanized Iron	80	34.76
P-32	J-38	J-40	Galvanized Iron	80	3.91

P-33	J-40	J-41	Galvanized Iron	80	31.83
P-34	J-41	J-42	Galvanized Iron	80	15
P-35	J-42	J-43	Galvanized Iron	80	46.07
P-36	J-42	J-44	Galvanized Iron	63	40.96
P-37	J-44	J-45	Galvanized Iron	80	40.05
P-38	J-45	J-46	Galvanized Iron	80	53.74
P-39	J-111	J-12	Galvanized Iron	63	54.63
P-40	J-38	J-39	Galvanized Iron	63	54.66
P-41	J-25	J-26	Galvanized Iron	80	162.23
P-42	J-49	J-1	Galvanized Iron	90	4.82

Appendix 5. Base Demand Computation

Label	Population n	Demand d
J-1	543	0.000

J-2	212	0.049
J-3	170	0.039
J-4	0	0
J-5	129	0.030
J-6	140	0.032
J-7	143	0.033
J-8	245	0.057
J-9	1086	0.251
J-10	543	0.126
J-11	250	0.058
J-12	423	0.098
J-13	490	0.113
J-14	1024	0.237
J-15	1011	0.234
J-16	1000	0.231
J-17	1011	0.234
J-18	230	0.053
J-19	1170	0.271
J-20	670	0.155
J-21	1200	0.278
J-22	940	0.218
J-23	1074	0.249
J-24	132	0.031
J-25	450	0.104
J-26	1054	0.244
J-27	95	0.022

J-28	173	0.040
J-29	79	0.018
J-30	1016	0.235
J-31	360	0.083
J-32	155	0.036
J-33	230	0.053
J-34	120	0.028
J-35	560	0.130
J-36	23	0.005
J-37	100	0.023
J-38	72	0.017
J-39	25	0.006
J-40	114	0.026
J-41	140	0.032
J-42	250	0.058
J-43	540	0.125
J-44	212	0.049
J-45	34	0.008
J-46	211	0.049
J-47	120	0.028
J-48	200	0.046
J-49	160	0.037
J-50	360	0.083
J-51	155	0.036
J-52	230	0.053
J-53	120	0.028

J-54	560	0.130
J-55	360	0.083
J-56	155	0.036

Appendix 6 . Demand at different Scenarios

Lebel	Base Demand(A)	Maximum Day Demand(A*1.2)	Peak Hour Demand(A*2)	Low Demand(A*0.25)
J-1	0.000	0.00	0.0	0.000
J-2	0.049	0.06	0.1	0.012
J-3	0.039	0.05	0.1	0.010
J-4	0.00	0.00	0.0	0.000
J-5	0.030	0.04	0.1	0.008
J-6	0.032	0.04	0.1	0.008
J-7	0.033	0.04	0.1	0.008
J-8	0.057	0.07	0.1	0.014
J-9	0.251	0.30	0.5	0.063
J-10	0.126	0.15	0.3	0.032
J-11	0.058	0.07	0.1	0.015
J-12	0.098	0.12	0.2	0.025
J-13	0.113	0.14	0.2	0.028

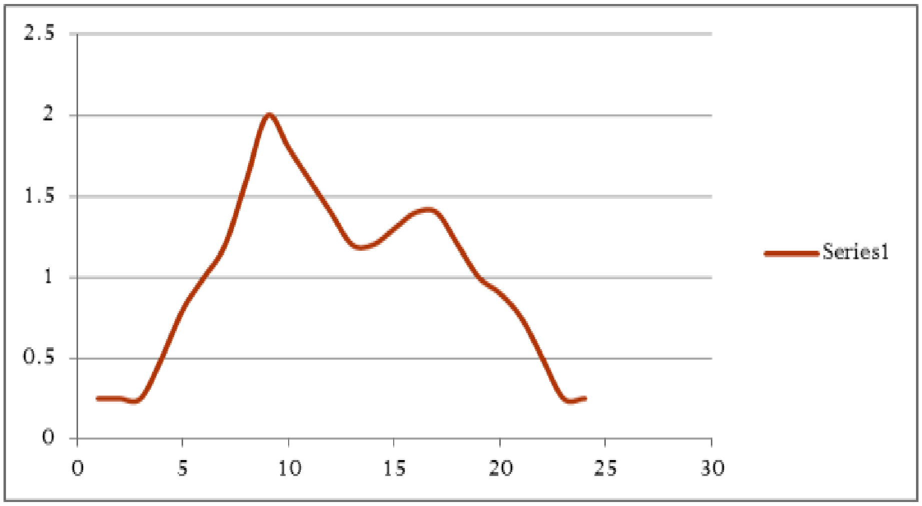
J-14	0.237	0.28	0.5	0.059
J-15	0.234	0.28	0.5	0.059
J-16	0.231	0.28	0.5	0.058
J-17	0.234	0.28	0.5	0.059
J-18	0.053	0.06	0.1	0.013
J-19	0.271	0.33	0.5	0.068
J-20	0.155	0.19	0.3	0.039
J-21	0.278	0.33	0.6	0.070
J-22	0.218	0.26	0.4	0.055
J-23	0.249	0.30	0.5	0.062
J-24	0.031	0.04	0.1	0.008
J-25	0.104	0.12	0.2	0.026
J-26	0.244	0.29	0.5	0.061
J-27	0.022	0.03	0.0	0.006
J-28	0.040	0.05	0.1	0.010
J-29	0.018	0.02	0.0	0.005
J-30	0.235	0.28	0.5	0.059
31	0.083	0.10	0.2	0.021
J-32	0.036	0.04	0.1	0.009
J-33	0.053	0.06	0.1	0.013
J-34	0.028	0.03	0.1	0.007
J-35	0.130	0.16	0.3	0.033

J-36	0.005	0.01	0.0	0.001
J-37	0.023	0.03	0.0	0.006
J-38	0.017	0.02	0.0	0.004
J-39	0.006	0.01	0.0	0.002
J-40	0.026	0.03	0.1	0.007
J-41	0.032	0.04	0.1	0.008
J-42	0.058	0.07	0.1	0.015
J-43	0.125	0.15	0.3	0.031
J-44	0.049	0.06	0.1	0.012
J-45	0.008	0.01	0.0	0.002
J-46	0.049	0.06	0.1	0.012
J-47	0.028	0.03	0.1	0.007
J-48	0.046	0.06	0.1	0.012
J-49	0.037	0.04	0.1	0.009
J-50	0.125	0.15	0.3	0.031
J-51	0.049	0.06	0.1	0.012
J-52	0.008	0.01	0.0	0.002
J-53	0.049	0.06	0.1	0.012
J-54	0.028	0.03	0.1	0.007
J-55	0.046	0.06	0.1	0.012

J-56	0.037	0.04	0.1	0.009

Appendix 7. Hourly demand factor

Time	Factor	Time	Factor	Time	Factor
00-01	0.25	08-09	2	16-17	1.4
01-02	0.25	09-10	1.8	17-18	1.2
02-03	0.25	10-11	1.6	18-19	1
03-04	0.5	11-12	1.4	19-20	0.9
04-05	0.8	12-13	1.2	20-21	0.75
05-06	1	13-14	1.2	21-22	0.5
06-07	1.2	14-15	1.3	22-23	0.25
07-08	1,6	15-16	1.4	23-24	0.25



Appendix 8. Model Output of Pressure junction

EPANET 2 - Bonosha Town WDSN F.net - [Network Table - Nodes]

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Node ID	Elevation m	Base Demand LPS	Demand LPS	Head m	Pressure m	Quality
Junc J1	1960	0	0.00	499.34	-1460.66	0.0
Junc J2	1971	0.049	0.05	499.32	-1471.68	0.0
Junc J3	1970	0.039	0.04	499.30	-1470.70	0.0
Junc J4	1970	0	0.00	-2491772.00	-2493742.00	0.0
Junc J5	1970	0.030	0.03	1971.50	1.50	0.0
Junc J6	1966	0.032	0.03	1971.49	5.49	0.0
Junc J7	1963	0.033	0.03	1971.48	8.48	0.0
Junc J8	1963	0.057	0.06	1971.48	8.48	0.0
Junc J9	1958	0.251	0.25	1971.48	13.48	0.0
Junc J10	1954	0.126	0.13	1971.48	17.48	0.0
Junc J11	1960	0.058	0.06	1971.48	11.48	0.0
Junc J12	1958	0.098	0.10	1971.48	13.48	0.0
Junc J13	1956	0.113	0.11	1971.48	15.48	0.0
Junc J14	1956	0.237	0.24	1971.48	15.48	0.0
Junc J15	1957	0.234	0.23	1971.48	14.48	0.0
Junc J16	1969	0.231	0.23	-2491772.00	-2493741.00	0.0
Junc J17	1966	0.234	0.23	-2491772.00	-2493738.00	0.0
Junc J18	1970	0.053	0.05	-2491772.00	-2493742.00	0.0
Junc J19	1961	0.271	0.27	-2491772.00	-2493733.00	0.0
Junc J20	1962	0.155	0.16	-2491772.00	-2493734.00	0.0
Junc J21	1959	0.278	0.28	-2491772.00	-2493731.00	0.0
Junc J22	1958	0.218	0.22	-2491772.00	-2493730.00	0.0
Junc J23	1957	0.249	0.25	-2491772.00	-2493729.00	0.0
Junc J24	1954	0.031	0.03	-2491772.00	-2493726.00	0.0

Node ID	m	LPS	LPS	m	m	
Junc J25	1954	0.104	0.10	-2491772.00	-2493726.00	0.00
Junc J26	1952	0.244	0.24	-2491772.00	-2493724.00	0.00
Junc J27	1947	0.022	0.02	-2491772.00	-2493719.00	0.00
Junc J28	1949	0.040	0.04	-2491772.00	-2493721.00	0.00
Junc J29	1969	0.018	0.02	-2491772.00	-2493741.00	0.00
Junc J30	1966	0.235	0.23	-2491772.00	-2493738.00	0.00
Junc J31	1965	0.083	0.08	-2491772.00	-2493737.00	0.00
Junc J32	1963	0.036	0.04	-2491772.00	-2493735.00	0.00
Junc J33	1958	0.053	0.05	-2491772.00	-2493730.00	0.00
Junc J34	1962	0.028	0.03	-2491772.00	-2493734.00	0.00
Junc J35	1962	0.130	0.13	-2491772.00	-2493734.00	0.00
Junc J36	1958	0.005	0.00	-2491772.00	-2493730.00	0.00
Junc J37	1955	0.023	0.02	-2491772.00	-2493727.00	0.00
Junc J38	1954	0.017	0.02	-2491772.00	-2493726.00	0.00
Junc J39	1954	0.006	0.01	-2870659.00	-2872613.00	0.00
Junc J40	1952	0.032	0.03	-2870659.00	-2872611.00	0.00
Junc J41	1949	0.058	0.06	-2870659.00	-2872608.00	0.00
Junc J42	1948	0.125	0.12	-2870659.00	-2872607.00	0.00
Junc J43	1954	0.049	0.05	-2870659.00	-2872613.00	0.00
Junc J44	1946	0.008	0.01	-2870659.00	-2872605.00	0.00
Junc J45	1957	0.049	0.05	-2491772.00	-2493729.00	0.00
Junc J46	1957	0.028	0.03	-2491772.00	-2493729.00	0.00
Junc J47	1958	0.046	0.05	-2491772.00	-2493730.00	0.00
Junc J48	1951	0.037	0.04	-2491772.00	-2493723.00	0.00
Junc J49	1954	0.008	0.01	-2491772.00	-2493726.00	0.00
Junc J50	1952	0.049	0.05	-2491772.00	-2493724.00	0.00
Junc J51	1952	0.028	0.03	-2491772.00	-2493724.00	0.00
Junc J52	1951	0.046	0.05	-2491772.00	-2493723.00	0.00
Junc J53	1956	0.037	0.04	1971.48	15.48	0.00
Junc J54	1949	0.049	0.05	-2491772.00	-2493721.00	0.00
Junc J55	1948	0.028	0.03	-2870659.00	-2872607.00	0.00
Junc J56	1951	0.046	0.05	-2870659.00	-2872610.00	0.00
Resvr R1	250	#N/A	-2.40	250.00	0.00	0.00
Tank T1	1970	#N/A	-1.31	1971.50	1.50	0.00

Auto-Length Off | LPS | 100% | X,Y: 400046.35, 830398.79

Appendix 9. Model Output of Velocity at Pipe

EPANET 2 - Bonosha Town WDSN F.net - [Network Table - Links]

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Link ID	Length m	Diameter mm	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor	Reaction Rate mg/L/d	Quality	Status
Pipe P1	6	90	2.40	0.38	2.63	0.033	0.00	0.00	Open
Pipe P2	6	90	2.35	0.37	2.54	0.033	0.00	0.00	Open
Pipe P3	6	80	1.28	0.25	1.44	0.035	0.00	0.00	Open
Pipe P4	6	80	1.24	0.25	1.39	0.036	0.00	0.00	Open
Pipe P5	6	80	0.41	0.08	0.17	0.040	0.00	0.00	Open
Pipe P6	6	80	0.16	0.03	0.02	0.037	0.00	0.00	Open
Pipe P7	6	80	0.13	0.03	0.02	0.062	0.00	0.00	Open
Pipe P8	6	80	0.74	0.15	0.52	0.038	0.00	0.00	Open
Pipe P9	6	63	0.23	0.08	0.20	0.044	0.00	0.00	Open
Pipe P10	6	80	0.45	0.09	0.22	0.044	0.00	0.00	Open
Pipe P11	6	63	0.24	0.08	0.20	0.042	0.00	0.00	Open
Pipe P12	6	80	0.11	0.02	0.00	0.000	0.00	0.00	Open
Pipe P13	6	80	1.55	0.31	0.00	0.000	0.00	0.00	Open
Pipe P14	6	80	0.27	0.05	0.00	0.000	0.00	0.00	Open
Pipe P15	6	80	1.39	0.28	0.00	0.000	0.00	0.00	Open
Pipe P16	6	63	0.28	0.09	0.00	0.000	0.00	0.00	Open
Pipe P17	6	80	0.96	0.19	0.00	0.000	0.00	0.00	Open
Pipe P18	6	63	0.25	0.08	0.00	0.000	0.00	0.00	Open
Pipe P19	6	80	0.49	0.10	0.00	0.000	0.00	0.00	Open
Pipe P20	6	80	0.10	0.02	0.00	0.000	0.00	0.00	Open
Pipe P21	6	80	0.35	0.07	0.00	0.000	0.00	0.00	Open
Pipe P22	6	80	0.11	0.02	0.00	0.000	0.00	0.00	Open
Pipe P23	6	80	0.09	0.02	0.00	0.000	0.00	0.00	Open
Pipe P24	6	80	0.04	0.01	0.00	0.000	0.00	0.00	Open

Pipe P25	6	80	1.25	0.25	0.00	0.000	0.00	0.00	Open
Pipe P26	6	80	1.02	0.20	0.00	0.000	0.00	0.00	Open
Pipe P27	6	80	0.93	0.19	0.00	0.000	0.00	0.00	Open
Pipe P28	6	80	0.45	0.09	0.00	0.000	0.00	0.00	Open
Pipe P29	6	80	0.32	0.06	0.00	0.000	0.00	0.00	Open
Pipe P30	6	63	0.02	0.01	0.00	0.000	0.00	0.00	Open
Pipe P31	6	80	0.05	0.01	0.00	0.000	0.00	0.00	Open
Pipe P32	6	80	0.29	0.06	0.00	0.000	0.00	0.00	Open
Pipe P33	6	80	0.24	0.05	0.00	0.000	0.00	0.00	Open
Pipe P34	6	80	0.21	0.04	0.00	0.000	0.00	0.00	Open
Pipe P35	6	63	0.04	0.01	0.00	0.000	0.00	0.00	Open
Pipe P36	6	80	0.13	0.03	0.00	0.000	0.00	0.00	Open
Pipe P37	6	80	0.12	0.02	0.00	0.000	0.00	0.00	Open
Pipe P38	6	80	0.07	0.01	0.00	0.000	0.00	0.00	Open
Pipe P39	6	63	0.05	0.01	0.00	0.000	0.00	0.00	Open
Pipe P40	6	80	0.37	0.07	0.00	0.000	0.00	0.00	Open
Pipe P41	6	80	0.24	0.05	0.00	0.000	0.00	0.00	Open
Pipe P42	6	80	0.21	0.04	0.00	0.000	0.00	0.00	Open
Pipe P43	6	80	0.09	0.02	0.00	0.000	0.00	0.00	Open
Pipe P44	6	80	0.06	0.01	0.00	0.000	0.00	0.00	Open
Pipe P45	6	80	0.10	0.02	0.00	0.000	0.00	0.00	Open
Pipe P46	6	63	0.05	0.02	0.00	0.000	0.00	0.00	Open
Pipe P47	6	63	0.01	0.00	0.00	0.000	0.00	0.00	Open
Pipe P48	6	80	0.85	0.17	0.00	0.000	0.00	0.00	Open

Auto-Length Off | LPS | 100% | X,Y: 399555.56, 830422.90