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EVALUATING THE HYDRAULIC PERFORMANCE OF EXISTING WATER SUPPLY DISTRIBUTION SYSTEM /THE CASE OF TEBELA TOWN OF WOLAITA ZONE, SNNPR, ETHIOPIA

BY

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ABSTRACT

The adequate and reliable water supply in the developing towns of Ethiopia is becoming a challenge for most water utilities *especially public service providers like Tebela town.* The main objective of this research was to check the hydraulic performance of Tebela town's water distribution system by evaluating water demand and production, losses of water, and hydraulic parameters. Both secondary and primary data sources were used for this study. Primary data was collected through field surveys and photographs of relevant sites and infrastructures. The secondary data was collected from design documents, literature, journals, and office. Moreover, to analyse the existing water distribution system, a model was developed by using WaterGEMSv8i software. The model simulation run was performed for peak and low demand scenarios to analyze the distribution system. The analysis shows that the water supply coverage was 25.9%.The water loss of the town was 17.2% from the total water production. *Modeling results showed violation of maximum and minimum pressure and velocity criteria at different junctions and pipes. After modifying the existing water distribution system, 96.23% of the junctions are in the recommended pressure range and 73.50% of the pipes are in the recommended velocity range at minimum hourly consumption. Generally the result of the analysis shows that the overall hydraulic performance of water distribution of the town was moderate, which is reflected by medium water production rate, water consumption, and nonrevenue water, also low water coverage, some velocity and pressure is not in permissible range.* Therefore, it is important to rehabilitate and improve the water distribution system capacities, establish pressure zones, increase pumping rates, and drill additional boreholes to meet the current water demand and future demand.

Key word: *WaterGEMSv8i, Hydraulic performance, Simulation, Water distribution system, water losses.*

1. INTRODUCTION

1.1 Background and Justifications

Water is an essential and life-sustaining natural resource and is critical for the survival of all living organisms, food production and economic development. Problems with providing a satisfactory water supply to the rapidly growing population, especially in developing countries, are increasing from time to time. The sustainable provision of adequate and safe drinking water is the most important of all public services (Dassalew, 2017).

The adequate and reliable water supply in the developing towns of Ethiopia is becoming a challenge for most water utilities. Problems with providing a satisfactory water supply to the rapidly growing population, especially in developing countries, are increasing from time to time (Asmelash, 2014).

Water is the primary resource needed to sustain life. Every citizen in the country has the right to have access to potable water. Access to safe drinking water supplies and sanitation services in Ethiopia are among the lowest in Sub-Saharan Africa (Seifu, 2012). Access to clean and safe drinking water is a fundamental human need. However, in many areas of the world, natural water sources have been impacted by a variety of biological and chemical contaminants. The ingestion of these contaminants may cause acute or chronic health problems. To prevent such illnesses, many technologies have been developed to treat, disinfect and supply safe drinking water quality (Dawit, 2015).

This research evaluated the performance of Tebela's water supply system in terms of main performance indicators such as water supply coverage, water loss, hydraulic performance, and recommended solutions for improving the water supply service.

1.2 Statement of the Problem

Tebela town has water supply and demand related problems. Presently, Tebela faces a serious deficit in its water supply due to an increased population and expanded economic activity in and around the subsystems.

The performance of the urban water supply system is to improve the water supply service level, and the main activities are to find the gap or to fill it, between the demand and the existing water supply system, and to analyze whether the distribution system is working as per design or not. Consequently, this study investigated the water supply and distribution conditions of the town in terms of water supply coverage, water demand, water supply and distribution system, and the gap associated with water scarcity to start intervention measures in order to address the aforementioned problems in the study area. Finally, this research attempts to evaluate the present status of water supply distribution system performance and its outlook to provide base line information for decision makers and further research.

1.3 Objectives of the Study

1.3.1 Major Objective

The main objective of this research was to evaluate the hydraulic performance of the existing water distribution system in Tebela town.

1.3.2 The Specific Objective

The specific objective of this research includes:

- To evaluate the existing water supply situation of the town,
- To evaluate water losses of existing water distribution system,

3. MATERIALS AND METHODS

3.1 Description of the Study Area

3.1.1 Location and demography

The study area is Tebela Town, Humbo Woreda of Wolaita Zone, Southern Nations Nationalities and Peoples Regional State (SNNPRS) of Ethiopia. It is located 346km away from the country's capital, southwest of Addis Ababa, and 149km away from the northeast of the capital city of the region, Hawassa, which is also located 20km away from the south of Soddo town. It shares a boundary with the Eastern Damot Woyde Woreda, in the North with Sodo Zuria Woreda, in the South with Mirab Abaya Woreda, and in the West with Offa Woreda. The town lies between a latitude of 6^0 42' 40" N and a longitude of 37^0 46' 40" E. Tebela town altitude ranges between 1200 and 1900 m.a.s.l.

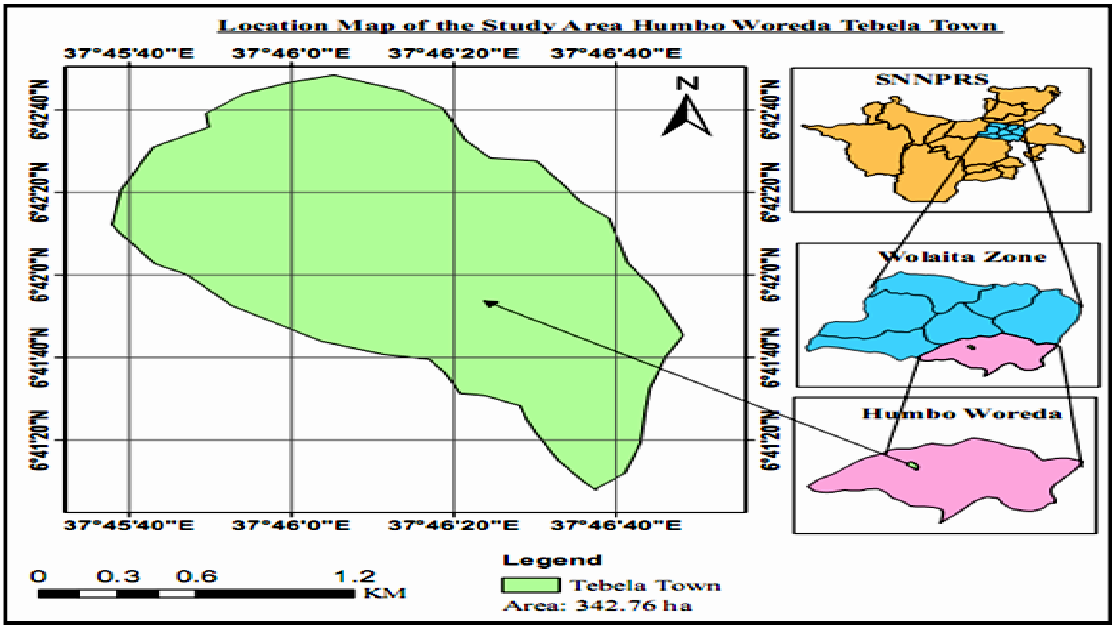

Figure 3.1 Location map of the study area (Source: WSU GIS Lab, 2020).

According to the results of the housing and population census, the current total population of Tebela town in 2020 is 43,314 of which 22,485 are males and 20,829 are females with an average family size of 5 people per household. The town administrative has two kfile-ketema, Alata and Abana. Alata kfile ketema contains (01, Ambe shoya, Abbela sipa, and Ela qebela) kebeles and Abana kfile ketema contains (02, Koysha ogodama and Shochore ogodama) kebeles.

Teble 3.1 Tebela town administration population in each kebele (2018-2020)

Year	2018			2019			2020		
Population									
Kebeles	M	F	Total	M	F	Total	M	F	Total
01	5060	4687	9747	5019	4648	9667	3965	3673	7638
02	4279	3963	8241	4787	4434	9221	3874	3592	7466
Ambe shoya	3215	3471	6686	3496	3238	6734	3534	3273	6807
Abbela sipa	909	981	1890	1585	1712	3297	2309	2138	4447
Koysha ogodama	1656	1788	3445	1819	1964	3783	2647	2452	5099
Schchore ogodama	1944	2099	4043	1937	2091	4028	2797	2591	5388
Ela qebela	2592	2793	5385	2215	2385	4600	3359	3110	6469
Total	**19,655**	**19,782**	**39,437**	**20,858**	**20,472**	**41,330**	**22,485**	**20,829**	**43,314**

The population of Alata kfile ketema is 25,361 of which 13,167 are males and 12,194 are females. Abana kfile ketema is 17,953 of which 9,318 are males and 8,635 are females. According to the Tebela town administration office, the majority of the town's population is growing at a rate of 4.8 percent per year (CSA, 2020).

3.2 Material

This research was conducted to evaluate the hydraulic performance of the existing water supply in the distribution system. To achieve the goal of the research, the materials that were used were computers, endnote, Arc GIS Version 9.3, WaterGEMS V8i, and GPS Garmin72.

3.3 The Research Design

This study was designed to evaluate the hydraulic performance of the water distribution system. First, estimating the current and future water demand of the town was conducted by considering each mode of service. After that, the per capita water consumption of the town can be evaluated against annual consumption in a specific year. Then the percentage of the water loss was estimated. The total water produced and actual water consumption as aggregated from the individual contracts (customer meters) was used as an input for water loss analysis. The water loss analyses, both apparent and real, were carried out by using the top down water balance approach. Then they analyse the simulation, calibration, and validation of the model and determine the hydraulic parameters. Finally, the performance of the water distribution network was evaluated by the standard guidelines. Also, based on calculated performance indicators, key statistical comparisons are made, and strategies for loss reduction are developed for international experiences.

3.5 Data Analysis

To analyse the data which is collected from different sources, both qualitative and quantitative methods were used. From the quantitative methods, the descriptive statistical methods like percentage, graphs and cross tabulation was used in order to come up with the appropriate result. In addition to this, qualitative methods like narration were employed in the study. The computer software applications Origin8 and Excel were used to analyse the data obtained from the office. The field survey data for the distribution system was evaluated by using the engineering software called WatergemsV8i.

3.5.3 Modeling of the existing distribution system

3.5.3.1 WaterGEMS V8i

WaterGEMSV8i was used for the purpose of understanding the pressure regime, demand, velocity, head loss, and overall systematically studding and better understanding network operation (CAD/GEMs, 2008). Hydraulic performance analysis was carried out for an extended period of time using WaterGEMS. A GIS location map showing the town's water sources, reservoirs, and boost stations is produced by taking galvanized steel pipe (GPS) readings of the existing water sources, reservoirs, and pumping stations.

3.5.4.1 Sample size

Ideally, during the water distribution model calibration process, each link and node was adjusted for each link and node. However, 2%-10% of representative sample measurements can be made available for the use of model calibration due to limited financial and labour requirements for data collection. In general, international proposed guidelines stipulate that for a medium to highly detailed network model (medium to low skeletnoization), the following limits should be adopted (AWWA, 2005):

- ► 3% of the nodes in the network should be tested for pressure readings.
- ► 5% of the pipes in the network should be tested for flow readings. In the study area, there are 106 total junctions in the network. However, the minimum acceptable sample size was 3% of the total junction.

Hence, the sample size of the network was 0.03 x 106 = 3.10, which is approximately 3 junctions. Therefore, for this study area, three representative sample measurements were taken from the whole water distribution system for calibration.

4. RESULTS AND DISCUSSION

4.1 Evaluating the Current Water Supply Performance of Tebela Town

4.1.1 Water supply coverage

Water supply coverage can be defined as the percentage of people in access of water supply service in the town. To address the need of highly rising water supply needs in the urban population, the water supply service utilities have to manage the existing water supply systems in a manner to efficiently address the need. It is observed that the financial constraint, poor management of the water supply system and the low capacity of human resource has a great impact in the low coverage of water supply provision.

Table 4.2 Water supply coverage of Tebela Town for the years 2019-2020

Year	Annual water consumption (m^3/yr)	Total population no	Consumption l/person/day	Total population served by water from utility	percentage of water coverage
2019	322,871.8	41,330	21.40	11,796	28.6%
2020	337,230.3	43,314	21.33	11,207	25.9%

(Source: Tebela town water supply and sewerage enterprise data, and own study analysis)

- Percentage of Water Supply Coverage = (Population served/Total population)*100

In 2010 the Ethiopian Government presented the equally ambitious Growth and Transformation Plan (GTP) 2011-2015, which aims at increasing drinking water coverage, from 68.5% to 98.5%. In comparison with this plan, the water supply coverage is behind the plan (Ministry of Finance and Economic Development, 2010) and also in comparison, the Ethiopia's capital city, Addis Ababa water supply coverage during 2009 was found to be 60.67% (Shimelis Kabeto, 2011).

4.1.3 Water production and consumption capacity of the service

Table 4.3 Water production and Consumption

No	Description	Units	2016	2017	2018	2019	2020
1	Total population living in the service area	No	35,907	37,630	39,437	41,330	43,314
2	Population served public water taps	No	8,530	6,438	4,811	3,531	2,712
3	Population served yard water taps	No	7,181	7,525	7,887	8,265	8,495
4	Average daily per capita water cons.	(l/c/d)	21.52	21.47	21.41	21.40	21.33
5	Number of water point	No	15	13	12	12	12
6	Duration of supply	Hr/d	8	9	10	11	12
7	Volume of water production	m^3/y	338,441.1	361,091.5	373,251.8	392,742.7	407,267.0
8	Volume of water sold	m^3/y	281,947.0	294,845.6	308,131.0	322,871.8	337,230.3

(Source: Tebela town water supply utility office data and own study analysis)

Water production and consumption are indicated as below in figure 4.1.

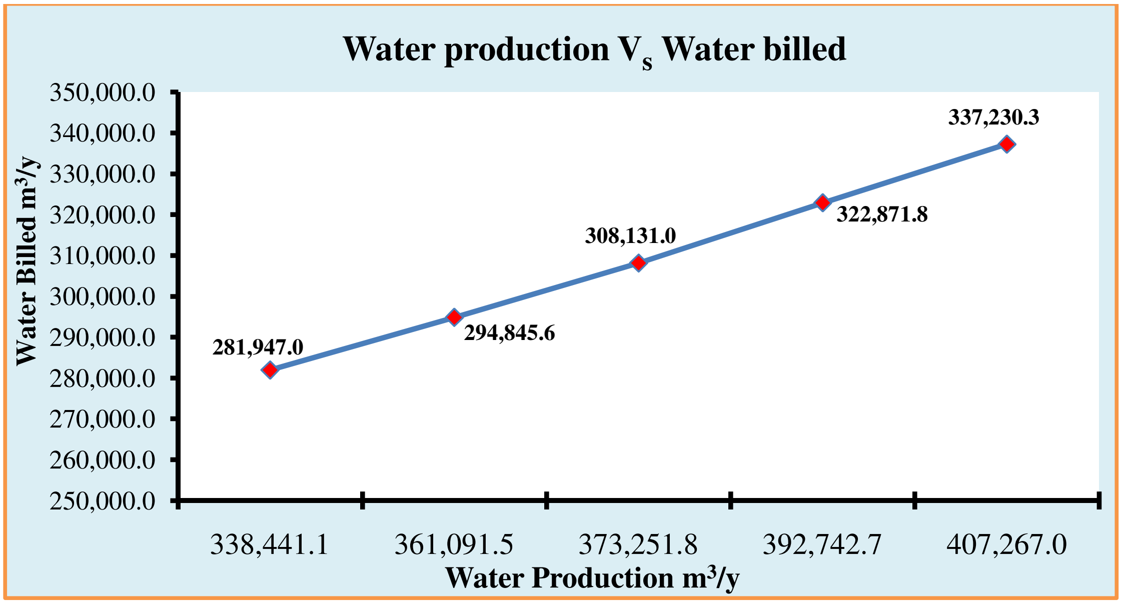

Figure 4.1 Total volume of water production and water billed

4.2 Water Losses Analysis

One of the major challenges facing water utilities is the high volume of water lost in distribution networks. If a large quantity of supplied water is lost; it is difficult to meet the level of satisfaction of the user community. Whereby, water loss for Tebela town's water supply system was assessed and discussed as below; because of the type of the system, the water is distributed equally to the community but, with the similar challenge mentioned above, it performed inefficiently against its aim.

The major sites for the loss are:

- Taps on every water point are open, overflowing the reservoir.
- Malfunctioning or damage of valves
- Vandalism and illegal disconnection
- Disconnection of joints
- Leakage occurred due to high pressure in some points.

The current production of water supply for Tebela town depends on Abana bora boreholes and koysho ogodama natural spring water that are administrated by Tebela town water supply and sewerage enterprise office. The designed water production capacity of the borehole system is 24.92l/s ($717m^3$/day) and the spring capacity is 4.6 l/s ($398.1m^3$/day). However, the actual production of water has been lower than the maximum capacity. The volume of billed water (consumption) for five consecutive years was depicted in figure 4.3.

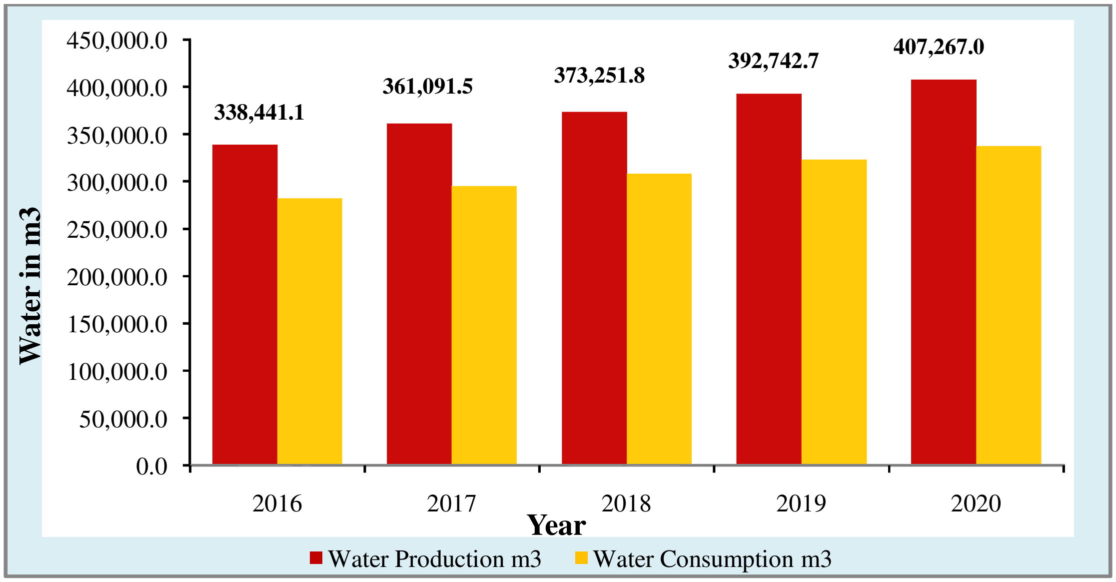

Figure 4.3 Water production and consumption for Tebela Town's water supply

Water loss from water distribution systems (WDSs) has long been a feature of water distribution system operations management. According to (Motiee *et al.*, 2007), total water loss or unaccounted for water (UFW) is the difference between the volume of water produced and the volume that is billed or consumed. The percentage of water loss in the town water distribution system is given in figure 4.4. According to figure 4.4, the water loss in 2020 (17.2%) is less than 2019 (17.79%) due to pump, pipe and fitting maintenance. The average amount of water, which actually reaches the consumers, therefore, accounts for only 82.8% of the total water produced. According to (Mckenzie *et al.*, 2006), the system efficiency is good (acceptable) if above 75% of water produced reaches the consumer. Thus, Tebela town water supply system is good (acceptable). Figure 4.4 shows that nonrevenue water from the system varies from year to year due to pipe aging, which causes leakage, pipe bursting, installation (excitation of network in new area), and illegal connection

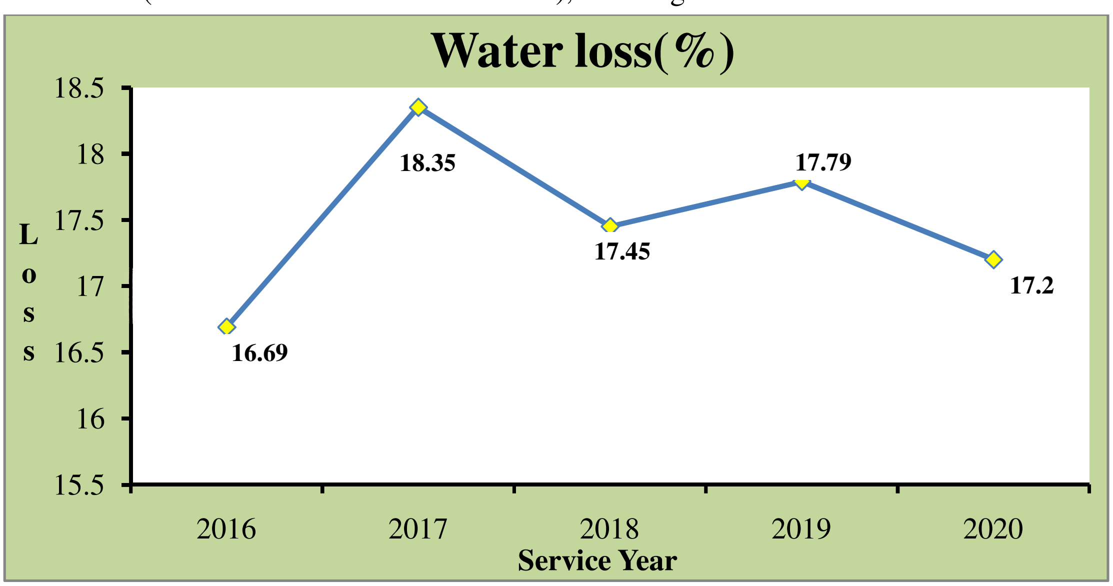

Figure 4.4

Water losses in percentage

4.2.1 Total water loss expressed as a percentage (UFW)

The total annual water produced and consumed within the specified year (2020) was 407,267.0m^3 and 337,230.3m^3 respectively, and the annual total water loss was 70,036.7m^3. That accounts for 17.2%. (Saroj, 2008) gives classification and descriptions of UFW as acceptable, which could be monitored and controlled, when the loss is < 10%, as intermediate, which could be control when the loss is 10-25% and as a matter of concern that reduces the water supply when the loss is > 25%. According to this study, the average water loss in Tebela town was 17.2%, showing that a controlling mechanism was needed.

4.2.1.2 Water loss expressed as the length of the main pipe

One of the best indicators of water loss in the distribution network system was determining loss as per the length of the main pipe. According to the town's water utility report, the total length of the water distribution line was estimated at around 22.592km. The water loss per kilometre length of the main pipe was determined as 70,036.7m^3/year ÷ (22.592km 365days)=8,493.33liters/km/day. According to (Farley *et al.*, 2008), the performance indicator of the physical loss target matrix describes a good condition system if water loss per length of main pipe is <1000 liters/km/day, an average condition system is between 10,000-18,000 liters/km/day and a bad condition system is > 18,000 liters/km/day. In line with this, the town's water loss per length of main pipe was 8,493.33 liters/km/day, which is shown to be in good condition.

4.2.1.2 Water loss expressed as per the number of service connections

Tebela town total number of service connections was 1730, which were obtained from the town's water utility. The water loss per number of service connections was determined as 70,036.7m3/year x1000liters ÷ (1,730×365days) = 110.91liters/connection/day. According to (Farley *et. al.*, 2008), the performance indicator of the physical loss target matrix describes a good condition system if water loss per length of main pipe <150 liters/connection/day, an average condition system is between 150-450 liters/connection/day and a bad condition system is > 450liters/connection/day. In line with this, the town's water loss per length of main pipe was 110.91liters/connection/day, which is shown to be in good condition.

4.2.1.3 Unbilled authorized consumption

Unbilled authorized consumption is the volume of water used for operational purposes, such as fire fighting, and water produced for free by water supply service workers. According to the Tebela town utility report (2020), the total volume of unbilled authorized consumption of water was 0m^3/year.

4.2.1.4 Estimating apparent losses

Apparent losses consist of unauthorized consumption, metering inaccuracies, and data handling errors (Lambert and Taylor, 2010) and are aggregated into 2,859.6m^3/ year + 7,526.92m^3/year + 843.01m^3/year, which is equal to 11,229.53m^3/year. This loss amount was 2.76% of the total production of water, which is about 16.03% of the total system loss as detailed in the following sub-section.

4.2.1.4.1 Unauthorized consumption

Unauthorized consumption includes illegal connections, unauthorized use of fire hydrants, meter bypassing, and a deficient billing collection system. It is difficult to estimate unauthorized consumption. According to the water service office's 2020 report, the amount of unauthorized consumption in the town was 2,859.6m^3/year.

4.2.1.4.2 Customer meter inaccuracies

Water meter inaccuracies are considered to be a significant component of apparent losses in the water supply system (Rizzo and Cilia, 2005). Water losses as a result of metering inaccuracies were analyzed using the comparison of testing bench values and the average water reading value of customer meters that were obtained from authorized consumption water in 2020. The total customer metering inaccuracies lost in the town's water utility was estimated at 7,526.92m^3/year taken from the utility office.

Generally, in the case of Tebela town, the main reason for this high meter under registration is the deterioration of water meters with age, resulting in inaccurate readings. This is highly influenced by the lack of a water meter testing and replacement program and an unlimited service year for meters in the distribution system.

4.2.1.4.3 Systematic data handling error

Data handling errors in the mater reading and billing systems contributed to the apparent losses. It includes billing system entry errors, account adjustments, invalid meter consumption readings, poor accounting, and others. It is difficult to estimate the value of the volume of data handling errors. Therefore, it is recommended to take the default value, which is 0.25% of the billed meter volume (Saroj, 2008). Based on the above recommended value, the total lost volume of data handling error of Tebela town was 0.25%*337,230.3m^3/year, which is equal to 843.01m^3/year.

4.2.1.5 Estimating real losses

This category includes the volume of water lost through all types of leaks, bursts, and overflows on the main, service reservoir, and service connection, up to the point of customer metering. Real losses can be calculated as the volume of NRW minus the sum volume of apparent losses and unbilled authorized consumption. Based on this definition, the volume of total real loss was 58,807.17m^3/year, which covers 14.44% of the total production, which is 83.96% of the total system loss. This result signifies more of the loss in the system as real loss which is mainly caused due to deterioration of the existing distribution system infrastructure.

4.2.2 Quantifying water loss by the water balance method

To estimate the water loss by using the water balance method for Tebela town in the year 2020 based on international water association (IWA) standards, the water balance components are obtained by using the available data and estimated in the above. The results are summarized in the table below.

Table 4.4 Water balance (m^3/year) for Tebela town year 2020

System in put volume =407,267 m^3/year	Authorized consumption =337,230.3 m^3/year	Billed authorized consumption =337,230.3 m^3/year	Billed metered consumption m^3/year =337,230.3 Billed unmetered consumption m^3 /year=0	Revenue water =337,230.3 m^3/year
		Unbilled authorized consumption =0 m^3/year	Unbilled metered consumption m^3 /year=0 Unbilled unmetered consumption m^3 /year=0	
	Water loss =70,036.7 m^3/year	Apparent loss =11,229.53 m^3/year	Unauthorized consumption m^3/year=2,859.6	Non-revenue water =70,036.7 m^3/year
			Customer meter inaccuracies =7,526.92 m^3 /year	
			Systematic data handling errors =843.01 m^3 /year	
		Real loss =58,807.17m^3/year		

4.4 Hydraulic Performance of Water Supply System

The performance of water distribution networks does not depend only on the ability to deliver adequate flows and pressures, but also on its efficiency in doing so. Previous measures equate demand satisfaction to performance and apply alternate reliability measures that are proportional to pressure surplus. The water distribution network is a loop network; it has one reservoir tank at higher elevations. The level of the reservoir tank is 1678.38m and the flow distribution is supplied by a gravity system from the reservoir to customer end taps. Flow plays a role in supplying flow. Pumping is still required from the source (BH) to the reservoir tank. The pump curve is defined by one head versus flow coordinate of 86 m for 24.9l/s. The network configurations were modelled, the original (looped) system, the same system with fewer loops, and with increased diameters.

4.4.1 Network simulation

To build and simulate the hydraulic model, WaterGEMSV8i water distribution modeling software was used. The water distribution network map was obtained from the Tebela town water supply and sewerage enterprises which were prepared with a feasibility study design document report of the town's water supply distribution network.

4.4.2 Pipe network maps development

In building system models, it is typically used to draw a system map for the water distribution system because it illustrates a wide variety of valuable characteristics. System maps may include information such as:

Pipe alignment, connectivity, material, diameter, and so on.

The locations of other system components, such as tanks and valves,

Miscellaneous notes or references for tank characteristics

Pressure zone boundaries, elevations

In my study, the map was developed using data collected from the site and analyzed by WaterGEMS software.

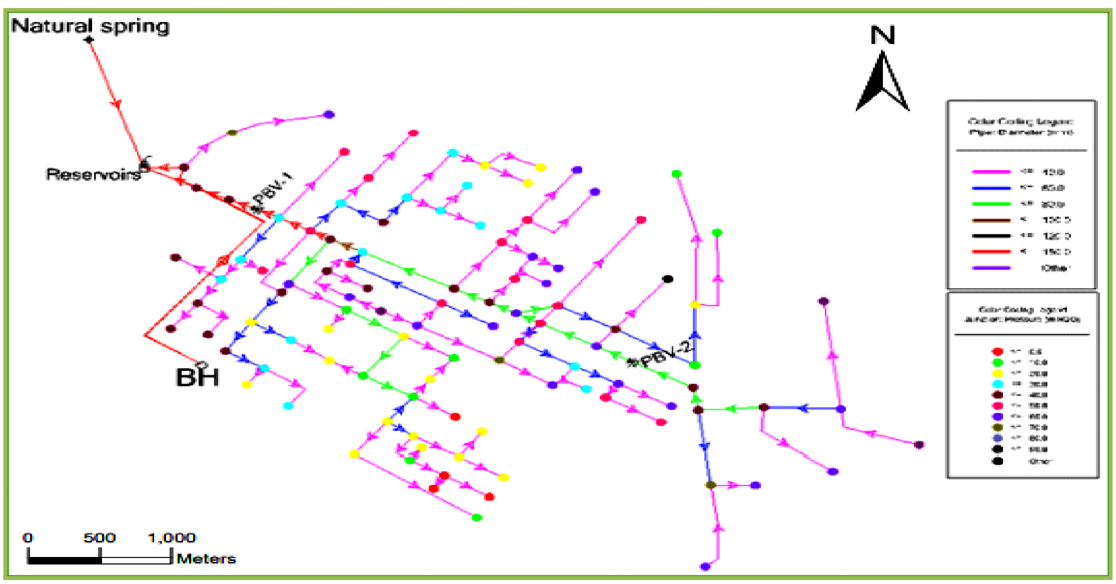

Figure 4.5 Tebela town's existing water distribution pipe network (Source: Own software analysis)

4.4.3 Analysis of pressure, flow and velocity of existing water distribution system

4.4.3.1 Junction report of WaterGEMS out put

The evaluation of the analysis results of the given existing water distribution system in Tebela town nodes reports analysis in a hydraulic model of WaterGEMS V8i. According to the model result, most areas of the distribution system have low velocity. This low velocity causes several problems. And some of them are shortage of source (Q in l/s), large size of pipe diameter, and topography of the area. So some parts of the distribution net work system need modification of pipe diameter during minimum day demand time to adjust pressure within the (MoWR, 2006) guide line standard as in Apendix_B1.

4.4.3.2 Pipe report of WaterGEMS output

Water velocities shall be maintained at less than 2.2 m/sec, except in short sections and velocities in small diameter pipes (evaluation results analysis of hydraulic parameters of pipe sizes with velocities by the use of hydraulic model (WaterGEMS V8i) standard are discussed. The velocity of 0.42 m/s–2.2 m/s during analysis is listed in appendix_A2 and B2 according to the (MoWR, 2006) guide line standard. As well as all velocity greater than 0.6 m/s of the distribution network listed in appendix_A2 during peak hour time from 6:00-9:00 AM.

4.4.3.3 Pressure analysis

In Ethiopia's water supply distribution system network, the minimum and maximum operating pressures were 15m and 70m, respectively (MoWR, 2006a). Pressure influences the water supply capacity of the distribution system. In order to achieve a 15m minimum and 70m maximum operating pressure, it is necessary to provide a pressure control valve, establish a boosting station, and replace the old pipe with the new one. The maximum pressure in the main is considered not to exceed 80m to limit leakage and stress on pipes (Mosissa, 2008). There is no defined maximum and minimum pressure ranges designed by the town's water utility. Therefore, a literature review based recommendation for optimal and minimum pressure was used to assess the hydraulic performance of the water distribution system. According to (Totsuka *et al.,* 2004), those consumers further away from supply points always collect less water than those nearer to the source due to pressure losses in the network, which increase as far as the source. Pressure was increased as elevation decreased and vice versa. Households located at a higher elevation and close to the reservoir site have access to water at low water pressure (Mekonnen, 2014). During hydraulic modeling of the water pressure of Tebela town, 106 nodes and 117 pipes were identified. With regard to the current simulation, the result of pressure at peak consumption was summarized in table 4.7, and detailed in appendix A1.

Table 4.7 Distribution of pressure at peak hour consumption

Pressure (m of H2O)	Number of nodes	Percentage
<=15	26	24.53
<=25	10	9.43
<=35	19	17.92
<=45	17	16.04
<=55	18	16.98
<=65	16	15.09
<=70	0	0
Above	0	0
Total	106	100

After hydraulic analysis using BenetlyWaterGEMSV8i as shown that table (4.7), 24.53% of the nodes are under desirable minimum pressure and 0% of the nodes are exceeding maximum allowable pressure during peak hour consumption. At peak time consumption, junctions 43, 51, 52, and 54 were all under negative pressure. Thus, only 75.47% of nodes have pressure within the recommended limit (15m to 70m).

Therefore, from the above table result, 24.53% should be improved in the distribution system to meet the permissible pressure. Lower pressure can cause reduction of quantities of water supplied to the consumer and entry of a contaminant or self deterioration of water quality within the network itself severe damage to public health.

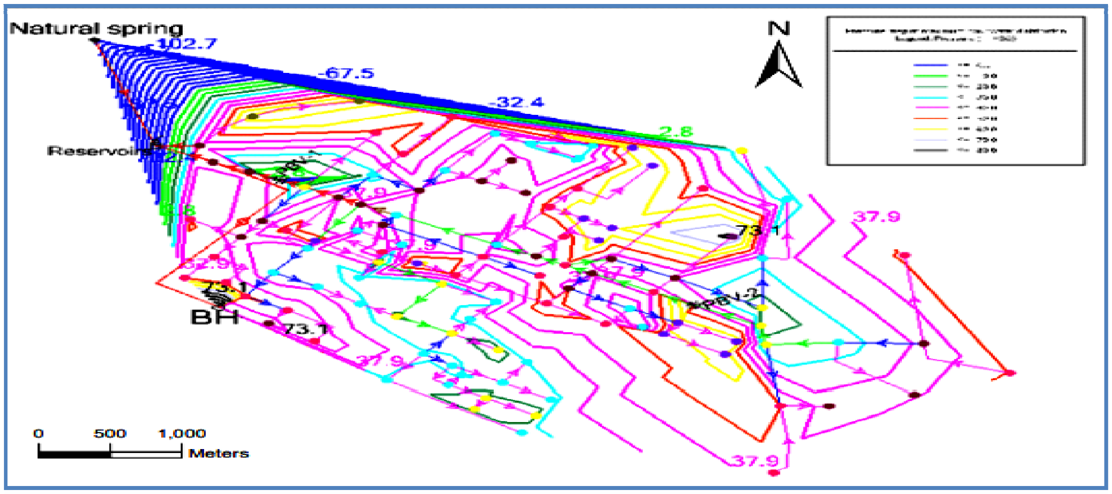

Figure 4.6 Pressure contour map of Tebela town at peak hour consumption

Table 4.8 Distribution of pressure at minimum consumption time

Pressure (m of H2O)	Number of nodes	Percentage
<=15	4	3.77
<=25	21	19.81
<=35	25	23.58
<=45	27	25.47
<=55	15	14.15
<=65	9	8.49
<=70	3	2.83
Above	2	1.89
Total	106	100

As shown in table 4.8 above, 3.77% of the nodes are under desirable minimum pressure and 1.89% of the nodes are exceeding maximum allowable pressure during minimum hour consumption. There is no negative pressure during a minimum consumption time, while, 94.34% of the nodes are in the permissible pressure range of a minimum of 15m and a maximum of 70m. However, 1.89% of the nodes were getting water above standard pressure (>70m) due to low consumption at midnight when most of the consumers are sleeping and not using water. Higher pressure may cause the pipe to burst.

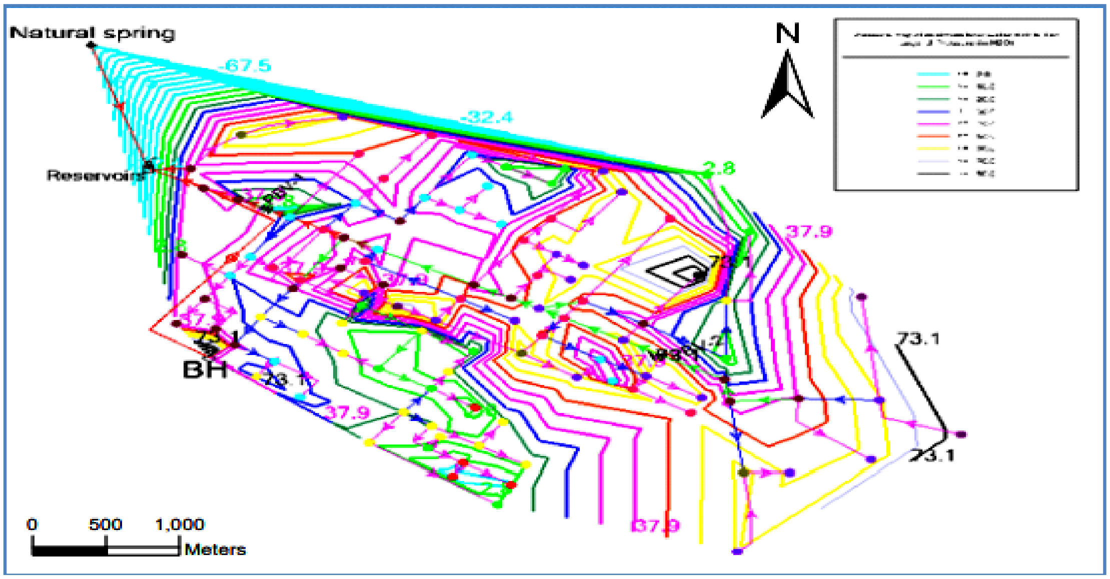

Figure 4.7 Pressure contour map of Tebela town at minimum hourly consumption

In the case of Tebela town, the main causes of water supply interruption were shortage of water from the source, lack of maintenance, improper function of the pump, and interruption of electric power in the pumping pressure system. In conclusion, to achieve a 15m minimum and 70m maximum pressure, it is necessary to give a pressure control valve, establish a boosting station, and replace the old pipes with new ones that have the required diameter.

4.4.3.4 Velocity analysis

The velocity of water flow in a pipe is also one of the important parameters for evaluating the hydraulic performance of a water supply distribution system. According to (Andey and Kelkar, 2007), flow in the pipe below 0.6m/s causes water stagnation, sediment accumulation, and bacterial growth in the pipe. On the other hand, the velocity of flow in the pipe above 2m/s causes head loss as well as water hammer. As the demand pattern changed, so did the velocity of the water distribution system. At peak consumption time, the values are different as compared to those at low consumption time. The town's water supply distribution system's velocities during peak consumption time were summarized in table 4.9 below.

Table 4.9 Velocity for water supply distribution system during peak hour consumption

Velocity (m/s)	Number of pipes	Percentage	Effect
<=0.5	1	0.85%	Water stagnation happens
<=0.6	34	29.06	Sedimentation happens
<=2	78	66.67	An acceptable level
Above	4	3.42	Head loss and water hammer
Total	117	100%	

As indicated in table 4.9 above, 0.85% of the pipes are below the desirable minimum velocity and 3.42% of the pipes are exceeding the maximum allowable velocity during peak hour consumption. While 95.73% of the pipes are in the recommended velocity range of minimum 0.6 m/s and maximum 2 m/s velocities. In this study area, 4.27% of the velocity is not in a suitable range based on Ethiopia's urban water supply design guideline criteria.

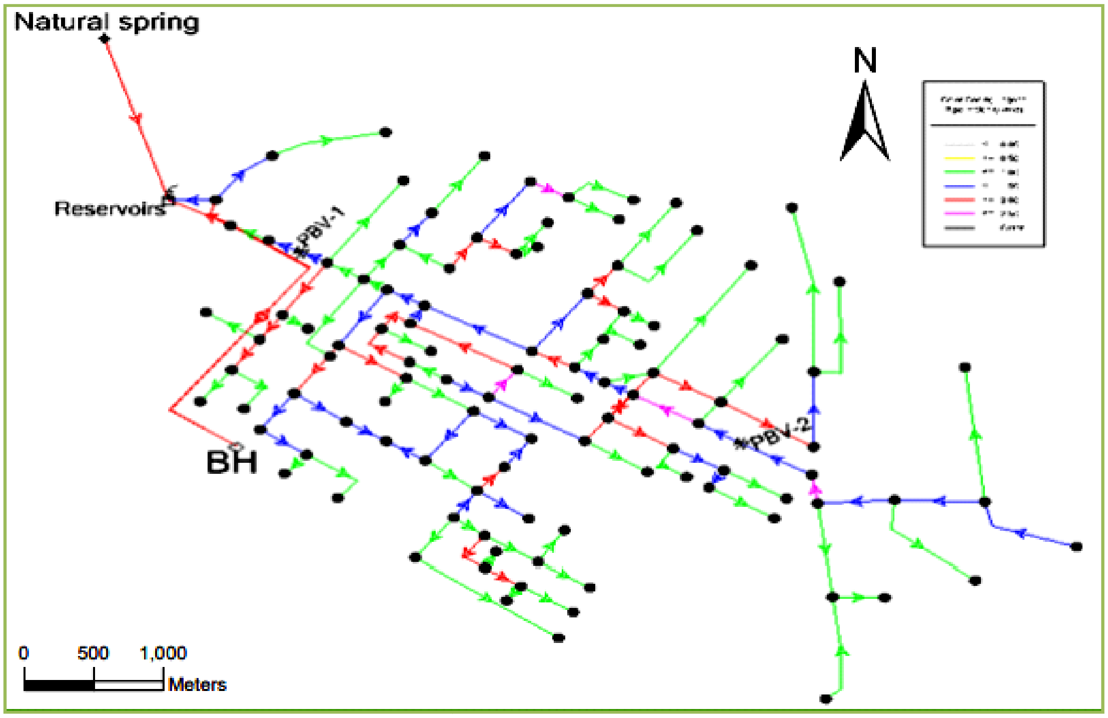

Figure 4.8 Velocity for water supply distribution network during peak hour consumption

Table 4.10 Velocities for water supply distribution system during min consumption time

Velocity (m/s)	Number of pipes	Percentage	Effect
<=0.5	40	34.18	Water stagnation happens
<=0.6	9	7.69	Sedimentation happens
<=2	68	58.12	An acceptable level
Above	0	0	Head loss and water hammer
Total	117	100%	

As shown in table 4.10 above, 34.18% of the pipe was below the desirable minimum velocity and no pipe velocity exceeded the maximum allowable velocity during low consumption time. While 65.81% of the pipes are in the recommended velocity range of minimum 0.6 m/s and maximum 2 m/s velocities. In this study area, 34.18% of the velocity is not in a suitable range based on Ethiopia's urban water supply design guideline criteria. There is no head loss and water hammer in this study area during the minimum consumption time.

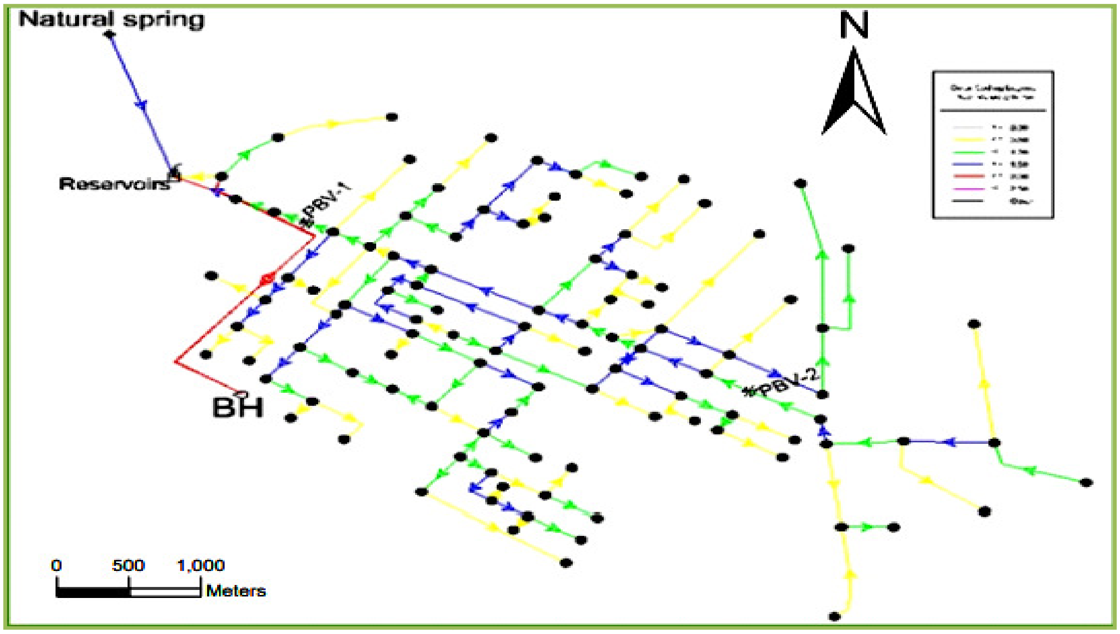

Figure 4.9 Velocity for water supply distribution network during minimum hour consumption.
Generally, the town's water distribution system's velocity was inadequate since velocity in major pipe parts during the minimum consumption time and peak hour consumption, as shown in figures (4.8) and (4.9). Therefore, control of the flow velocity in water distribution networks should be maintained in order to avoid pipe breaks, water hammer, and water stagnation which cause sediment deposition in the pipe and head loss. Velocity range can also be adopted as a design criterion. Low velocities are undesirable for sanitary reasons, while excessively high velocities result in significant head losses. Figure 4.10 shows that velocity is decreasing from the main line to the sub distribution line (for selected pipe). Figure 4.10 Velocity variations between the node at the main line and the sub distribution line and with respect to distance.

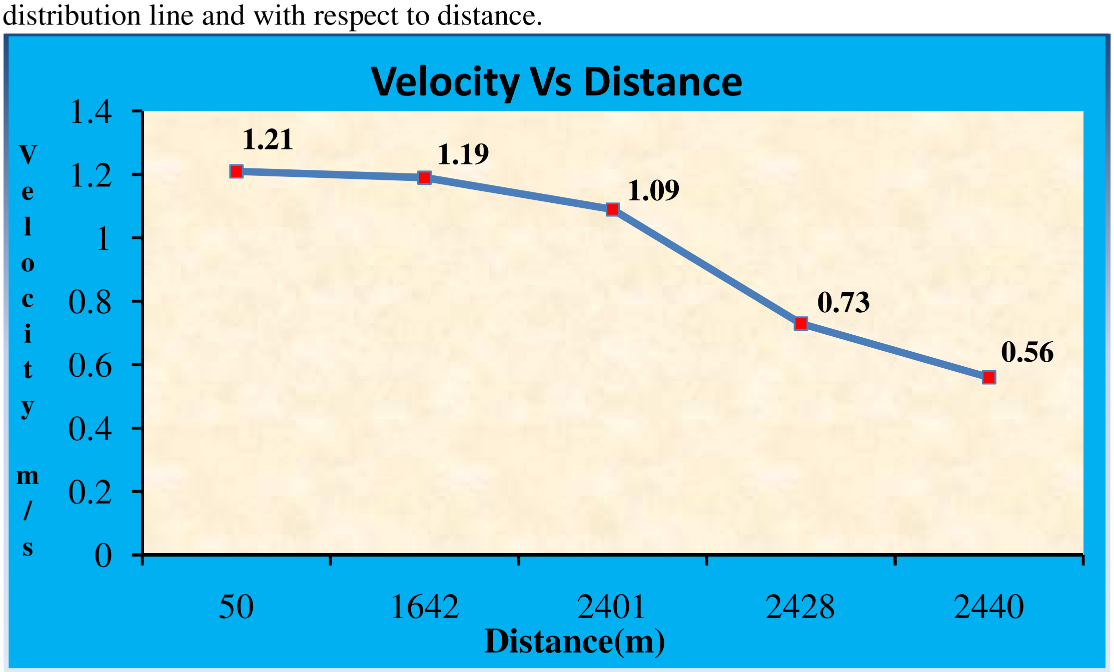

Figure 4.10 Velocity variations between the node at the main line and sub distribution line
According to (Yitayal, 2019), the velocity of flow in the pipe below 0.6 m/s causes sediment

accumulation. Town distribution system velocity in pipe was also inadequate. Figure 4.11 below shows that velocity in the distribution network is high during high consumption time for selected pipes. Detailed in appendix A2, during high consumption the faucets open, so the velocity of water increases.

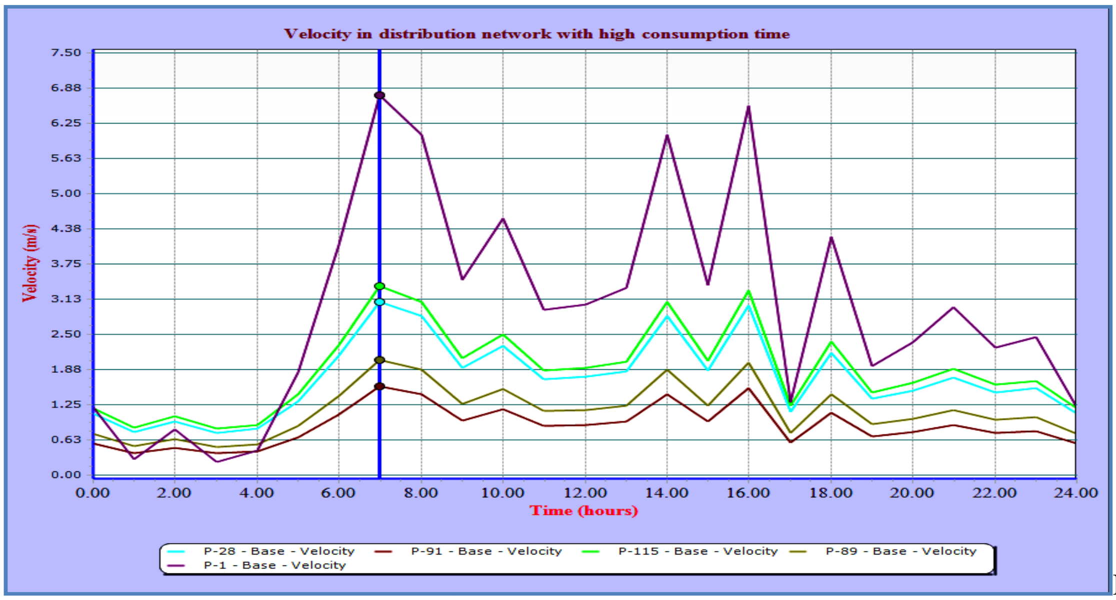

Figure 4.11 Velocity distributions for selected pipe

4.5 Calibration Model Result

Calibration is an iterative procedure of parameter evaluation and adjustment by comparing simulated and observed values. The pressured field measured data at J-28, J-51 and J-88 were used for calibration of the model. As the model gives an automatic C value for GSP pipes, it is 130. Since the existing pipe age is seven years, the roughness coefficient of pipes is not less than the model values. Calibration can be carried out on the base scenarios within the acceptable level. The simulated pressure value of simulated scenarios, the peak demand and low demand time was calibrated until the result was approximately equal to the measured pressure value. For calibration of a water distribution system, pipe roughness was considered as the primary calibration parameter in this study. In addition, nodal demand was also adjusted for model calibration. An extended period simulation has be performed and compared with the values at three observed points. Following analysis, it was discovered that model data and observed data were reasonably matched for a C value of 130, as per the AWWA guideline (2005). The model output and observed data (from three observation points) matched within a reasonable extent for a C value of 130 for given pipes.

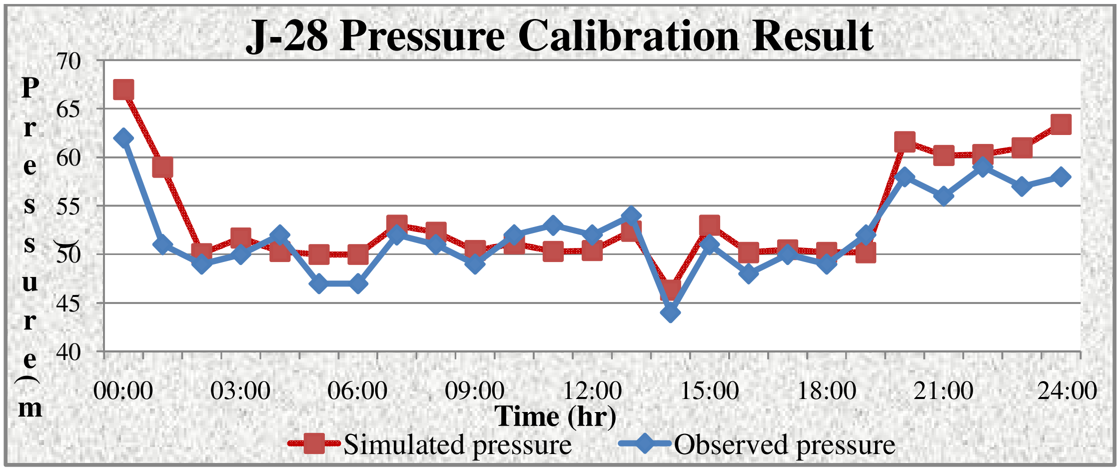

Figure 4.12 J-28 Pressure calibration result

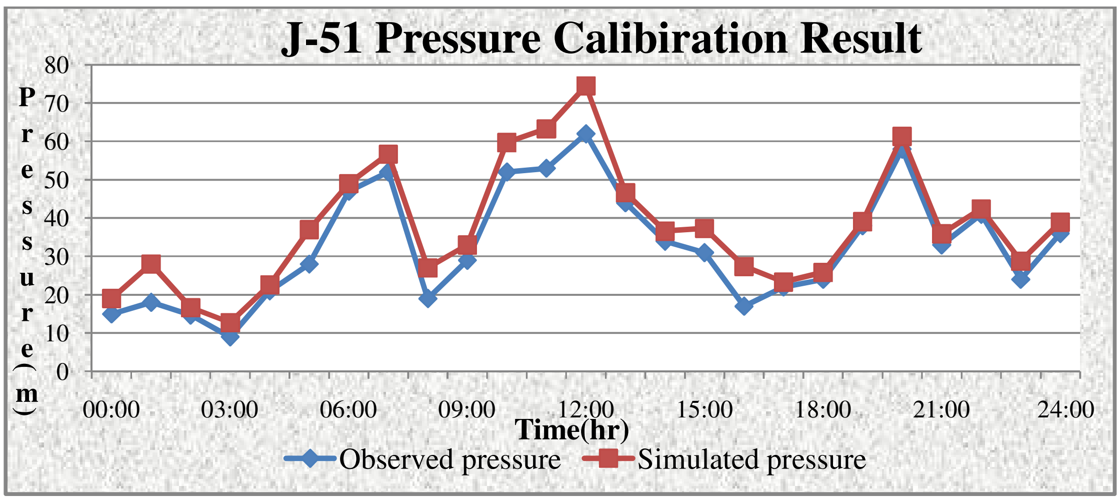

Figure 4.13 J-51 Pressure calibration result

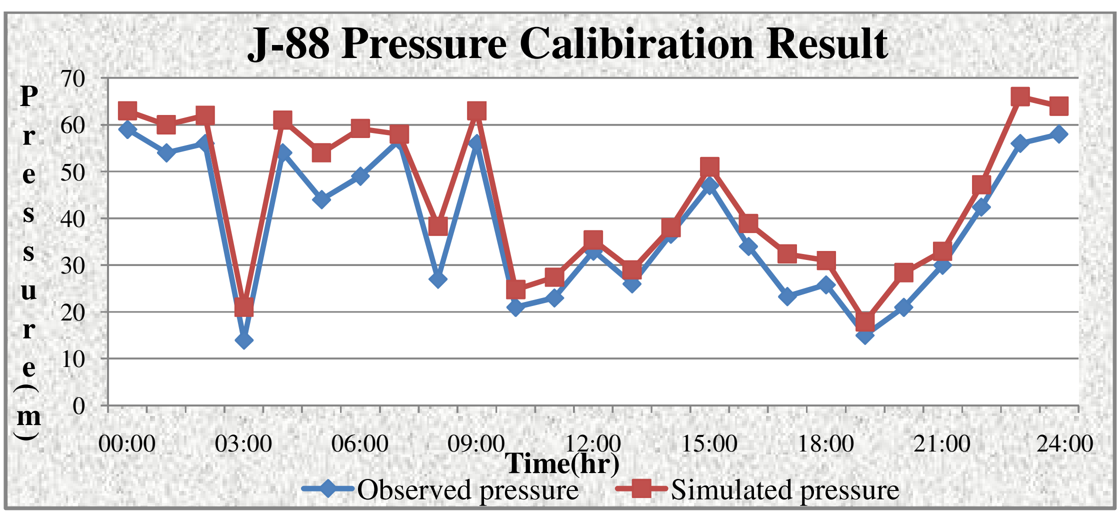

Figure 4.14 J-88 Pressure calibration result

The degree of accuracy varies depending on the size of the system and the amount of field data and testing available to the modeler.

Table 4.11 Junction pressure calibration based on difference pressure errors

Time(hr)	Junctions	Observed pressure(m H_2O)	Simulated pressure(mH_2O)	Differences pressure error (mH_2O)
3:00 AM	J-28	50	51.7	1.7
	J-51	15	16.6	1.6
	J-88	71	72.6	1.6
6:00 AM	J-28	47	50	3
	J-51	6	5.1	-0.9
	J-88	45	47	2
2:00 PM	J-28	46	46.3	0.3
	J-51	10	12.8	2.8
	J-88	44	46.6	2.6
6:00 PM	J-28	49	50.2	1.2
	J-51	11	12.7	1.7
	J-88	72	75.1	3.1
Average				1.725

As shown in table 4.11 above the computed pressure values have an average error of 1.725m pressure from simulated to observed values. As a result, the model is calibrated to meet the criteria for pressure calibration at the average level (average 1. 5m to maximum 5m).

4.6 Model Validation

Methods used to get unbiased estimates of the future performance of statistical prediction models and classifiers include data splitting and re-sampling. Model validation is a means of assessing the applicability of a given model with respect to field measured data. The pressure field measured values at J-28, J-51, and J-88 were used for validation of the model. The model validation work was taken manually using the correlation coefficient equation (R^2) method and the graph figure presented 4.15 below.

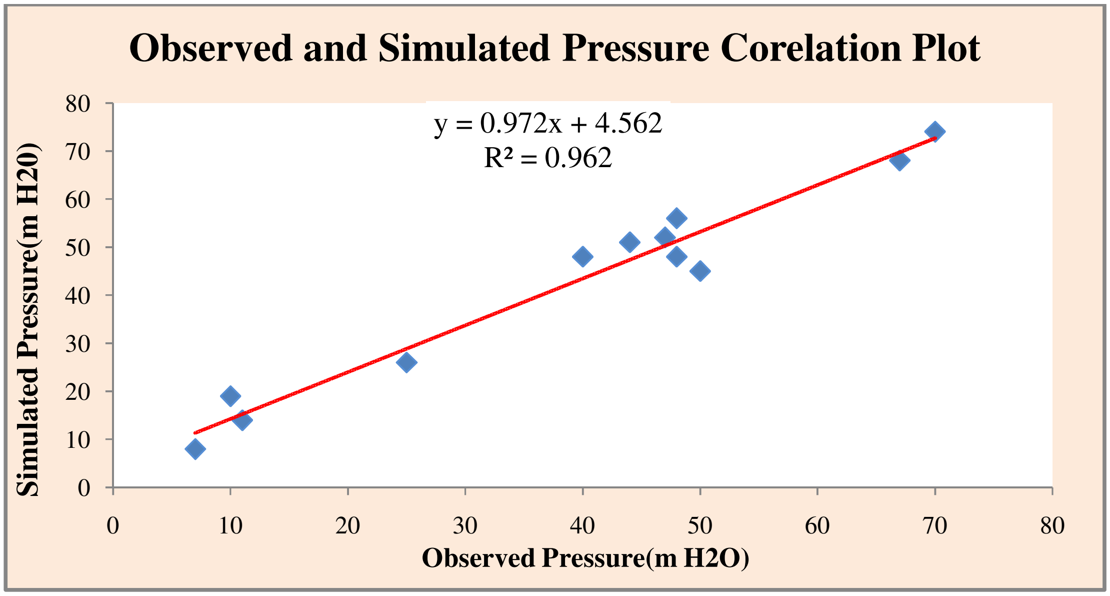

Figure 4.15 Relation between field measured and simulated pressure

From the above figure 4.15, the correlation (R^2) value for peak low demand time was 0.962. Since the value of R^2 approaches 1 for this scenario, that indicates a good correlation between the field's measured pressure and simulated pressure. R^2 shows how much of the variance in the dependent variable would be accounted for if the model was derived from which the sample was taken.

4.7 Hydraulic Network Improvement

There are three sets of design criteria to be considered in designing or improving a system. These are pressure, residual chlorine, and velocity. The design criteria used in the design of water supply distribution system components, nodal pressure during the period of peak demand, and optimum velocities of the transfer and distribution mains. Modification to the problems is made by creating new alternatives and scenarios, trial and error procedures until a solution appears to meet the design criteria.

4.7.1 Adding a pressure reducing valve to the network

The best operational practice to optimize the operation of the water distribution system was to control the pressure in the network. This management of pressure has been reflected in the aspect of reducing excessive pressure by installing a pressure reduction valve. By controlling the pressure, it is possible to reduce the amount of water loss from the system, the occurrence of internal damage, and power consumption related to high pressure at a minimum hourly demand pressure was high at lower elevation areas. Installing pressure-reduced valves around Humbo St.George Church at junction (J-17) and Mehal Kebele area at junction (J-88) links that have maximum pressure was used to reduce excessive pressure to the desired allowable value seen in the table below.

Table 4.12 Excessive pressure in the improved system at minimum consumption hour

Junction	Elevation (m)	Pressure(mH_2O) before adding PRV	Pressure(mH_2O) after adding PRV
J-17	1608.40	70.3	58.4
J-88	1596.35	75.1	60.7

Table 4.13 Improved system nodes with pressure at minimum consumption hour

Pressure (mH_2O)	Number of nodes	Percentage
<15	4	3.77
15-70	102	96.23
Total	106	100

After modifying the existing water distribution system by adding a pressure reducing valve, 96.23% of the junctions are in the recommended pressure range of a minimum of 15m of H_2O and a maximum of 70m of H_2O, and only 3.77 % of the junctions are not in the recommended pressure range.

4.7.2 Adding a pressure boosting pump to the network

According to (Swamee *et al.,* 2008) the minimum design nodal pressures are prescribed to discharge design flows onto the properties. It is based on population served, types of dwellings in the area, and fire fighting requirements. The general consideration is that the water should reach up to the upper stories of low rise buildings in sufficient quality and pressure, considering fire fighting requirements. In case of high rise buildings, booster pump (valves) are installed in the water supply system to water for the pressure head requirements. Installing pressure booster pump at Evangelical lutheran church, Tebela elementary school, Shoya and Jegera village in order to increase the lower pressure of 22 junctions, the area of low pressure value seen in the table below.

Table 4.14 Low pressure in the system at peak consumption hour

Junctions	Elevation (m)	Low pressure (mH_2O)	Reason	Village
J-29	1,628.27	5	high topography area	Lutheran Church
J-30	1,611.01	10	high topography area	Lutheran Church
J-31	1,610.27	8.5	high topography area	Lutheran Church
J-32	1,610.89	13.9	high topography area	Lutheran Church
J-39	1,612.34	14.1	high topography area	Lutheran Church
J-40	1,620.55	10.7	high topography area	Lutheran Church
J-41	1,622.32	5.2	high topography area	Elementary School
J-42	1,619.85	6.6	high topography area	Elementary School
J-45	1,616.97	12.1	high topography area	Elementary School
J-46	1,616.20	9.4	high topography area	Elementary School
J-48	1,619.04	13.1	high topography area	Elementary School
J-49	1,618.05	11.7	high topography area	Elementary School
J-50	1,617.49	8.9	high topography area	Elementary School
J-53	1,613.46	12.7	high topography area	Elementary School
J-55	1,617.46	14.2	high topography area	Elementary School
J-58	1,614.26	9.8	high topography area	Elementary School
J-59	1,613.25	9.7	Long distance from source	Shoya
J-63	1,612.56	9.7	Long distance from source	Shoya
J-89	1,610.49	13.7	Long distance from source	Shoya
J-90	1,611.46	3.4	Long distance from source	Jegera

J-91	1,612.38	5.6	Long distance from source	Jegera
J-102	1,619.53	5.3	Long distance from source	Shoya

4.7.3 Improving pipe size

Increasing the diameter of the pipe in a water distribution model results in a corresponding decrease in velocity and an increase in pressure. At peak hour consumption, the velocities outside of the design range are modified by resizing the pipe diameter. The pipe which does not meet the allowable minimum and maximum velocity was selected for modification to improve the water distribution system.

Table 4.15 Modified pipe size in main distribution system at minimum hour consumption

Label	Existing pipe size(mm)		Modified pipe size(mm)	
	Diameter (mm)	Velocity(m/s)	Diameter (mm)	Velocity(m/s)
P-1	150	0.45	80	0.80
P-6	40	0.42	25	0.61
P-7	40	0.42	25	0.61
P-16	40	0.42	25	0.60
P-18	40	0.42	25	0.60
P-22	40	0.42	25	0.60
P-25	50	0.47	32	0.64
P-27	40	0.42	25	0.62
P-29	40	0.42	25	0.60
P-31	40	0.42	25	0.61
P-33	40	0.42	25	0.60
P-34	40	0.42	25	0.60
P-35	40	0.42	25	0.62
P-36	40	0.42	25	0.61
P-40	40	0.42	25	0.60
P-47	40	0.42	25	0.60
P-48	40	0.42	25	0.60
P-50	80	0.42	50	0.71
P-55	40	0.42	25	0.63
P-59	40	0.45	25	0.61
P-61	40	0.42	25	0.60
P-63	40	0.42	25	0.60
P-67	40	0.42	25	0.60
P-69	40	0.45	25	0.62
P-70	40	0.42	25	0.61
P-73	50	0.49	32	0.64
P-79	40	0.42	25	0.60
P-80	40	0.42	25	0.60
P-81	40	0.42	25	0.62
P-82	40	0.42	25	0.61
P-85	40	0.42	25	0.60

P-86	40	0.42	25	0.61
P-88	40	0.42	25	0.60
P-91	40	0.42	25	0.60
P-94	50	0.4	32	0.64
P-97	80	0.49	50	0.71
P-104	40	0.42	25	0.60
P-105	40	0.42	25	0.60
P-107	40	0.45	25	0.62

Table 4.16 Improved system velocity in the distribution system at minimum hourly consumption

Velocity (m/s)	Number of pipes	Percentage	Effect
<=0.6	31	26.49	Sedimentation happens
0.6-2	86	73.50	An acceptable level
>2	0	0	Head loss and water hammer
Total	117	100	

After modifying the pipe sizes in the existing distribution system as shown in the table above, 73.50% of the pipes are in the recommended velocity range of above 0.6 m/s and below 2 m/s.

5. CONCLUSION AND RECOMMENDATION

The purpose of this study was to evaluate the performance of the Tebela town water distribution system under existing and projected future demand conditions. Based on the evaluations presented, the following conclusions and recommendations were forwarded.

5.1. Conclusion

The main sources of water for the people living in Tebela town were borehole which gives 717m^3/day of water and natural springs with distribution that provided 398.1m^3/day. The town has no water treatment plant, but the reservoirs water is treated with chlorine monthly. The water coverage of the area is about 25.9%, which is moderate level.

Total water loss was calculated using a percentage of the system input volume, the length of the mains, and the number of connections. Generally, based on the analysis results, the total water loss from the system was 70,036.7m^3/year, which accounted for 17.2% of the total water production in the study area. The total apparent loss volume includes the loss due to unauthorized consumption, metering inaccuracies, and data handling errors and was aggregated to 11,229.53m^3/year, which covers 16.04% of the total losses. Real loss includes the volume of water lost through all types of leaks, bursts and overflows on service reservoirs. In this study, the real loss volume was found to be 58,807.17m^3/year, which covers 83.96% of the total losses. Real losses are the dominant component of water losses in Tebela town's water distribution system. High levels of water losses have a serious impact on Tebela water service finance as well as on available water resources in water scarce environments. During hydraulic modeling of the town's water pressure, 106 nodes and 117 pipes were identified. At peak hour consumption, 24.53% of the nodes are under desirable minimum pressure, no nodes are exceeding maximum allowable pressure, junctions 43, 51, 52, and 54 are negative pressure, and 75.47% of the nodes have pressure within the recommended limit. During minimum time consumption, 3.77% of the nodes are below the desirable minimum pressure, 1.89% of the nodes are getting water above standard pressure, and 94.34% of the nodes are in the permissible pressure range. For peak hour consumption, 0.85% of the pipes are below the desirable minimum velocity, 3.42% of

the pipes velocity is exceeding the maximum allowable velocity, and 95.73% of the pipes are in the recommended velocity range. During low consumption times, 35.04% of the pipes' velocity is below the desirable minimum velocity, and 64.96% of the pipes are in the recommended velocity range. After modifying the existing water distribution system, 96.23% of the junctions are in the recommended pressure range of a minimum of 15m of H_2O and a maximum of 70m of H_2O. Only 3.77% of the junctions are not in the recommended pressure range. Also, 73.50% of the pipes are in the recommended velocity range of above 0.6 m/s and below 2 m/s at minimum hourly consumption; only 26.50% of the pipes are less than or equal to 6m/s.
Therefore, the result of the analysis showed that the overall technical performance of the existing water distribution system in the town was moderate, which was reflected by the medium water production rate, water consumption, level of nonrevenue water, water coverage, and same velocity and pressure not in the permissible range.

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