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BAHIR DAR INSTITUTE OF TECHNOLOGY

SCHOOL OF RESEARCH AND GRADUATE STUDIES

FACULTY OF CIVIL AND WATER RESOURCE ENGINEERING

Master of Science in Water Supply and Sanitary Engineering

MSc Thesis on

**Evaluation of Hydraulic Performance of Kombolcha Town Water Distribution
Systems**

By

Selamawit Workineh Tsige

Advisor: Dr. Tamru Tesseme

March, 2022

Bahir Dar, Ethiopia



BAHIR DAR UNIVERSITY
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FACULTY OF CIVIL AND WATER RESOURCE ENGINEERING
**Evaluation of Hydraulic Performance of Kombolcha Town Water Distribution
Systems**

By

Selamawit workineh

A thesis submitted

In Partial Fulfillment of the Requirements for the Degree of Master of Science in Water
Supply and Sanitary Engineering

Main Advisor: Tamru Tesseme (Ph.D.)

March, 2022

Bahir Dar, Ethiopia

Declaration

I, the undersigned, declare that the thesis comprises my own work. In compliance with internationally accepted practices, I have acknowledged and refereed all materials used in this work. I understand that non-adherence to the principles of academic honesty and integrity, misrepresentation/ fabrication of any idea/data/fact/source will constitute sufficient ground for disciplinary action by the University and can also evoke penal action from the sources which have not been properly cited or acknowledged.

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Approval of thesis for defense result

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Abstract

It's mandatory to assess the hydraulic performance of water supply distribution systems to address water distribution issues in an urban water supply system. This can be accomplished by analyzing the current status of the water distribution system. This study was aimed to analyze the existing water distribution system in kombolcha town using WaterGEMS (Geospatial Engineering modeling software) software. WaterGEMS model was used for automated calibration and analyzing the hydraulic parameters of water distribution system. To reduce the difference between measured and predicted pressure, the model was calibrated at the selected nodes within good performance. The result showed that the water supply coverage was low which covers only 69.4%. besides water demand and supply of kombolcha town were not balanced. The people used daily water of 50 l/day with the billed water amount is 67.1% of production and 32.9% water is considered to be non-revenue (NRW). The simulation result of the existing water distribution system showed that, during peak hour demand 23.1% of junction have low pressure ($<15\text{m H}_2\text{O}$) and during minimum demand hour 67.4% of the junctions have high pressure ($>60\text{m H}_2\text{O}$). The hydraulic performance analysis also revealed that 83.5% and 24% of total pipes have velocity below 0.6m/s at minimum and peak demand respectively. The study concluded that the hydraulic performance of Kombolcha town water supply distribution system is running below the expected level. Thus, the high pressures in the distribution should be managed by using pressure reducing valves and a low pressure and velocity should be improved by making rehabilitation of existing boreholes, which satisfy the existing peak water demand in the town.

Keywords: *Hydraulic performance, WaterGEMS, Darwin Calibrator, Water loss*

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Abbreviations

ADD	Average daily demand
ADSWE	Amhara Design and supervision work Enterprise
ALC	Active Leakage Control
AWRDO	Amhara Water Resource Design Office
AWWA	American Water Works Association
BABE	Bursts and Background Estimate
CAD	Computer-Aided Design
CARL	Current Annual real loss
CSA	Central Statistical Agency
DCI	Ductile Iron
DMA	District metered area
DN	Diameter
DS	Distribution system
DWSA	Dessie water and sewerage authority
EPA	Environmental protection agency
ETB	Ethiopian Birr
GEMS	Geospatial engineering modeling system
GI	Galvanized iron
GIS	Geographic information system
GPS	Global Positioning System
HDPE	High density polyethylene
HGL	Hydraulic gridline
ILI	Infrastructure Leakage Index
IMF	International monetary fund
IWA	International Water Association
KWSA	Kombolcha water and sewerage authority
KWSS	Kombolcha Water supply and Sewerage service
MDD	Maximum daily demand
MDF	Maximum day factor
MHD	Maximum hour demand

MNF	Minimum night flow
MOWIE	Ministry of water irrigation and energy
NC	Number of connection
NRW	Non-revenue water
PHD	Peak hour demand
PHF	Peak hour factor
PI	Performance indicator
PRV	Pressure reducing valve
PVC	Polyvinyl chloride
RMSE	Root mean square Error
SCADA	Supervisory Control and Data Acquisition system
TADD	Total average daily demand
UARL	Unavoidable annual real loss
UFW	Unaccounted for water
UPVC	Poly vinyl Chloride un- plasticized
USAID	United state aid
UWD	Unaccounted water demand
WDN	Water distribution network
WDS	Water distribution system
WHO	World health organization
WLA	Water loss analysis

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Chapter one

1 Introduction

1.1 Background

Water is unique to the necessities for the existence of living things in general and human beings in particular. Although human life can exist for many days without food, the absence of water for only a few days has fatal consequences (Senthil & Yaashikaa, 2019). For any municipal town, an efficient water supply distribution system is an essential service for supplying water to people. Without meeting the water supply-demand of the town, the enhancement of developmental activities and improving the health condition of communities is impossible (Brhane, 2019). Water supply system is used for supplying water to different customers. It is a means of collecting, storing, pumping and transporting water through water distribution system (Trifunovic, 2019).

Ghorpade & Vaidya (2018) states the function of water distribution system as, it is a complicated combination of hydraulic control parameters connected together to transmit of water from sources to consumers while simultaneously satisfying demand, pressure, and water quality requirements. It consists of pipes, pumps, junctions, valves, fittings, and storage tanks. Water distribution systems are designed to adequately satisfy the water requirements for a combination of domestic, commercial, industrial, and firefighting purposes (Mehta et al., 2017). A best performing system should provide safe, sufficient and affordable water supply service, with low water loss and good quality of water which fulfills national and international standards (Agunwamba, 2018). The system should be capable of meeting the demands placed on it at all times and at satisfactory performance. However, one-third of urban water supply systems in Africa were not operated continuously (Abebe, 2020). High population growth rate, scarcity of source water, treatment plant size, reservoirs and storage tank capacity, power outages to run water pumps, intermittent pumping, high leakage problems, or some combination of these conditions were the main causes for the loss of performance of water distribution system (Alexandra et al., 2009; Grady et al., 2014). The failure to provide safe drinking water puts public health at risk. (Massoud & Zia, 2018). Depending of the availability of water most utility adapt intermittent pumping system by divide the water distribution networks in the

serving area into several pressure zones through which water is pumped alternatively (Dubasik, 2017). This way of operating the municipal water supply networks will affect the expected performance of the network by affecting the pressure values and the velocities (Dahasahasra, 2007). It also increases pipes breakage rates. The breakage in mains results from oscillating pressures due to providing a large number of homes with a high quantity of water in a short period (Grady et al., 2014)

Jalal (2008) defined Performance of WDS as its ability to deliver a required quantity of water under sufficient pressure and an acceptable level of quality during different normal and abnormal operational situations. The water distribution system performance can be evaluated or measured based on physical (Hydraulic) and chemical (qualitative) characteristics of the supplied water. The physical (Hydraulic) performance can be measured based on quantity of pressure and quantity of outflows in the service life. The quality of water is can be evaluated based on chemical characteristics of the water and its constituents in service life (Tabesh & Doulatkhah, 2006).

Sivakumar & Prasad (2012) states hydraulic performance is the first and most obvious element to address in improving water distribution system. The procedure of designing, building and running a WDS is primarily driven by the need to satisfy a given set of demand points with sufficient flow and pressure. Performance of water distribution system to consumers' satisfaction is the major challenge for water authorities all over the world. Spatially, most of the developing country water distribution systems are frequently interrupted (Grady et al., 2014). The other major factor which affecting water utilities are the considerable difference between the amounts of water produced into the distribution system and the amount of water loss in the systems. The large quantity of water is losses through leaking pipes, joints, valves and fittings of the distribution systems (Shao et al., 2019). According to the USAID (2010) the global volume of non-revenue water (NRW) is staggering. Each year more than 32 billion m³ of treated water are lost through leakage from distribution networks. An additional 16 billion m³ per year are delivered to customers but not invoiced because of theft, poor metering, or corruption.

Hydraulically water distribution modeling helps to predict the performance of the system to solve a wide variety of design and operational problems (Thomas, 2000).

The analysis of water distribution network (WDN) modelling consists of computation of flow and pressure at nodes. It should be designed carefully and maintained properly to reach the consumer (Vaidya Deepali R., 2019). Water distribution system computer models have been in use since the middle 1960s and have evolved into complicated, user-friendly tools that are capable of simulating large distribution systems (Mehta, 2016). There are various modeling software available in the market free and commercially used for designing of water distribution network with appropriate ways (Awe et al., 2019).

WaterGEMS is one of the hydraulic modelling software, which is used for analysis and design of water distribution network and it is the latest software developed by Bentley (Roy et al., 2015). Compared to other software (such as Epanet, water CAD etc). WaterGEMS has strong design algorithm to meet the criteria of accuracy in design of water distribution networks, control of distribution network variables like flow, pressure, and velocity along with their optimization (Sonaje, 2015). It is a super set of water cad software. It provides synchronized database connections, geospatial links, and advanced model-building modules that connect with virtually any digital data format (Rai et al., 2019). It includes Load Builder and Terrain Extractor (TRex) modules to help engineers allocate water demands and node elevations based on geospatial data found in shape files, various types of DEMs, and even CAD drawings. These modules help engineers to avoid manual-input mistake during data entering process (Roy et al., 2015). In addition, this software is easy to use and simple for understanding and data entering. It can integrate with ArcGIS software (Mehta et al., 2017).

Kombolcha town is one of the towns that have been getting potable water supply system since 1963 (Metaferia, 2011). The rapid increase in population, large volume of unaccounted for water due to breakage of the aging pipe lines, economic development and awareness of health benefits of improved water and sanitation cause to rise in water demand and the water shortage. Due to burden on the water distribution system now it is facing many problems like pipe breakage due to aging of infrastructure, low coverage, intermittent water supply system. Those problem in the city affect the hydraulic performance of water distribution system. Previously there is no study conducted for the town related to this. Therefore, analyses of distribution systems using models are a recent

approach to evaluate the performance. So, the aim of this study is to investigate the state of the existing water distribution systems (Kombolcha water distribution system as a case study) and to evaluate the hydraulic performance of the supply network under varying conditions of supply.

1.2 Statement of the problem

In kombolcha town it is observed that consumer's lives at higher elevation area don't get a sufficient water. The water has no power to reach to the top boundary areas. And also, customers live far from the distribution reservoir or source get water intermittently (after other customers are satisfied). In addition to this, there are kebeles in the town which are out of distribution pipe networks. Due to this the people try to get water after walking a long distance and peoples use surface water (Borkena river and its tributary) for different purposes. Based on the information obtained from town Municipality, kombolcha town have been getting potable water supply system since 1963 because of this most of water distribution system components are old without replacements and the pipe expansion was done without considering pressure and flow constraints. This situation leads to increase the amount of leakage and pipe failure through water distribution system.

Those problems of water supply and water loss affects town water distribution system performance related to quantity and quality of water. Those problems initiates to evaluate the hydraulic performance status of town water distribution system. So, this study aimed to analyze of WDS performance using WaterGEMS connect edition software.

1.3 Objectives of the study

General Objective

The general objective of the study is to evaluate the hydraulic performance of kombolcha town water distribution networks using statistical analysis and WaterGEMS software.

Specific Objective

- ✓ To evaluate the existing water supply coverage of the town.
- ✓ To evaluate the water losses in distribution system.
- ✓ To investigate the hydraulic parameters in the existing water distribution system using WaterGEMS software.

1.4 Research Questions

The objectives of the study will be achieved by way of seeking answers to the following questions

- What is the current status of kombolcha town water supply coverage?
- How much water is losses in the water distribution networks of kombolcha town?
- What are the conditions of the actual design and operation the existing water supply system of kombolcha town?
- Does the existing water distribution system fulfil the permissible limit of pressure and velocity?

1.5 Significance of the study

The model output in this study can be used to solve existing problems, analyze proposed operational changes, and prepare for unusual events. By comparing model results with field operations, the operator can determine the cause of problems in the system and formulate solutions that will work correctly the first time, instead of resorting to trail-and-error changes in the actual system, like: low-pressure problems, finding closed valves, and low demand problems.

In general, the research will be significant for town utility to improve the performance of the existing water distribution system and to reduce the amount of non-revenue water. Modeling and analysis practice for secondary distribution line enables an engineer or site supervisor to control a site from practical engineering point of view; enables identification and characterization of the system hydraulically.

It serves as baseline data for any further investigation, town water supply and sewerage office, Government organization, NGO's, and academic purposes. It also may help to mitigate the problem by locating the place where the problem (inadequate pressure and velocity) is taking place.

1.6 Scope of the study

The aim of this research is to present the fundamental concept of hydraulics applied to kombolcha town water supply network, in order for municipal officials of the town to a better evaluation and decision making of water distribution and delivery systems. Therefore, it mainly focuses and will assess to identify the hydraulic performance and water loss of the town water distribution system. This were achieved with hydraulic modeling (WaterGEMS software), water loss analysis. But, due to lack of enough budgets, chemical reagents, and distance of the study area from the laboratory these researches exclude the water quality analysis in the distribution system of the study area and also this research doesn't include extended period calibration.

1.7 Limitation of the study

The main limitation faced during the study was availability of documented data which describe all components of distribution systems. Some of data not organized in the office were

- ✚ location or Coordinate (x, y) of the customer meter.
- ✚ Organized data in computerized system.
- ✚ The utility does not have any recorded data related with average leak flow, number of reported bursts, minimum night flow and average leak duration
- ✚ Time series water consumption data's for 24hour.

Due to the above reason there was some sort of limitation during the preparation of this document. So the analysis was done based on the available data.

Chapter two

2 Literature Review

2.1 Water supply coverage

The coverage of water supply is usually evaluated based on the quality, quantity, paying capacity of the people, distance, etc. It provides a picture of the water supply situation of one specific country or city and it helps to compare one country with others. The percentages of the population with or without piped water connection are a relevant indicator to compare the coverage of water supply in urban areas. Although the water supply coverage, better in urban areas while compared with the rural (Grady et al., 2014).

The actual water supply coverage in cities of developing countries is very low while compared to the developed country (Howard, 2005; Marobhe, 2010). According to the Global Water Supply and Sanitation Assessment 2015 Report, the sub-Saharan Africa country urban Piped water coverage have only 33%. The minimum quantity of 25 liters of potable water per person per day provided at a minimum flow rate. In addition, the supply not interrupted for more than seven days per year (i.e. water should be available 98% of the time) is considered as a basic service for southern African cities' domestic water supply (Kabeto, 2011). In Ethiopia, the minimum quantity of urban water supply is 40 liters per capita/day within a distance up to 0.25km. However, the recommended quantity is different from each other depending on their activities and development (Federal Democratic Republic of Ethiopia, 2015).

2.2 Urban water demand

Water demand is the volume of water requested by users to satisfy their needs. In most developing countries, the theoretical water demand considerably exceeds the actual consumption water use (Asmamaw, 2018). According to Abu-Madi & Trifunovic (2013) water demand was the algebraic sum of the quantity of water utilized by consumer and the amount of water physically lost from the system. Water consumption or water demand is the driving force behind the operation of a water distribution system.

Any location at which water leaves the system can be characterized as a demand on the system (Walski & Chase, 2003). Water demand forecast supplies a basis for operational, tactical and strategic decision-making and can help improve the performance of water

distribution systems (Ristow & Henning, 2021). There are two primary approaches to water demand modeling: deterministic water demand estimation, and stochastic demand estimating. Within the deterministic approach, the actual water demand for all major users is evaluated certainly based on anticipated water consumption over the service time. Be that as it may, stochastic water demand estimating considers questionable changes on water demand over the time and area ranges (Al-Zahrani, 2015).

Demands are described using three water use scenarios such as average daily demand, peak Hour demand and maximum day demand in cubic meter per day (Giustolisi & Walski, 2012; Thomas,2000). The average daily demand is the total annual (average) water demand distributed over 365 days. It obtained by simply summing up the domestic and non-domestic demands as well as unaccounted for water (UFW), The maximum day water demand is considered to meet water consumption changes with seasons and days of the week and peak hour demand is the highest demand of any one hour over the maximum day. It represents the diurnal variations in water demand resulting from the behavioral patterns of the local population. It is obtained by multiplying the maximum day demand with the peak hour factor (Swamee & Sharma, 2008).

Diao & Sitzenfrei (2019) state the variations in water usage for water supply systems typically follow a 24-hour cycle. However, in reality, water demand varies continuously over time and for extended-period simulations, the model requires both baseline demand data and information on how demands vary over time. These demands can be determined by applying a multiplication factor or a peaking factor (Kowalska et al., 2018; Thomas 2000). In Ethiopia most consultants used a Multiplication/ Peaking factors from average day to maximum day tend to range from 1.0 to 1.3, and factors from average day to peak hour are typically between 1.6 and 2.0 (MoWIE, 2006). Of course, these values are system-specific, so they must be determined based on the demand characteristics of the system at hand. Therefore, when a particular junction serves more than one demand type, the total demand for a junction at any given time is equal to the sum of each baseline demand times with its respective pattern multiplier. It is used in most software packages to assign a different pattern to the different components of the composite demand (M. Walski & Chase, 2003)

Water demand Types

Al-Zahrani (2015) stated the number of water demand results from the number of factor types in water consumption varies according to the mode of services, climatic conditions, socio-economic condition, and other related factors. Water demand can be classified as;

1. Domestic water demand

Howard (2005) defines Domestic water demand as; it is the water required in private building for drinking, cooking, bathing, flushing and washing clothes. Domestic water consumption was various according to the living standard to consumer economic status of the community, climatic condition, mode of service, affordability and accessibility of service. Daily per capita water consumption in Ethiopia is generally very low throughout the country. Actual water demand is expected to be greater than present consumption if greater supplies were available to the community (Ministry of water & Energy, 2011)

Climate condition and the socio-economic situation of the area is the main factors that affects water demand of the population under consideration. Therefore, the water demand should be adjusted for those condition. The demand adjustment factors for both situations were given in table 2.1 & 2.2.

Table 2.1: Demand Adjustment for climate condition

Group	Mean annual precipitation(mm)	Factor
A	600 or less	1.10
B	601-900	1.05
C	>901	1

Table 2.2: Adjustment factor for socioeconomic situation

Description	Factors
Towns enjoying high living standard and very high potential development	1.10
Towns having a very high potential for development, but lower living standard at present	1.05
Towns under normal Ethiopia condition	1

Source: (MOWIE, 2006)

2. Non-domestic Demand

Non-domestic demand comprises Industrial, Commercial, and Institutional, Firefighting demands, Unaccounted Water Demand (UWD). Industrial water demand: represents the amount of water demand required by industries and factories in the cities. Commercial and institutional water demand; in addition, those of household consumers, the water requirement of towns includes the need of such commercial and institutional consumers as public schools, clinics, hospitals, offices shops and hotels. Unaccounted water demand: is the amount of water physically lost from the system and (Motiee et al., 2007). According to Ministry of water & Energy (2011) water losses are a function of the quality of construction the type and age of pipes in the distribution network and pressure within the system.

2.3 Water distribution system

Water distribution networks are very important lifeline infrastructure systems, where failures are inevitable. Typical water distribution networks (WDNs) consists of network of pipes, nodes linking the pipes, storage tanks, reservoirs, pumps, additional appurtenances like valves (Walski & Chase, 2003; NRC, 2007). Water distribution systems represent a major portion of the investment in urban infrastructure and a critical component of public work. The main goal is to design water distribution systems to deliver potable water over spatially extensive areas in required quantities and under satisfactory pressures. Therefore, hydraulic models for water distribution networks have become indispensable tools for understanding system behavior by simulating pressures and flows at different locations and times in the networks (Thomas, 2000).

2.3.1 Water supply mode in DS

Water can be supplied to the consumers by the following two systems/operational ways:

Continuous Supply: This is the best system and water is supplied for all 24 hours. This system is possible when there is adequate quantity of water for supply. In this system sample of water is always available for firefighting and due to continuous circulation water always remains fresh. In this system less diameter of pipes is required and rusting of pipes will be less. Losses will be more if there are leakages in the system. Operating of system components, pumps and reservoirs, in the continuous supply systems is a result of consumer's needs: with reduced demand in the night periods, pumps may operate at lower

level and balancing reservoirs may be refilled, whereas during the maximum demand periods, the pumps will operate at their maximum capacity and the reservoirs will supply parts of the distribution network (Jeevan & Mjp, 2012).

Intermittent Supply: Drinking water systems should invariably be operated on continuous pattern for twenty-four hours a day. But due to financial constraints it is not practically possible to operate drinking water systems for twenty-four hours a day in developing countries. Generally, a period of eight hours or less is considered adequate to supply the drinking water (Battermann & Macke, 2001).

In most developing countries water supply is not continuous but intermittent, and this could have been foreseen at the design stage. This has resulted in severe supply, pressure problems in the network and great inequities in the distribution of water. Technically, these problems can be mitigated or eliminated if the intermittent supply conditions are considered while designing the water distribution systems (Vairavamoorthy and Elango, 2002). Because of the low supply rate of water and the intermittent Nature of supply, the demand for water at the nodes in the network are not based on notions of diurnal variations of demand related to the consumer behavior (as with networks in developed countries), but on the maximum quantity of water that can be collected during supply hours (Vairavamoorthy et al., 2001).

The factor that is most related to contamination is duration of supply (Vairavamoorthy et al., 2001). Both continuous and intermittent water distribution systems might suffer from the contaminant intrusion problem, and the intermittent systems were found more vulnerable of contaminant intrusion (Yan, 2001).

The design of water transport and distribution systems under intermittent conditions should not be based on the minimum and maximum peaks of the demand pattern but on the peaks of supply and delivery patterns that are governed by availability of sufficient water at the source, the capacity of the supply system, and the number of supply and delivery hours (Abu-Madi & Trifunovic, 2013).

2.3.2 Factors causing loss of hydraulic integrity in WDN

In most of the developing regions, the design of water distribution systems is based on the assumption of direct supply, although most of these systems are intermittent systems which result in severe supply, insufficient pressure in the distribution system (pressure losses in several areas in the network), inequitable distribution of the available water and very short duration of supply. However, the purpose of hydraulic principle in the water distribution system is to supply water at adequate/acceptable pressure and flow (Dighade et al., 2014). The most common factors for intermittent water supply and loss of hydraulic integrity in the distribution system (Jeevan & Mjp, 2012) are;

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

High pressure during low demand conditions: High pressure during low demand conditions can cause pipe bursting, leakage and large amount of water losses through the distribution networks. Therefore, when dealing with high pressures, PRVs should be used to reduce and regulate pressure in the system (Walski & Chase, 2003). Accordingly, pipes and pumps must be sized to overcome these problems and to provide acceptable pressure in the system.

Pump Capacity: A pump is device in which mechanical energy is applied and transferred to the water as total head, and these head is a function of flow rate through the pump (Walski & Chase, 2003). While, the failure, location, size and capacity of pumps in water distribution are the major impacts for low flow or negative pressure raised in the system, and this can lead to intermittent water supply in the distribution system.

2.4 Water loss in distribution system

Water losses occur in all water distribution networks, even new one and it is only the volume that varies. Thereby, the volume of this losses reflects the capacity of water authorities to manage their distribution networks (AL-Washali et al., 2020). To most water utilities, the level of Non-Revenue Water (NRW) is a key performance indicator of efficiency. The total annual water produced and distributed to the distribution system and the water billed that was aggregated from the individual customer meter readings is used to quantify the total water losses of the town (EPA, 2010). Utility managers should use the water balance to calculate each component and determine where water losses are occurring. By quantifying NRW from the water balance concept, volumes of lost water into system can be calculate and they will then priorities and implement the required policy changes and operational practices which lead to the proper understood and take the required actions (Malcolm Farley et al., 2010).

There are two different approaches to assess water losses. The first is top-down annual water balance and the second is bottom-up approaches based on minimum night flow analysis. In developing countries, mostly top-down water audit is suggested, because of the assessment of real loss of bottom-up approach during the night does not hold the best result since, in intermittent supply system consumer always keeps tap open in the house connection to fill storage tank (AL-Washali et al., 2016).

According to Lambert (2000) the actual quantity of water lost from the water distribution system was varied from utility to utility depending upon local factors such as topography, length of mains, number of connection and standard of service and upon how well the system is being operated and maintained.

Dighade et al (2014) have discussed in analyzing the existing water loss in the distribution pipe network, it is considered appropriate to relate the percentage of losses to both the distribution system and to the percentage of the total pipeline length, which lead to impacts on water quality, flows and pressure, whereas unauthorized connections can lead to contamination through poor or missing backflow prevention.

The general steps to be followed for understanding and managing Losses in Water Distribution networks are Analysis of network characteristics and operating practices, Quantification of water losses, and Use of appropriate tools and mechanisms to suggest appropriate solutions (AL-Washali et al., 2020). Quantifying and characterizing water loss and leakage in a city water supply is by its nature a complex task. Leakage identification needs detailed field investigation sometimes using sophisticated equipment (Sileshi, 2011). In developing countries more than 40% of treated water was lost as non-revenue water (Bogale & Sahilu, 2016). AWWA Leak Detection and Accountability Committee recommended 10% as a benchmark for Water loss. i.e.

- ✚ < 10% Acceptable, monitoring and control,
- ✚ 10-25% Intermediate, could be reduced
- ✚ > 25% Matter of concern, reduction needed (Liemberger & Farley, 2010).

2.4.1 Causes of water loss

Leakage in Water Distribution Systems: Leakage is often a large source of unaccounted for water (UFW). Leakage may be caused for poor management of pressure zones, which result in pipe or pipe-joint failure. Although some leakage may go unnoticed for a long time, detection of visible leakage also required good reporting which also needs a strong public participation (Farley et al., 2010). From one municipality to another and even from one location to another, the causes of leaks will vary depending on the nature of the soil, the quality of construction, the materials used the pressure level and the utilities operating and maintenance practice (Xin et al., 2014).

Pressure in the distribution system: It has proved that pressure is directly proportional to leakage and it is one of the main contributing factors for a leakage, thus depending on the network profile reducing pressure might be an effective method in cost minimization, reduction of leakages and pipe cracks through effective measurement of flows and pressure values which supports to find average pressure (Garmendia et al., 2017; Vicente et al., 2015). As pressure increases, leakage increases and money is then spent to transport and process a product that is wasted. Reducing pressure on the other hand may make existing leaks more difficult to find, because they make less noise, or do not come up to the surface. Therefore, pressure reduction should be coordinated with leakage detection and repaired operations (Farley et al., 2010). To reduce the problems, some water authorities develop a

zoning scheme whereby the complete water distribution network is broken down in to manageable segments that can be easily metered and monitored and analyzed (Desalegn, 2005).

Age of Pipes Network: Pipe age and material are important factors contributing to the burst probability of pipes that as a result cause lots of water loss (Abebe et al., 2021). However, as this information is mostly not available especially for aged pipes, it is usually estimated using the history of the urban development. Newer systems may have as little as 5 percent leakages, while older systems may have 40 percent leakage or higher (Walski & Chase, 2003).

Effects of Corrosion: Corrosion is the problem that is created as water supply pipelines are in continuous contact with soil surrounding it and the water moving through it. The water itself or the surrounding soil may cause problems that will affect the performance and life of the distribution pipes in the system. Therefore, ductile-iron or steel pipelines placed in aggressive soils must be protected by coatings with corrosive resistant materials (Sahilu, 2015).


2.4.2 System of water balance

Findings of strongly suggest that discussion on water losses must be preceded by a clear definition of water balance components (Farley & Liemberger, 2005). The water balance calculation provides a tool for water utility to estimate how much water is being lost due to number of reasons. According to the International Water Association (IWA) represents the best practice water balance with accepted terminology which is illustrated in Table 2.3

Table 2.3: Water Balance Showing Water Losses and NRW Components

‘IWA Best practice’ Water balance terminology				
System input volume	Authorized consumption	Billed Authorized consumption	Billed meter consumption	Revenue water
		Unbilled	Billed non-metered consumption	
		Unbilled Authorized consumption	Unbilled metered consumption	Non-revenue water
			Unbilled non-metered consumption	
	Water losses	Apparent losses	Unauthorized consumption	
			Metering in accuracy	
		Real losses	Leakage on transmission and distribution mains	
			Leakage and over flow from tanks	
			Leakage on service connection up to costumer meters	

According to IWWA the above terminologies are defined as below

 Real or physical losses

Physical losses, sometimes called ‘real losses or ‘leakage’, includes the total volume of water losses minus commercial losses. Real losses are “the annual volumes lost from transmission and distribution system through all types of leaks, bursts and overflows on mains, service reservoirs and service connections up to the point of customer metering”. Real losses are attributed to varying pressure, inefficient leak detection system, poor workmanship and maintenance of the distribution network (Sharma & Bereket, 2008)

A high level of real losses reduces the amount of precious water reaching customers, increase the water utility production costs and water resource since they represent water that is extracted and treated but never reaches beneficial use.

It is recommended as a standard practice to split water losses into real and apparent losses when studying water losses (McKenzie & Seago, 2005). Components of real loss are Leakage on transmission and /or distribution mains, Leakage and Overflows at Utility's Storage Tanks, Leakage on Service Connections up to Point of Customer Metering

✚ Apparent losses

Apparent losses are commercial losses. It includes water that is consumed but not paid for by the user. which are from an improper recording of total water consumed due to meter errors, inaccurate assumption of unmeasured and unauthorized consumption and these deficiencies is attributed to administrative inefficiencies of the water utility (Motiee et al., 2007). Apparent loss may consider as all water that was successfully delivered to the consumer but which is not metered or recorded correctly and these causes an error of consumer consumption (Lambert & Taylor, 2010).

Commercial losses can amount to a higher volume of water than physical losses and often have a greater value, since reducing commercial losses increases revenue, whereas physical losses reduce production costs. Components of apparent loss are Unauthorized Consumption, Customer Metering Inaccuracies and Data Handling Errors. (Xin et al., 2014)

✚ Authorized consumption

Authorized consumption is the volume consumption by the registered consumers; also it is referred as revenue water because of it produce revenue and depending on the country low it can be metered or unmetered for the items of firefighting, flushing of mains, street cleaning, watering of municipal gardens, building water, etc. (Farley et al., 2010).

✚ Unauthorized Consumption

It is that quantity of water which is removed from the system without authorization. Unauthorized consumption includes theft by illegal meter by-passes, vandalism, or un-metered hydrant use for construction or recreation.

✚ Unbilled Metered Consumption

It is the quantity of water that does not generate revenues but which is accounted and not lost from the system. Water used in the treatment process or water provided without charges are examples of these quantities. The Public Water System does not bill a charge for this water.

✚ Unbilled authorized consumption

Unbilled authorized consumption include water used by the utility for operational purposes like water that is used to flush the mains after fixing a break, water provided for free to certain consumer groups, water that is used for street cleaning, firefighting and fire flow tests. Unbilled authorized consumption consists metered and unmetered consumption and it can be assessed using the metering and billing output (Farley et al., 2010).

✚ Revenue Water

It is water that is consumed and for which the utility receives payment. Revenue water consumption volume is measured or estimated. Revenue water includes metered and unmetered billed authorized consumption.

✚ Non-Revenue Water (NRW)

It is water that is not billed and no payment is received. It can be either authorized, or result from apparent and real losses.

2.4.3 Water Losses evaluation

In a developed country water loss have been assessed before and after leakage detection surveying by two different approaches; which are top-down annual water balance and bottom-up approach based on minimum night flow analysis (Dighade et al., 2014).

In developing countries mostly top-down water losses auditing was conducted as per the guide-line suggested by IWA. Due to lack of accurate input data and advanced technology and equipment, it is very difficult to detect real losses in the distribution system as a result; separation of water loss into real and apparent loss was a big challenge for strategic planning (Dighade et al., 2014). The assessment of real loss by the bottom-up approach during the night does not hold the good result because in intermittent water supply system consumer always keep tap open in-house connections to fill storage tank (Al-washali et al., 2020).

The system input volume, billed consumption and unbilled metered authorized consumption are usually metered. In contrast, unbilled authorized unmetered consumption and the apparent losses are estimated (Farley et al., 2010).

The steps to quantify water loss are:

- A. The unbilled authorized consumption (metered and unmetered) is usually a small component and thus typically assumed in the range from 0.5 % to 1.25% (Lambert & Taylor, 2010) of the system input volume or estimated by the utility.
- B. The apparent losses estimation starts with assuming the unauthorized consumption at 0.25 % or 1% of billed authorized consumption as in and (AWWA, 2013).
- C. Afterwards, the customer meter inaccuracies should be estimated according to meter tests at different flow rates representing typical customer water use and meter guidance manuals (AWWA, 2013).
- D. The next step is to estimate systematic data handling errors by exporting and analyzing historic billing data trends for a certain period (Farley et al., 2010). Eventually, the component of apparent losses is estimated by summing its subcomponents.
- E. Calculating Real Losses: The Calculation of real losses in its simplest form is now easy Volume of NRW minus volume of apparent losses and this figure is useful for the start of the analysis in order to get a feeling which magnitude of real losses can be expected. However, it always has to be kept in mind that the water balance might have errors and therefore it is important to verify the real loss figure by one of the following three methodologies (AL-Washali et al., 2016; McKenzie & Seago, 2005).

Minimum Night Flow Analysis (MNF): It is only method that provides valuable actual measurements whose accuracy can be evaluated. It helps a utility to downscale WLA to a level that enables the utility to better manage the network and control leakage. However, MNF requires intensive field work that limits its use for regular assessment or for baseline assessment. MNF needs investment, sophisticated equipment and advance technical awareness of the network components. The accuracy of this method depends on the technical capacity of the utility manpower, the average pressure which is always questionable. MNF analysis is appropriate only for systems with zoned networks and where several DMAs are established, the water balance should be the first option to be used in general (AL-Washali et al., 2016).

Bursts and Background Estimates (BABE): The BABE concept is the only approach that breaks down real losses into sub-components. It allows a utility to better understand the nature and types of leaks in the network. Assessing real losses through BABE factors is questionable. According to AL-Washali et al (2016) the method shouldn't be applicable for utilities with no regular active leakage control such as those in developing countries. WLA shouldn't be conducted through BABE factors unless there is no other option due to its excessive assumptions that lead to underestimating the volume of real losses.

Top-Down Water Balance: The Top-Down Water Balance is neither pressure-dependent nor an extensive-field-work method. It is a cost-effective assessment that should be used first and conducted annually; allowing regular internal and external monitoring of real losses. In this approach, apparent losses are quantified first. Next, real losses are estimated as the calculated residual, i.e., total losses minus apparent losses. In developing countries mostly top-down water losses auditing was conducted as per the guide-line suggested by IWA. Due to lack of accurate input data and advanced technology and equipment, it is very difficult to detect real losses in the distribution system as a result; separation of water loss into real and apparent loss was a big challenge for strategic planning (Dighade et al., 2014).

2.4.4 Performance indicators for water losses

For measuring the changes and evaluate water loss performance indicator (PI) is used test how well the network is managed in relation of comparing the achievement done in the reduction of NRW (Makaya, 2017). There are different measures for assessing the water supply system performance yet they have not been subjected to comparative interpretation over a range of different water utility condition worldwide (Jernigan, 2019). According to Farley & Liemberger (2005), the key elements of performance indicators are investigated and appropriate recommendations have been made in real losses, apparent losses, total losses and non-revenue water by IWA approach.

The most widely used performance indicator in developing countries for water loss performance is Non-Revenue Water (NRW) which is expressed in terms of system input volume. Although, it is an important PI, many practitioners tend to overlook its shortcomings for properly assessing water losses because it is highly dependent on supply time, average operating pressure and level of consumption (Dighade et al., 2014).

The IWA approach of selecting different Performance Indicators (PIs) for different purposes (financial, operational, and water resources) are a clear step forward (see in Table 2.4).

Table 2.4: Selected performance indicators

component	Type	Basic performance indicators (PI)	Detailed (PI)
Non-Revenue water	financial	Volume of NRW as % of system input volume	Value of NRW as % of cost of running system
Real losses	Water resource	Volume of real losses as % of system in put volume	
Real losses		Liters/service connection/day for system with 20 or more service/km main	The infrastructure leakage index defined as the ratio of current annual real losses to the unavoidable annual real loss=CARL/UARL
		Use ^{m3} /km mains/day for systems with fewer than 20 service/km main	
Apparent losses	operational		m ³ /service

❖ **Unavoidable Annual Real Losses**

Real losses cannot be eliminated totally. The lowest technically achievable annual volume of real losses for well-maintained and well-managed systems is known as unavoidable annual real losses (UARL). It represents the minimum level of real losses that could technically be reached, for most utilities it will not be economic to reduce real losses to this level. According to AWWA (2013); Kadu (2015); USAID (2010) the UARL volume is given by equation

$$UARL \left(\frac{l}{d} \right) = (18 * Lm + 0.8 * Nc + 25 * Lp) * p \text{-----} 2.1$$

Where: L_m = length of transmission and distribution system (km)

N_C= number of service connection

L_P= total length of private pipe between the street property boundary and customer meters (km) and P= average pressure in the zone (m)

❖ **Infrastructure Leak Index (ILI)**

This is the new and most advanced real loss performance indicator recommended by IWA and AWWA. However, poor data quality as well as low operating pressures; particularly in developing countries, are often cited as motivation for not using the ILI in which cases the % of system input tends to re-appear (McKenzie & Seago, 2005). ILI is a measure of how well a distribution network is managed for the control of real losses, at a current operating pressure and the value is estimated by dividing the current annual real loss in the water balance (CARL) by the unavoidable annual real loss (UARL) for a system of the same size as the assessed system (Farley & Liemberger, 2005). In developed countries NRW is used as financial indicator and not as an operational indicator, whereas the Infrastructure leakage index (ILI) is used as a technical performance indicator for real losses. (Dighade et al., 2014)

$$ILI = \frac{\text{Current annual real loss(CARL)}}{\text{Unavoidable annual real loss(UARL)}} \text{-----} 2.2$$

2.4.5 Strategy for water loss management in developing countries

In many developing countries the concept of water loss management has received very little attention compared with the severity of the problem of water losses (Dighade et al., 2014; Mutikanga et al., 2010). A problem-solving approach was implemented for water loss management, which is practicable and achievable, can be applied to any water distribution system in developing countries (M. Farley & Liemberger, 2005). Water loss reduction should be the aim of every water utility since it leads to improved economic and ecological efficiency and better service for customers (Fallis et al., 2011). Quantifying the water losses items of both physical and non-physical water losses in the network were to improve the system efficiency to and increase the income of utility (Motiee et al., 2007). There are different components to effective management water loss strategy (USAID, 2010):

- i. **Water auditing:** Water auditing determines the amount of water lost from a water distribution system due to leakage and other reasons. To a quantify water lost from water distribution systems, water utilities should undertake water auditing where the total water supplied was compared with the amount of water billed (Dighade et al., 2014; Lambert & Taylor, 2010).

- ii. **Intervention:** The intervention process puts the options selected in to action. More than one action selected was beneficial to water utilities. Selecting the order of these actions should be based on budget constraint, public benefits and priority of other scheduled capital improvements (EPA, 2010). Intervention can include: Gathering future information, Metering assessment, testing or meter replacement program, Detecting and locating leaks, Repairing or replacing pipe, Operational and maintenance program, Administrative processes or policy changes (ADB, 2013).
- iii. **Upgrading the network:** The aim of upgrading the network was to bring the infrastructure management to a stage where utility practitioners can be reduced losses and improve network performance. One of the fundamental network managements was zoning. The principle of zoning is well established by dividing the network into smaller parts that the utility workers can be understood and more easily analysis pressure and flow profiles and problem areas (Fallis et al., 2011; M. Farley & Liemberger, 2005).
- iv. **Evaluation:** Evaluation is necessary to ensure that whatever the intervention was, it succeeded in its goal. The major portion of the evaluation was benchmarking (EPA, 2010).

2.5 Performance evaluation of urban water distribution system

Performance of a water distribution system can be defined as its ability to deliver a required quantity of water under sufficient pressure and an acceptable level of quality during different normal and abnormal operational situations (Tabesh & Doulatkhah, 2006). The system performance can be classified as physical and chemical characteristics of the supplied water into two primary aspects of quantity and quality (Jalal, 2008).

On the other hand, quantity of supplied water can be measured quantity of pressure and quantity of outflows in the service life. The quality of water is depends on chemical characteristics of the water and its constituents in service life (Jalal, 2008). The other main performance indicators are the amount of water loss in the systems and reliability of the systems within the given service life.

The performance of a WDS depends on efficient and reliable working of all physical components of the system including pipes, pumps, control valves, reservoirs and tanks (Mehta, 2016). Using a computer model are advantageous for assessing the hydraulic parameter behavior of town water distribution network (Haider et al., 2014). Therefore, making hydraulic simulation software, especially from hydraulic point view using engineering approach is one of the methods used for discussion and decision measure on the system, either is the system within level of service based on pressure consideration or not (Awe et al., 2019).

This thesis was focused on the physical characteristics of water distribution systems. The physical characteristics of the system performance can be evaluated by using computational prediction based on simulation models and field measurements. Those systems are used interrelated with each other. Computational prediction based on simulation models to predict the performance of the system to solve a wide variety of design, operational, and water quality problems. For instance, a simulation model can predict the flow rates and pressures within a water system to evaluate a design and compare system performance against design standards. And also, it helps to operational studies to solve problems, such as evaluating storage capacity, investigating control schemes, and finding ways to deliver water under difficult operating scenarios (Bhoyar & Mane, 2017).

The field data are used to calibrating water distribution models. Calibration is the process of comparing the results of the model simulation to actual field data and making corrections and adjustments to the model to achieve close agreement between computer-predicted values and field measurements. Typical comparison values include pressures, flow rate, and reservoir water levels. If the field data and model results are reasonably close, the model is considered calibrated (NRC, 2007). After calibrating the water distribution model, the performance of the system was evaluated by comparing with the water distribution design standard (MoWIE, 2006).

2.5.1 Water distribution system modelling

A model is a tool that can be used to determine the likely response of a system to a given set of stimuli without having to actually impose those stimuli on the system. Today, water distribution modeling is a critical part of designing and operating water distribution systems that are capable of serving communities reliably, efficiently, and safely, both now and in the future(Hurst, 1945; M. Walski & Chase, 2003).

Thomas (2000) has recognized as a water distribution system model is created using a link-node formulation that is governed by two conservation laws, namely mass balance at nodes and energy management round hydraulic nodes. The node is an idea where water drinking is allocated and defined as demand which treated as the nodal hydraulic head can be solved. This design is valid only if the hydraulic pressure at all nodes is acceptable so that the demand is autonomous of pressure

According to Thomas (2000) With today's technology and expedient software packages, we are able to model a system relatively quickly and saves us from the repetitive iterations that determine the flows and pressures. Advantages of Computerized Network Analysis are: -

Feasibility: The most important advantage of computer modeling is that it makes network analysis feasible. Without computerized techniques, analysis is slow and impractical, except for very simple systems. Before computerized network analysis, over design was the common reaction to uncertainty caused by the inability to accurately predict flows and pressures in a system. By providing reliable information, network analysis enables the user to more efficiently analyze, design, and plan water systems (M. Walski & Chase, 2003).

Cost: The cost of computer hardware and software is relatively small compared to the amount of money that can be saved by designing a water system efficiently and accurately. Water facilities are expensive, and the cost savings achieved through proper sizing and installation can range from thousands of dollars to millions of dollars. Efficient operating strategies developed through use of network analysis can also be used to minimize energy costs, thus increasing cost savings (M. Walski & Chase, 2003).

2.5.2 Basic principles of hydraulic modelling

The main reason for modeling a system is to assist designers, managers and planners to explore the governing laws of such systems and accurately analyze their behavior. Hence, models are employed to resolve problems in the systems design and operation (Bogale & Sahilu, 2016). Hydraulic principles water flow in a distribution network satisfies two basic hydraulic principles such as: conservation of mass, and conservation of energy.

Conservation of mass: The principle of conservation of mass dictates that the fluid mass entering through any pipe will be equal to the mass leaving the pipe. In network modeling; all out flows are lumped at the nodes or junctions(M. Walski & Chase, 2003).

Mathematically, the principle can be expressed as follows

$$\sum(Q_i - U_i) - \frac{ds}{dt} = 0 \text{ ----- 2.3}$$

Where, Q_i is the inflow to node in the i -th pipe (m^3/s), U_i is the water used or leaving at the i -th node (m^3/s) and ds/dt is change in storage (m^3).

Law of conservation of energy: According to Bernoulli's equation; the principle of conservation of energy dictates that the difference in energy between two points must be the same regardless of the path that is taken (M. Walski & Chase, 2003). For hydraulic analysis, this principle can be represented in terms of a head as follows:

$$Z_1 + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} + \sum hp = Z_2 + \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + \sum hl + \sum hp \text{ ----- 2.4}$$

Where,

$Z_{1\&2}$ = Elevation at point 1 and 2 respectively (m)

g = Acceleration due to gravity (m/s^2)

$P_{1\&2}$ = Pressure at point 1 and 2 respectively (N/m^2)

hp = pumping head gain (m)

γ = Water specific weight (N/m^3)

hl =Head loss in pipes (m)

$V_{1\&2}$ = Elevation at point 1 and 2 respectively (m/s)

hm =head loss due to minor loss (m)

2.5.3 Modeling a system using Water GEMS

Mehta et al (2017), states that several computer programs are available for modeling, analyzing, performance evaluated under various physical and hydraulic condition including, Epanet, Bentley WaterCAD v8i, and Bentley WaterGEMS v8i for modeling distribution network systems have been designed for each of the pressure zone based on the design top and bottom water levels of the associated reservoirs and the agreed design criteria using the hydraulic modeling computer program.

In recent years the software had a great evolution especially in features such as interoperability, ease of use, productivity tools, connection to external data; consultation processes multi-criteria, operations of spatial analysis, graphics capabilities, integration with Systems Geographic information (GIS) (Awe et al., 2019). Within the most recent developments include the following features like Data Exchange with other Information Systems, Electronic Devices and / or other management programs, Using Genetic Algorithms for automated processes hydraulic calibration, optimal design and energy optimization, Analytical Leakage Detection, Vulnerability Plans to Pollution Events, Systems integration with SCADA, Multi-parameter Quality Analysis, Network Renewal Planning, Integration with Analysis of Hydraulic Transients and Waterfall (Mehta et al., 2017) .

Sonaje (2015) state as WaterGEMS is hydraulic modeling application for water distribution systems design with advanced interoperability, geospatial model building, optimization, and asset management tools. Once the spatial model is built, the parameters that need to be defined for each model components. It includes state-of-the-art genetic algorithm optimization engines for automated calibration, design and rehabilitation, and pump operations. Darwin Calibrator evaluates millions of possible solutions to let users quickly find a calibration hypothesis that best matches measured flows, pressures, and on/off status, empowering users to make reliable decisions based on accurate hydraulic simulation of the real world. Darwin Designer automatically finds maximum benefit or minimum cost

designs and rehabilitation strategies, based on available budget, construction cost, and pressure and velocity constraints (Mehta et al., 2017).

The unique aspect of WaterGEMS is the data presentation mode which is highly appealing and comprehensible and it can be integrated with GIS, SCADA, and Its versatility also allows presentation with variety of graphical tools (Awe et al., 2019). It provides synchronized database connections, geospatial links, and advanced model-building modules that connect with virtually any digital data format. It includes Load Builder and TRex modules to help engineers allocate water demands and node elevations based on geospatial data found in shape files, geodata bases, various types of DEMs, and even CAD drawings. These modules help engineers avoid potential manual-input mistakes (Mehta et al., 2017). Additionally, it has the capability of performing the analysis of the system for the steady state scenarios and for an extended period of any length. The other capabilities of the software are as follows:

- ✓ Evaluate the hydraulics for different demands at a single node with varying time patterns
- ✓ Solve for different frictional head losses using Hazen-William, Darcy-Weisbach or the Chezy -Manning equations (M. Walski & Chase, 2003).
- ✓ Determine fire flow capacities for hydrants
- ✓ Model tanks, including those which are not circular
- ✓ Model various valve operations
- ✓ Provides control-based operations

A research conducted by Mehta (2017) & Rai et al (2019), WaterGEMS software used for the design has the capability to handle various pipe network problems without changing in model of or mathematical formulation. The software used was viable alternative to other methods particularly in view of accuracy and its results in a simpler algorithm, without any iterative process. And also Berhane (2020); Bhojar & Mane (2017) proved that waterGEMS have a better performance for finding optimal pipe diameter to supply adequate quantity of water at satisfactory pressures to the end users.

2.5.4 Water distribution system simulation

Simulation refers to the process of imitating the behavior of one system through the functions another. In our case, the term simulation refers to the process of using a

mathematical representation or real system, called a model. Simulation can be used to predict system responses to under a wide range of conditions without disrupting the actual system, and solutions can be evaluated before time, money, and materials are invested in a real-world project. There are two most basic types of simulations that a model may perform, depending on what the modeler is trying to observe or predict. These are:

- ✚ **Steady State Simulation:** represent a particular view of point in time and are used to determine the operating behavior of a system under static conditions. It computes the hydraulic parameters such as flows, pressures, pump operating characteristics, and others by assuming that demands and boundary conditions were not change with respect to time (Walski & Chase, 2003).
- ✚ **Extended Period Simulation:** is determining the dynamic behavior of a system over a period of time, and it analyze the system on assumption that the hydraulic demands and boundary conditions were change with respect to time. Hence, extended period analysis used to evaluate system performance over time and allows the user to model pressures and flow rates changing, tanks filling and draining, and regulating valves opening and closing throughout the system in response to varying demand conditions and automatic control strategies formulated by the modeler (Thomas M. Walski, 2000).

2.5.5 Calibration and validation

Apaydin (2013) define Calibration as, it is the process of comparing the model results to field observations and, if necessary, adjusting the data describing the system until model-predicted performance reasonably agrees with measured system performance over a wide range of operating conditions. The process of calibration may include changing system demands, fine-tuning the roughness of pipes, altering pump operating characteristics, and adjusting other model attributes that affect simulation results (Lingireddy et al., 2005).

The input degree of calibration required for a model should depend upon the intended use of the model (Lingireddy et al., 2005). Some applications such as design and water quality analysis typically require a high degree of calibration, while other uses, such as master planning, can be performed with a model that has not been calibrated to such a high standard(Engineering Computer Applications Committee, 1999). According to M. Walski & Chase (2003); hydraulic model calibration is the necessary process of modeling and to

have better confidence, understanding and identifying errors made during the model building process.

Calibrating water distribution network models relies upon field measurement data, such as junction pressures, water levels in storage facilities, valve settings, and pump operating status (on/off). Among all the possible field observation data, junction HGL and pipe flows are most often used to evaluate the goodness-of-fit of the model calibration (Berardi & Giustolisi, 2021). Other parameters, such as tank levels, valve settings, and pump operating status/speed are used as boundary conditions that are recorded when collecting a set of calibration observations of junction pressures and pipe flow rates (Berardi & Giustolisi, 2021; Grayman et al., 2006; Jadhao & Gupta, 2018).

Pressure measurement

Collecting pressures data throughout the water distribution system used to indicate the level of service. Pressure readings are done using pressure gauge commonly taken at pump stations, storage tanks, reservoirs, fire hydrants, home faucets, air release and other types of valves (Kowalska et al., 2018). However, different factors can contribute to deviation between model simulation and actual field data. Therefore, calibration can be accomplished by adjusting only internal pipe roughness values or estimates of nodal demands until an agreement between observed and computed pressures and flows is obtained (WALSKI, 2017). The basis for this claim is that unlike pipe lengths, diameters, and tank levels, which are directly measured, pipe roughness values and nodal demands are typically estimated, and thus have room for adjustment (Kowalska et al., 2018; WALSKI, 2017).

This process requires both macro calibration (eliminating large discrepancies) and micro calibration (fine-tuning the model's accuracy) (AWWA, 2012). Macro calibration focuses on large difference between observed and predicted values and helps identify gross model errors. Possible causes of such errors are many but may include: closed and partially closed valve, nodal elevation, inaccurate pump curves or tank telemetry data, incorrect pipe sizes, incorrect pipe lengths, incorrect network geometry (Lingireddy et al., 2005).

After the model results and the field observations are in reasonable agreement, a micro-level model calibration should be performed. As discussed previously, the two parameters adjusted during this final calibration phase will normally include pipe roughness and nodal

demands. The research done by (Abebe, 2020; Chaki, 2017), For calibration of a water distribution system, pipe roughness was considered as the primary calibration parameter. The calibrated model may be used as a functional model to analyze different hydraulic scenarios and water quality analysis (Chaki,2017).

AWWA Engineering and Computer Applications Committee proposed a minimum hydraulic model calibration guidelines specify that for a medium to highly detailed network model (medium to low skeletonization), the following sampling limits should be adopted (Grayman et al., 2006). According to M. Walski & Chase (2003),the number of pressure reading is related to the level of detail as illustrated in the table 2.5.

The goodness-of-fit of model calibration is evaluated by the discrepancy between the model simulated and field measured junction HGL and pipe flow. The goodness-of-fit score is calculated by using minimize the sum of difference squares (Apaydin, 2013).

Table 2.5: Criteria for Hydraulic Network Model Calibration

Intended Use	Level of Detail	Number of pressures reading	Accuracy of pressure reading
Long-range Planning	Low	10%of nodes	±5 Psi for 100% of Readings
Design	moderate to high	5%-2%of nodes	±2 Psi for 90% of Readings
Operations	low to high	10%-2% of nodes	±2 Psi for 90% of Readings
Water Quality	high	2%of nodes	±3Psi for 70% of Readings

2.6 Review of Related works

Evaluating the performance of the system is indirectly estimating the circumstances and rehabilitation needs to ensure continuous and reliable working of all of system components during their entire service life before the occurrence of a failure (Chaki, 2017). Several researches have made to study the performance of water distribution systems, and to reach an optimal solutions and assumptions in order to improve the hydraulic performance and cost effective of the water supply networks.

Zyoud (2003) studied performance of Jenin water distribution systems by using hydraulic modelling software. The researcher design Jenin water distribution system as intermittent and continuous system. The result revealed that the existing water distribution system exposed to unstructured network made of old pipes, Excessive service pressure and insufficient source and the intermittent supply affects the hydraulic performance of the network and exposes it to high values of pressure and velocities.

Mehta et al (2017) studied the operations of Punagam area Water Distribution system in Surat city in terms of pressure Variations from the Treatment Works to the consumer points using WaterGEMS V8i software. He analyzed the pressure at each node, track the flow of water in each pipe and height of the water in each tank during simulation using water GEMS and the simulated water pressure did not vary significantly with the actual values indicating that the pipes still had their hydraulic capacities even though some sections of the network need augmentation.

Agunwamba (2018) evaluated the performance of Wadata sub-zone water distribution system with respect to pressure, velocity, hydraulic head loss and nodal demands using WaterCAD and Epanet. The researcher examined there is no statistical difference between the results of Epanet and WaterCAD. The results revealed that the performance of the water distribution system of Wadata sub-zone under current demand is inefficient.

Adane (2020) conducted to evaluate existing water supply distribution system hydraulic modelling software. The result showed that network exposed to excessive and minimum pressure due to high elevation and low elevation respectively. There is also high amount of water loss. The researcher recommends that to achieve a 15m minimum and 80m maximum pressure, it is necessary to provide pressure controlling valve and establishing

boosting station, and to meet the current and future water demand. Finding for additional water source is required.

Abebe (2020) conducted research titled “Hydraulic Performance of water supply distribution system in case of Aiytyef subsystem (Dessie, Ethiopia)”. His intention was to evaluate the performance of the Aiytyef subsystem in Dessie town water supply networks; using WaterGEMS Software. The study shows that the network is exposed to relatively high and low values of pressure and velocity.

Berhane (2020) studied the optimum design of water distribution networks using WaterGEMS model. The Darwin Designer and scheduler in WaterGEMS was applied for finding optimal pipe diameter to supply adequate quantity of water at satisfactory pressures to the end users and for optimal control and operation of pump systems. The results show that the optimized networks reduce the cost by 9.6% than those of before optimization networks by traditional hydraulic. In addition to this, the optimal tanks filling/emptying arrangement decreased the daily cost of energy consumptions by 12.5% compare as a currently scheduled pump. The researcher suggest application of WaterGEMS models can be used effective tool for least cost design and calibration of WDS.

Hunde & Itefa (2020) studied hydraulic performance of water distribution systems of Addis Ababa Science and Technology University (AASTU). A computerized technique was developed for the analysis and optimal design of water distribution networks. The results show that the network is unmet minimum pressure requirement and the water demand projection showed an increment of 2.5-liter per capita demand (LPCD) in every five years. Finally, the researcher redesigned the network according to design guide line by improving water source, pump capacity and by resizing pipe diameter.

Kuma (2021) conducted a case study was conducted ‘Evaluating the hydraulic performance of water distribution system of Tulu Bolo town’. The hydraulic model of water distribution network was developed using GIS integrated with WaterGEMS hydraulic model. According to the analysis, about 92.6% of nodes have optimized pressure ranged between 15m to 70m and about 1.27% is under permissible pressure. Model calibration was performed by comparing simulated data with field data, the result of pressure calibration has a linear correlation coefficient of 0.93 and the hydraulic model in WaterGEMS was

calibrated and optimized with a field data. The researcher recommends that for reinforcing the water network geospatial databases like mapping household water connection and its relational class between the parcels and water customers.

Bahre (2021) studied the hydraulic modeling of the Aksum town water distribution system which is located in the Central Tigray region of Ethiopia. The objective was to evaluate the hydraulic performance of the water supply distribution system by assessing the situation of the existing water supply distribution system. The researcher used WaterGEMS V8i software to model the water distribution system. The study shows that the network is exposed to relatively high and low values of pressure and velocity. Which have effects on the performance of the network. The model performance measures were checked based on the coefficient of determination. The researcher improved the network by using pressure reducing valve and by upgrading pipe diameter.

Ebsa (2021) & Gudeta (2017) studied the Hydraulic performance Analysis of water supply distribution network by using WaterGEMS software. The water distribution system has been analyzed for steady state and extended period simulation for the present population scenario. From analysis result, the observed problems in the system were aged pipes, undersized pipes, low pressures and low velocity. This problem has been solved by replacing the aged pipes with new one and using the design criteria of velocity and pressure for undersized pipes, low pressure and low velocity.

Chapter 3

3 Methodology

3.1 Description of the Study Area

The study was conducted at Kombolcha town which is located in the eastern part of Amhara region in South Wollo Administrative Zone at 11°06' north latitude and 39°45' east longitude. It exists at a distance of 23km from Dessie town (zonal capital), 503 km from the regional capital Bahir Dar and 378 km from the Ethiopian capital city Addis Ababa. The altitude of town varies from 1500-2300m above sea level meters. Figure 3.1 shows the location of Kombolcha city.

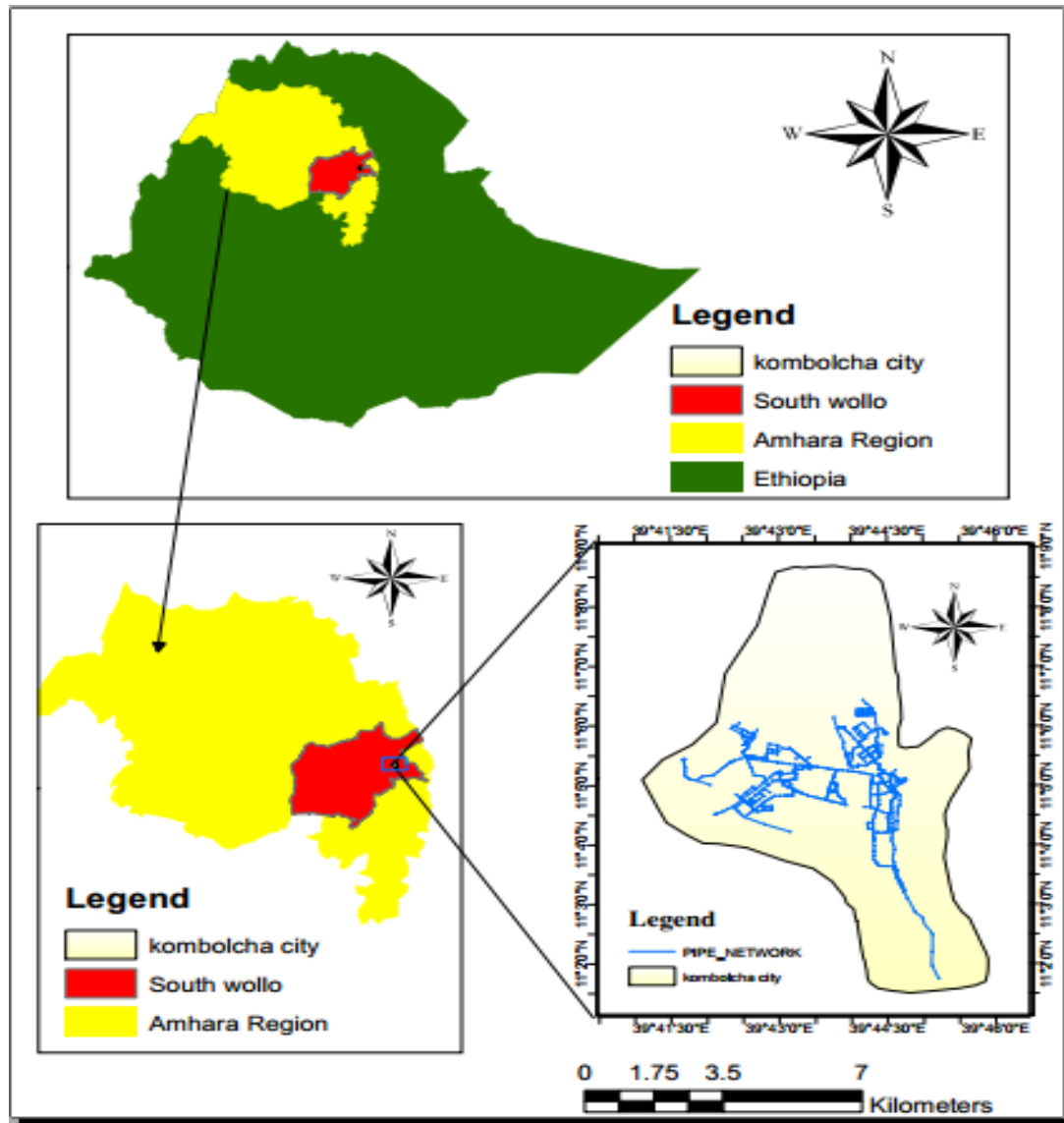


Figure 3.1: Location of the study area

The mean monthly temperature varies between 16.1°C (in December) to 22.1°C (in June) and the mean annual rainfall in the area is 1248mm. The eastern and western part of the town is blocked by mountains and the southeast of the town is flat and swampy. There is also a river known as Borkena crossing to the center of the town flowing from north to south. Considering Ethiopian CSA population projection estimation (CSA, 2013), the current total population of the town is 134,599 with annual growth rate of 4%.

Existing water supply system of Kombolcha town.

According to the information the town started getting water supply in 1963 with three deep well (belonging to the Dewa area). Which are located north of the town. Before the piped water supply installation, the people of the town used Borkena and Berberi rivers as water sources. Because of population growth and people raised the question of clean water service the city municipality office started to expand and prove the water supply system services which include construction of boreholes, reservoirs, distribution lines.

During the year between 1963 and 2015 three water supply projects have been implemented that the drilling of additional eight bore holes in three well fields (Dewa Meda, shasheber and birarao well fields). All wells have single submersible pumps on each well. The total boreholes yield during construction discharge 303 lit/sec but the currently total discharge 155lit/sec with all eleven boreholes with 16hr average pumping rate but their present yield has exhibited a reduction as shown on Annex B.

There are seven circular type storage reservoir /tank with the total capacity of 2950m³. All storage reservoirs are put on elevated ground surface to maintain enough pressure in the downstream pipe network to provide a good flow to the area being supplied, and to enable the water to be raised up to the top of buildings. The existing storage reservoirs history and locations are summarized in Annex D.

Transmission pipe of kombolcha town is independent and separate from the distribution network. Thus, there is no any connection for the customers from this transmission line. WDS of the town was examined through a typically branched and looped network made of a mixture of DCI, HDPE, PVC and GI pipe materials with sizes range from DN 40mm to DN 315mm starting from borehole to every consumer with a total estimated length of distribution lines is covered about 67.5km.

As indicated in figure 3.3, the major pipe in the distribution system of kombolcha town is HDPE with the coverage of distance in length is 54.38%. Where the other pipes in the distribution system are Galvanize iron, ductile iron and polyvinyl chloride which covers in the distributions system a smaller percentage 24.56%, 1.6% and 19.45% respectively.

Water distribution system components are including; Reservoirs, Tanks, Pipes and Pumps as match as possible and the resulting sketch fairly represent the actual water network. Figure 3.2 illustrates the layout of water distribution system for study area.

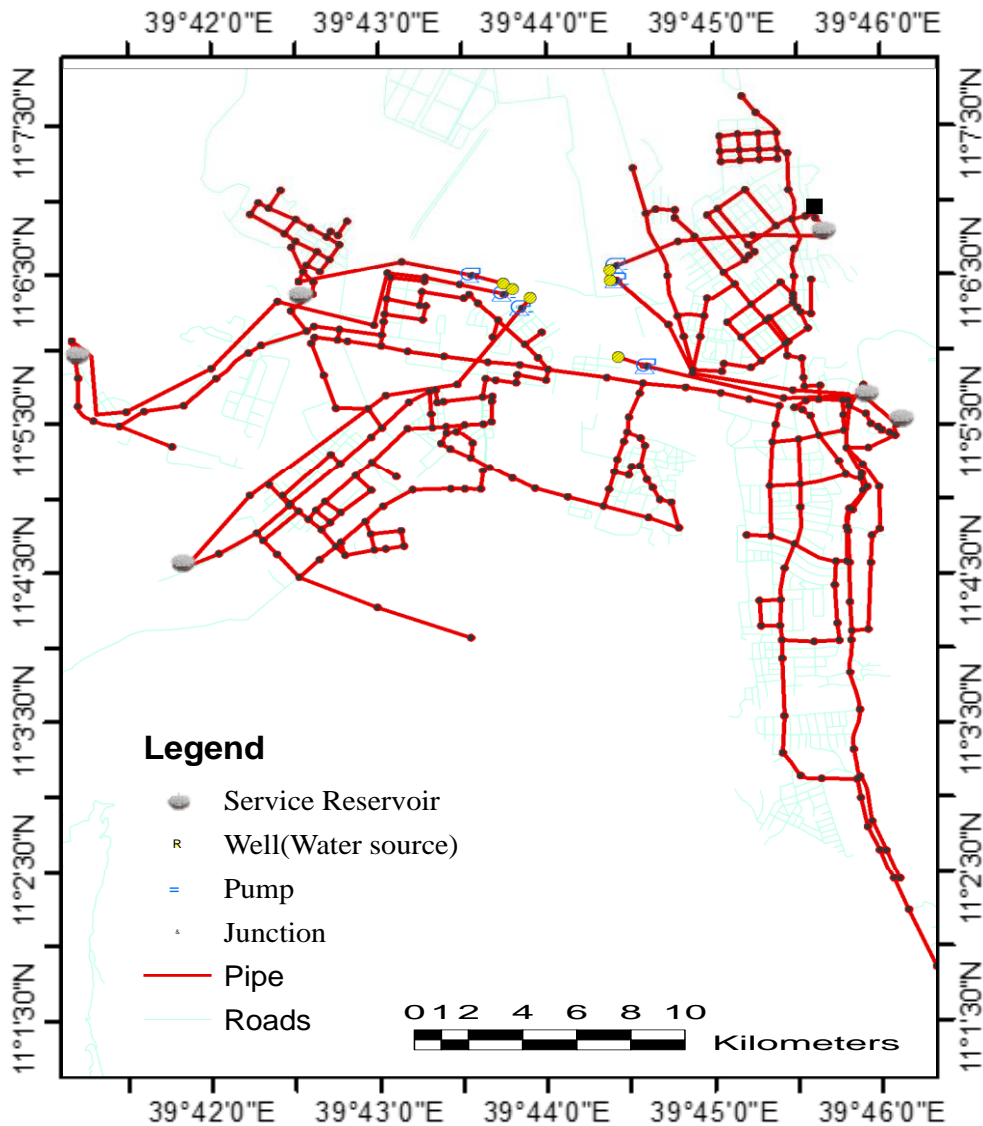


Figure 3.2: Components of kombolcha water distribution network

3.2 Materials

This research was mainly conducted to evaluate existing water supply distribution system performance in kombolcha town. To achieve the goal of the research the Equipment's that were used were, pressure gauge with a size of 1/2" or 15mm to measure pressure at selected points, together with, GPS instrument was used to collect the required elevation data during pressure reading to identify the exact location of the sample point's water distribution modeling for calibration purpose.

Software's like: WaterGEMS CONNECT Edition, Arc GIS version 10.4.1, Global Mapper v20, AutoCAD 2011 and Microsoft Excel 2013 were used to collect, organize and analyze the data. WaterGEMS CONNECT Edition Software used in modeling and analyzing of hydraulic modeling application of water distribution systems. It provides an easy-to use environment for engineers to analyze, design, and optimize water distribution systems. Figure 3.3 shows the work space of waterGEMS software. Arc GIS version 10.4.1, was used to display the overlapped shape file of the distribution network on the topographic map of the town and also used for delineation of the study area. GIS allows mapping, modelling, querying, analyzing and displaying large quantities of such diverse data. It can create maps in different scales, projections and color. Global Mapper v20 was used to check the elevation of the junction, storage tank and water source. While, Microsoft Excel sheet was used to analyze water supply coverage, to organize elevation data, to calculate nodal base water demand.

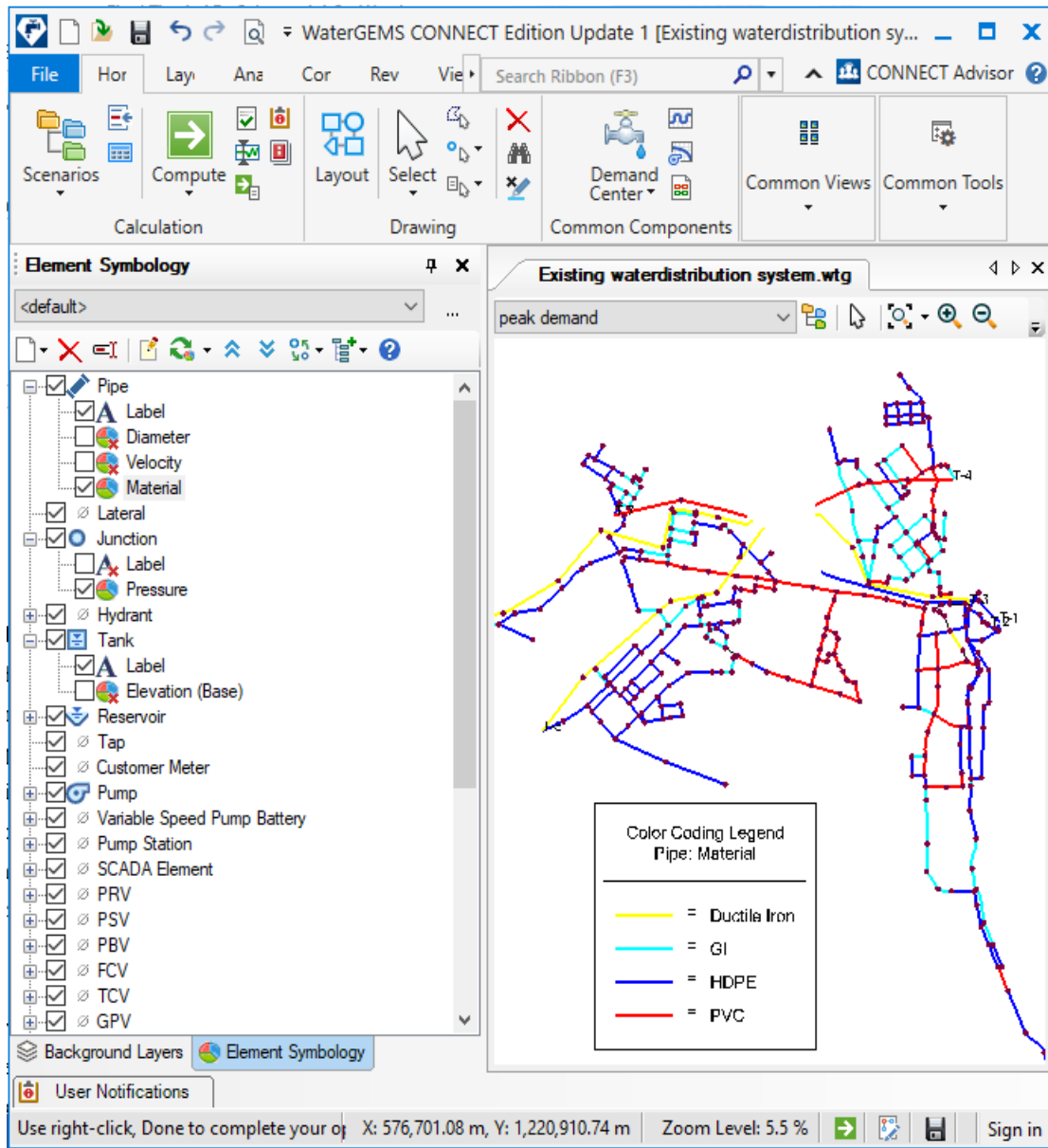


Figure 3.3: WaterGEMS connect edition overview with town pipe material description

3.3 Data collection

The data collection was based on Primary and secondary data collection methods.

Primary data was collected through field survey (include field measurement and direct observation). Measurement of junction elevation using hand GPS, flow thorough pipe line and junction pressure by using pressure gage ware done in April 2021 at higher and lower elevation points for model calibration process. A field visit to the city of kombolcha was conducted on the March & April 2021 to collect additional relevant data.

While, secondary data were collected from different literature reviews, town water supply service office existing documents and, Amara design and supervision work office (ADSW) design reports. Major Information and data gathered from ADSW were System Map/layout of WDN in AutoCAD format and diurnal curve. System map is the most useful documents for gaining an overall understanding of a water distribution system because they illustrate Pipe alignment, connectivity, material, diameter, the locations of other system components, such as tanks and reservoirs. Diurnal curve is a type of pattern that describes changes in demand for a daily cycle. It indicates the variation in water consumption over a 24-hours and it can be developed by measuring the discharge from the outlet of service reservoir. The data collected from kombocha town administration office were town population data with growth rate, topographic map and cadastral map. Cadastral map includes information on buildings, parcels and blocks for each Kebeles which are located in study area. The data was updated in 2019 and it has the recent documents of the town and it is used for demand allocation purpose. From topographic map, contour lines with an interval of two meter & a scale of 1: 50,000 have been collected. Water source/Reservoir type (detail is present in Annex B), storage reservoir (elevations, capacity diameter and height) detail present in Annex D, Water production and consumption Records from 2015 to 2020 (detail describe in Annex F) and node elevation data were collected from kombolcha town water service office.

3.4 Data Analysis

3.4.1 Water Supply Coverage and Water loss analysis

To analyze the data which is collected from different sources, quantitative methods was used. From the quantitative methods, the descriptive statistical methods, percentage and graphs were used in order to come up with the appropriate result. Use Micro soft excel to analyze the data obtained from the office.

A. Water Supply Coverage

In this study effort is made to evaluate the coverage related to the quantity of the supply and level of connection that are related to the performance. The number of domestic connections per family and the average daily per capita consumption is used to analyze the water supply coverage for the entire study area.

But before calculating supply coverage the town population should be projected to current year by using Exponential increase method. In this method percentage error is the lowest value (minimum value) than other methods and this method is good for fast growing town or city like kombolcha town. The population growth rate was taken from Amhara water, mineral and energy office which is 4% and the base population used for projection of population of the town is the 2012 population data (i.e. 95,395 at the time of Town Water Supply design).

Then, the Average daily per capital consumption of the town was calculated by dividing annual water consumption recorded data with total projected population. According to kombolcha town water supply system billed data, the total numbers of connections or water meters within the study area were about 20,766 and According to CSA (2019) average family size of 4.5 was used for calculating the average number of connections per family.

B. Water loss

The total water loss in the town water supply distribution system was evaluated based on the percentage of Nonrevenue Water, Water loss per pipe length, water loss per number of connections.

Percentage by input volume is still recommended as a basic financial PI for non-revenue water (Malcolm Farley et al., 2010). The data obtained from the town water service office, the yearly (2015 to 2020) and monthly water production and consumption (billed water

volume) in the system was identified as shown in Annex F. Using this data total percentage of water loss in the system was calculated using equation 3.1.

$$\text{System Input} = \text{Authorized Use} + \text{Water Losses} \dots\dots\dots 3.1$$

According to AWWA (2013), NRW consists of the sum of Unbilled Metered and Unbilled Unmetered Consumption, Apparent Losses, and Real Losses. The components are not identified or it is not broken down as it requires detail system data and sophisticated equipment's for field measurements. During field visit from March 15 up to April 12/2021, it was observed that the utility does not have any recorded data related with average leak flow, number of reported bursts and average leak duration. Due to this physical or apparent loss in the main was assessed base on the available data, and it was adopted by considering the minimum achievable annual physical losses (unavoidable annual real loss) in the system (Dighade et al., 2014; Usaid, 2010).

It estimates a volume of leaks that are undetectable or would be uneconomical to repair during the year. (Liemberger & Farley, 2010). It is a useful concept as it can be used to predict, with reasonable reliability, the lowest technically annual real losses for any combination of mains length, number of connections, customer meter location and average operating pressure. According to AWWA (2013) the UARL volume is given by equation 2.1.

As per data collected from town water service office, and during the year 2020/21 the town water system covers; the Total main distribution pipe length of the system was 65.7 Km, the Number of registered service connections or water meters were 20,766 and the water utility has laid an average length of 10m (0.01km) private pipe from distribution Line to customer boundary depending on the field measured average value. Therefore, for this work, the total length of private pipe was taken by multiplying the number of service connections and the average length of private pipe, and it was used 207.6 Km. Finally, the Average pressure was taken from the simulated pressure value of 24-hour duration and the average value of 54.05 m was used.

3.4.2 Hydraulic Performance Analysis

The field survey data for distribution system was evaluated by using the engineering software WaterGEMS. The method of analysis was based on nodal pressure and velocity parameters. During data analysis, nodal pressure and pipe link velocity will have determined to identify higher or lower pressure zone of the area.

WaterGems software was used for the purpose of calculating pressure for customer's demand, velocity, and head loss. The modeling process was model building, data entry, model test and problem analysis.

The first step for model water distribution system using WaterGEMS software is importing of all the existing data at once detail at section 3.4.2.1. After importing water distribution network next step is entering of all relevant data like pipe characteristics (material, diameter, length and roughness coefficient), junction (elevation and demand), pump elevation, tank and reservoir elevation data. Except junction demand and elevation of some boundary elements (like pumps, reservoirs and tank), All of those data were collected from town water service office. Junction/node demand allocation steps are presented in portion 3.4.2.2 and undefined elevation of nodes or boundary elements have been directly measured by using hand geographical positioning system.

Then run the model with EPS and steady state simulation. The model was performed in steady state run for the average daily demand, which is the Demand at every node not changing throughout 24 hours of a day and the system condition has been computed over 24 hours with a specified time increment of one hour and starting model run at time 12:00 PM. The software simulates dynamic state hydraulic calculation based on mass and energy conservation principle. However, for the analysis the peak and minimum hour demand have been simulated to identify the current performance of the system related to system parameter like pressure and velocity. The model has been performed 12:00 PM to 5: 00 AM for minimum hour consumption and 6:00 AM to 8:00 AM for the peak hour consumption. It is noted that minimum hour model run has been made at 1:00PM from starting time and peak hour model has been made at 7:00AM from the starting.

Finally, Evaluate the model performance by performing calibration processes of the resulting model, the detail is described in portion 3.4.2.3.

3.4.2.1 Creating the Model in WaterGEMS

One very good feature that WaterGEMS offers is the Model Builder. Using this tool one can directly import all the ArcGIS shape files, Excel files and AutoCAD files at once to create a well-defined system. This enable to import system map directly from other files without drawing each component as shown in figure 3.4.

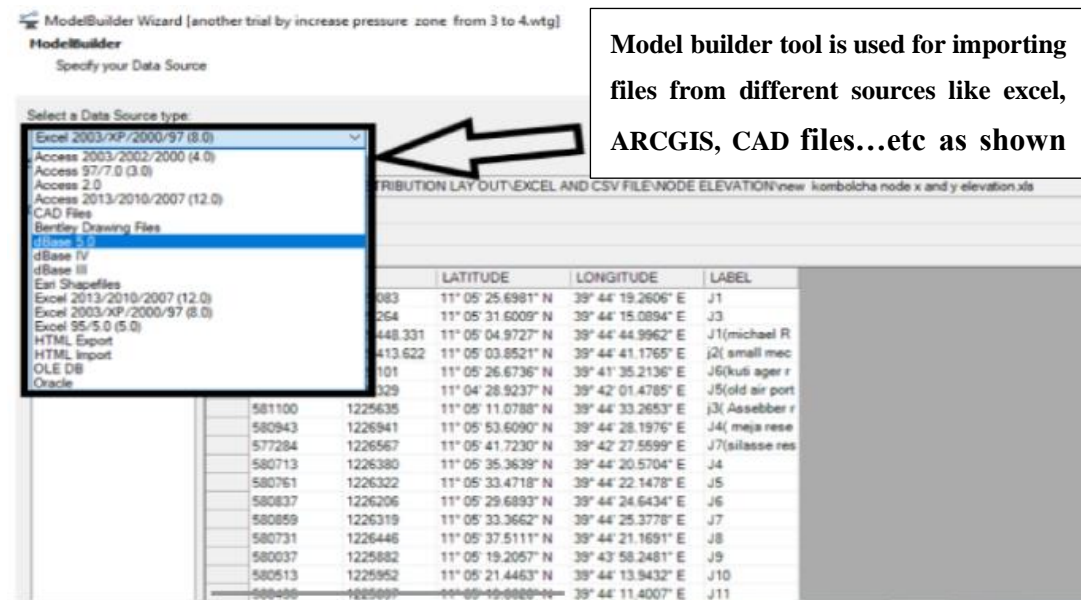


Figure 3.4: Data entering using model builder tool in Water gems software

3.4.2.2 Demand allocation

The first procedure after the setting up of all the required information is allocation of estimated base demand to each node using categories of residential and non-residential consumption and population coverage in each junction.

Allocation of calculated base demands to the nodes in the network though WaterGEMS hydraulic model is consisting of several methods (i.e. point load data, area load data, and Land use data). The method used in assigning demands through point load data needs detailed household survey in order to find the exact number of georeferenced billing meter data, nearest node and pipe within a service polygon. In area load data method, the corresponding area is influenced by the node within the given network.

From the methods of assigning demands by area load data, the method through proportional distribution by population practically applicable to most of the study areas where the demands to the nodes in the network are assigned by creating the Thiessen polygons to the

corresponding nodes. The last method in assigning demands through land use data methods the demands are calculated by land use and population, and assigned to nodes in the network based on land use classification. Hence, point load data and land use data were not used. Because there is no data like georeferenced billing meter data, nearest node and pipe, land use classification in the study area.

So, for this study depends on the quantity and quality of collected data, Water demand was allocated using proportional distribution by population method. Calculated Base demand for each node is present in Annex A. The steps to assign base demand to each supply node are:

i. Identify the number of houses around each node

To know the number of houses for each node, first create the service area for each node by Thiessen polygon method in waterGEMs software and export this to Arc Map software, the created Thiessen polygons help to identify the exact pressure zone boundaries. Topographic map was collected from kombolcha town administration office. Then in Arc Map, the cadastral map shape file was opened and the shape file of the distribution network was overlapped on it. The Thiessen polygon shape file was added. From this, the number of houses inside each polygon were physically counted from the overlapped cadastral map and assigned to node inside polygon. Figure 3.5 shows the overlapped shape files of Thiessen poly gone, distribution layout and cadastral maps to identify number of house served from each node.

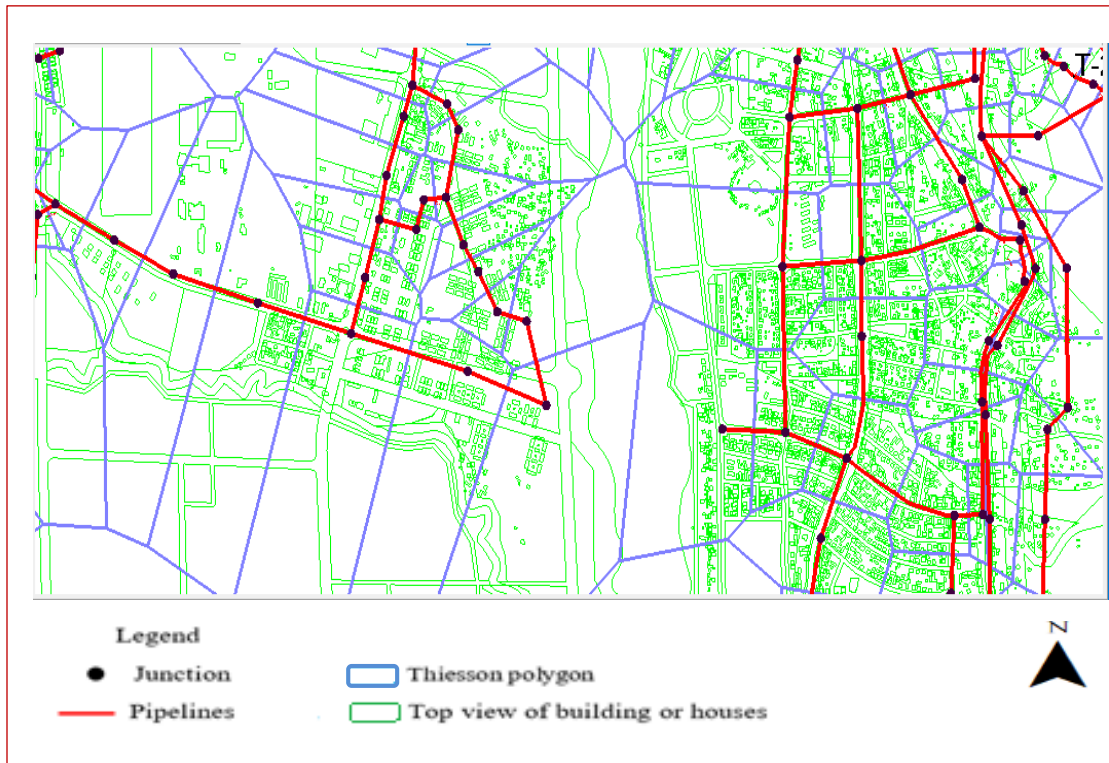


Figure 3.5: Partial view of service area for each node together with houses

ii. Calculate the number of peoples served from each supply node

The average number of a person in each house (person per housing unit) was obtained from the revised design report of the town. Number of people can be calculated by multiplying average number of people in each house with number of houses for node.

iii. Calculate base water demand for each node

Water demand refers to all water required by the system including domestic and non-domestic demands.

Domestic demand can be estimated by multiplying per-capita demand with projected population. Climate condition and the socio-economic situation of the area is the main factors that affects water demand of the population under consideration. Therefore, the water demand should be adjusted for those condition. According to table 2.1, Kombolcha having a mean annual precipitation of 1248mm, with a climate adjustment factor of 1.00, and by using table 2.2 towns having a very high potential for development, but lower living standard at present, therefore the socio-economic adjustment factor is 1.05, Then adjusted domestic demand can be calculated by multiplying domestic demand with climatic and socio-economic factors.

Estimation of non-domestic demand needs a detail study of past, present and proposed public, commercial, institutional and industrial establishments in the town. This technique, however, desires well organized evidence recording of the type and scale of water demands of various sectors. However, there's no well-organized information indicating the kind, scale and water consumption pattern of various sectors found within the town.

Because of this, the quantity of nondomestic demand is expressed as a proportion of domestic demand the value is fixed after the forecasting of population for identified the category of the town on CSA. The non-domestic water demand for each town was calculated by multiplying the domestic water demand with the non-domestic water demand percentage. According to MoWIE (2006), estimated domestic water demand is 30% for institutional and commercial demand, 10% for industrial water demand and add NRW percentage.

The average daily demand is the total annual (average) water demand distributed over 365 days. It obtained by simply summing up the domestic and non-domestic demands as well as unaccounted for water (UFW) and other formula is multiply per capita demand by total population of the system. Once the average daily water demand of the system was determined, to calculate base water demand for the particular supply node, divide population served from node by total population of the town, then multiply the value with average daily demand (Bhadbhade, 2009).

The population figures, which have been adopted in this model, depend on the statistics of the figures derived from the number of house connections multiplied with the average household size and estimated as 93,447. By using the town cadastral map, the total number of houses identified were 20766. Finally, by following the listed procedures the base water demand was assigned to all nodes in the excel sheet as shown Annex A and imported to waterGEMS by using model builder tool.

Water demand in a distribution system fluctuates over time. This variation in demand over time can be modeled using demand pattern. Figure 3.6 shows the diurnal curve of Kombolcha town, which was gained from Amhara water works and design office, and was developed by measuring primary data from supplying discharge in the outlet of the service reservoir.

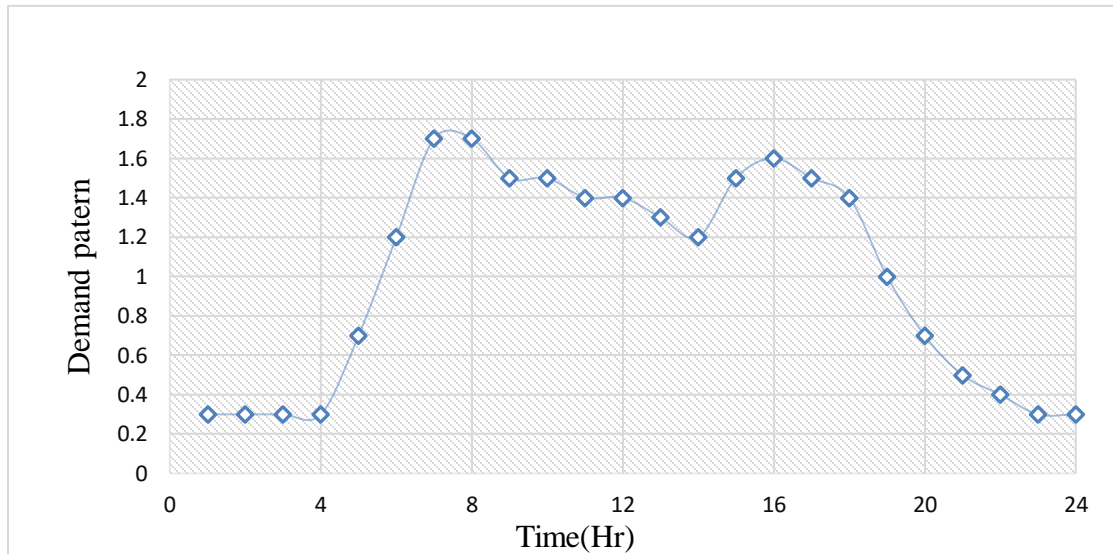


Figure 3.6: Variation of domestic water demand during the day (ADSWE, 2011)

3.4.2.3 Model calibration and validation

Model calibration was determined based on the results of model pressure and measured pressure in the selected nodes (Jadhao & Gupta, 2018). Collection of pressure at various selected locations within the physical network was mandatory for calibration.

To achieve this, limiting sampling criteria for the constructed model was developed as indicated in the sample size and the requisite field test points selected.

Pressure Measurement

The Pressure readings were taken with a pressure gauge in Kombolcha town on April 2021, at high and low-pressure zone of the selected points in distribution network at end user taps. All sampling points were selected after the computed model was simulated, knowing the pressure variation area and after reconnaissance of the town water distribution Network.

🚦 Sample size

Typical network representation of a water network may include hundreds or thousands of links and nodes.

However, only a small percentage of representative sample measurements can be made available for the use of model calibration and validation due to the limited financial and labor requirements for data collection. Based on the intended use, for this study, the study area water supply network has already existed. In this case prefer intended use have redesign and operational purpose. The entire number of junctions in the network system of kombolcha town is 334 junctions, Hence as per (AWWA, 2012; Thomas M. Walski, 2000), 5% (16 samples) have been taken for the calibration & validation purpose during the peak (7:00AM to 9:00AM) and minimum (3:00AM to 5:00AM) hour demand respectively.

Sampling location

Sites should be spread throughout the study area and should reflect a variety of situations of interest, such as local lines, areas farthest to the tank, and areas under the influence of tanks, different types of pipes, low and high elevation areas. In addition, sampling taps should be placed close to mains (AWWA, 2012). Based on this Pressure sample was taken from customer connection direct from the main line from higher elevation, lower elevation, and high and low pressure areas. Figure 3.7 shows the sampling location of pressure measurement in kombolcha town distribution system.

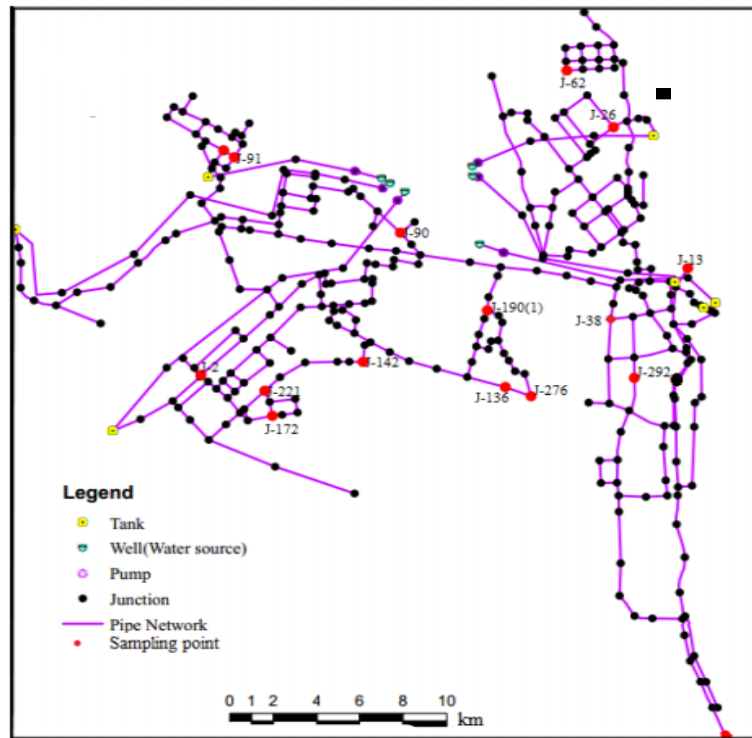


Figure 3.7: Sample location for pressure measurement

Darwin calibration method

WaterGEMS Darwin calibrator tool was used to calibrate the network. Darwin Calibrator evaluates millions of possible solutions to return the best possible calibration hypothesis (Bentley, 2006). It uses optimization methods search for a solution describing the unknown calibration parameters that minimizes an objective function, while simultaneously satisfying constraints that describe the feasible solution. The objective function was minimizing the sum of the squares of differences (RMSE), between observed and model-predicted values.

After constricting of the hydraulic model inserted the recorded field data to the Darwin calibrator and deciding the adjustment groups in the most important issues for automatic calibration of the study area system. According to (Apaydin, 2013) pipes are grouped based on their diameter, materials, age and velocities. In this cause, the pipe was grouped depends on their materials and diameters.

In most cases of the steady state, calibration will be more sensitive to changes in pipe roughness while the extended period calibration will be more sensitive to changes in the distribution of demands (Walski, 2017). For this study, performing a steady state calibration of the model parameters have been adjusted to match pressures associated with field observation. The model result Hazen William's coefficient (C factor) is adjusted. The pipe roughness coefficients for the group of pipes by the same amount to minimize the summation of the sum of square of difference between the measured and simulated values of the pressure. The initial estimation of pipe roughness coefficient values obtained from literature values as shown in Annex E. This coefficient is used to calculate frictional loss in pipe line using Hazen-Williams Equation (Thomas M. Walski, 2000).

The acceptable level of accuracy was determined based on the AWWA published guidelines as indicated in the literature review Table 2.5. The calibration process was performed by adjusting pipe roughness coefficient until it has become within the acceptable limit of 90% of field test measurements (it should be within $\pm 1.5m$ or $\pm 5m$). After completed the calibration process for study area, the consistency or performance of the model was then evaluated by validating it with test data obtained under different conditions Using the minimum demand field measurements of pressure data.

Figure 3.8 shows the steps of overall research work (flow chart) for the study.

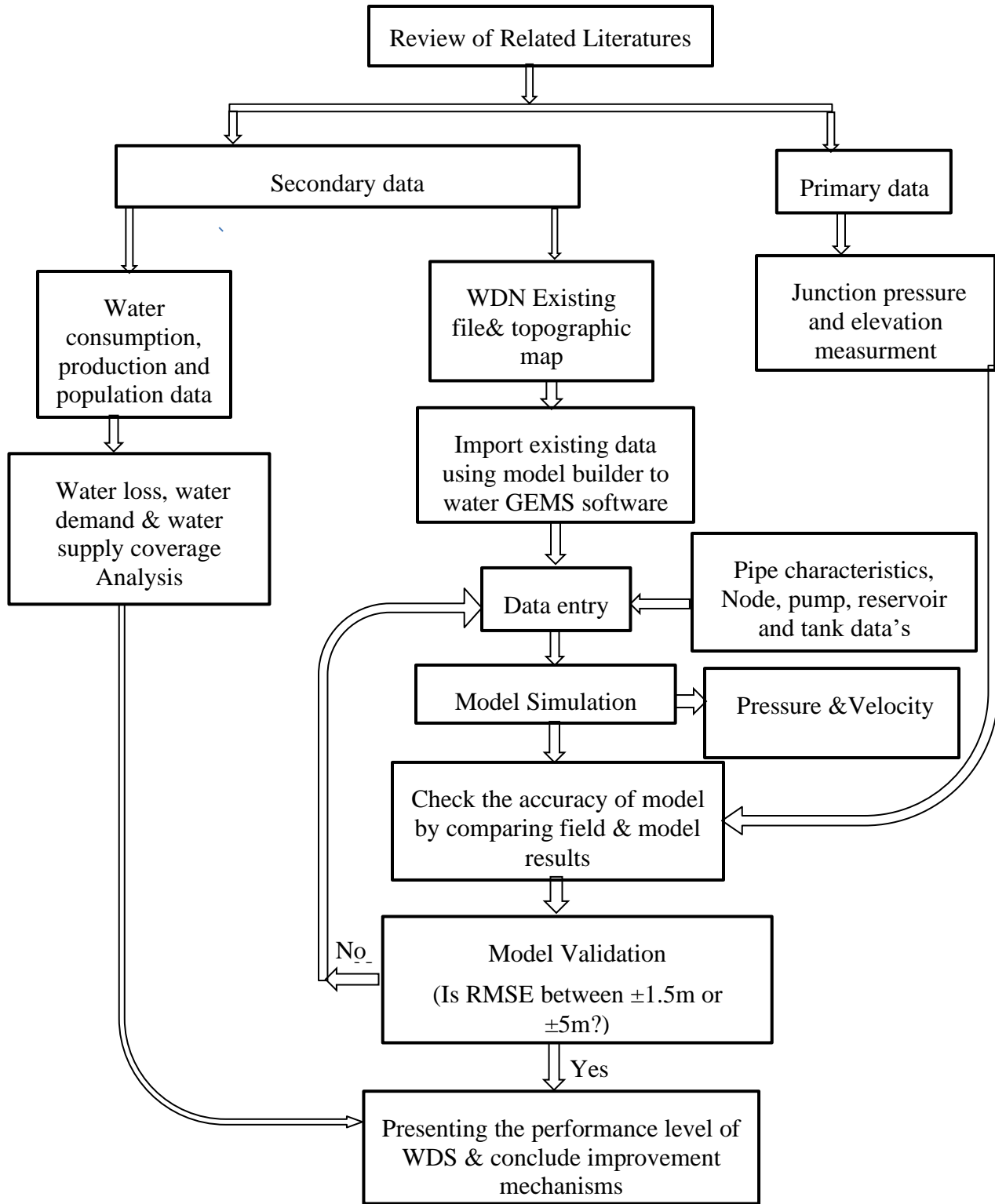


Figure 3.8: Flow chart for overall methodology

Chapter Four

4 Results and Discussion

4.1 Water Supply Coverage

According to the water supply service of kombolcha town, the existing design has the per capita demand that had been adopted as 102 l/c/d with the population of 95,395 in 2012. However, With the current projected population of 134,599 and water consumption data of 2,450,167m³ for 2020/21, the per capita consumption is 50 l/c/d. According to the WHO (2011), the minimum optimal water consumption rate per person per day is given as 100 liters. Regarding to this value, the water consumption of the town was satisfying only 50% of the standard value. And also, it doesn't satisfy MoWIE (2006) recommended value of 70l/c/d. This indicates the present domestic per capita consumption in Kombolcha town is low showing that the demand is suppressed due to inadequacy of the existing water supply.

The calculated total average daily demand for kombolcha town was 11,980 m³/d in 2020/21 the detail is in Annex C, but the existing water production is 10,063.4 m³/d (116.5l/s). This shows that the estimated demands and production of the existing sources are not feet. Additional water quantity is required in the system to balance the system supply and demand gaps.

The percentage population of kombolcha that has access to safe water from the total population has been estimated by using total town active connection (i.e. 20,766) with relative to the total population of the town it was 69.4%. In other words, the average in-house or yard connection of the town was 69%. The remaining people use traditional and often unprotected sources.

4.2 Water Loss

Analyzing water loss and leakage needs a detail data due to its complex nature. The data are usually scarce in developing countries that the case of Kombolcha town is also similar. The six year's actual production and consumption figures obtained from the kombolcha town water supply and sanitation service were used to calculate water loss as presented in figure 4.1 by using equation 3.1.

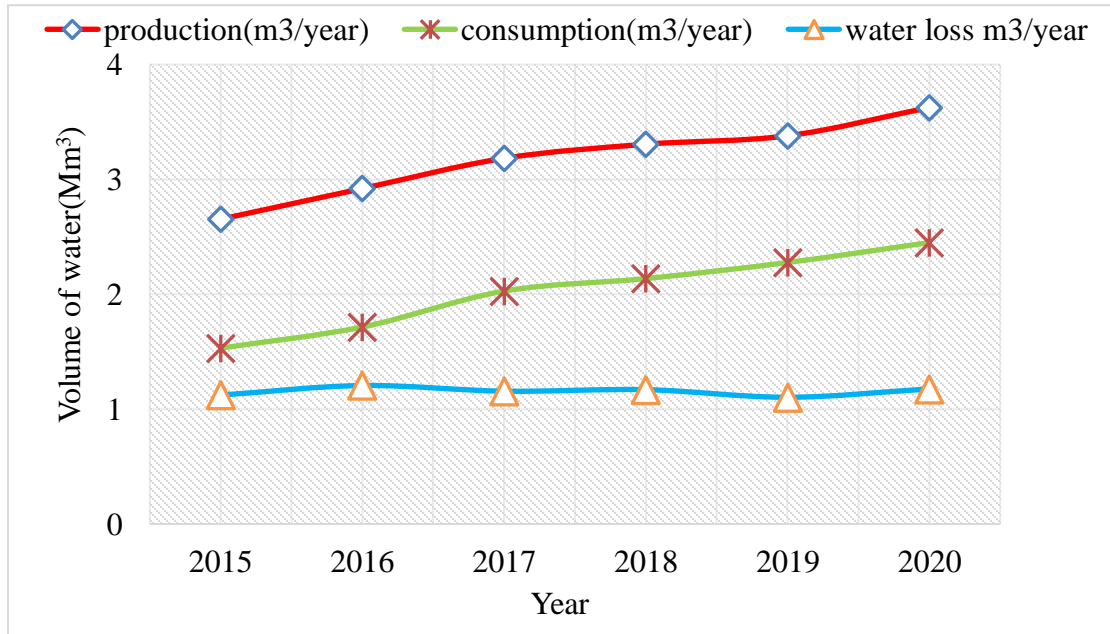


Figure 4.1: Water production, consumption and water loss of the town

The research done by A. kebede (2015), the water loss result was 26% of total production, But as shown in figure 4.1 the annual water distributed to the system in a year 2020/21 was 3,625,836 cubic meters and annual water loss was 1,175,669 cubic meters which account to 32.4% of the total production. This indicates the amount of loss trend increased year to year. That means the utility doesn't take action for water loss reduction strategy.

According to MoWIE (2006) classification and descriptions of water losses as acceptable, if the loss is <15%, Thus, an average water loss in kombolcha town was 32.4%, which is greater than the standard value, that showing a matter of concern, the reduction is needed.

The average amount of water which is actually reached to the consumers was only 67.7% of the total production. According to AWWA (2013) the water supply system efficiency is acceptable if above 75% of water produced reaches the consumer Based on this, the

efficiency of study area water supply system is not acceptable because of only 67.7% of the total production reaches the consumer which is below 75%. This High levels of water loss seriously affect the financial viability of water utilities through lost revenues and increased operational costs.

Water loss expressed as the length of the main pipe

According to the town water utility, the total length of water distribution line including both main and privet pipe line (from property boundary to customer's meter) were estimated about 273.3 km. The water loss per kilometer length of main pipe is 11785.6l/km/day. As per revised literature of EPA (2010) the general rule for water loss level guide line for water loss level in distribution 'the average condition of water loss as per pipe length is (10,000 - 18,000)liter/km/day, and bad condition of system is >18,000 liter/km/day.' Therefore, this figure shown that if distribution line is expanded, real loss also increases in the pipe network.

Water loss expressed as per number of service connection

The total number of service connection of kombolcha town was 20,766 which were obtained from town water utility. The water loss per number of service connection is 155l/c/d. According to Malcolm Farley et al. (2010) performance indicator physical losses target matrix: kombolcha town is water loss per number of connections were found in good condition system which is less than < 250 litter/connection /day.

Quantifying the Components of Non-Revenue Water

Non-revenue water is water that is not billed and no payment is received. It is the sum of unbilled authorized consumption and total water loss (apparent and real losses). Total loss is the sum of real loss (avoidable and unavoidable loss) and apparent loss. However, for this study unable to compute total real loss due to limitation of leakage data average leakage flow, number of pipe burst, number of water loss due to customer meter inaccuracy, illegal connection and etc. instead of this unavoidable real loss is computed by using equation 3.2. The unavoidable real loss (UARL) for kombolcha town is 453491.31 m³/year. So, Total apparent losses plus avoidable real loss in the system was determined from the town water balance as:

$$\text{Apparent loss plus avoidable real loss} = \text{Total water loss} - \text{UARL}$$

From the above description the apparent loss and avoidable real loss were large in volume which is 722,177.7 m³/year and it covers 20% of total volume of water production. Also, it covers 61.43% of total water loss in study area.

As shown in figure 4.2 the proportion indicates that the huge amount of water loss is leakage due to the presence of high average pressure in the study area. This shows that the distribution system of study area is more susceptible for burst and leakage. The unbilled authorized consumption (metered and unmetered) is usually a small Component and thus typically assumed in the range from 0.5 % to 1.25% of the system input volume (AWWA, 2013). So for kombolcha city this volume of water is 18129.18 m³/year. This volume of water is considered as non- revenue water.

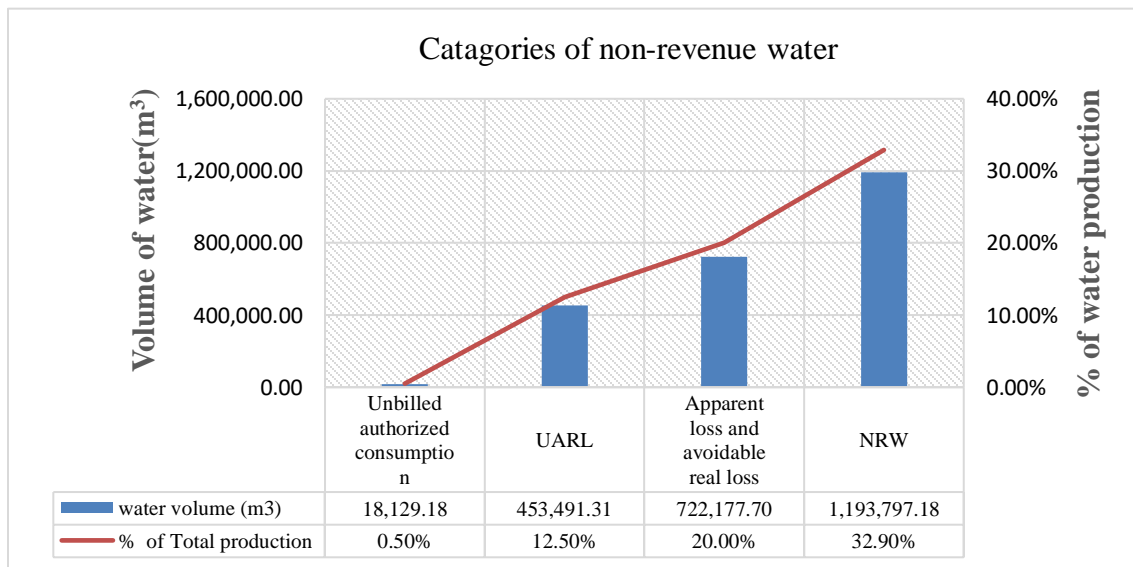


Figure 4.2:Category of water loss during year 2020

Quantifying loss by Water Balance Method

To estimate the water loss by using water balance method for Kombolcha town in the year 2020/21 based on international water association (IWA) the water balance components are obtained by using the available data and estimated in the above. The results are summarized in table 4.1.

Table 4.1: Water balance (m³) for fiscal year 2020/21

System input volume 3,625,836	Authorized consumption 2468296.18 (68.1%)	Billed Authorized Consumption 2,450,167 (67.6%)	Billed metered consumption 2,450,167 (67.6%)	Revenue water 2,450,167 (67.6%)
			Billed unmetered consumption (0.0%)	
		Unbilled authorized consumption 18129.18 (0.5%)	Unbilled metered and un-metered consumption 18129.18 (0.5%)	Non-Revenue Water 1,193,797.18 (32.9%)
	Water loss 1,175,668 (32.42%)	Apparent loss plus avoidable real loss 824595.9 (22.74%)	Unavoidable real loss 351072.1 (9.68%)	

As shown on the above table 4.1, the results of Non-Revenue water by water balance method high levels of NRW (33% of System Input Volume) and water losses (32.42% of System Input Volume) have serious impact on kombolcha water supply service office's finances and available water resources. The average tariff of water in the kombolcha town is 10 ETB /m³, based on this average water tariff value the water utility lost 11.5 million ETB for every year.

4.3 Hydraulic model Results

4.3.1 Calibration and validation Results

A. Calibration

For check model performance pressure data was measured at customer's home faucet. The size of the water main in the study model integrates a size greater than 2.5 inch. Because of this it was difficult to take a measurement at a direct connection to the water main nodes, due to the size of pressure gauge available. So, the head loss between the supply main and the site where a pressure is measured had been considered. The head loss included the elevation head difference between two corresponding locations and pipe friction loss as shown in table 4.2.

Table 4.2: Locations of samples of a supply main node and the corresponding home faucet

No	junction (ID)	sample node elevation		head loss between two locations
		software	during field measurement	
1	J-13	1,921.25	1924.3	-3.00
2	J-26	1,853.22	1855.2	-2.02
3	J-62	1,845.00	1847	-2.00
4	J-190(1)	1,830.60	1830.7	-0.09
5	J-221	1,887.90	1891.4	-3.46
6	J-263	1,893.24	1892.1	1.14
7	J-276	1,821.00	1821.7	-0.74
8	J-292	1,835.40	1835	0.40

The Relation between measured pressure and simulated pressure in the model was described below in table 4.3. According to AWWA (2012) the observed pressure and simulated pressure difference was out of the minimum permissible difference values ($\pm 1.5\text{m}$) for design and operations evaluations. As a result of this, calibration of the existing simulated systems was done depends on the observed value by using Darwin calibrator in WaterGEMS.

Table 4.3: Simulated and observed pressure at the sample node

Junction (ID)	Time	Simulated pressure	Measured pressure at customer tap	Head lose between two location(m)	Correct observed pressure at supply main	Pressure difference error(m)
J-13	(8:00:00 AM) Peak demand	50.84	56	-3.00	53	-2.16
J-26		39.13	40	-2.02	37.98	1.15
J-62		10.04	20	-2.00	18	-7.96
J-190(1)		17.49	16	-0.09	15.91	1.58
J-221		5.05	12	-3.46	8.54	-3.49
J-263		45.96	46	1.14	47.14	-1.18
J-276		20.02	27	-0.74	26.26	-6.24
J-292		53.78	45	0.40	45.4	-8.38

As shown in the table 4.3 the correlation of observed and simulated hydraulic grade is poor in J-62, J-292, J-221 and J-276, in this case there in no pipe roughness adjustment. The roughness value taken from design document for city. Initial C value was taken from literature (140,130,140 and 110 for HDPE, DI, PVC and GI pipes respectively).

Then by using Darwin calibrator to find the optimal solution by adjusting C-factors. To reduce the difference error between observed and simulated value the C-factor of HDPE pipe increased to 150 and DI, PVC and GI pipes reduced to 110,130 and 66 respectively. This indicates that ductile and galvanized iron pipes are old pipe because the internal roughness was increase and hazen-Williams's value is decrease due to the corrosion effect.

As shown in Figure 4.3 and Annex G, the RMSE and coefficient ($R^2 = 0.98$) values were within the limit. According to AWWA (2012) the liner regression relationship of pressure which showed a typical $R^2 > 0.5$ and difference error is between $\pm 1.5m$ or $\pm 5m$ is considered acceptable level of model performance.

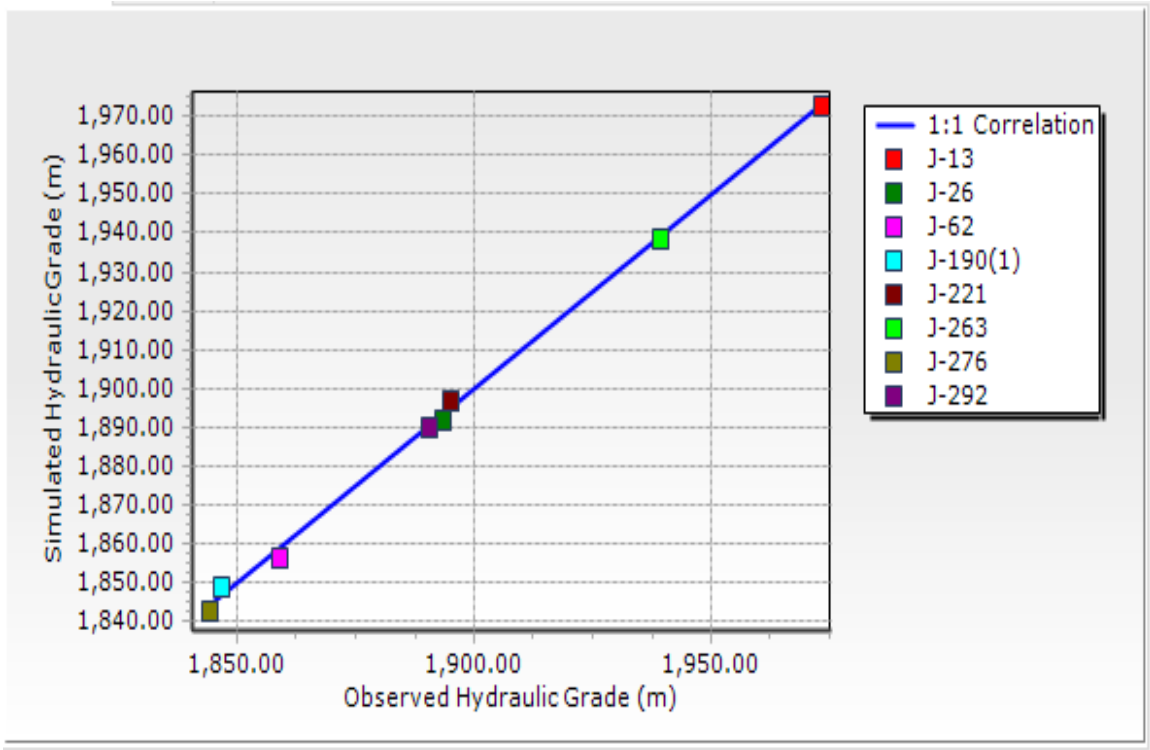


Figure 4.3: Correlation of observed and simulated HGL new optimized run-1

B. Validation

Model validation is the steps that follows calibration and uses an independent observed data set to verify that the model is well calibrated (AWWA, 2012). In the validation step, the calibrated model is run under conditions differing from those used for calibration and the results compared to field data. If the model results closely approximate the field results (visually) for an appropriate time period, the calibrated model is considered to be validated (Thomas M. Walski, 2000). Eight samples have been taken at minimum demand scenario to verify the model is well calibrated. The validation result shown in Figure 4.4.

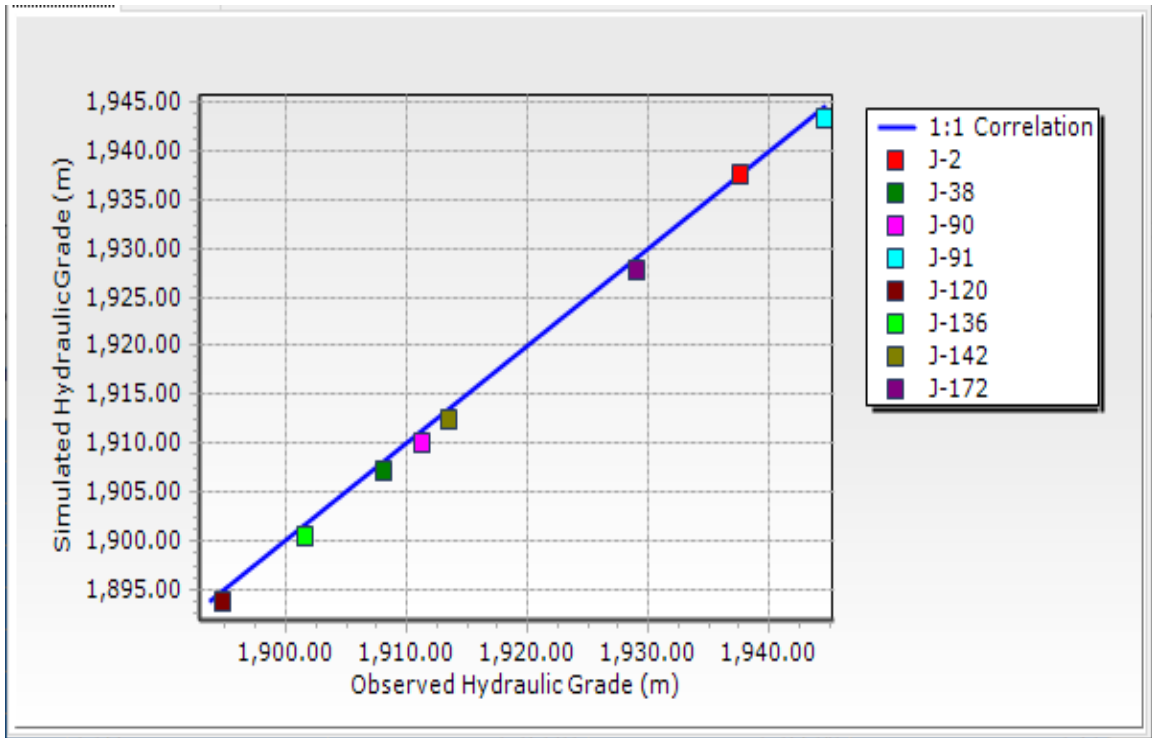


Figure 4.4: correlation between observed and simulated pressure parameters

As shown in the Figure 4.4 & Annex H, the simulated and measured hydraulic grade correlated With each other, which indicates that less error variance between simulated and observed Pressure in the system (within an average error between $\pm 1.5\text{m}$). Hence, the model is acceptable validated.

4.3.2 Pressure and velocity

The WDS kombolcha town includes 401 pipes of different materials, 334 junctions, 6 pumps, 6 water source reservoirs, and 7 tanks as shown in Figure 3.2. After the model was calibrated, the current water production in the existing WDS was evaluated under consideration of the required water patterns in the study area.

The WDN of town was classified using a color coding, which area is high, medium, and small-pressure and velocity parts. With concerning current simulation, the result for pressure using the estimated average daily demand, during peak hour consumption and minimum consumption is summarized in Table 4.4 and detail in Annex I.

Table 4.4: Pressure and velocity variation in different scenarios

Existing distribution system Hydraulic parameters (%)							
No	Scenarios	Junction Pressure(mH ₂ O)			Pipe Velocity(m/s)		
		P<15	15<P<60	P>60	V<0.6	0.6<V<2	P>2
1	Avg.day demand	2.4	74.6	23.1	8.2	91.3	0.5
2	low hour demand	1.5	31.1	67.4	83.5	13.5	3
3	peak hour demand	23.1	64.1	12.9	43.4	46.6	10

1) Pressure

The Ethiopian urban water supply guideline criteria for the minimum and maximum operating pressure value in the distribution network were 15 and 60 m respectively (MoWIE, 2006).

The pressure was low during day time with the increasing of the water demand of the customers and high during night hours when the demand is low. Figure 4.5 shows the variation of pressure in different junctions is caused by changing the demand of recurrent starts. It illustrates the demand and pressure at selected nodes. This is because, the increasing of losses and demand in the distribution system will decrease the values of pressure.

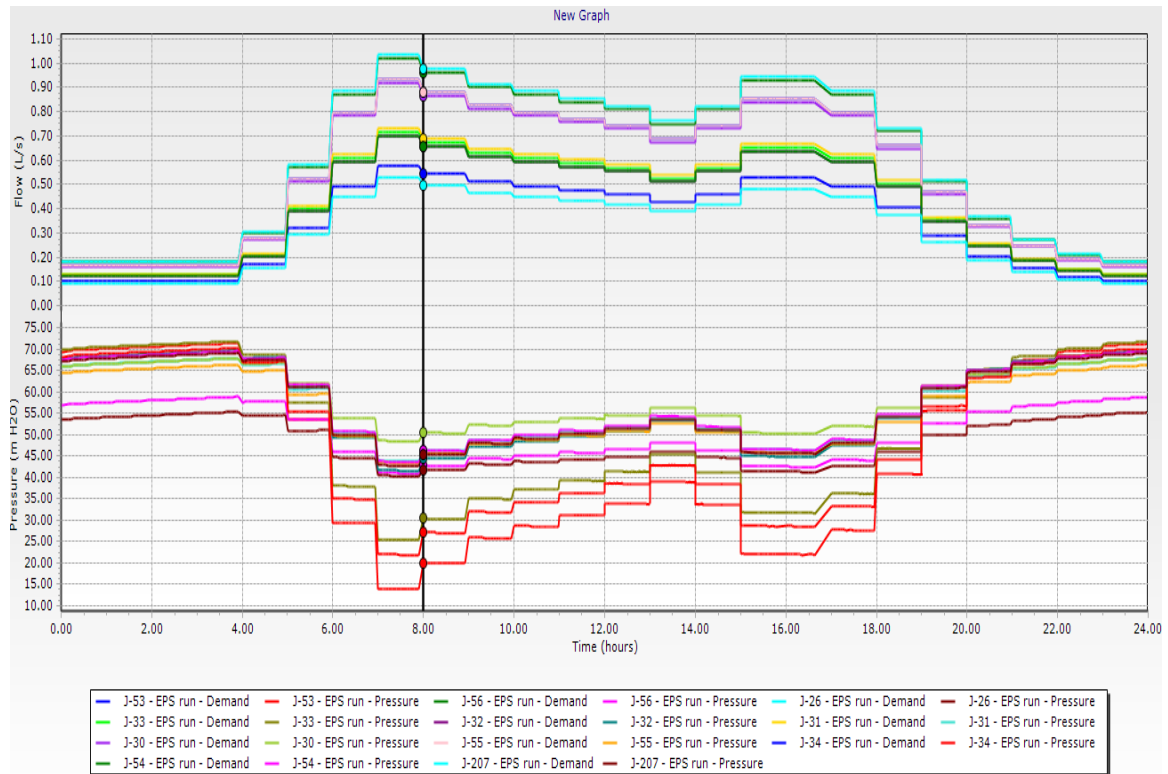


Figure 4.5: Pressure and demand relationship for the selected Junctions

i. **Pressure at minimum consumption hour**

High pressure during low demand conditions can cause pipe overflowing, leakage and large amount of water loss through the distribution network. As per model analysis, maximum water pressure in the transmission main and distribution line was found at the Downstream of the town where the elevation in the area is low around Broken River and university area. As showed in Table 4.4, Figure 4.6 and detailed in Annex I, 67.4% of nodes are liable to extremely high pressure. And 31.1% of nodes are received the water of optimum pressure at low consumption hour.

One of the main causes of high values of pressure in the system is the wide range of levels between the sources of water (reservoirs) and the consumption nodes, also the absence of pressure reducing valves. These high values of pressure may affect adversely the hydraulic Performance of the water distribution network. Also they produce high velocities, which accelerate the deterioration and corrosion of the pipes in the distribution system (M. Farley & Liemberger, 2005). Generally, reducing excess pressures in distribution system reduces flow rates from existing leaks and extends infrastructure working life.

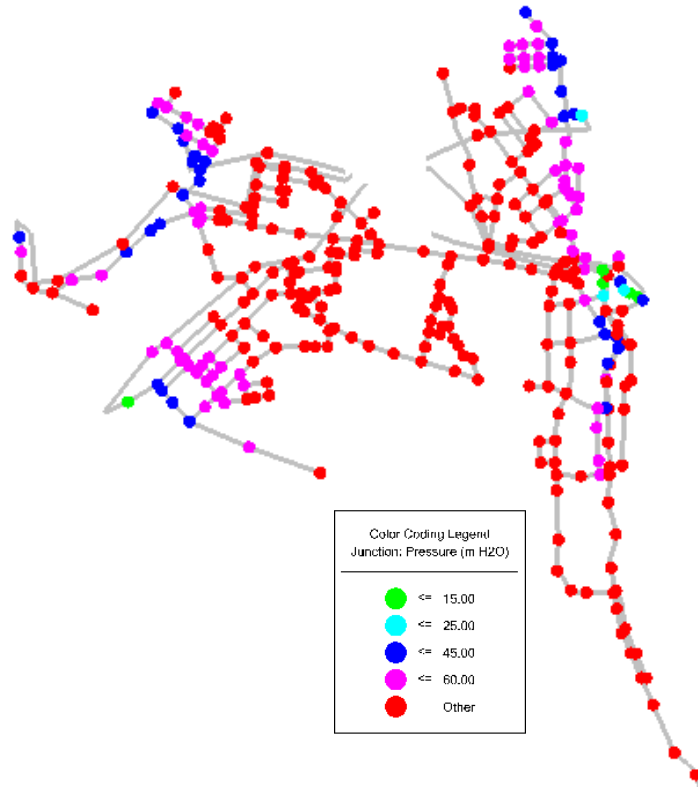


Figure 4.6: Junction Pressure map of the water distribution system at MHD

ii. Pressure at peak hour demand

As indicated in table 4.4 and figure 4.9, during peak demand 23.1% of nodes were below minimum acceptable pressures (15 m H₂O). This occur around old airport and Tebase in the study area due to the increments of water demands during this scenario the detailed description is put in Annex I. And also there are 12.9% of nodes exceeded the maximum allowable pressures of 60 m H₂O. While 64.1 % of nodes were in the permissible pressure ranges of minimum and maximum pressure. Pressure on this peak hour is the most important for design and improvement and expansion of the existing system, updating and installation of new water supply distribution schemes. Households located on higher elevations relative to supply point and far away from source site gets water at low water pressure. The water users located in higher elevation relative to supply points, get less water and they fetch after users located in lower elevation are satisfied. Effects of distance and elevation in pressure distribution of selected nodes are shown in Figure 4.7 and 4.8.

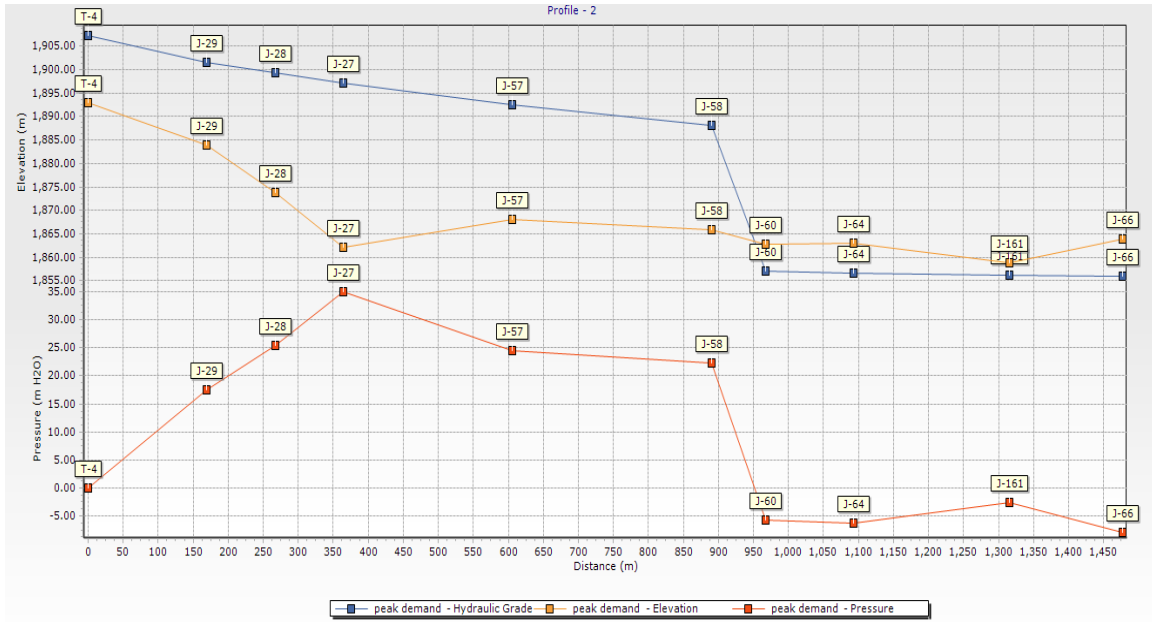


Figure 4.7: The effects of elevation difference and distance from source on pressure

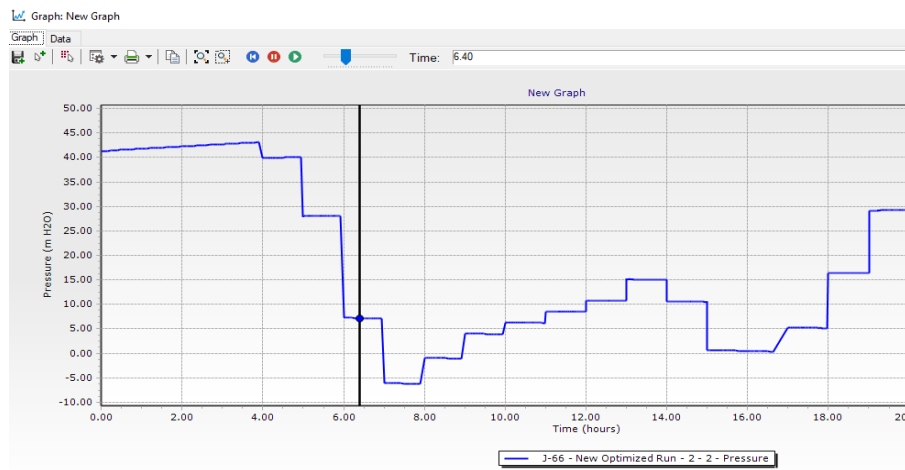


Figure 4.8: Pressure versus Time at Junction 66

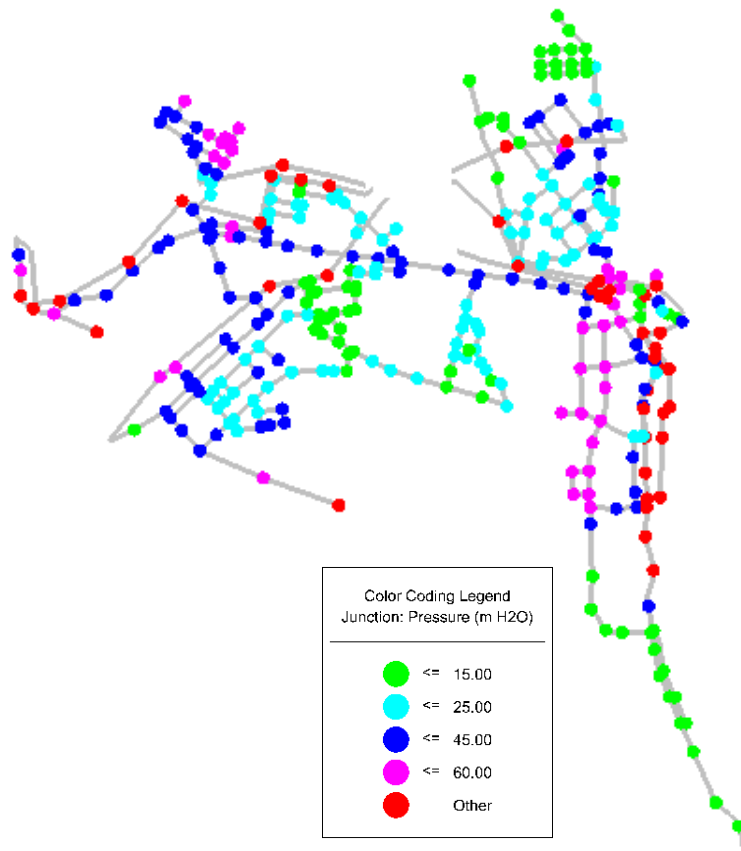


Figure 4.9: junction Pressure of the water distribution system at PHD

2) Velocity

The velocity of water flow in a pipe is also one of the important parameters in hydraulic modeling performance evaluation of the efficiency of water supply distribution and transmission line. The velocity ranges can also be adopted as the design criteria, low velocities for hygienic, while too high-velocity cause exceptional head loss reason are not preferred. Velocity distribution is also varying with demand pattern changes as shown in figure 4.10.

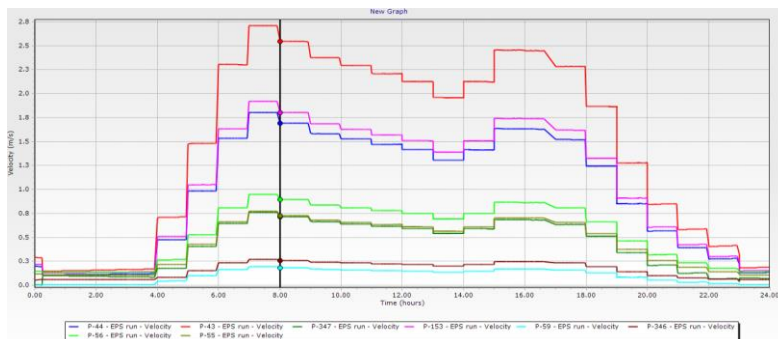


Figure 4.10: velocity and demand relationship for selected pipes

The result of velocity and head loss of kombolcha town is not desirable or adequate with respect to MoWIE (2006) of urban water supply design criteria. It is summarized in the table 4.4, figure 4.11(a&b) and the detail in Annex J. The analysis shows that systems in minimum hour demand scenario 83.5% pipes were characterized by velocity less than 0.6 m/s, and also at peak hour demand scenario 43.4% pipes were in the velocity class below 0.6 m/s. The other 13.5% and 46.6% of pipes had a velocity between 0.6 and 2 m/s during low and peak hour demand scenarios respectively. Most of the piped network was detected by low velocity. Which shows that the water distribution network is over designed compared to the available source of water in the system. This can result in damage of the network pipes since low velocity can cause deposit build up because of settling.

Therefore, control of the flow velocity in water distribution networks should be maintained in order to avoid pipe break, water hammer, water stagnation which causes sediment deposition in the pipe and head loss.

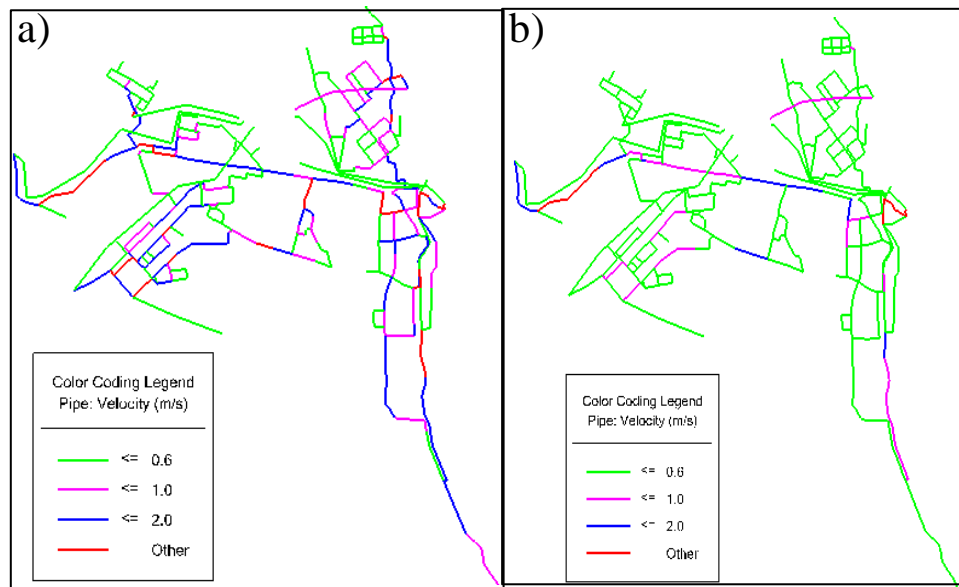


Figure 4.11: Flow velocity (a) at PHD and (b) at MHD

4.3.3 Identified Problem and Improvement mechanisms

1) Identified Problem

Models are the footprint in finding the cause of the interruptions or problems. WaterGEMS results revealed that the designed pressure and velocity did not meet the whole network of the existing WDS in the study area. As a result of this, the required water demand did not deliver to the end users in the study area. In general, the study area of water distribution system has both design and operational problems.

The major problems with respect to hydraulic network modeling as mentioned below:

- ❖ Under sized and oversized service pipe diameter
- ❖ Low pressure occurs Properties on high ground elevation and customers live far from the tank or source
- ❖ High pressure during low demand conditions this condition will affect the life of pipes and increase the leakage and breakage rates.
- ❖ Low velocity: Which shows that the water distribution network is over designed compared to the available source of water in the system. This can result in damage of the network pipes since low velocity can cause deposit build up because of settling.

For this study, improving the existing water supply system has been done peak and low consumption hour, because the minimum and maximum pressures are found than average demand.

2) Improvement Mechanisms

For improving a system there are sets of design criterion to be considered, for pressure and velocity. The improvement has been done depending on number of assumptions and facts considering with the future water consumption, availability of water, reducing the losses and high pressures, to satisfy the requirements of demands, limits of velocities and pressures in order to provide the water by acceptable quantity and quality in the future.

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 are as follows (MoWIE, 2006).

- ❖ Minimum static head is 15 m, but which can supply a 4-storey building from the distribution system is 20 m.
- ❖ Maximum static head within a pressure zone was limited to 60 m.
- ❖ The absolute minimum and maximum velocity of flow in a pipeline is in the range 0.6m/s-2m/s, in order to avoid stagnation, water quality problems and pipe breakage in the water system (Bentley Institute, 2008).

The system is redesigned depends on at peak hour flow to minimize the low-pressure nodes and low hour demand to minimize high-pressure nodes in steady state simulation. So before improving the distribution system identification of nodes with low pressure at peak demand scenario and high pressure at minimum demand scenario was made as shown table 4.4, and Annex I.

a. Controlling of high pressure beyond the allowable limit

According to WaterGEMS result analysis as shown table 4.4 at minimum demand scenario 67.4% of junction pressure is above allowable pressure value and needs reduction of excessive pressure to the desired allowable value. The best operational practice to optimize the operation of water distribution system was controlling the pressure in the network. Therefore, this excessive pressure is reduced by installing pressure reduce valves (PRV) at links which has maximum pressure. PRVs are instruments that are installed at strategic points in the network to reduce or maintain network pressure at set level as shown in table 4.5.

The valve maintains the pre-set downstream pressure regardless of the upstream pressure or flow-rate fluctuations. Normally a pressure reducing valve (PRV) which is used to control the maximum pressure entering a zone. This is possibly the simplest and most straightforward form of pressure management as it involves the use of a PRV with no additional equipment. The advantages of this form of pressure control are simple to install, have low cost and Maintenance and operation is relatively simple.

This way of manage the system will affect positively the hydraulic performance of the distribution network, especially that it will reduce the losses and velocities in the system and consequently reduce the adverse effects of the high velocities which cause deterioration of the pipes in the system, and save the water meter of the customers from blowing up due to the high pressures.

Table 4.5: Location of pressure reduce valves in improved system

Label	Elevation (m)	Diameter (Valve) (mm)	X (m)	Y (m)
PRV-1	1,842.1	100.0	579,088.2	1,225,874.2
PRV-2	1,831.8	100.0	579,626.5	1,225,788.4
PRV-3	1,863.8	100.0	578,651.3	1,225,094.1
PRV-4	1,832.3	60.0	579,344.7	1,224,828.2
PRV-5	1,846.5	100.0	578,828.0	1,225,919.0
PRV-6	1,849.4	100.0	578,658.1	1,225,799.9
PRV-7	1,862.2	100.0	578,044.0	1,226,024.0
PRV-8	1,849.9	80.0	578,534.4	1,226,397.6
PRV-9	1,899.8	225.0	581,147.3	1,225,278.7
PRV-10	1,866.1	225.0	581,117.4	1,224,618.5
PRV-11	1,867.0	80.0	581,277.6	1,224,371.9
PRV-12	1,856.8	225.0	581,131.1	1,224,066.1
PRV-13	1,847.6	80.0	581,142.3	1,223,452.7

The results of the model for improved systems are illustrated in figure 4.12 and Annex I

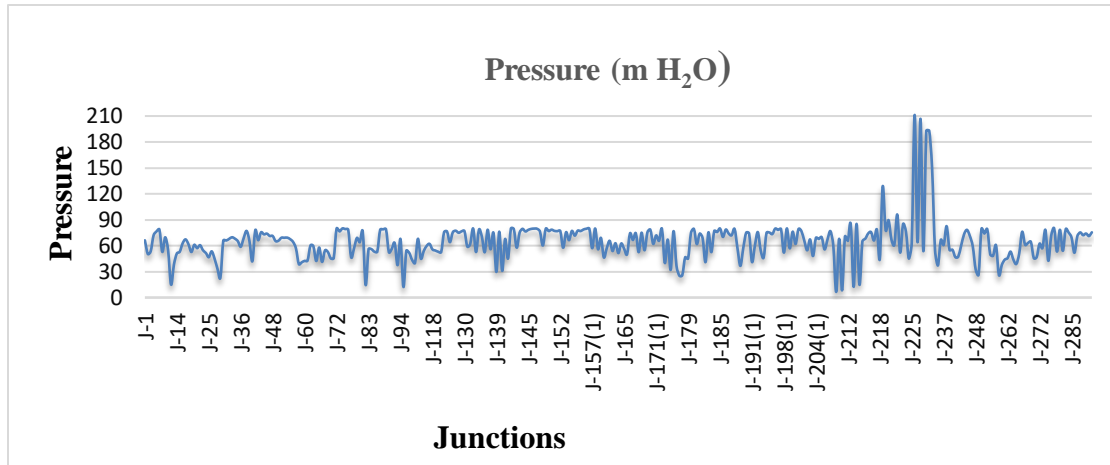


Figure 4.12: Junction pressure value for improved system

The results of improved systems are summarized in above figure 4.12, only 8% of nodes have maximum pressure above allowable (60m) in the system.

b. Managing low pressure and velocity

As we see from the table 4.4 actual velocity and pressure in the distribution system is less than 0.6m/s and 15m respectively. And the reason for this may be due to inadequate diameter (oversized or under size) of pipe, less water flow in the system and the existence of high-water loss. The improvement measure for those problem is discussed in below section.

This hydraulic performance analyzing was depending on the currently existing source of water. The available sources are lower than the needed demand. So, improving the yield of existing source of water and analyzing the performance of the existing hydraulic structure have been made rather than resizing the over size of pipe to improve the system. Although many of the scholars' states that improving velocity and pressure in water distribution system is done by resizing the pipe diameter in the system, but in this study resizing the pipe diameter in order to improve the velocity is not recommended. Because, this pipe material is an asset of the utility.

For full fill the current demand of town the flow supplied must be greater than or equal to the flow demanded. But for current scenario as shown in Figure 4.13 the current flow demand (peak flow) of the town is 236.18L/s and flow supplied is 116L/s. So, 120.08L/s or more than will required to fill the current demand and future demand.

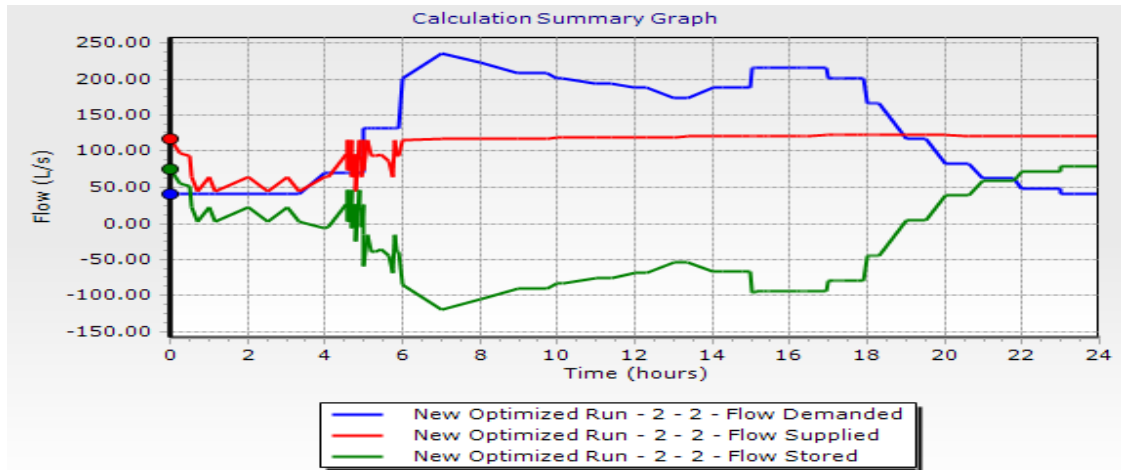


Figure 4.13: Extended period simulation of current flow supplied vs. flow demanded
 Since the main source of water supply in the town is ground water. According to hydrological study conducted by ADSW the area around kombolcha have high ground water potential with high yield. In order to satisfy the current and future water demand doing rehabilitation work for boreholes improve the yield capacity and also using booster stations in the hill areas (2000m³ service reservoir) is required in order to meet the demand for users lives at higher elevation.

To determine the size of reservoirs, it was adopted the commonly practiced in many water supply systems and based on the urban water supply design criteria of the ministry of water resources; it was used for sizing the reservoir volume as one third of the maximum daily demand. As shown Annex C the maximum day demand of kombolcha town is 14386.8m³/day. Accordingly, the current required reservoirs volume capacity is 4795.6m³.The existing 2950 m³ reservoirs can be incorporated to the system and an additional 2000 m³ new reservoirs should be constructed to deliver adequate water in the distribution networks.

When the town water demand and storage requirements are satisfied, velocity and pressure distribution in both peak and minimum day consumption becomes under allowable velocity ranges. The improved velocity and pressure distribution in the model is illustrated in the figure 4.14a, Annex J and figure 4.14b, Annex I respectively.

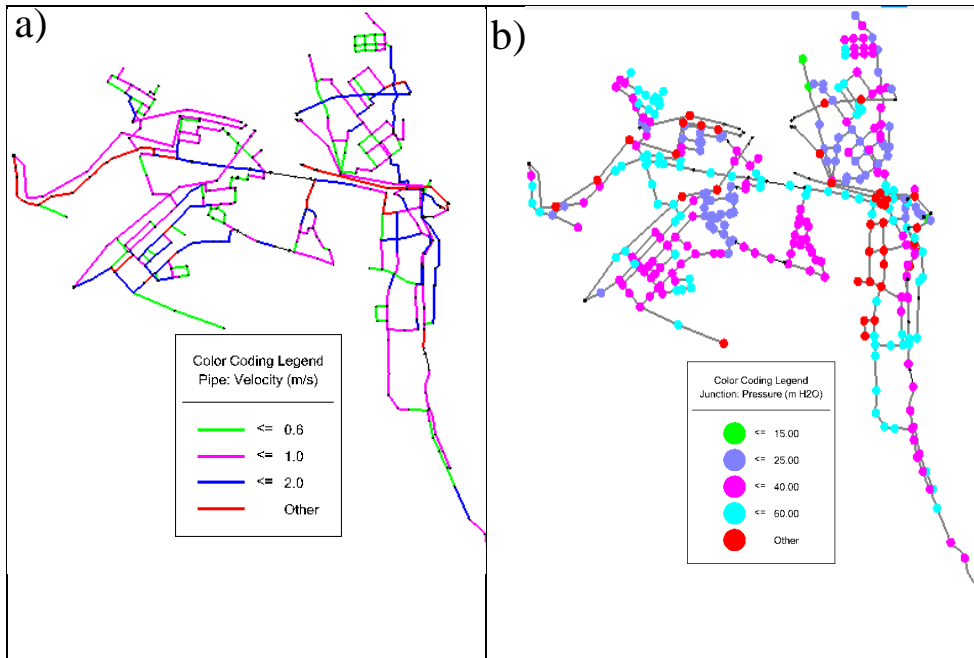


Figure 4.14: Flow velocity (a) and pressure (b) in the current needed demand (PHD)

c. Zoning and district metering

Zoning is used for better controlling high water loss in the distribution system. To divide the distribution network into a number of zones or DMAs, each with a defined and permanent boundary, so that night flows into each district can be regularly monitored, enabling the presence of unreported bursts and leakage to be identified and located. DMA is essential to ensure a minimum amount of NRW and to minimize the loss of precious water through system leakage.

Pressure zones are set up to regulate pressure in locations where large grade changes will create too much pressure at lower end of the system and not enough pressure in the higher ends allocating nodes to their appropriate pressure zoning would give the chance to the nodes getting better flow and pressure head. In the pressure zoning, the software WaterGEMS that we use is highly responsible for categorizing the system. As shown in figure 4.15, the boundaries of the 3 pressure zones proposed for kombolcha city water distribution system were delineated. The network analysis identified and confirmed the preferred locations for reservoirs and pumping stations.

Pressure Zone	Nodes <Count>	Isolation Elements <Count>	Pipes <Count>	Boundary Pipes <Count>	Customer Meters <Count>	Length (m)	Fluid Volume (L)	Color
0 Pressure Zone - 1	174	4	201	0	0	39,255	548,683.0	255,0,0
1 Pressure Zone - 2	100	4	112	0	0	25,487	484,906.8	0,0,255
2 Pressure Zone - 3	67	2	84	0	0	15,027	104,923.2	0,255,0

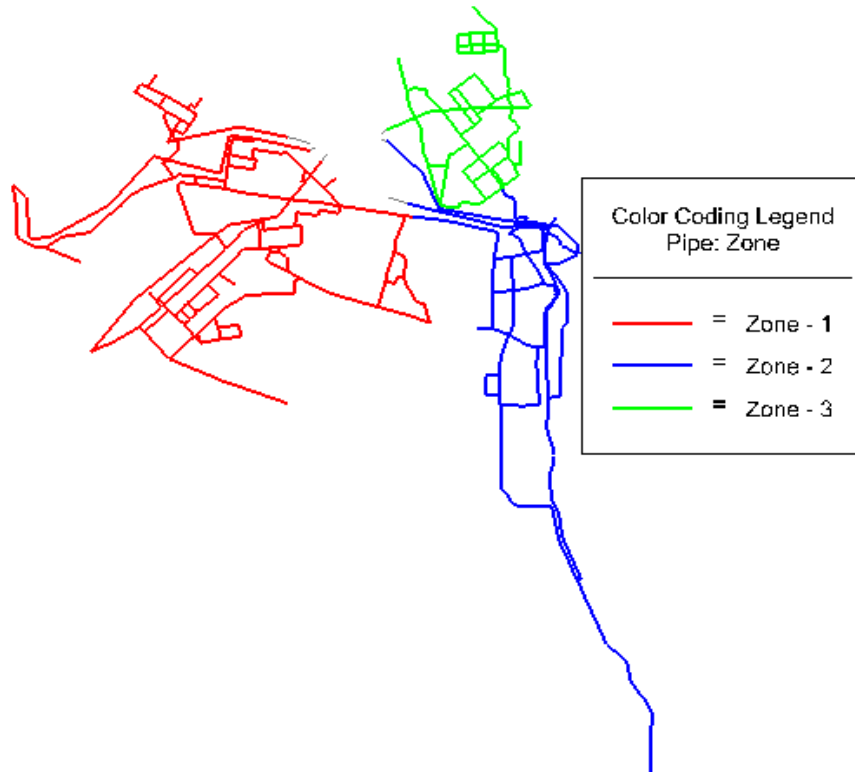


Figure 4.15: pressure zone boundaries for the kombolcha town

Chapter Five

5 Conclusion and Recommendation

5.1 Conclusion

In this study, the existing WDS is simulated through the construction of a model using Bentley WaterGEMS Connect edition software. The system was evaluated for the existing design and operation of the water distribution network, especially in various abnormal situations. The study confirmed WaterGEMS modeling procedure is the best than the conventional trial and error simulation approach. After computing the existing system, about 23.1% of junctions are failed to satisfy desirable minimum pressure during peak hour consumption and 67.4% of junctions have extremely high pressure during minimum consumption hour. Due to this, the distribution system was exposed to risks of high leakage, repeated pipe breakage and large volume of water loss. In addition to water pressure, the flow Velocity in distribution network is without the allowable limits. During minimum use 83.5 % of the pipe had less than 0.6 m/s velocity. Even if in peak hour demand scenario 43.4% pipes have velocity below allowable limit. These extreme amounts of low velocity show that the systems have over designed compared to the available source of water in the system.

Generally, the model results shown that the existence of both design and operational problems. Even during peak hour flow there is the pressure which exceed maximum allowable pressure (60m H₂O) and during low flow pressures which is lower than minimum allowable pressure (15m H₂O) clearly proved the existence of design problems. While generated excessive negative pressures and low velocity clearly proved the existence of operational problems.

In the modified system, the system shows the ability to cope the future extension in the case of providing the necessary requirements of developing and/or replacing the old pipes, providing the needed quantities of potable water and overcoming the problems of high pressures by using pressure reducing valves.

5.2 Recommendation

To improve the current conditions of water distribution system in the town, the following Recommendations were drawn to kombolcha town existing water supply system.

- As computed the current water demand in the town is much greater than of the daily water production of the system, so it is necessary to do periodic rehabilitation for the existing wells, to improve the yielding capacity of boreholes. This action can fill the gap between demand and supply. Moreover, additional booster stations should be constructed to deliver adequate water in the distribution networks at appropriate location.
- Pressure management is one of the fundamental elements of a well-developed leakage management strategy. The rate of leakage in water distribution networks is a function of the pressure applied by pumps or by gravity. To minimize the pressure in the distribution system to an acceptable level it is recommended to install PRV in the WDN at different location where the elevation is low compared to source.
- For better controlling, managing the network and also to make the assessment of water loss complete and in a reliable way, it is appreciable to divide the existing network in to sub systems and DMA for easy monitoring and managing the system. Additionally, the utility should make rehabilitation of old pipe network, to reduce the amount of water loss.
- The utility should be collected Reliable static and continuous data of X and Y coordinates of its customer water meters for proper water demand allocation of node, minimum night flow data to calculate real loss through water distribution system.

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Appendix

Annex A: Assigned base water demand to each supply node

ID	Label	X (m)	Y (m)	Number of houses	Number of people served from each node	Base demand (l/s)
33	J-1	580,373	1,226,134	121	544.5	0.52
30	J-2	577,224	1,224,828	28	126	0.29
31	J-3	577,076	1,224,976	52	234	0.35
34	J-5	577,543	1,225,582		0	0.22
248	J-6	581,215	1,225,056	0	0	0.00
251	J-7	581,149	1,224,785	56	252	0.36
256	J-8	581,142	1,223,770	65.0	292.5	0.38
258	J-9	581,133	1,223,515	450.0	2025	1.36
262	J-10	581,158	1,222,917	290.0	1305	0.95
282	J-11	581,277	1,225,465	33.0	148.5	0.30
287	J-13	581,225	1,225,767	82	369	0.42
289	J-14	581,126	1,225,601	39	175.5	0.31
306	J-15	580,912	1,225,652	100	450	0.47
308	J-16	580,823	1,225,641	28	126	0.29
310	J-17	580,743	1,225,590		0	0.22
312	J-18	580,856	1,225,525	82	369	0.42
321	J-19	580,923	1,225,760	200	900	0.72
323	J-20	580,814	1,225,752	70	315	0.39
325	J-21	580,794	1,225,972	100	450	0.47
327	J-22	580,694	1,225,991	125	562.5	0.53
333	J-23	580,733	1,226,470	40	180	0.32
335	J-24	580,741	1,226,610	40	180	0.32
337	J-25	580,741	1,226,826	126	567	0.54
339	J-26	580,614	1,227,005	157	706.5	0.61
341	J-27	580,725	1,227,058	73	328.5	0.40
343	J-28	580,819	1,227,081	86	387	0.43
345	J-29	580,888	1,227,068	120	540	0.52
355	J-30	580,420	1,226,857	126	567	0.54
357	J-31	580,465	1,226,802	85	382.5	0.43
359	J-32	580,383	1,226,742	77	346.5	0.41
361	J-33	580,249	1,226,640	80	360	0.42
363	J-34	580,326	1,226,547	49	220.5	0.34
365	J-35	580,442	1,226,391	86	387	0.43

367	J-36	580,646	1,226,562	120	540	0.52
372	J-37	580,276	1,226,251	69	310.5	0.39
386	J-38	580,591	1,225,320	90	405	0.44
390	J-39	580,799	1,225,572		0	0.22
406	J-40	581,126	1,224,793	87	391.5	0.44
416	J-41	580,656	1,223,768	150	675	0.60
440	J-43	580,440	1,225,900	50	225	0.34
443	J-44	579,971	1,226,140	45	202.5	0.33
446	J-45	580,113	1,226,299	70	315	0.39
448	J-46	580,086	1,226,139	64	288	0.38
451	J-47	580,134	1,226,453	100	450	0.47
453	J-48	579,873	1,226,650	200	900	0.72
455	J-49	579,706	1,227,101	120	540	0.52
457	J-50	579,774	1,227,134	138	621	0.57
459	J-51	579,908	1,227,129	140	630	0.57
461	J-52	579,907	1,227,067	110	495	0.50
463	J-53	580,046	1,226,922	130	585	0.55
466	J-54	580,134	1,227,088	77	346.5	0.41
468	J-55	580,208	1,227,138	130	585	0.55
471	J-56	580,391	1,227,287	150	675	0.60
475	J-57	580,700	1,227,287	200	900	0.72
477	J-58	580,692	1,227,571	95	427.5	0.46
481	J-60	580,622	1,227,604	47	211.5	0.33
483	J-61	580,623	1,227,527	38	171	0.31
485	J-62	580,230	1,227,504	90	405	0.44
487	J-63	580,224	1,227,583	50	225	0.34
490	J-64	580,616	1,227,729	86	387	0.43
492	J-65	580,220	1,227,701	150	675	0.60
496	J-66	580,373	1,228,015	200	900	0.72
498	J-67	580,363	1,227,514	60	270	0.37
501	J-68	580,346	1,227,718	98	441	0.46
505	J-69	580,497	1,227,520	46	207	0.33
508	J-70	580,487	1,227,724	98	441	0.46
526	J-72	579,873	1,224,844	160	720	0.62
536	J-73	579,608	1,225,125	230	1035	0.80
542	J-74	578,569	1,225,090	25	112.5	0.28
555	J-75	578,043	1,225,423	24	108	0.28
558	J-76	578,342	1,225,264		0	0.22

567	J-77	577,429	1,224,392	110	495	0.50
582	J-78	577,833	1,224,022	0	0	0.00
591	J-79	577,790	1,224,934	64	288	0.38
593	J-80	577,681	1,225,049	48	216	0.34
601	J-81	577,965	1,225,041	86	387	0.43
607	J-82	576,726	1,224,439	0	0	0.00
627	J-83	579,026	1,225,883		0	0.22
629	J-84	579,010	1,225,801	32	144	0.30
631	J-85	578,810	1,225,841	14	63	0.25
633	J-86	578,802	1,225,780	45	202.5	0.33
635	J-87	578,588	1,225,812	60	270	0.37
652	J-88	577,863	1,226,150	23	103.5	0.27
657	J-89	578,473	1,226,466	8	36	0.24
661	J-90	578,863	1,226,078	40	180	0.32
690	J-91	577,497	1,226,742	60	270	0.37
692	J-92	577,567	1,226,857	36	162	0.31
695	J-93	577,430	1,226,646	13	58.5	0.25
700	J-94	577,620	1,227,039	89	400.5	0.44
717	J-95	581,079	1,225,648	80	360	0.42
722	J-96	577,503	1,224,682	34	153	0.30
726	J-97	577,545	1,224,496	24	108	0.28
768	J-106	581,201	1,223,226	250	1125	0.85
775	J-108	581,101	1,225,060	84	378	0.43
788	J-110	580,612	1,225,454	32	144	0.30
813	J-113	580,495	1,227,598	57	256.5	0.36
816	J-114	580,359	1,227,594	54	243	0.35
37	J-115	580,680	1,226,081	88	396	0.44
146	J-117	580,547	1,226,264	100	450	0.47
158	J-118	580,733	1,226,402	20	90	0.27
170	J-119	580,763	1,226,367	43	193.5	0.32
182	J-120	580,837	1,226,206	110	495	0.50
194	J-121	580,859	1,226,319	78	351	0.41
205	J-122	579,670	1,225,351	80	360	0.42
218	J-123	580,037	1,225,882	79	355.5	0.42
39	J-124	580,513	1,225,952	107	481.5	0.49
50	J-125	579,700	1,225,290	170	765	0.65
67	J-127	579,587	1,225,055	120	540	0.52
78	J-128	579,450	1,224,942	88	396	0.44

88	J-129	579,488	1,225,079	100	450	0.47
108	J-130	579,667	1,225,131	108	486	0.49
823	J-131	579,928	1,226,298	88	396	0.44
826	J-132	580,568	1,226,283	55	247.5	0.36
128	J-132(1)	579,744	1,224,950	200	900	0.72
830	J-133	579,805	1,224,861	120	540	0.52
136	J-133(1)	580,787	1,226,329	76	342	0.41
833	J-134	580,252	1,225,929	52	234	0.35
138	J-135	580,860	1,226,584	100	450	0.47
139	J-136	579,725	1,224,721	700	3150	2.00
842	J-137	580,916	1,225,372	156	702	0.61
141	J-138	579,412	1,224,810	380	1710	1.18
850	J-139	579,161	1,224,882	78	351	0.41
142	J-139(1)	581,055	1,225,173	320	1440	1.03
853	J-140	578,934	1,224,950	74	333	0.40
143	J-140(1)	581,223	1,224,933	100	450	0.47
856	J-141	578,617	1,225,115	64	288	0.38
144	J-141(1)	581,108	1,224,649	0	0	0.00
145	J-142	578,555	1,224,944	190	855	0.70
859	J-142(1)	581,043	1,223,901	62	279	0.37
148	J-143	578,079	1,224,938	190	855	0.70
149	J-144	578,485	1,225,196	54	243	0.35
865	J-144(1)	580,580	1,224,577	370	1665	1.16
150	J-145	578,497	1,225,311	52	234	0.35
869	J-145(1)	580,410	1,224,585	350	1575	1.11
151	J-146	578,385	1,225,398	20	90	0.27
152	J-147	578,442	1,225,452	38	171	0.31
874	J-147(1)	580,643	1,223,881	252	1134	0.86
153	J-148	579,614	1,227,454	150	675	0.60
877	J-148(1)	580,512	1,223,879	140	630	0.57
154	J-149	578,278	1,225,307	50	225	0.34
879	J-149(1)	580,499	1,224,076	159	715.5	0.62
155	J-150	578,316	1,225,388	30	135	0.29
156	J-151	578,312	1,225,438	30	135	0.29
157	J-152	578,217	1,225,430	56	252	0.36
886	J-152(1)	581,256	1,222,314	310	1395	1.00
160	J-153	578,195	1,225,709	20	90	0.27
888	J-153(1)	581,220	1,225,146	40	180	0.32

161	J-154	578,240	1,225,722	25	112.5	0.28
890	J-154(1)	581,336	1,224,964	30	135	0.29
162	J-155	578,250	1,225,627	40	180	0.32
892	J-155(1)	581,339	1,224,636	140	630	0.57
163	J-156	578,295	1,225,633	36	162	0.31
894	J-156(1)	581,284	1,224,584	71	319.5	0.40
164	J-157	578,567	1,225,670	20	90	0.27
896	J-157(1)	581,260	1,223,854	130	585	0.55
165	J-158	578,561	1,225,466	36	162	0.31
898	J-158(1)	581,135	1,223,851	48	216	0.34
902	J-159	581,278	1,224,372	100	450	0.47
168	J-160	577,795	1,225,158	60	270	0.37
905	J-160(1)	581,117	1,224,619	57	256.5	0.36
169	J-161	580,472	1,227,886	100	450	0.47
908	J-161(1)	581,131	1,224,066	160	720	0.62
172	J-162	577,571	1,224,739	90	405	0.44
911	J-162(1)	581,434	1,221,884	85	382.5	0.43
174	J-163	577,467	1,224,845	57	256.5	0.36
915	J-163(1)	581,387	1,222,130	0	0	0.00
175	J-164	577,404	1,224,776	12	54	0.25
918	J-164(1)	581,340	1,222,132	44	198	0.33
921	J-165	581,208	1,222,538	170	765	0.65
177	J-166	577,351	1,224,707	56	252	0.36
924	J-166(1)	579,664	1,225,699		0	0.22
178	J-167	577,896	1,224,810	180	810	0.67
927	J-167(1)	579,611	1,225,514	119	535.5	0.52
179	J-168	577,579	1,224,528	58	261	0.36
930	J-168(1)	579,553	1,225,322	150	675	0.60
180	J-169	577,609	1,224,427	104	468	0.48
933	J-169(1)	579,507	1,225,182	130	585	0.55
936	J-170	579,714	1,225,019	165	742.5	0.64
939	J-171	580,773	1,225,341	80	360	0.42
185	J-171(1)	578,776	1,225,030		0	0.22
186	J-172	577,811	1,224,469	102	459	0.47
942	J-172(1)	578,443	1,224,944	120	540	0.52
945	J-173	580,638	1,225,602		0	0.22
187	J-173(1)	577,133	1,224,433	80	360	0.42
188	J-174	581,210	1,225,030		0	0.22

190	J-176	578,018	1,224,498	28	126	0.29
952	J-176(1)	577,036	1,224,542	34	153	0.30
191	J-177	581,242	1,225,542	67	301.5	0.39
192	J-178	581,448	1,225,373	0	0	0.00
957	J-178(1)	577,282	1,224,769	60	270	0.37
961	J-179	580,784	1,224,982	260	1170	0.88
193	J-179(1)	577,217	1,224,813	61	274.5	0.37
196	J-180	580,572	1,224,968	246	1107	0.84
197	J-181	577,788	1,224,591	66	297	0.38
965	J-181(1)	577,790	1,225,356	320	1440	1.03
968	J-182	581,111	1,224,384		0	0.22
198	J-182(1)	577,458	1,225,838	40	180	0.32
971	J-183	581,182	1,222,684	66	297	0.38
199	J-183(1)	578,053	1,225,628	38	171	0.31
974	J-184	580,788	1,222,710	370	1665	1.16
200	J-184(1)	578,209	1,225,536	40	180	0.32
977	J-185	580,663	1,222,888	280	1260	0.93
201	J-185(1)	578,634	1,225,674	30	135	0.29
979	J-186	580,673	1,223,174	550	2475	1.62
202	J-186(1)	578,629	1,225,477	46	207	0.33
203	J-187	577,998	1,224,617	120	540	0.52
981	J-187(1)	578,632	1,225,635	36	162	0.31
204	J-188	581,052	1,223,781	56	252	0.36
206	J-189	576,992	1,224,599	18	81	0.26
992	J-190	579,577	1,225,395	80	360	0.42
207	J-190(1)	579,435	1,225,820		0	0.22
995	J-191	577,288	1,224,257		0	0.22
208	J-191(1)	577,625	1,226,102	246	1107	0.84
998	J-192	578,658	1,225,800	42	189	0.32
1001	J-193	579,687	1,225,777		0	0.22
210	J-193(1)	577,370	1,226,086		0	0.22
1004	J-194	581,034	1,224,381	200	900	0.72
211	J-194(1)	578,953	1,225,975		0	0.22
1007	J-195	578,486	1,223,786	67	301.5	0.39
301	J-195(1)	578,534	1,226,398	0	0	0.00
1010	J-196	580,744	1,224,516	360	1620	1.13
213	J-196(1)	577,898	1,226,430	0	0	0.00
1013	J-197	580,648	1,224,082	250	1125	0.85

216	J-197(1)	578,171	1,226,377	50	225	0.34
1015	J-198	581,203	1,222,707	160	720	0.62
217	J-198(1)	578,127	1,226,381	40	180	0.32
1019	J-199	581,288	1,222,358	115	517.5	0.51
219	J-199(1)	578,149	1,226,269	38	171	0.31
1021	J-200	580,933	1,222,689	230	1035	0.80
220	J-200(1)	577,884	1,226,313	11	49.5	0.24
1025	J-201	578,161	1,226,568	32	144	0.30
1027	J-202	581,481	1,221,904	0	0	0.00
222	J-202(1)	577,876	1,226,254	9	40.5	0.24
1035	J-203	581,543	1,221,659	160	720	0.62
223	J-203(1)	577,365	1,226,989	75	337.5	0.41
1038	J-204	581,738	1,221,218	300	1350	0.98
224	J-204(1)	576,933	1,226,011	200	900	0.72
1041	J-205	581,940	1,221,016	430	1935	1.31
225	J-205(1)	575,851	1,225,482	50	225	0.34
1431	J-206	580,734	1,226,437	20	90	0.27
1434	J-207	580,426	1,226,774	37	166.5	0.31
1437	J-208	580,659	1,223,626	426	1917	1.30
1440	J-209	580,807	1,225,821	100	450	0.47
41	J-210	581,371	1,225,404	16	72	0.26
42	J-211	581,327	1,225,387		0	0.22
43	J-212	581,259	1,225,276		0	0.22
44	J-213	581,086	1,225,532	44	198	0.33
45	J-214	581,089	1,225,411	49	220.5	0.34
47	J-215	581,107	1,225,275	50	225	0.34
48	J-216	581,252	1,224,963	22	99	0.27
51	J-218	581,128	1,224,384	61	274.5	0.37
52	J-219	580,230	1,225,701		0	0.22
53	J-220	577,891	1,224,477	53	238.5	0.35
54	J-221	577,749	1,224,689	88	396	0.44
55	J-222	577,443	1,224,626	30	135	0.29
57	J-224	577,865	1,225,428	120	540	0.52
58	J-225	577,573	1,225,148	46	207	0.33
59	J-226	577,507	1,225,219	222	999	0.78
60	J-227	577,170	1,224,891	136	612	0.56
69	J-234	575,744	1,225,814	55	247.5	0.36
70	J-235	575,725	1,225,949	0	0	0.00

71	J-236	575,742	1,225,595	32	144	0.30
72	J-237	576,032	1,225,440	44	198	0.33
73	J-238	576,399	1,225,281		0	0.22
74	J-239	576,205	1,225,556	0	0	0.00
75	J-240	576,480	1,225,601	80	360	0.42
76	J-241	576,707	1,225,813	100	450	0.47
77	J-242	577,019	1,226,070	180	810	0.67
79	J-243	577,393	1,226,132	0	0	0.00
80	J-244	577,554	1,226,113		0	0.22
81	J-245	577,768	1,225,578		0	0.22
83	J-246	577,567	1,226,194		0	0.22
84	J-247	577,390	1,226,217	0	0	0.00
85	J-248	577,380	1,226,469	51	229.5	0.34
86	J-249	577,385	1,226,563	0	0	0.00
87	J-250	577,860	1,226,066	170	765	0.65
89	J-251	577,928	1,226,598	0	0	0.00
90	J-252	578,434	1,226,438	0	0	0.00
93	J-255	578,675	1,226,267	103	463.5	0.48
94	J-256	578,638	1,226,135	91	409.5	0.45
95	J-257	578,604	1,225,940	29	130.5	0.29
96	J-258	578,044	1,226,024	28	126	0.29
97	J-259	577,314	1,226,643	60	270	0.37
98	J-260	577,336	1,226,697	15	67.5	0.25
99	J-261	577,231	1,226,837	36	162	0.31
100	J-262	577,261	1,226,883	44	198	0.33
101	J-263	577,411	1,226,802	76	342	0.41
102	J-264	577,183	1,226,944	140	630	0.57
103	J-265	576,944	1,227,093	160	720	0.62
104	J-266	577,262	1,227,053	26	117	0.28
40	J-267	577,478	1,226,918		0	0.22
105	J-267(1)	577,506	1,226,959	32	144	0.30
106	J-268	577,557	1,226,930	54	243	0.35
107	J-269	577,001	1,227,183	38	171	0.31
109	J-270	577,073	1,227,142	55	247.5	0.36
110	J-271	577,156	1,227,279	82	369	0.42
111	J-272	577,338	1,226,182	100	450	0.47
112	J-273	578,144	1,226,488	40	180	0.32
113	J-274	577,229	1,226,338	27	121.5	0.28

114	J-275	578,979	1,226,172	130	585	0.55
836	J-275(1)	579,191	1,225,865	50	225	0.34
137	J-276	578,345	1,224,945	300	1350	0.98
116	J-277	578,298	1,225,985	110	495	0.50
118	J-278	579,985	1,225,744		0	0.22
119	J-279	580,428	1,225,652	180	810	0.67
120	J-280	580,881	1,223,757	49	220.5	0.34
121	J-281	581,025	1,224,198		0	0.22
122	J-282	580,683	1,224,313		0	0.22
125	J-285	580,785	1,224,803	150	675	0.60
130	J-289	578,828	1,225,919	160	720	0.62
131	J-290	577,156	1,227,279	220	990	0.78
133	J-292	577,338	1,226,182	430	1935	1.31
230	J-294	578,144	1,226,488		0	0.22
227	J-295	577,229	1,226,338	190	855	0.70
228	J-296	578,979	1,226,172	50	225	0.34
226	J-297	579,191	1,225,865	180	810	0.67
117	J-299	578,345	1,224,945	82	369	0.42

Annex B: Water Source of the town water supply system

Location of source	Design yield(l/s)	Yield(l/s)	Daily production (m ³)	Depth (m)	Service year
Deway -1(BH -1)	10	10	648	100	1963
Deway -2 (BH -2)	10	10	648	100	1963
Deway -3(BH -3)	5.5	5.5	356.4	110	1963
Deway -4(BH -4)	25	20	1296	140	2007
Deway -5(BH -5)	60	0	0	112	2007
Deway -6(BH -6)	50	20	296	112	2007
shishaber -7(BH -7)	19	14.8	959.04	144	2012
Shishaber -8(BH -8)	19	15	972		2012
Shishaber -9(BH -9)	30	20	1296		2012
Shishaber -10(BH -10)	25	15	972		2012
Biraro(BH-11)	50	25	1620	120	2015
TOTAL	303.5l/s	155.3	10063.44		

Source: From KWSSO document

Annex C: Current and projected domestic water demand of Kombolcha own

Description	Unit	2020	2022	2027	2032	2040
Urban growth rate		4.1	4	3.8	3.7	3.7
Total population	No	134599	141354	161487	184904	233462
Per capita demand	l/c/d	45.9	48.4	55.4	63.5	78
Average Domestic Demand (ADD)	m ³ /d	6178.1	6841.5	8946.4	11741.4	18210
Adjusted total domestic Demand	m ³ /d	6116.3	6773.1	8857	11624	18028
University community	No	8500.0	8500.0	8500.0	8500.0	8500
University community demand(60l/c/day)	m ³ /d	510.0	510.0	510.0	510.0	510
Public and commercial demand (30% of ADD)	m ³ /d	1834.9	2031.9	2657.1	3487.2	5408.4
Industrial demand (10% of ADD)	m ³ /d	611.6	677.3	885.7	1162.4	1802.8
Average daily demand (ADD)	m ³ /d	9072.8	9992.3	12909.7	16783.6	25749
Unaccounted flow of water (UFW)	%	32.4	30.0	29.0	27.0	26.0
UFW	m ³ /d	2930.5	2997.7	3743.8	4531.6	6694.8
Total average day water demand (TADD)	m ³ /d	11989.0	12990.0	16653.5	21315.2	32443.8
Total average day water demand (TADD)	L/S	138.8	150.3	192.7	246.7	375.5
Max day factor (MDF)		1.2	1.2	1.2	1.2	1.2
max day demand (MDD)	m ³ /d	14386.8	15588	19984	25578	38932.6
max day demand	l/s	166.5	180.4	231.3	296.0	450.6
peak hour factor (PHF)		1.7	1.7	1.7	1.7	1.7
peak hour demand (PHD)	m ³ /d	20381	22083	28311	36235.8	55154.5
peak hour demand	l/s	235.9	255.6	327.7	419.4	638.4

Annex D: Reservoirs in the Study Area Distribution System

No	Name	Volume (m ³)	Height (m)	Diameter (m)	Location		
					Elevation (Masl)	X(m)	Y(m)
1	Michael	1000	6.5	14.0	1986	581453	125442
2	Small michael	50	2	5.6	1938	581347	1225650
3	Asseber	500	4.5	12	1897	581099	1226960
4	Asselel	500	4.5	12	1840	577304	1226569
5	Meja	300	3	11.3	1895	580959	1226960
6	Old airport	300	3	11.3	1940	576505	1224329
7	Kutiager	300	3	11.3	2051	575696	1226101

Source: kombolcha town water office

Annex E: Roughness coefficient, C- factor for different pipe material

Types of pipes	Hazen-Williams C Value			
	UPVC	GI	DCI	HDPE
new	150	110	120	150
existing	100-120	90-110	100-110	

Source (Water GEMS, 2018)

Annex F: Monthly water productions, consumption and loss

S/N	Year	Production(m3)	Consumption(m ³)	Water loss (m ³ /month)
1	January	225,100	200147	24,953
2	February	248,589	213586	35,003
3	March	250,127	214125	36,002
4	April	258,789	236196	22,593
5	May	262,251	224751	37,500
6	June	296,251	214000	82,251
7	July	310,120	195140	114,980
8	August	329,410	189145	140,265
9	September	343,587	170458	173,129
10	October	375,251	160619	214,632
11	November	358,147	209447	148,700
12	December	368,214	222553	145,661
	Total	3,625,836	2450167	1,175,669

Annex G: Observed and simulated hydraulic grade Difference results for calibration

Solution		Simulated Results			
Simulated Results Browser					
Attribute		Snapshot	Hydraulic Grade RMSE (m)		
Hydraulic Grade		1 J-13	0.47		
Flow		2 J-26	1.53		
		3 J-62	2.73		
		4 J-190(1)	1.71		
		5 J-221	1.53		
		6 J-263	1.00		
		7 J-276	1.55		
		8 J-292	0.46		
Simulated Results					
	Field Data Snapshot	Junction	Observed Hydraulic Grade (m)	Simulated Hydraulic Grade (m)	Difference (m)
1	J-13	J-13	1,973.35	1,972.88	-0.47
2	J-26	J-26	1,893.30	1,891.77	-1.53
3	J-62	J-62	1,859.02	1,856.29	-2.73
4	J-190(1)	J-190(1)	1,846.63	1,848.34	1.71
5	J-221	J-221	1,894.91	1,896.44	1.53
6	J-263	J-263	1,939.33	1,938.33	-1.00
7	J-276	J-276	1,844.04	1,842.49	-1.55
8	J-292	J-292	1,890.51	1,890.05	-0.46

Annex H: Observed and simulated hydraulic grade Difference results for validation

Solution		Simulated Results			
Simulated Results Browser					
Attribute		Snapshot	Hydraulic Grade RMSE (m)		
Hydraulic Grade	1	J-2	0.03		
Flow	2	J-38	0.22		
	3	J-90	0.12		
	4	J-91	0.97		
	5	J-120	0.23		
	6	J-136	0.09		
	7	J-142	0.65		
	8	J-172	1.13		
Simulated Results					
	Field Data Snapshot	Junction	Observed Hydraulic Grade (m)	Simulated Hydraulic Grade (m)	Difference (m)
1	J-2	J-2	1,937.57	1,937.60	0.03
2	J-38	J-38	1,908.18	1,907.96	-0.22
3	J-90	J-90	1,911.33	1,911.21	-0.12
4	J-91	J-91	1,944.61	1,943.64	-0.97
5	J-120	J-120	1,894.77	1,895.00	0.23
6	J-136	J-136	1,901.70	1,901.61	-0.09
7	J-142	J-142	1,913.59	1,912.94	-0.65
8	J-172	J-172	1,929.01	1,927.88	-1.13

Annex I: Junctions pressure result at peak (PHD) and Minimum hour demand (MHD)

ID	Label	X (m)	Y (m)	Elevation (m)	Pressure (m H ₂ O)			
					Existing		Improved	
					MHD	PHD	MHD	PHD
33	J-1	580,373.0	1,226,134.0	1,839.8	66.2	22.3	50.2	24.6
30	J-2	577,236.0	1,224,833.3	1,899.0	48.8	26.9	50.5	27.5
31	J-3	577,075.9	1,224,975.7	1,896.6	53.4	51.6	53.5	49.4
34	J-5	577,543.0	1,225,582.0	1,878.4	67.7	42.9	52.0	52.5
248	J-6	581,214.1	1,225,065.8	1,881.7	109.6	108.4	58.5	67.5
251	J-7	581,149.9	1,224,782.2	1,879.4	111.8	110.5	54.7	69.1
256	J-8	581,141.7	1,223,770.1	1,863.0	122.8	94.1	52.8	48.2
258	J-9	581,133.0	1,223,515.4	1,845.8	136.8	94.9	59.7	30.7
262	J-10	581,158.1	1,222,916.6	1,842.4	122.1	31.6	54.7	35.2
282	J-11	581,276.9	1,225,464.9	1,945.2	19.8	18.7	15.0	17.0
287	J-13	581,224.8	1,225,766.7	1,921.3	53.9	51.5	38.0	32.0
289	J-14	581,125.9	1,225,600.7	1,908.3	66.8	64.5	50.9	45.5
306	J-15	580,911.9	1,225,652.2	1,859.4	53.0	52.0	52.9	52.1
308	J-16	580,822.7	1,225,641.2	1,849.6	62.8	61.5	52.7	61.6
310	J-17	580,742.8	1,225,589.8	1,845.0	67.4	65.8	47.3	65.9
312	J-18	580,855.8	1,225,525.1	1,850.5	61.9	59.5	41.7	59.7
321	J-19	580,922.7	1,225,760.4	1,859.2	52.8	47.2	52.7	47.6
323	J-20	580,813.6	1,225,751.9	1,850.3	61.1	52.0	60.0	52.6
325	J-21	580,794.4	1,225,972.1	1,851.7	57.2	32.0	57.2	33.9
327	J-22	580,693.9	1,225,991.4	1,846.9	60.7	27.8	50.6	30.4
333	J-23	580,732.9	1,226,470.3	1,851.9	54.7	26.3	54.7	30.7
335	J-24	580,740.5	1,226,610.2	1,855.2	51.6	31.2	51.6	30.3
337	J-25	580,740.5	1,226,826.2	1,860.2	46.6	30.9	46.6	30.4
339	J-26	580,614.4	1,227,005.3	1,853.2	53.5	38.5	53.5	38.5
341	J-27	580,725.0	1,227,057.7	1,862.0	44.9	35.1	44.9	34.9
343	J-28	580,819.1	1,227,081.2	1,873.8	33.2	25.4	33.2	25.3
345	J-29	580,887.8	1,227,068.0	1,884.0	23.1	17.5	23.1	17.4
355	J-30	580,420.4	1,226,857.1	1,840.6	66.0	46.6	56.0	46.9
357	J-31	580,464.8	1,226,802.4	1,840.4	66.0	42.1	56.0	42.5
359	J-32	580,396.3	1,226,740.8	1,838.4	67.9	39.8	57.9	40.5
361	J-33	580,249.3	1,226,639.8	1,836.0	69.8	23.8	49.8	25.7
363	J-34	580,326.3	1,226,547.4	1,837.6	68.1	20.3	58.1	22.5
365	J-35	580,442.0	1,226,391.4	1,840.8	65.3	21.9	55.3	24.9
367	J-36	580,645.9	1,226,561.8	1,847.6	58.8	24.2	58.8	28.1

372	J-37	580,275.9	1,226,250.8	1,837.4	68.6	24.7	48.6	26.4
386	J-38	580,590.8	1,225,319.8	1,833.0	81.8	53.6	57.1	65.9
390	J-39	580,798.7	1,225,572.0	1,846.9	65.5	63.8	55.4	63.9
406	J-40	581,126.2	1,224,793.3	1,869.9	42.6	34.0	41.9	36.0
416	J-41	580,656.2	1,223,767.6	1,832.2	80.5	48.6	57.9	60.0
440	J-43	580,439.6	1,225,899.6	1,839.5	66.4	19.3	46.4	20.5
443	J-44	579,970.6	1,226,139.7	1,830.0	75.5	21.1	55.5	23.4
446	J-45	580,113.0	1,226,299.0	1,832.5	73.1	21.2	53.1	23.5
448	J-46	580,086.4	1,226,138.5	1,831.6	74.0	21.6	44.0	22.7
451	J-47	580,133.9	1,226,437.4	1,834.4	71.2	20.9	41.2	23.1
453	J-48	579,857.7	1,226,612.7	1,834.1	71.3	12.5	51.3	17.6
455	J-49	579,706.2	1,227,101.2	1,839.9	65.4	4.3	45.3	5.4
457	J-50	579,774.3	1,227,134.0	1,839.4	65.8	4.7	55.8	18.8
459	J-51	579,907.8	1,227,128.9	1,836.0	69.2	8.2	59.2	18.0
461	J-52	579,907.2	1,227,066.8	1,836.0	69.2	8.3	49.2	17.2
463	J-53	580,046.4	1,226,921.8	1,836.0	69.4	12.3	49.3	18.1
466	J-54	580,134.1	1,227,088.1	1,838.8	67.6	41.8	57.6	42.4
468	J-55	580,207.6	1,227,138.2	1,842.0	64.4	41.5	54.4	41.9
471	J-56	580,398.4	1,227,285.7	1,849.6	57.0	39.2	57.0	39.3
475	J-57	580,700.3	1,227,286.7	1,868.0	38.7	24.5	38.7	24.4
477	J-58	580,691.7	1,227,571.2	1,865.8	40.8	22.3	40.8	22.1
481	J-60	580,621.9	1,227,604.0	1,862.8	42.5	-5.8	42.5	25.0
483	J-61	580,623.3	1,227,527.4	1,862.5	42.8	-5.7	42.8	25.0
485	J-62	580,230.5	1,227,504.1	1,845.0	60.2	11.3	60.0	41.6
487	J-63	580,223.6	1,227,583.5	1,845.8	59.5	10.5	59.5	40.8
490	J-64	580,616.5	1,227,728.6	1,863.0	42.3	-6.4	42.3	24.5
492	J-65	580,219.5	1,227,701.2	1,847.3	58.0	9.0	58.0	39.1
496	J-66	580,372.8	1,228,014.6	1,864.0	41.3	-8.0	41.3	21.7
498	J-67	580,362.6	1,227,514.0	1,850.7	54.5	5.6	54.5	36.3
501	J-68	580,345.9	1,227,717.9	1,852.6	52.7	3.8	52.7	34.1
505	J-69	580,497.3	1,227,520.4	1,860.0	45.3	-3.5	45.3	27.2
508	J-70	580,487.5	1,227,724.1	1,859.7	45.6	-3.2	45.6	27.4
526	J-72	579,884.0	1,224,838.7	1,825.4	102.2	15.6	59.9	29.4
536	J-73	579,607.7	1,225,125.0	1,828.9	98.7	14.7	46.5	29.0
542	J-74	578,569.4	1,225,090.1	1,867.9	72.3	5.4	60.0	16.9
555	J-75	578,042.6	1,225,422.7	1,869.0	73.4	18.0	59.2	28.5
558	J-76	578,342.0	1,225,263.6	1,869.2	70.5	2.0	58.6	19.5
567	J-77	577,429.5	1,224,392.2	1,902.8	45.2	25.2	46.7	32.0

582	J-78	577,833.1	1,224,022.0	1,893.0	56.5	50.1	57.0	46.1
591	J-79	577,790.2	1,224,934.3	1,879.9	66.1	30.2	49.0	40.4
593	J-80	577,680.6	1,225,049.2	1,884.8	61.0	23.6	54.1	33.8
601	J-81	577,965.0	1,225,040.7	1,872.0	72.7	28.6	56.6	38.4
607	J-82	576,726.1	1,224,438.8	1,935.7	14.5	13.8	14.6	18.8
627	J-83	579,025.6	1,225,883.4	1,843.5	93.0	27.3	56.1	43.5
629	J-84	579,010.1	1,225,801.2	1,843.6	93.0	27.0	56.0	42.6
631	J-85	578,809.6	1,225,841.3	1,846.5	90.6	23.9	53.1	38.8
633	J-86	578,801.6	1,225,779.8	1,846.5	90.8	23.8	53.1	38.6
635	J-87	578,587.7	1,225,811.9	1,869.3	68.6	1.1	48.4	16.3
652	J-88	577,863.2	1,226,150.4	1,865.5	77.2	33.4	48.9	49.2
657	J-89	578,473.4	1,226,465.6	1,854.0	83.9	17.6	59.1	19.6
661	J-90	578,863.0	1,226,078.2	1,847.1	89.5	23.1	52.5	38.2
690	J-91	577,497.2	1,226,741.6	1,890.0	56.1	49.0	56.4	47.7
692	J-92	577,567.0	1,226,856.5	1,883.3	62.7	54.4	53.0	53.4
695	J-93	577,430.2	1,226,645.8	1,909.0	37.1	30.7	37.5	32.9
700	J-94	577,620.1	1,227,039.3	1,878.6	67.4	57.6	57.7	49.5
717	J-95	581,078.7	1,225,648.4	1,900.2	12.3	12.1	12.3	21.1
722	J-96	577,503.3	1,224,681.8	1,895.0	51.8	21.1	54.2	31.5
726	J-97	577,544.7	1,224,495.7	1,897.4	49.5	19.6	51.7	32.7
768	J-106	581,200.7	1,223,225.8	1,854.0	125.5	73.4	43.5	34.8
775	J-108	581,101.4	1,225,060.0	1,872.0	40.5	35.1	40.1	36.1
788	J-110	580,611.8	1,225,454.4	1,841.3	76.5	38.9	58.0	51.8
813	J-113	580,494.7	1,227,598.2	1,860.2	45.1	-3.7	45.1	27.0
816	J-114	580,359.3	1,227,593.9	1,851.4	53.8	4.9	53.8	35.5
37	J-115	580,679.6	1,226,080.6	1,846.8	59.7	21.8	59.7	25.1
146	J-117	580,547.0	1,226,264.0	1,844.0	62.1	19.2	52.1	23.9
158	J-118	580,732.9	1,226,402.3	1,850.8	55.7	20.3	55.7	24.1
170	J-119	580,762.6	1,226,367.1	1,852.1	54.4	17.9	54.4	21.7
182	J-120	580,839.6	1,226,203.4	1,853.5	53.0	15.4	53.0	17.8
194	J-121	580,859.0	1,226,319.0	1,853.9	52.6	15.0	52.6	18.0
205	J-122	579,670.0	1,225,351.0	1,830.0	97.7	16.3	55.6	30.4
218	J-123	580,037.0	1,225,882.0	1,829.1	76.5	24.0	56.5	25.3
39	J-124	580,513.0	1,225,952.0	1,842.0	64.1	19.7	44.1	21.0
50	J-125	579,700.0	1,225,290.0	1,830.0	97.7	15.2	45.5	29.3
67	J-127	579,587.0	1,225,055.0	1,828.1	99.6	15.8	47.3	30.0
78	J-128	579,450.0	1,224,942.0	1,830.4	97.3	13.9	55.0	28.0
88	J-129	579,488.0	1,225,079.0	1,828.9	98.8	15.7	56.5	29.8

108	J-130	579,667.0	1,225,131.0	1,828.4	99.2	15.2	57.0	29.0
823	J-131	580,706.0	1,226,102.4	1,847.5	59.1	21.5	49.0	24.8
826	J-132	580,570.5	1,226,286.3	1,844.6	61.9	25.2	41.8	28.7
128	J-132(1)	579,754.2	1,224,955.7	1,825.4	102.2	16.2	60.0	30.0
830	J-133	580,787.5	1,226,329.3	1,853.8	52.7	15.3	52.7	19.0
136	J-133(1)	579,805.0	1,224,861.0	1,826.8	100.8	14.4	58.5	28.2
833	J-134	580,251.6	1,225,928.6	1,835.2	70.5	19.2	60.5	20.5
138	J-135	580,860.0	1,226,584.0	1,854.0	52.5	14.8	52.5	15.2
139	J-136	579,725.0	1,224,721.0	1,826.8	100.9	14.2	58.5	28.4
842	J-137	580,915.6	1,225,372.4	1,856.3	56.2	52.4	55.9	52.6
141	J-138	579,412.0	1,224,810.0	1,830.5	97.4	13.7	54.9	27.8
850	J-139	581,054.7	1,225,172.5	1,882.5	30.0	25.1	29.6	25.9
142	J-139(1)	579,161.0	1,224,882.0	1,837.4	95.7	15.5	56.3	28.4
853	J-140	581,222.8	1,224,932.8	1,881.1	31.4	24.7	30.9	26.1
143	J-140(1)	578,934.0	1,224,950.0	1,846.0	92.9	22.6	57.8	34.1
856	J-141	581,108.1	1,224,648.6	1,866.7	45.9	35.8	45.1	38.4
144	J-141(1)	578,617.0	1,225,115.0	1,867.3	72.4	4.0	59.8	17.5
145	J-142	578,555.0	1,224,944.0	1,866.9	74.7	12.9	49.3	22.8
859	J-142(1)	581,043.1	1,223,900.9	1,852.9	59.8	34.0	47.7	44.4
148	J-143	578,079.0	1,224,938.0	1,873.4	70.7	21.4	45.1	33.8
149	J-144	578,485.0	1,225,196.0	1,868.3	71.4	2.9	59.5	16.2
865	J-144(1)	580,579.8	1,224,577.3	1,834.0	78.9	53.0	56.5	63.7
150	J-145	578,497.0	1,225,311.0	1,869.5	70.2	1.8	58.3	20.0
869	J-145(1)	580,410.3	1,224,584.7	1,831.0	81.8	55.6	59.4	66.4
151	J-146	578,385.0	1,225,398.0	1,867.7	72.0	3.6	49.8	16.8
152	J-147	578,442.0	1,225,452.0	1,867.3	72.4	4.0	59.6	16.7
874	J-147(1)	580,643.1	1,223,881.2	1,833.8	78.9	47.9	56.4	59.0
153	J-148	579,614.0	1,227,454.0	1,845.0	60.2	-1.1	60.2	-1.9
877	J-148(1)	580,511.7	1,223,878.8	1,830.5	82.2	51.1	59.7	62.0
154	J-149	578,278.0	1,225,307.0	1,870.9	68.8	0.4	46.9	15.3
879	J-149(1)	580,499.5	1,224,075.9	1,831.7	81.0	50.0	48.5	61.0
155	J-150	578,316.0	1,225,388.0	1,870.7	69.1	0.8	57.1	19.9
156	J-151	578,312.0	1,225,438.0	1,871.0	68.9	1.2	56.8	16.9
157	J-152	578,217.0	1,225,429.8	1,863.7	77.7	17.2	47.0	20.5
886	J-152(1)	581,255.7	1,222,314.1	1,839.5	86.3	-57.8	57.6	31.3
160	J-153	578,195.2	1,225,708.8	1,872.4	68.8	7.4	55.6	16.8
888	J-153(1)	581,219.6	1,225,145.9	1,891.4	99.7	96.4	46.4	52.1
161	J-154	578,240.2	1,225,721.8	1,870.8	69.6	6.2	57.1	16.0

890	J-154(1)	581,335.5	1,224,964.0	1,885.6	105.1	99.5	51.6	50.6
162	J-155	578,250.0	1,225,627.0	1,870.4	68.8	2.0	47.4	17.3
892	J-155(1)	581,338.9	1,224,636.0	1,879.7	110.6	102.1	46.9	46.7
163	J-156	578,294.8	1,225,632.6	1,859.9	79.2	12.2	48.8	17.4
894	J-156(1)	581,284.1	1,224,584.3	1,876.9	113.4	104.4	59.6	47.7
164	J-157	578,566.6	1,225,670.0	1,868.0	70.5	2.4	59.7	16.0
896	J-157(1)	581,259.8	1,223,854.2	1,859.1	130.6	119.7	57.1	47.4
165	J-158	578,571.4	1,225,458.0	1,867.8	71.7	3.0	60.0	16.5
898	J-158(1)	581,145.4	1,223,843.1	1,860.3	129.4	118.5	55.9	46.1
902	J-159	581,277.6	1,224,371.9	1,867.0	123.1	113.3	49.1	31.5
168	J-160	577,795.0	1,225,158.0	1,879.6	65.1	21.2	46.5	28.2
905	J-160(1)	581,117.4	1,224,618.5	1,866.1	125.0	123.7	57.1	32.1
169	J-161	580,472.2	1,227,886.4	1,858.8	46.5	-2.7	55.7	39.8
908	J-161(1)	581,131.1	1,224,066.1	1,856.8	132.9	122.1	53.8	28.9
172	J-162	577,576.0	1,224,759.9	1,892.0	54.6	22.0	42.9	42.1
911	J-162(1)	581,438.6	1,221,904.8	1,831.1	96.5	-49.6	51.5	28.6
174	J-163	577,467.5	1,224,844.8	1,895.2	51.3	18.7	52.6	36.4
915	J-163(1)	581,386.6	1,222,129.5	1,833.9	97.2	-44.7	56.2	29.9
175	J-164	577,404.0	1,224,776.0	1,897.7	49.2	18.6	50.1	29.3
918	J-164(1)	581,339.9	1,222,132.4	1,834.4	92.2	-52.8	74.3	43.6
921	J-165	581,208.0	1,222,539.6	1,841.1	84.0	-58.6	56.6	33.4
177	J-166	577,351.0	1,224,707.0	1,899.1	48.1	20.2	54.5	35.6
924	J-166(1)	579,663.7	1,225,698.9	1,832.2	95.5	28.9	52.5	32.0
178	J-167	577,873.1	1,224,808.0	1,882.1	63.1	20.7	55.1	30.9
927	J-167(1)	579,587.9	1,225,512.5	1,831.5	96.2	21.2	54.7	33.8
179	J-168	577,579.0	1,224,528.0	1,896.6	50.2	18.8	55.4	29.6
930	J-168(1)	579,553.3	1,225,321.8	1,830.5	97.2	16.7	48.6	29.2
180	J-169	577,609.0	1,224,427.0	1,894.4	52.4	20.9	52.1	29.4
933	J-169(1)	579,507.2	1,225,182.2	1,830.1	97.6	15.5	52.0	69.6
936	J-170	579,713.9	1,225,018.8	1,826.8	100.8	15.3	45.7	40.7
939	J-171	578,775.7	1,225,029.9	1,851.6	87.6	18.0	49.6	29.6
185	J-171(1)	580,773.1	1,225,340.7	1,839.0	74.1	55.2	40.0	31.8
186	J-172	577,814.2	1,224,465.8	1,883.2	63.1	27.7	57.1	45.8
942	J-172(1)	578,443.4	1,224,944.3	1,867.8	74.9	18.3	32.0	27.9
945	J-173	577,132.6	1,224,433.3	1,910.1	39.6	34.8	46.9	51.6
187	J-173(1)	580,638.0	1,225,602.0	1,841.2	79.8	32.2	36.6	30.3
188	J-174	581,210.4	1,225,029.8	1,880.1	32.5	26.9	25.3	21.8
190	J-176	578,018.0	1,224,498.0	1,872.0	74.2	38.4	25.8	24.2

952	J-176(1)	577,035.6	1,224,542.0	1,913.5	36.4	32.5	46.6	25.7
191	J-177	581,242.0	1,225,542.0	1,934.5	34.4	32.6	45.1	43.3
192	J-178	581,448.0	1,225,373.0	1,966.3	25.4	25.1	44.9	67.1
957	J-178(1)	577,282.4	1,224,768.8	1,903.0	45.1	25.7	59.5	68.8
961	J-179	577,209.4	1,224,813.9	1,905.0	45.0	43.4	41.9	36.0
193	J-179(1)	580,784.0	1,224,982.0	1,836.0	76.9	56.2	44.0	45.2
196	J-180	580,572.0	1,224,968.0	1,831.0	82.2	56.9	58.8	50.4
197	J-181	577,788.0	1,224,591.0	1,887.0	59.2	23.2	40.9	29.6
965	J-181(1)	577,789.9	1,225,355.8	1,875.3	71.3	41.9	55.4	25.5
968	J-182	577,458.5	1,225,837.8	1,881.7	64.4	40.4	52.5	26.4
198	J-182(1)	581,110.6	1,224,383.5	1,870.2	42.5	23.9	57.0	19.2
971	J-183	578,053.5	1,225,628.1	1,863.0	79.8	27.6	55.8	57.7
199	J-183(1)	581,181.6	1,222,684.2	1,845.0	79.7	-61.6	49.9	20.0
974	J-184	578,208.5	1,225,536.3	1,871.1	70.1	8.8	50.1	51.7
200	J-184(1)	580,788.0	1,222,710.0	1,823.4	98.7	-29.9	48.7	16.2
977	J-185	578,634.3	1,225,673.6	1,867.8	70.7	2.4	53.9	50.0
201	J-185(1)	580,663.0	1,222,888.0	1,831.0	88.8	-24.2	52.0	46.7
979	J-186	578,629.4	1,225,476.6	1,867.6	71.5	2.4	49.4	16.5
202	J-186(1)	580,673.0	1,223,174.0	1,830.0	87.0	-2.8	57.1	41.1
203	J-187	577,998.0	1,224,617.0	1,876.8	69.4	33.4	36.7	35.0
981	J-187(1)	578,631.6	1,225,635.0	1,868.3	70.2	1.9	57.4	46.8
204	J-188	581,057.3	1,223,766.9	1,853.2	59.5	30.2	55.1	31.9
206	J-189	576,991.5	1,224,599.5	1,913.5	36.6	35.0	44.1	55.8
992	J-190	579,434.8	1,225,819.8	1,834.8	96.4	31.5	41.2	30.3
207	J-190(1)	579,577.0	1,225,395.0	1,830.6	97.1	17.7	50.1	42.0
995	J-191	577,624.5	1,226,101.9	1,876.3	68.1	35.1	55.7	47.5
208	J-191(1)	577,288.0	1,224,257.0	1,908.8	40.7	34.3	54.4	41.5
998	J-192	578,658.1	1,225,799.9	1,849.4	88.4	21.0	46.2	34.1
1001	J-193	577,370.4	1,226,086.0	1,890.4	55.7	31.9	54.9	64.0
210	J-193(1)	579,687.0	1,225,777.0	1,830.9	96.9	32.8	55.2	20.1
1004	J-194	578,953.4	1,225,974.5	1,845.2	91.3	25.2	43.4	61.0
211	J-194(1)	581,034.0	1,224,381.0	1,864.5	48.2	23.9	49.8	23.4
1007	J-195	578,534.4	1,226,397.6	1,849.9	87.2	20.5	58.3	61.6
301	J-195(1)	578,486.0	1,223,786.0	1,875.0	74.5	68.1	59.2	21.3
1010	J-196	577,898.2	1,226,430.1	1,861.0	79.3	17.8	52.0	34.9
213	J-196(1)	580,744.0	1,224,516.0	1,837.2	75.5	51.0	60.0	23.9
1013	J-197	578,170.8	1,226,376.6	1,859.3	80.3	16.1	56.9	37.5
216	J-197(1)	580,648.0	1,224,082.0	1,832.0	80.7	50.8	56.3	26.7

1015	J-198	578,126.9	1,226,381.4	1,859.5	80.0	15.6	62.0	46.8
217	J-198(1)	581,203.0	1,222,707.0	1,845.0	110.4	3.5	59.2	20.9
1019	J-199	578,148.8	1,226,269.5	1,859.0	80.6	16.9	56.9	29.5
219	J-199(1)	581,287.9	1,222,358.0	1,840.0	100.8	-28.7	55.8	44.7
1021	J-200	577,883.6	1,226,313.3	1,862.0	79.0	23.3	54.7	47.1
220	J-200(1)	580,933.2	1,222,688.6	1,836.0	87.9	-49.9	57.2	41.3
1025	J-201	578,161.0	1,226,568.4	1,865.0	75.3	13.5	48.0	47.4
1027	J-202	577,875.8	1,226,254.4	1,863.0	78.2	23.6	59.0	37.8
222	J-202(1)	581,481.4	1,221,904.5	1,831.0	98.2	-46.1	47.8	46.8
1035	J-203	577,364.9	1,226,988.5	1,891.7	54.3	45.8	59.8	36.7
223	J-203(1)	581,543.0	1,221,659.0	1,829.4	98.0	-52.4	55.7	26.7
1038	J-204	576,932.7	1,226,011.3	1,908.7	44.9	30.9	47.2	41.5
224	J-204(1)	581,738.0	1,221,218.0	1,827.4	99.8	-55.9	56.8	55.9
1041	J-205	575,851.3	1,225,482.5	1,978.3	67.5	62.6	59.8	49.7
225	J-205(1)	581,940.0	1,221,016.0	1,826.5	100.6	-57.0	60.0	2.1
1431	J-206	580,734.3	1,226,436.9	1,850.8	55.7	22.7	58.0	77.4
1434	J-207	580,425.8	1,226,774.3	1,839.1	67.2	40.9	8.3	19.4
1437	J-208	580,658.8	1,223,626.3	1,831.2	82.4	28.8	50.6	74.1
1440	J-209	580,806.9	1,225,821.0	1,851.3	59.9	48.9	45.6	63.8
41	J-210	581,407.1	1,225,396.9	1,959.9	6.4	6.3	54.8	88.1
1475	J-210	580,669.0	1,225,661.6	1,844.1	148.7	148.7	12.4	20.1
42	J-211	581,327.8	1,225,439.8	1,952.2	10.1	9.6	55.3	87.7
1477	J-211	581,226.0	1,225,681.8	1,922.3	70.6	70.6	14.9	18.6
43	J-212	581,259.0	1,225,276.0	1,926.4	65.0	64.3	44.8	65.7
1501	J-212	579,864.2	1,226,232.4	1,830.0	84.8	84.8	48.0	59.6
44	J-213	581,086.1	1,225,532.4	1,900.0	12.5	11.0	54.1	78.4
1503	J-213	580,033.2	1,225,852.8	1,828.9	85.3	85.3	55.7	66.5
45	J-214	581,089.1	1,225,410.8	1,897.5	15.0	12.5	56.0	68.1
1505	J-214	580,731.5	1,225,723.4	1,848.3	64.8	64.8	48.7	78.8
47	J-215	581,107.3	1,225,274.8	1,890.2	101.1	100.1	44.6	39.0
1509	J-215	579,931.2	1,226,883.6	1,836.8	74.1	74.1	28.7	129.5
48	J-216	581,252.0	1,224,963.0	1,882.5	108.8	107.6	48.1	56.1
1511	J-216	580,453.2	1,226,931.1	1,843.0	66.0	66.0	59.6	90.0
1515	J-217	578,001.9	1,226,723.3	1,867.6	78.7	78.7	47.1	42.0
51	J-218	581,128.4	1,224,372.9	1,871.6	118.1	107.4	41.0	34.5
1520	J-218	576,673.1	1,225,889.4	1,930.5	128.7	128.7	56.1	97.2
52	J-219	580,230.0	1,225,700.7	1,830.0	91.6	42.4	52.0	31.0
1522	J-219	576,079.9	1,225,551.2	1,968.4	89.6	89.6	55.0	85.8

53	J-220	577,891.0	1,224,477.0	1,881.8	64.5	28.9	46.3	45.0
54	J-221	577,749.0	1,224,689.0	1,887.9	58.2	21.6	45.0	45.3
1528	J-221	578,389.8	1,225,767.2	1,856.4	96.1	96.1	43.5	37.7
55	J-222	577,442.5	1,224,626.3	1,897.1	49.8	20.0	211.3	213.1
1530	J-222	577,891.6	1,225,680.3	1,867.0	85.0	85.0	54.0	38.4
57	J-224	577,865.0	1,225,428.0	1,873.0	72.7	39.8	206.8	208.6
1534	J-224	576,941.6	1,224,892.5	1,905.9	45.0	45.0	53.6	30.0
58	J-225	577,573.0	1,225,148.0	1,885.9	61.4	36.1	192.9	194.5
1559	J-225	578,406.1	1,226,544.5	1,852.3	211.3	211.3	193.2	194.6
59	J-226	577,507.0	1,225,219.0	1,885.4	62.0	36.8	149.2	150.2
1561	J-226	578,163.8	1,226,598.9	1,856.3	206.8	206.8	55.2	45.8
60	J-227	577,176.6	1,224,897.6	1,895.8	51.9	29.2	37.1	32.1
1565	J-228	577,902.4	1,226,632.4	1,869.8	192.9	192.9	56.6	50.5
1567	J-229	577,808.8	1,226,227.7	1,868.6	193.2	193.2	50.9	16.8
1569	J-230	577,135.0	1,226,410.4	1,911.4	149.2	149.2	52.6	37.4
69	J-234	575,744.0	1,225,814.0	1,996.7	55.0	52.7	55.2	19.2
70	J-235	575,725.0	1,225,949.0	2,016.9	37.0	35.8	55.4	30.3
71	J-236	575,742.0	1,225,595.0	1,982.0	66.4	62.5	46.8	34.0
72	J-237	576,032.0	1,225,440.0	1,971.8	60.2	52.4	47.3	44.9
73	J-238	576,399.0	1,225,281.0	1,943.5	88.4	80.6	60.1	42.5
74	J-239	576,205.0	1,225,556.0	1,963.0	54.0	44.2	43.1	55.7
75	J-240	576,480.0	1,225,601.0	1,943.3	53.6	41.3	48.4	51.8
76	J-241	576,707.0	1,225,813.0	1,930.6	44.4	30.4	51.2	56.1
77	J-242	577,019.2	1,226,070.2	1,908.0	44.2	30.3	59.3	49.8
79	J-243	577,393.4	1,226,132.0	1,890.5	55.6	32.2	32.5	28.7
80	J-244	577,554.4	1,226,112.9	1,877.4	68.7	45.4	26.5	23.6
81	J-245	577,768.0	1,225,578.0	1,871.4	74.5	44.7	49.5	54.5
83	J-246	577,567.0	1,226,194.0	1,879.4	66.9	45.5	54.0	17.6
84	J-247	577,390.0	1,226,217.0	1,891.6	55.7	40.6	59.1	19.6
85	J-248	577,379.9	1,226,468.9	1,915.6	31.2	24.4	50.1	35.8
86	J-249	577,385.2	1,226,563.5	1,920.9	25.6	20.5	48.3	33.4
87	J-250	577,859.6	1,226,065.8	1,870.7	72.0	30.1	60.5	47.6
89	J-251	577,927.7	1,226,597.6	1,860.5	79.8	18.2	26.0	24.2
90	J-252	578,434.5	1,226,437.9	1,855.0	82.9	16.7	37.7	33.8
93	J-255	578,675.0	1,226,267.0	1,849.6	87.2	20.6	43.8	39.0
94	J-256	578,638.0	1,226,135.0	1,851.5	85.4	18.7	45.7	40.3
95	J-257	578,604.0	1,225,940.0	1,851.7	87.4	31.3	53.2	48.0
96	J-258	578,044.0	1,226,024.0	1,862.2	79.6	33.8	43.7	37.3

97	J-259	577,314.0	1,226,643.0	1,920.6	25.7	22.4	39.0	32.1
98	J-260	577,336.0	1,226,697.0	1,908.8	37.4	31.8	51.2	43.9
99	J-261	577,230.8	1,226,837.3	1,902.7	43.4	37.3	56.1	36.7
100	J-262	577,260.5	1,226,882.7	1,900.8	45.3	38.6	41.1	47.9
101	J-263	577,410.8	1,226,801.7	1,893.2	52.8	46.1	43.1	49.4
102	J-264	577,183.0	1,226,944.0	1,902.7	43.4	35.7	54.5	48.4
103	J-265	576,944.0	1,227,093.0	1,907.4	38.6	30.4	45.8	38.0
104	J-266	577,262.0	1,227,053.0	1,895.2	50.8	42.6	46.8	38.7
40	J-267	582,355.0	1,219,092.0	1,820.0	106.9	-54.9	52.3	53.7
105	J-267(1)	577,478.0	1,226,918.0	1,885.2	60.8	51.9	57.3	49.8
106	J-268	577,506.3	1,226,959.5	1,883.2	62.7	53.5	58.4	19.3
107	J-269	577,557.0	1,226,930.0	1,881.8	64.2	54.7	42.6	36.8
109	J-270	577,001.0	1,227,183.0	1,900.6	45.4	37.2	51.5	20.6
110	J-271	577,073.0	1,227,142.0	1,899.6	46.4	38.1	40.3	29.0
111	J-272	577,156.0	1,227,279.0	1,884.0	62.0	53.6	53.2	38.3
112	J-273	577,338.0	1,226,182.0	1,893.6	54.0	41.2	58.5	29.8
113	J-274	578,144.0	1,226,488.0	1,859.8	78.7	12.9	54.1	48.0
114	J-275	577,229.0	1,226,338.0	1,906.9	40.3	30.3	59.1	54.2
836	J-275(1)	580,236.6	1,225,869.8	1,834.1	71.5	19.3	45.2	53.5
137	J-276	579,937.0	1,224,641.0	1,825.0	102.6	15.9	49.5	52.5
116	J-277	578,979.0	1,226,172.0	1,846.5	90.1	23.7	51.8	39.1
119	J-279	578,345.0	1,224,945.0	1,869.8	73.2	17.7	41.0	56.4
120	J-280	578,298.0	1,225,985.0	1,858.2	82.4	31.7	45.3	65.2
121	J-281	579,984.6	1,225,744.2	1,828.3	96.1	40.0	52.1	35.0
122	J-282	580,428.0	1,225,652.0	1,833.0	88.3	39.9	44.1	34.8
125	J-285	580,881.0	1,223,757.0	1,840.7	72.0	41.4	51.2	35.3
130	J-289	581,025.0	1,224,198.0	1,858.9	53.9	28.9	45.5	21.6
131	J-290	580,674.7	1,224,327.4	1,839.5	73.2	45.9	53.0	46.0
133	J-292	580,785.0	1,224,803.0	1,835.4	77.4	54.7	51.0	55.0
230	J-294	578,828.0	1,225,919.0	1,846.5	91.2	29.8	49.3	29.3
227	J-295	582,184.0	1,219,952.0	1,824.0	102.9	-58.7	40.5	18.6
228	J-296	582,186.0	1,219,508.0	1,822.0	104.9	-56.9	56.1	15.4
226	J-297	582,196.0	1,220,502.0	1,825.0	102.0	-58.4	42.3	19.3
117	J-299	579,928.0	1,226,298.0	1,830.0	75.5	20.5	54.0	20.0

Annex J: Pipe velocity Result; during minimum and peak hour demand

Label	Start Node	Stop Node	Material	Hazen-Williams C	Diameter (mm)	Length (m)	Velocity (m/s)	
							MHD	PHD
P-1	T-1	J-178	HDPE	150	250	99	2.1	2.5
P-2	J-178	J-212	HDPE	150	225	216	0.5	0.9
P-3	J-212	J-215	HDPE	150	225	156	0.5	0.9
P-4	J-215	J-6	HDPE	150	225	235	0.1	0.4
P-5	J-6	J-216	HDPE	150	225	110	0.1	0.4
P-6	J-216	J-7	HDPE	150	225	208	0.1	0.4
P-7	J-7	J-160(1)	HDPE	150	225	177	0.1	0.4
P-8	J-198(1)	J-199(1)	GI	66	80	363	0.8	1.4
P-9	J-199(1)	J-163(1)	GI	66	80	249	0.8	1.3
P-11	J-203(1)	J-204(1)	HDPE	150	80	482	0.2	1
P-12	J-204(1)	J-205(1)	HDPE	150	80	292	0.1	0.8
P-13	J-205(1)	J-297	HDPE	150	80	586	0.1	0.7
P-14	J-297	J-295	HDPE	150	80	550	0.1	0.4
P-15	J-295	J-296	HDPE	150	80	444	0	0.2
P-16	J-296	J-267	HDPE	150	80	449	0	0.1
P-17	J-178	J-210	GI	66	150	50	4.9	4.9
P-18	J-210	T-2	HDPE	150	150	56	4.9	4.9
P-21	J-211	J-11	HDPE	150	80	59	2.2	1.9
P-22	J-11	J-177	HDPE	150	80	85	2.2	2
P-23	J-177	J-14	HDPE	150	80	133	2.2	2.1
P-25	J-13	J-14	HDPE	150	80	194	0	0.1
P-26	J-14	J-215	HDPE	150	80	327	2.2	2.3
P-27	J-213	J-214	HDPE	150	80	122	0.1	0.9
P-29	T-3	J-95	DCI	110	250	42	0	1.2
P-32	J-15	J-16	DCI	110	250	91	0.1	0.9
P-33	J-16	J-17	DCI	110	250	95	0.1	0.9
P-34	J-15	J-19	PVC	144	80	109	0.6	2
P-35	J-19	J-20	PVC	144	80	110	0.5	1.7
P-36	J-20	J-209	PVC	144	80	69	0.5	1.6
P-37	J-21	J-22	GI	66	80	102	0.5	1.3
P-38	J-22	J-115	GI	66	80	91	0.4	1.1
P-39	J-115	J-131	GI	66	80	34	0.1	0.4
P-40	J-133	J-119	GI	66	80	45	0.1	0.6
P-41	J-119	J-118	GI	66	80	46	0.1	0.7

P-42	J-118	J-206	GI	66	80	35	0.1	1.5
P-43	J-23	J-24	PVC	130	80	140	0.3	2.7
P-44	J-24	J-25	PVC	130	100	221	0.2	1.8
P-46	J-26	J-27	PVC	130	90	124	0.3	2.1
P-47	J-27	J-28	PVC	130	150	98	0.3	2.3
P-48	J-28	J-29	PVC	130	150	99	0.4	2.4
P-49	J-26	J-30	PVC	130	90	244	0.2	1.3
P-50	J-30	J-31	GI	66	80	71	0.2	1.1
P-51	J-31	J-207	GI	66	80	48	0.1	1
P-52	J-32	J-33	GI	66	80	178	0.2	1.4
P-53	J-33	J-34	GI	66	80	120	0	0.5
P-54	J-34	J-35	GI	66	80	194	0.1	0.6
P-55	J-35	J-36	GI	66	80	266	0.1	0.8
P-56	J-36	J-23	GI	66	80	126	0.1	0.9
P-57	J-115	J-117	GI	66	80	226	0.1	0.6
P-58	J-117	J-35	GI	66	80	165	0.1	0.2
P-59	J-35	J-37	GI	66	80	218	0	0.2
P-60	J-37	J-1	GI	66	80	152	0	0.1
P-61	J-1	J-124	GI	66	80	230	0	0.2
P-62	J-124	J-115	GI	66	80	211	0.2	0.8
P-64	J-121	J-135	GI	90	90	265	0	0.1
P-65	J-173(1)	J-282	PVC	130	150	216	0.5	0.6
P-66	J-282	J-219	PVC	130	150	204	0.5	0.6
P-71	J-173(1)	J-110	PVC	130	80	150	1.6	2.3
P-72	J-110	J-38	PVC	130	80	137	1.6	2.4
P-73	J-38	J-171(1)	HDPE	150	80	184	0.9	2
P-76	J-17	J-39	DCI	110	250	59	0.1	0.9
P-77	J-39	J-18	DCI	110	200	74	0.1	1.4
P-79	J-108	J-179(1)	PVC	130	80	329	0.3	1.9
P-80	J-179(1)	J-180	PVC	130	80	213	0.3	1.3
P-85	J-174	J-140	HDPE	150	125	101	0.1	1.4
P-86	J-40	J-141	HDPE	150	125	148	0.1	1.2
P-87	J-182(1)	J-194(1)	HDPE	150	80	77	0.3	2.8
P-88	J-196(1)	J-292	PVC	130	100	298	0.2	0.8
P-89	J-292	J-179(1)	PVC	130	100	179	0.2	1.1
P-92	J-289	J-142(1)	HDPE	150	125	298	0	0.6
P-93	J-188	J-285	PVC	130	100	177	0.1	0.8
P-94	J-285	J-41	PVC	130	100	225	0.1	0.7

P-95	J-147(1)	J-197(1)	PVC	130	100	201	0	0.7
P-96	J-197(1)	J-290	PVC	130	100	248	0	1
P-105	J-186(1)	J-185(1)	GI	66	80	286	0.4	1.1
P-106	J-185(1)	J-184(1)	GI	66	80	218	0.4	1.1
P-107	J-184(1)	J-200(1)	GI	66	80	150	0.4	0.9
P-108	J-200(1)	J-183(1)	HDPE	150	80	249	0.4	0.9
P-114	J-43	J-124	GI	66	80	90	0.1	0.7
P-115	J-43	J-134	GI	66	80	190	0.1	0.6
P-116	J-123	J-44	GI	66	80	266	0.1	0.3
P-117	J-44	J-299	GI	66	80	164	0	0.2
P-118	J-299	J-45	GI	66	80	189	0.1	0.5
P-119	J-45	J-46	GI	66	80	163	0	0.2
P-120	J-46	J-123	GI	66	80	261	0	0.1
P-121	J-45	J-47	HDPE	150	80	140	0.1	0.9
P-122	J-299	J-48	GI	66	80	326	0.1	0.6
P-123	J-48	J-49	GI	66	80	519	0.1	0.4
P-124	J-49	J-50	HDPE	150	80	76	0	0
P-125	J-50	J-51	HDPE	150	80	134	0	0.2
P-126	J-51	J-52	HDPE	150	80	62	0.1	0.4
P-127	J-52	J-53	GI	66	80	201	0.1	0.6
P-128	J-53	J-33	GI	66	80	347	0.1	0.8
P-129	J-32	J-54	HDPE	158	80	435	0.1	0.7
P-130	J-54	J-55	GI	66	80	89	0.1	0.8
P-131	J-55	J-30	GI	66	80	353	0.1	0.4
P-132	J-55	J-56	GI	66	80	241	0.1	0.6
P-133	J-56	J-26	PVC	150	80	354	0.1	0.8
P-134	J-27	J-57	HDPE	150	100	241	0.3	1.5
P-135	J-57	J-58	HDPE	150	100	285	0.2	1.4
P-138	J-60	J-61	HDPE	150	80	77	0.1	0.5
P-139	J-61	J-69	HDPE	150	80	126	0.1	0.4
P-140	J-69	J-67	HDPE	150	80	135	0.1	0.3
P-141	J-67	J-62	HDPE	150	80	133	0	0.2
P-142	J-62	J-63	HDPE	150	80	80	0	0
P-143	J-113	J-60	HDPE	150	80	127	0.1	0.6
P-144	J-60	J-64	HDPE	150	90	126	0.1	0.6
P-145	J-64	J-70	HDPE	150	80	129	0	0.2
P-146	J-70	J-68	HDPE	150	80	142	0	0.3
P-147	J-68	J-65	HDPE	150	80	128	0	0.2

P-148	J-65	J-63	HDPE	150	80	118	0	0
P-149	J-64	J-161	HDPE	150	80	222	0.1	0.4
P-150	J-161	J-66	HDPE	150	80	162	0	0.2
P-151	J-67	J-114	HDPE	150	80	80	0	0
P-152	J-114	J-68	HDPE	150	80	125	0	0.1
P-153	J-25	J-27	PVC	173	100	253	0.2	1.9
P-154	J-49	J-148	HDPE	150	80	365	0	0.2
P-155	J-219	J-281	PVC	130	100	249	1	1.3
P-156	J-281	J-193(1)	PVC	130	100	299	1.1	1.3
P-157	J-193(1)	J-166(1)	PVC	130	100	82	0.1	2.2
P-159	J-190(1)	J-168(1)	PVC	130	100	77	0	1.2
P-161	J-129	J-128	PVC	130	100	142	0.2	0.4
P-162	J-128	J-138	PVC	130	100	137	0.2	0.3
P-165	J-136	J-276	PVC	130	80	227	0.1	0.2
P-166	J-276	J-72	PVC	130	80	205	0	0.2
P-167	J-72	J-133(1)	PVC	130	80	82	0	0.4
P-168	J-133(1)	J-132(1)	PVC	130	80	107	0	0.5
P-169	J-170	J-130	PVC	130	80	122	0.1	1
P-172	J-125	J-122	PVC	130	80	68	0.1	1.1
P-173	J-122	J-190(1)	PVC	130	80	103	0.1	1.2
P-174	J-127	J-129	PVC	130	80	102	0.2	0.7
P-175	J-127	J-73	PVC	130	80	73	0.1	0.5
P-176	J-73	J-130	PVC	130	80	60	0.1	0.2
P-177	J-138	J-139(1)	PVC	130	60	261	1.1	1.4
P-178	J-139(1)	J-140(1)	PVC	130	60	237	1.2	2
P-180	J-141(1)	J-74	GI	66	100	54	0.4	1
P-181	J-74	J-142	GI	66	100	147	0.4	1
P-182	J-142	J-172(1)	GI	66	100	112	0.5	1.2
P-183	J-279	J-143	HDPE	150	90	266	0.6	1.8
P-184	J-143	J-167	HDPE	150	90	244	0.7	1.9
P-186	J-144	J-145	GI	66	80	116	0	0.1
P-189	J-158	J-147	HDPE	150	80	130	0.3	0.6
P-190	J-147	J-151	HDPE	150	80	131	0.3	0.7
P-191	J-151	J-152	GI	66	80	96	0.5	1.3
P-192	J-152	J-75	HDPE	150	80	175	0.7	1.8
P-193	J-75	J-160	HDPE	150	80	363	0.7	1.9
P-194	J-144	J-76	GI	66	80	158	0	0
P-195	J-76	J-149	GI	66	80	77	0	0.1

P-196	J-149	J-150	GI	66	80	89	0.1	0.2
P-197	J-150	J-146	GI	66	80	70	0.1	0.2
P-198	J-146	J-147	HDPE	150	80	79	0	0.1
P-199	J-151	J-150	GI	66	80	50	0.2	0.5
P-200	J-167	J-221	HDPE	150	90	172	0.7	2.1
P-201	J-221	J-168	HDPE	150	90	234	0.5	1.7
P-202	J-168	J-97	HDPE	150	90	47	0.6	2
P-203	J-97	J-77	HDPE	150	90	155	0.9	2.9
P-204	J-168	J-169	HDPE	150	80	105	0.1	0.3
P-205	J-169	J-172	HDPE	150	80	209	0.4	1.4
P-206	J-172	J-220	HDPE	150	80	78	0.1	0.5
P-207	J-220	J-176	HDPE	150	80	129	0.1	0.4
P-208	J-176	J-187	HDPE	150	80	121	0.1	0.3
P-209	J-172	J-181	HDPE	150	80	128	0.2	0.7
P-210	J-181	J-221	HDPE	150	80	105	0.3	0.7
P-211	J-187	J-181	HDPE	150	80	212	0.1	0.1
P-212	J-77	J-191(1)	HDPE	150	90	196	0.9	3.1
P-213	J-191(1)	J-78	HDPE	150	80	594	0	0
P-214	J-78	J-195(1)	HDPE	150	80	694	0	0
P-215	J-222	J-166	HDPE	150	80	122	0.4	1.3
P-216	J-178(1)	J-2	HDPE	150	80	80	0.5	1.9
P-217	J-2	J-227	HDPE	150	80	88	0.3	0.9
P-219	J-222	J-96	HDPE	150	80	82	0.3	1
P-220	J-96	J-162	HDPE	137	80	107	0.4	1.2
P-221	J-162	J-79	HDPE	150	80	276	0.4	1.1
P-222	J-79	J-80	HDPE	150	80	159	0.3	1
P-223	J-163	J-164	HDPE	150	80	94	0.5	1.5
P-224	J-164	J-166	HDPE	150	80	87	0.6	1.9
P-225	J-163	J-80	HDPE	150	80	295	0.4	1.3
P-226	J-80	J-160	HDPE	150	80	158	0.8	2.2
P-227	J-160	J-81	HDPE	150	80	207	0	0.1
P-228	J-225	J-226	HDPE	150	80	97	0.2	0.5
P-229	J-226	J-227	HDPE	150	80	461	0.2	0.7
P-231	J-2	J-225	HDPE	150	80	461	0.3	0.9
P-232	J-189	J-82	HDPE	150	250	311	0.3	1
P-233	J-82	T-5	HDPE	150	250	249	0.3	1
P-235	T-6	J-235	HDPE	150	125	159	1.6	2
P-236	J-235	J-234	HDPE	150	125	138	1.6	2

P-237	J-234	J-236	HDPE	150	125	220	1.6	1.9
P-238	J-236	J-205	HDPE	150	125	169	1.6	1.9
P-239	J-237	J-238	HDPE	150	90	401	0	0.1
P-240	J-237	J-239	HDPE	150	90	208	3	3.2
P-241	J-239	J-240	HDPE	150	90	279	3	3.2
P-242	J-240	J-241	HDPE	150	90	311	2.9	3
P-243	J-204	J-242	HDPE	150	125	105	1.5	1.4
P-244	J-193(1)	J-190	PVC	130	100	256	1.2	1
P-246	J-83	J-84	PVC	130	80	84	0.5	0.4
P-247	J-84	J-85	PVC	130	80	204	0.5	0.3
P-248	J-85	J-86	PVC	130	80	62	0.5	0.2
P-249	J-86	J-192	PVC	130	80	145	0.5	0.1
P-250	J-87	J-157	HDPE	150	80	144	0.6	0.1
P-251	J-187(1)	J-186	GI	66	80	158	0.2	0.1
P-252	J-157	J-156	HDPE	150	80	275	0.4	0.7
P-253	J-156	J-155	HDPE	150	80	46	0.4	0.8
P-254	J-155	J-154	GI	66	80	95	0.5	0.9
P-255	J-154	J-153	GI	66	80	47	0.5	1
P-258	J-83	J-294	PVC	130	100	201	0.8	1.7
P-259	J-294	J-257	PVC	130	100	225	0.8	1.8
P-260	J-257	J-280	PVC	130	100	309	0.8	1.8
P-261	J-280	J-258	PVC	130	100	257	0.8	1.9
P-262	J-258	J-250	PVC	130	100	189	0.8	2
P-263	J-250	J-88	GI	66	90	85	0.2	0.7
P-264	J-200	J-196	GI	66	80	118	0.3	1
P-266	J-274	J-252	HDPE	150	80	295	0.4	0.5
P-267	J-252	J-89	HDPE	150	80	48	0.4	0.5
P-268	J-195	J-255	HDPE	150	80	192	0.3	0.3
P-271	J-194	J-83	HDPE	150	80	116	0.2	0.5
P-272	J-90	J-277	HDPE	150	80	149	0	0.2
P-276	J-255	J-256	HDPE	150	80	137	0	0.2
P-277	J-224	J-245	GI	66	80	179	0.1	0.5
P-278	J-245	J-5	GI	66	80	225	0.1	0.6
P-279	J-5	J-182	HDPE	150	90	269	0.1	0.5
P-280	J-250	J-191	PVC	130	100	238	1	2.7
P-281	J-244	J-243	HDPE	150	125	163	0.1	0.3
P-282	J-244	J-246	HDPE	150	125	82	0.7	2.1
P-283	J-246	J-88	GI	66	80	300	0.4	1.3

P-284	J-242	J-273	HDPE	150	125	338	1.5	1.3
P-285	J-273	J-247	HDPE	150	125	63	0.9	2.7
P-286	J-246	J-247	HDPE	150	125	179	0.9	2.7
P-287	J-273	J-275	HDPE	150	125	191	0.6	1.4
P-288	J-275	J-248	HDPE	150	125	200	0.6	1.5
P-289	J-248	J-249	HDPE	150	125	95	0.6	1.5
P-290	T-7	J-259	HDPE	150	125	86	0.4	2.6
P-291	J-259	J-260	GI	66	80	59	0.1	0.8
P-292	J-260	J-263	GI	66	80	130	0.1	0.4
P-293	J-263	J-91	GI	66	80	105	0	0.2
P-294	J-91	J-92	GI	66	80	135	0.1	0.4
P-295	J-92	J-267(1)	GI	66	80	108	0	0.3
P-296	J-260	J-93	GI	66	80	107	0.1	0.4
P-297	J-93	J-91	GI	66	80	118	0.1	0.3
P-298	J-267(1)	J-268	GI	66	80	50	0.1	0.4
P-299	J-268	J-269	GI	66	80	59	0	0.3
P-300	J-269	J-94	GI	66	80	126	0	0.1
P-301	J-259	J-261	HDPE	150	90	212	0.2	1.2
P-302	J-261	J-262	HDPE	150	90	54	0.2	1.1
P-303	J-262	J-263	GI	66	80	171	0	0
P-304	J-267(1)	J-203	GI	66	80	133	0	0.2
P-305	J-264	J-266	HDPE	150	80	135	0.1	0.6
P-306	J-262	J-264	HDPE	150	90	99	0.2	1
P-307	J-265	J-270	HDPE	150	80	107	0.1	0.3
P-308	J-270	J-271	HDPE	150	80	83	0	0.2
P-309	J-271	J-266	HDPE	150	80	209	0	0.1
P-310	J-271	J-272	HDPE	150	80	161	0	0.2
P-311	J-225	J-181(1)	HDPE	150	80	301	0.4	1.2
P-312	J-224	J-183	GI	66	80	275	0.4	1.2
P-313	J-184	J-152	GI	66	80	107	0.2	0.4
P-314	J-249	J-259	HDPE	150	125	107	0.6	1.5
P-315	J-47	J-34	HDPE	150	80	229	0.2	1
P-316	J-213	J-95	HDPE	150	80	116	0.1	1
P-317	J-162	J-163	HDPE	150	80	138	0	0.1
P-318	J-164	J-96	HDPE	150	80	137	0.1	0.3
P-319	J-97	J-169	HDPE	150	80	94	0.4	1.3
P-320	J-222	J-97	HDPE	150	80	166	0	0.2
P-321	J-189	J-176(1)	HDPE	150	150	72	0.8	2.7

P-325	J-194(1)	J-196(1)	PVC	130	150	326	0.1	0.3
P-334	J-218	J-161(1)	HDPE	150	225	307	0.1	0.3
P-338	J-108	J-174	PVC	130	200	116	0	0.5
P-340	J-8	J-9	HDPE	150	80	257	1.1	2.6
P-342	J-9	J-106	HDPE	150	80	301	1	2.1
P-343	J-106	J-10	GI	66	80	317	0.9	1.8
P-345	J-10	J-198(1)	GI	66	80	216	0.9	1.5
P-346	J-1	J-117	GI	66	80	217	0.1	0.3
P-347	J-132	J-118	HDPE	158	80	200	0	0.8
P-348	J-29	T-4	GI	66	200	169	0.2	1.4
P-349	J-113	J-114	HDPE	150	80	136	0.1	0.3
P-350	J-63	J-114	HDPE	150	80	136	0	0.2
P-351	J-70	J-113	HDPE	150	80	126	0	0.2
P-352	J-113	J-69	HDPE	150	80	78	0	0
P-353	J-131	J-132	PVC	130	80	229	0	0.6
P-356	J-18	J-137	DCI	137	200	164	0.1	1.3
P-357	J-137	J-214	HDPE	150	80	182	0.1	0.8
P-358	J-137	J-171(1)	HDPE	150	80	147	0.6	3.2
P-361	J-139	J-108	DCI	137	200	122	0.1	0.9
P-361	J-140	J-40	HDPE	150	125	175	0.1	1.3
P-362	J-141	J-182(1)	PVC	130	100	266	0.2	1.9
P-363	J-142(1)	J-188	PVC	130	80	135	0.1	1.4
P-365	J-196(1)	J-144(1)	GI	66	80	175	0.1	0.3
P-366	J-144(1)	J-145(1)	HDPE	158	80	170	0.1	0.4
P-367	J-148(1)	J-149(1)	HDPE	158	80	198	0	0.1
P-368	J-149(1)	J-197(1)	GI	72	80	149	0	0.4
P-369	J-41	J-147(1)	HDPE	150	80	114	0.1	0.8
P-370	J-147(1)	J-148(1)	HDPE	150	80	131	0.1	0
P-371	J-165	J-152(1)	HDPE	150	80	230	0.5	0.5
P-372	J-215	J-153(1)	HDPE	150	80	171	0.3	1.2
P-373	J-153(1)	J-154(1)	HDPE	150	80	216	0.3	1.1
P-374	J-154(1)	J-155(1)	HDPE	150	80	328	0.3	1
P-375	J-155(1)	J-156(1)	HDPE	150	80	75	0.3	0.8
P-376	J-156(1)	J-159	HDPE	150	80	213	0.2	0.6
P-377	J-159	J-157(1)	HDPE	150	80	518	0.2	0.5
P-378	J-157(1)	J-158(1)	HDPE	150	225	115	0	0
P-379	J-161(1)	J-158(1)	HDPE	150	225	223	0.1	0.3
P-380	J-160(1)	J-218	HDPE	150	90	246	0.8	2.3

P-381	J-158(1)	J-8	GI	72	80	73	1.1	2.7
P-382	J-162(1)	J-202(1)	GI	66	80	43	0.8	1.2
P-383	J-163(1)	J-202(1)	HDPE	150	80	244	0.8	1.3
P-384	J-152(1)	J-164(1)	HDPE	150	80	200	0.6	0.2
P-385	J-183(1)	J-165	HDPE	150	80	147	0.5	0.8
P-386	J-164(1)	J-162(1)	HDPE	150	80	248	0.6	0.1
P-387	J-162(1)	J-203(1)	PVC	130	80	267	0.2	1.1
P-390	J-167(1)	J-190(1)	PVC	130	100	118	0.1	2
P-391	J-169(1)	J-129	PVC	130	100	105	0.1	0.9
P-392	J-132(1)	J-170	PVC	130	80	75	0.1	0.7
P-393	J-140(1)	J-171	HDPE	150	100	177	0.4	0.8
P-394	J-172(1)	J-279	HDPE	150	100	98	0.5	1.3
P-395	J-173	J-191(1)	HDPE	150	150	235	0.3	1.2
P-396	J-166	J-178(1)	HDPE	150	80	92	1	3.3
P-397	J-176(1)	J-173	HDPE	150	150	146	0.3	1.2
P-398	J-176(1)	J-178(1)	HDPE	150	110	335	0.8	2.8
P-399	J-189	J-179	HDPE	150	200	306	0	0.1
P-400	J-3	J-179	GI	66	80	210	0	0.1
P-401	J-181(1)	J-224	GI	66	80	104	0.4	0.9
P-403	J-183	J-153	GI	66	80	163	0.4	1.1
P-404	J-153	J-184	HDPE	150	80	173	0.1	0
P-405	J-157	J-185	HDPE	150	80	68	0.2	0.5
P-406	J-185	J-187(1)	HDPE	150	80	39	0.2	0.2
P-407	J-146	J-145	HDPE	150	80	147	0	0.2
P-408	J-158	J-186	GI	66	80	61	0.3	0.5
P-412	J-191	J-244	HDPE	150	80	71	1.5	4.2
P-413	J-192	J-87	HDPE	150	90	74	0.4	0
P-414	J-182	J-193	HDPE	150	125	264	0.1	0.3
P-415	J-193	J-243	HDPE	150	80	51	0.1	0.8
P-416	J-90	J-194	HDPE	150	80	138	0.2	0.3
P-417	J-89	J-195	GI	66	80	91	0.4	0.4
P-418	J-196	J-251	GI	66	80	170	0	0.1
P-419	J-88	J-202	GI	66	90	105	0.5	1.6
P-420	J-198	J-197	GI	66	80	44	0.2	0.3
P-421	J-251	J-201	GI	66	80	235	0	0.1
P-422	J-196	J-198	GI	66	80	234	0.2	0.5
P-423	J-199	J-200	GI	66	80	269	0.3	0.8
P-424	J-197	J-199	HDPE	150	80	109	0.2	0.6

P-425	J-274	J-198	GI	66	80	108	0.4	0.6
P-426	J-202	J-200	HDPE	150	90	59	0.5	1.6
P-427	J-203	J-266	HDPE	150	80	121	0.1	0.4
P-428	J-264	J-265	HDPE	150	90	282	0.1	0.4
P-429	J-241	J-204	HDPE	150	90	305	2.9	2.9
P-430	J-205	J-237	GI	66	125	188	1.5	1.7
P-431	J-120	J-133	GI	66	80	136	0	0.1
P-432	J-131	J-120	GI	66	80	168	0	0
P-433	J-134	J-275(1)	GI	66	80	65	0.1	0.5
P-434	J-275(1)	J-123	PVC	130	80	200	0.1	0.4
P-435	J-206	J-23	GI	66	80	33	0.1	1.6
P-436	J-207	J-32	GI	66	80	45	0.1	0.9
P-437	J-208	J-41	GI	66	80	141	0.3	1.7
P-438	J-209	J-21	GI	66	80	152	0.5	1.4
P-452	J-166(1)	J-167(1)	PVC	130	100	201	0.1	2.1
P-453	J-168(1)	J-169(1)	PVC	130	100	147	0	1.1
P-454	J-125	J-130	PVC	130	80	162	0.1	0.9
P-455	J-138	J-136	PVC	130	80	325	0.2	0.9
P-456	J-141(1)	J-171	PVC	130	100	181	0.5	0.9
P-457	J-144	J-141(1)	GI	120	80	155	0	0.1
P-458	J-255	J-90	HDPE	150	80	266	0.3	0
P-459	J-60	J-58	HDPE	150	50	77	0.9	5.2
P-460	J-133	J-121	GI	150	80	72	0.1	0.3
P-461	J-15	J-95	DCI	137	250	172	0	1.1
P-462	J-137	J-139	DCI	130	200	245	0.1	0.9
P-463	J-38	J-180	HDPE	150	80	352	0.6	0.5
P-464	J-171(1)	J-179(1)	PVC	130	100	359	0.2	0.7
P-465	J-180	J-144(1)	HDPE	150	80	391	0.3	0.4
P-466	J-196(1)	J-290	PVC	144	100	201	0	1.2
P-467	J-194(1)	J-289	HDPE	150	125	183	0	0.7
P-468	J-208	J-186(1)	GI	72	80	453	0.4	1.3
P-469	J-211	T-2	HDPE	150	80	36	2.1	1.8
P-470	R-1	PMP-1	HDPE	150	315	210	0.3	0.3
P-471	PMP-1	J-210	HDPE	150	315	987	0.3	0.3
P-472	J-210	J-211	HDPE	150	315	563	0.3	0.3
P-473	J-211	T-1	HDPE	150	315	322	0.3	0.3
P-475	J-212	J-213	DCI	110	250	416	0.6	0.6
P-476	J-213	J-214	DCI	110	250	710	0.6	0.6

P-477	J-214	T-3	DCI	110	250	442	0.6	0.6
P-479	J-215	J-216	PVC	150	150	524	0.8	0.8
P-480	J-216	T-4	PVC	150	150	495	0.8	0.8
P-481(1)	R-4	PMP-4	PVC	150	300	229	0.3	0.3
P-481(2)	PMP-4	J-217	PVC	150	300	501	0.3	0.3
P-482	J-217	T-7	PVC	150	300	741	0.3	0.3
P-483(2)	J-230	J-218	DCI	110	200	697	0.6	0.6
P-484	J-218	J-219	DCI	110	200	684	0.6	0.6
P-485	J-219	T-6	DCI	110	200	846	0.6	0.6
P-488(1)	R-5	PMP-5	DCI	110	200	99	0.4	0.4
P-488(2)	PMP-5	J-221	DCI	110	200	745	0.4	0.4
P-489	J-221	J-222	DCI	110	200	506	0.4	0.4
P-491	J-222	J-224	DCI	110	200	1,235	0.4	0.4
P-492	J-224	T-5	DCI	110	200	711	0.4	0.4
P-493	R-3	PMP-3	PVC	130	150	58	0.8	0.8
P-494	PMP-3	J-215	PVC	130	150	471	0.8	0.8
P-495	R-2	PMP-2	DCI	110	250	48	0.6	0.6
P-496	PMP-2	J-212	DCI	110	250	495	0.6	0.6
P-497	R-6	PMP-6	DCI	110	200	72	0.6	0.6
P-498	PMP-6	J-225	DCI	110	200	321	0.6	0.6
P-499	J-225	J-226	DCI	110	200	248	0.6	0.6
P-501	J-226	J-228	DCI	110	200	264	0.6	0.6
P-502	J-228	J-229	DCI	110	200	415	0.6	0.6
P-503	J-229	J-230	DCI	110	200	699	0.6	0.6
P-504	J-83	J-190	PVC	130	100	414	1.2	1.1