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Hydraulic Performance Modeling and Improvements of Water Supply Distribution Systems

(In the case of Ginchi town water supply)

A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfillment of the Degree of Master of Science in Civil and Environmental Engineering.

(Major in Hydraulic Engineering)

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This is to certify that the thesis prepared by **Zerihun Abiyu**, entitled: **Hydraulic Performance Modeling and Its Improvements of Water Supply Distribution Systems** in case of Ginchi Town in partial fulfillment of the requirement for the degree of Master of Science (Hydraulic Engineering) compiles with the regulations of the university and meets the accepted standards with respect to originality and quality.

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DECLARATION

I hereby declare that this submission is my thesis work and it is not containing material previously published by another person nor which has been accepted for the award of any other academic degree of the university, except I have been used and where due acknowledgement has been made and cited in the reference part of this work.

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Abstract

Water Supply Distribution Systems Utilities are designed to deliver safe, reliable, affordable, and continuous supplying and distribution of potable water to end users throughout 24 hours. As time elapsed, meeting these commitments embraces multiple challenges in urban areas. The objectives of this work are to evaluate the hydraulic performance of water supply distribution system by assessing the situation of existing water supply distribution system. Also, it was mainly emphasized to estimate the physical water loss in the system to improve the efficacies of the water distribution systems networks to suggest some remedial measures. Water GEMS software was used as a tool to model water distribution system. The model can be used to identify the high pressure and low pressure in the junctions and magnitude of velocity and head loss through pipes were used as base to evaluate the hydraulic performance. Modeling results showed violation of maximum and minimum pressure and low velocity requirements. High pressures in the system occurred both during low demand and peak demand have to be identified and remedial measures is established using pressure reducing valves. It is also proposed to install air release valves at high elevation and sluice valves at low elevations. Modifications in operation and design will improve the current situation of the case study water supply distribution system.

Key Words: Water Distribution System, Water GEMS Modeling, Hydraulic Performance, Maximum pressure, Minimum pressure, Water Loss, Ginchi, Oromia, Ethiopia.

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Abbreviations and Acronyms

CSA - Central Statically Agency

CV- Check Valve

DEM - Digital Elevation Model

DCI - Ductile Cast Iron

DN- Nominal Diameter of Pipe

EPS - Extended-Period Simulation

GC - Gregorian calendar

GIS - Geographic Information System

GS - Galvanized Steel

GTWSSE- Ginchi Town Water Supply & Sewerage Enterprise

HDPE - High Density Polyethylene

HL - Head Loss

HGL – Hydraulic Gradient Line

HWC - Hazen –Williams Coefficient

KM - Kilo Meter

L – Length

L/S – Liter per Second

M – Meter

M^3/S – Meter cubic per Second

PN – Nominal Pressure

PVC - Polyvinyl Chloride Pipe

PRV - Pressure Reducing Valve

T-1 - Tank -1

UFW – Unaccounted for Water

WDN – Water Distribution Network

WDS - Water Distribution System

CHAPTER ONE

1. Introduction

1.1 Back Ground

In earlier times appearances of water distribution systems were not a complex network system. They were a system of single line or very few lines. As town grew to larger populations and much higher densities, there was a much greater need for water infrastructure.

This research focused on the hydraulic performance of water distribution network system at Ginch Town, West Shewa, Oromia, Ethiopia. The continuous and repeated deficiency in the performance of the Ginchi town water supply distribution networks became one of the most critical issues in the water supply sector that requires immediate action.

The Town Water supply distribution systems are designed to adequately satisfy the water requirements continuously for a combination of domestic, commercial, industrial, and firefighting purposes. The system should be capable of meeting the demands placed on it at all times and at satisfactory hydraulic performance. It should enable reliable operation during irregular situations and perform adequately under varying demand loads **(TAHAL, 2015).**

The majority of the WDS of the town were built decades ago; many are approaching or have exceeded their design lives. Thus analyzing the safe and secure operation of the old systems is crucial, particularly since performance has gradually declined and they require extensive upgrading. Many systems face aging problems over the long term operation and the challenges that come with the task of keeping their systems efficient. However, these needs far surpass the available resources **(EPA, 2002).** Also, some pipe network system in Ginchi town is served as over three decades. Population is growing rapidly with expansion of the town. Due to this hydraulic performance of water distribution system of the town facing some serious problems.

The above mentioned and poor water supply distribution system network with poor management way of operating the municipal water supply networks will affect the expected hydraulic performance of the network by affecting the pressure values and the velocities. It also increases pipes breakage rates.

Analysis of a pipe network system is essential to understand the hydraulic performance of Ginchi town water distribution systems network. In branched pipe network, the pipe discharges are unique and can be obtained simply by applying discharge continuity equations at all the nodes. However, in case of a looped pipe network, the number of pipes is too large to find the pipe discharges by merely applying discharge continuity equations at nodes. The analysis of looped network is carried out by using additional equations found from the fact that while traversing along a loop, as one reaches at the starting node, the net head loss is zero (Swamee et al. ,2008). Analyses of pipe system using Water GEMS models are a recent approach to evaluate the hydraulic performance of water distribution network.

Although much attention has been paid to the hydraulic performance assessment of water distribution systems, some important areas have not yet been investigated fully:

1. Various criteria in social and environmental costs resulting from the performance of water distribution systems.
2. The growing awareness of the quality of service requested by all customers and
3. Assessment of new approaches for improving performance of current operating systems to include more comprehensive public and economic issues.

However, measuring the hydraulic performance of a water distribution system as a multi-purpose system is not a straightforward task. Since it can be perceived from different viewpoints and related to a variety of parameters and properties of the system which are not usually quantifiable.

This study is to evaluate the hydraulic performance of the water distribution system network. To investigate the state of the existing water distribution system of Ginchi Town water supply distribution network system under varying conditions to suggest the remedial measure which support the sustainability of the town water supply systems. Rather, this research seeks to establish a broad perspective for the evaluation of the design and operation assessment of water distribution systems. The main goals are to challenge designers, and decision makers about those issues that have been often neglected in engineering practice, and also to test and verify new ideas for system design and operation.

1.2.Statement of the Problems

Currently Ginchi Town facing inadequacies and intermittent water distribution systems are the major challenge all over the town due to population growth and town expansion in all directions.

There is also high amount of leakage and pipe failure as well as provision of insufficient distribution. Hence, the water distribution system doesn`t provide the intended service effectively and equitably to its targeted community due to topographic condition of the town.

To propose appropriate recommendation, we have to study the hydraulic parameters, the variations, and the relations between them and other factors, which control the hydraulic performance of the water supply distribution networks. Also it is advisable to evaluate the effects of local conditions and improve them for increasing the efficiencies of the water distribution systems. So, in order to address the overall of Ginch Town water supply distribution problem and to give remedial measures, there is a need to evaluate the hydraulic performance of water distribution systems and to define the appropriate design requirements.

1.3.Research Questions

- What is the existing water supply situation in the town?
- What is the gap between water demand and available water supply?
- What are the key hydraulic parameters affecting the performance efficiency of water distribution network?
- What is the percentage of real water losses in the water system?
- What are the possible solutions to satisfy the demand?

1.4.Objective of the Study

1.4.1 General Objective

The main objective of the study is to analyses the hydraulic performance of Ginchi water supply distribution system network with its impacts, with considerations of the current practice to propose more efficient distribution system approaches.

1.4.2 Specific Objective

The specific objective of the study: -

- ☞ To evaluate the situation of existing water distribution system to increase the efficacies of the water distribution system network.
- ☞ To identify major factors that contributing for hydraulic performance deterioration of the distribution system network.
- ☞ To examine the levels of the physical water loss and apparent water losses in the system.
- ☞ To suggest the optimal solutions which support the sustainability of the town water distribution system?

1.5 Significance of the Study

The result of this research investigates the current situation of Ginchi Town water distribution network system. Also identify some of the present and the future problems. Based on findings, better system management has been proposed. Hopefully, the insights that can be draw from this study contribute to solve the hydraulic problems of Ginchi water distribution system and will imitate further research on similar other sites.

1.6 Structures of the Thesis

The thesis is organized into six parts. **The second** section discusses literature reviews related to water supply distribution network modeling and review others works in related to this thesis in both local and others. Determination of pipe roughness coefficients and nodal demands by calibrating a water distribution network model are discussed as well.

The third section presents the methodology used in data collection and preparation and analyzing modeling output for water distribution system performance used to reach at findings and conclusions. **The fourth section** discusses the results of the model outputs, analyzes water supply coverage, water loss, demand, gap identified in the system, calibration and validation, compare the results with actual value and proposed the feasible improving methods. **The fifth and sixth sections** deals with conclusion and recommendation of proposed solutions forwarding general approach GTWSSE has to follow to improve and design new and existing water supply based on results and findings of the study in the sub system distribution systems.

CHAPTER TWO

2. Literature Review

2.1 Introduction

Several researchers have been made to study the behavior of water distribution system, and to reach an optimal solutions and assumptions in order to improve the hydraulic performance, cost effective, and to increase the efficiencies of the water supply networks. Also provide here in details of the available hydraulic performance modeling technologies and software with model calibration and validation, their relevance, and details of their applicability in the GTWSSE situation. This chapter also gives the definitions for key terms adopted in the study and theoretical approaches about the main hydraulic network evaluation of governing parameters and conclusion.

2.2. Source of Water Supply Distribution Systems

GTWSSE use both surface water and ground water to supply water service. The case study suggests that ground water is preferable, because it requires less treatment and it is less subject to variations in terms of quality and quantity **(Wutich and Ragsdale, 2008).**Water that runs in streams or is found in depressions, such as rivers, lakes, man-made reservoirs, ponds or oceans is called surface water **(UN-HABITAT, 2003).**Ground water (wells), which indicate that the water is below ground level or below the earth`s surface. It is necessary to dig or bore a well in order to tap into the water source. This is also generally referred to as a “well-water source”. Interviewees and focus group discussants explained that compare to surface water, ground water has little organic matter (and hence has low turbidity) because of the fact that organic matters have been already filtered through natural physical parameters such as soil and rocks **(Gadgil, 1998; mersha, 2007).**

2.3 Water Supply System

Water supply distribution systems are designed and operated to provide water of a quality acceptable for human consumption. Another important factor is that in addition to providing drinking water, a major function of most distribution systems is to provide adequate standby fire flow. In order to satisfy these needs, most distribution systems use stand pipes, elevated tanks, storage reservoirs, and large sized pipes.

The effect of designing and operating a distribution system to maintain adequate fire flow and redundant capacity is that there are longer transit times between the treatment plant and the consumer than would otherwise be needed.

The distribution system is used to describe collectively the facilities used to supply water from its source to point of use. **Burchi and Andreas (2003)** define water distribution as the function of assigning water from a given source to given uses. The spatial demand and distribution of water resources vary depending on a number of factors such as willingness to pay, infrastructural capacity, modern technology distribution systems and components **(Warren, 1998).**

2.4 Types of Water Distribution System

There are two main layout of a distribution network: branched and looped **(walski et al., 2003).** Branched networks, or tree networks, are predominantly used to supply small areas, usually with few delivery points. For areas with many service points and high demand such as cities the looped configuration of the pipes is a more common feature of water distribution system. The loops provide alternative flow pathways; hence, consumers can be supplied from more than one direction. Looped networks can greatly improve the hydraulics of the distribution system in order to ensure the regularity of the water supply to the final customer. However, most of large distribution systems are essentially a combination of loops and branches with many interconnected pipes and valves. This is a result of a trade-off between loops for reliability and braches for infrastructure cost savings **(Walski et al., 2003).**

2.5 Water Demand

The amount of water needed for beneficial use is calculated based on a simple per capital taking into account the projected needs of the population, industrial, commercial and other uses supplied by the permit applicant. According to **UN-ESCAP, (2000),** Water demands are estimated using population projections, land use information and water production records. Historical population trends and projected growth patterns provide a comprehensive insight into the future water demands determined by the magnitude, direction and characteristic of population growth.

The growing population increases the demand of water for domestic use, food security and industrial development. The population growth trend has resulted in reduction of per capita water availability **(UN-WATER, 2006**). Water demand and use are directly related to the population. However, in rural areas, it is often difficult to estimate the population levels accurately due to lack of accurate and up to date census data; lack of up to date aerial photography or remote sensing data from which to estimate the number of settlements in an area and migratory labor with the male members of households often working in urban areas for long periods of time **(Wallingford, 2003).**

Population growth and economic development have placed stress on water resources **(Varma, 2010)** which has resulted in a decrease of per capita water availability. The international water management group **(UNFPA, 2007)** asserts that urbanization and industrialization which management group **(UNFPA, 2007)** asserts that urbanization and industrialization which commonly had high population densities has caused an adverse variability in quantity and quality of water resources therefore retards the demand and distribution of water **(WRI, 2007).**

The first consideration of a water distribution system is the determination of the quantity of water that will be required, with provision for the estimated requirements for the future. In terms of total quantity for domestic consumption, the water demand in a community usually is estimated on the basis of per capital demand. Water demand has been demonstrated: -

2.5.1 Average Daily Demand

This is the average of the total amount of water used each day during a 1-year period (usually expressed in cubic meter per day, **ADD**).

2.5.2 Maximum Daily Demand

This is maximum total amount of water used during 24-hour period in 3-year period. This number should consider and exclude any unusual and excessive identified used of water that would affect the calculation, such abnormal uses would include a water main break, a large scale fire, or an abnormal industrial demand. This is often referred to as the **MDD** rate **(Harry Hickey, 2008**).

2.5.3 Maximum Hourly Demand

This is the maximum amount of water used in any single hour, of any day, in 3-year period. It is normally expressed in cubic meter per day. It is determined in cubic meter per day by multiplying the peak hours by 24. This can also be express as **MHD(Harry Hickey, 2008).**

2.5.4 Peak Factor

When designing the distribution system hourly demand fluctuations must be considered. For example, during the night, people use less water, but in the morning and evening people use much more water. Peaking factors can be determined by dividing the maximum daily usage rate by the average daily usage rate as below.

Pf = Qmax/Qavg ………………………………………. (2.1)

Where, Pf = the peaking factor

QMax = the maximum daily demand, and

Qavg = the average daily demand.

This peaking factor can be applied to a system as a whole or specific peak factors can be developed and applied at a specific node **(Jeffrey and Gilbert, 2012)**.

If there are no recorded data's it is possible to determine peaking factors from average day to maximum day tend to range from 1.2 to 3.0 and factors from average day to peak hour are typically between 3.0 and 6.0. Of course, these values are system-specific, so they must be determined based on the demand characteristics of the system at hand (walski et al., 2003).

Peak factors for a water distribution designing can also estimated from the ratio of peak hourly demand on a maximum demand day during the year over the average hourly demand over the same period (Swamee, 1940).

Peak day and Peak hour factors:

Peak day factor

1.5 For population over 10,000

2 for population below 2000

Peak hour factor /peak factor

2 for population over 10,000

5 for population below 2000

2.5. 5. Fire flow demand

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

According to the ISO, the minimum creditable water supply is 250gpm (**56.78 m3/hr.)** for 2 hours or a total water supply of 30,000 gallons (**113.56 m3).** Most residential occupancies have a minimum water requirement of 500gpm (**113.56 m3/hr.),** and commercial properties can range up to 12,000gpm **(2,725.5 m3/hr.)** for 4 hours.

In similar way,Ysusi, (2000) fire flow requirements for different land uses, quantity of water and duration of firefighting events are listed in table below.

Table 2-1: Typical fire flow requirements (Ysusi, 2000)

Land use	Fire flows (LPS)	Durations (hr)
Industrial	350	4
High density area	320	4
Commercial	250	3
Multiple Family	220	2
Residential	125	2
Others	65	2

2.6 Hydraulic Modeling

Model-based simulation is a method for mathematically approximating the behavior of real water distribution systems. To effectively utilize the capabilities of distribution system simulation software and interpret the results produced, the engineer or modeler must understand the mathematical principles involved.

Hydraulic modeling functionality has therefore become essential in the global water industry and is now an integral part of most water system design, Master planning and fire flow analysis, particularly in the developed world **(walski et al. 2003).**To solve hydraulic system problems, there must be one equation for each pipe, pump, and valve, or for each junction, depending on the method used to solve for the unknowns in the hydraulic calculations. The number of equations that must be set up and solved in system hydraulics problem is very large, even for the most basic water distribution system. The value of a computer model is that tedious calculations are performed much more quickly and more accurately than manual calculations. In addition, the computer is an effective means of managing large amounts of data necessary to analyze a water distribution system. By using computer models, rather than focusing on the procedural mechanics of solving system equations, decision makers can focus more on communicating modeling results and formulating and comparing system design alternatives. Computer models of water distribution systems are not an end in them but are tools to help managers, engineers, planners and operations staff. When properly implemented, models become an integral part of the decision-making process for planning, design and operation of water distribution systems. Engineers and operators of a water system are still ultimately responsible for decision based on the results that computer models provide **(AWWA, 2012**).

2.6.1 Types of water Model Analysis

2.6.2.1 Steady-State Analyses: A steady state analysis provides a "**snapshot"** of pipe system conditions at any instant in time. Steady-state analyses are typically used to evaluate maximum day, peak hour, and fire flow conditions **(AWWA, 2012**).

2.6.2.2 Extended-Period Simulation: An extended period simulation is a series of steady-state simulations at specified intervals performed over a specified time period. This capability may be used, for instance, to model the operation of a water system over a 24-hour period with an analysis run for each hour **(AWWA, 2012**). Such a simulation is useful in modeling variations in demand, reservoir operations, water quality, and water transfers through transmission pipelines. Extended-period simulation requires that the system package model flow and pressure switches incorporate demand diurnal patterns for nodes and allow for varying tank configurations.

2.6.2 Principles of pipe network hydraulics

2.6.2.1 Conservation of mass

'The principle of conservation of mass dictates that the fluid mass entering any pipe will be equal to the mass leaving the pipe (since fluid is typically neither created nor destroyed in hydraulic systems)'. In network modeling, all out flows are lumped at the nodes or junctions` **(Walski et al., Thomas M., Chase, Donald V., and Savic , Dragan A., 2001).**

$$\sum_{Pipes} Q_i - U = 0 \quad \text{.. (2.2)}$$

Where, Qi =Inflow to node in i-th pipe (L^3/T)

U= Water used at node (L3/T)

Note that for pipe outflows from the node, the sign of Q is negative.

The term for accumulation of water at nodes is required to describe stored and withdrawn water from tanks. While extended period simulation is regarded **(Walski et al., Thomas , Chase, Donald , and Savic, Dragan , 2001).**

$$\sum_{pipes} Q_i - U - \frac{ds}{dt} = 0 \quad \text{.. (2.3)}$$

Where, ds/dt = changes in storage (L^3/T)

The conservation of mass equation is applied to all junction nodes and tanks in network, and one equation is written for each of them.

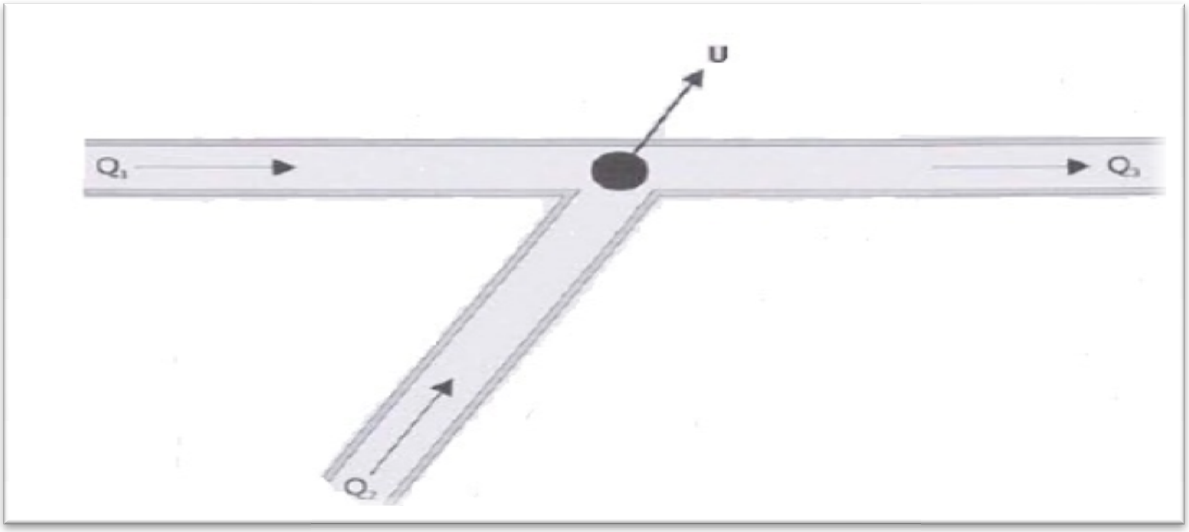

Figure 2-1 Conservation of mass principle

2.6.2.2 Conservation of energy

'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' (**Bernoulli, 1738 cited in (Walski et al., Thomas, Chase, Donald , and Savic, Dragan , 2001).**

The equation for conservation of energy is written it terms of head as follows:

$$Z1+\frac{P\,1}{\gamma}+\frac{(V1)2}{2g}+\sum hp = Z2+\frac{P2}{\gamma}+\frac{(V2)2}{2g}+\sum hL+\sum hm\text{........... }(2.4)$$

Where, Z = Elevation (L)

P = Pressure (M/L/T^2)

γ = Fluid specific weight (M/L/T^2)

V = Velocity (L/T)

g = gravitational acceleration constant (L/T^2)

h_p = head added at pump(L)

h_L= Head loss in pipes(L)

h_m = head loss due to minor losses(L)

Therefore, in connected network the difference in energy at any two point is equal to the energy increases from pumps and energy losses in pipes (frictional head loss) as well as energy losses in bending and fittings (minor head loss) that occur in the path between them.

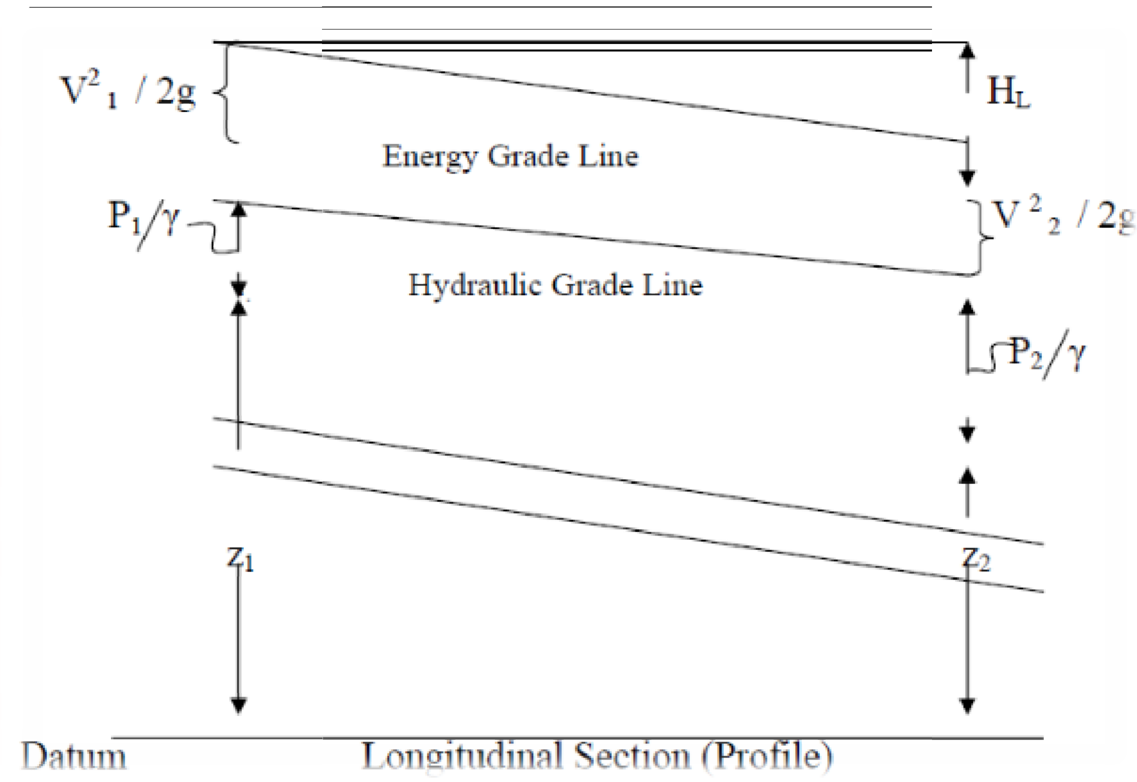

Figure 2-2: Energy principle

2.6.2.3 Energy losses

2.6.2.3.1 Friction head losses

In literature several equations are used to describe the friction head loss along pipe. The most popular formulas amongst practitioners are: Hazen-Williams formula, Manning`s formula, Darcy-weisbach`s formula and Colebrook-white formula. Note that through this work the Hazen –Williams and Darcy-Weisbach head loss expressions are employed as they are the predominant formulas used by practitioners.

Hazen-Williams equation

The Hazen–Williams formula uses a dimensionless roughness coefficient of pipe, denoted $\mathbf{C_{HW}}$. Higher values of $\mathbf{C_{HW}}$ represent smoother pipes and lower values of $\mathbf{C_{HW}}$ describe rougher pipes, **(walski et al., 2003).**

For Hydraulic calculation:

The Hazen-Williams formula is best for situations involving pressure conduits. The formula is:

$V= 0.84935\ C\ R^{0.63}\ I^{0.54}$ …………………………………………………………… (2.5)

For circular conduits, the formula is restated as

$hf = 10.666\ C^{-1.85}\ D^{-4.87}\ Q^{1.85}\ L$……………………………………………….(2.6)

Where, V=Velocity (m/s)

C= Hazen-Williams coefficient

R= Hydraulic Radius (m)

I = Hydraulic Gradient, hf/L

hf = Friction Head Loss (m)

D =Diameter of pipe (m)

Q= Discharge (m3/s)

L= Pipe Length (m)

For Hazen –Williams Coefficient (C value):

The manual recommends that the Hazen-Williams coefficient (C value) for new pipes made from cast iron, ductile iron or mild steel with cement mortar lining should be between 130 and 145. However, it is generally recommended that in the absence of specific data, a C value of 110 should be adopted. Therefore, a C value of 110 was adopted when designing the transmission system, including the existing pipelines (CPHEEO, MOUD, 1999).

2.6.2.4 Software for Hydraulic Analysis:

The hydraulic analysis using a computer software program called Water CADv6.5 works for a system networks below 500 pipes, which runs under the Auto CAD environment, Haestad Methods Inc.

2.7 Hydraulic design parameters

2. 7.1 Pressure

Pressure can be thought as a force applied normal, or perpendicular to a body that is in contact with a fluid. Pressure expressed:

$P=h\gamma$ ……………………………………………………………………… (2.7)

Where, P = Pressure ($M/L/T^2$)

h=depth of fluid above datum (L)

γ = fluid specific weight ($M/L^2/T^2$)

The quantity **P/γ** is called the **pressure head**, which is the energy resulting from water pressure **(Walski et al., Thomas , Chase, Donald , and Savic, Dragan , 2001).**

The pressure at nodes depends on the adopted minimum and maximum pressures within the network, topographic circumstances, and the size of the network.

The minimum pressure should be maintained to avoid water column separation and to ensure that consumer's demands are provided at all times. The maximum pressure constraints result from service performance requirements such fire needs or the pressure bearing capacity of pipes, also limit the leakage in the distribution system, especially that there is a direct relationship between the high pressure and the increasing of leakage value in the system.

The most current publication System Criteria set out by AWWA, the National Fire Protection Association (NFPA), the ISO and the Civil Engineering Handbook it should conform that:

- Normal minimum working pressure in the distribution system should be approximately **50 psi** and not less than **35 psi** during a maximum hour. A normal working pressure in most systems will vary between **50** and **56 psi.**
- System must be designed to maintain a minimum pressure of **20 psi** at ground level at all fire hydrants on the distribution system under required fire flow conditions.
- Maximum day demand is **1.5** time the average daily demand.

☞ Maximum hour demand is **2.25** times the average daily demand.

2.7.2 Flow rate

It is the quantity of water passes within a certain time through a certain section. Velocity is directly proportional to the flow rate. For a known pipe diameter and a known velocity, the flow rate through a section can be estimated. Low velocities affect the proper supply and will be undesirable for hygienic reasons (sediment formation may cause due to the longtime of retention).

2.7.3 Velocity

A velocity in the distribution network system is not lower than 0.6 m/s to prevent sedimentation and not be more than 3 m/s to prevent high head losses. In general, commonly used values are 1 - 1.5 m/s. A 1 m/s change in velocity corresponds to 100 m change in head **(Walski et al., Thomas, Chase, Donald, and Savic, Dragan , 2001).** The effect of the velocity on the diameters of pipe system can be observed from the following equation:

$$V = \frac{4Q}{\pi D^2} \quad \ldots\ldots (2.8)$$

$$D = \sqrt{4\; Q/\pi . V} \quad \ldots\ldots (2.9)$$

Where, D = Diameter of pipe (m)

Q = Discharge (m3/s)

V = Velocity (m/s)

From the above equation it is clear that the velocity increasing should decrease the diameter value.

Therefore, Designers need to determine the proper pipe size in order to meet peak demand and fire protection while maintaining an adequate dynamic pressure in the system **(Jeffrey and Gilbert, 2012)**.

2. 8 Model Calibration

Before a model is used, it must be adjusted to ensure that it will predict, with reasonable accuracy, the behavior of the system it models, i.e. it must be calibrated.

This is widely acknowledged by the research community and several studies on water distribution system calibration have been published in the past two decades **(Dragan , Savic Godfrey and Walters, 1995).**

The problem of WDS model calibration, even if only for water quantity, (Pressures and flows) is highly complex due to the large number of parameters examined and non-linear due to the flow equations. Several researchers have addressed this problem developing methods to minimize the difference between the values of the observed data and those computed by the network simulation model. These methods are based on the use of analytical equations, Simulation models and optimization techniques. Techniques based on analytical models may be applied to very small networks or may alternatively require large network to be simplified by considering only the skeleton network. Simulation techniques can handle larger networks but are generally restricted to a single loading condition. The most promising calibration procedure is based on optimization **(Dragan , Savic , Godfrey and Walters, 1995).**

Model calibration is the process of comparing model results with field results and making model modifications where appropriate to simulate the field results as closely as possible. Typical adjustments include adjustments to system connectivity, operational controls, facility configurations diurnal patterns, elevations, etc. Several indicators are utilized to determine if the model accurately simulates field conditions: water levels in storage tanks, the run times for pumps, and static and residual pressures from the fire flow tests and roughness coefficients for pipelines.

The hydraulic model is calibrated for two scenarios:

- Steady State Calibration: Simulating fire hydrant flow tests to match field results
- Extended Period Simulation (EPS) Calibration: Modifying the model until it mimics the field operations on the day of calibration.

2.8.1 Standard Calibration Procedures

The techniques and procedures for constructing a WDS model may vary but in they are summarized by the Water Research Center into the following activities.

☞ Inspection of supply, distribution and consumer records and maps.

☞ Site inspection of plant and equipment.

☞ Preliminary field measurement.

- Field measurement exercise.
- Entry of network data for a computer analysis and
- Calibration of the network model.

2.8.2 Calibration standards

The following issues are raised frequently in the field of distribution system modeling:

- Extent of calibration needed for various applications, and
- Standards for calibration.

There are no universally accepted standards but there are performance criteria for modeling distribution systems **(Water Authorities Association and WRc, 1989**). These are expressed in terms of the ability to reproduce field measured flows and pressures within the model, as shown below.

- **Flow**

1. ± **5** percent of measured flow when flows are more than ±**10** percent of total demand (transmission lines).
2. ± **10** percent of measured flow when flows are less than ±**10** percent of total demand(distribution line)

- **Pressure**

1. 0.5m (1.6ft) or 5 percent of head loss for 85 percent of test measurements.
2. 0.75m (2.31ft) or 7.5 percent of head loss for 95 percent of test measurements.
3. 2m (6.2ft) or 15 percent of head loss for 100 percent of test measurements.

2.9 Model Validation

Model validation is the step that follows calibration and uses an independent field data set to verify that the model is well calibrated. In the validation step, the calibrated model is run under conditions differing from those used for calibration and 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. Significant deviations indicate that further calibration is required **(EPA/600/R-06/028, 2005)**.

2.10 Water Loss and leakage

Water loss via leakage is acknowledged as one of the main challenges facing water distribution system operation. In any water supply system, the infrastructure will deteriorate with age whatever its nature. Water losses will increase over time if nothing is done, due to increased leakage from pipes, meter under registration or failure, and data handling errors (customer and network systems).

Water leakage is positively related to water pressure, and reduction in water pressure can be translated into reduction in water leakage. The total leakage in a pipe distribution network is often estimated according to a pressure-leakage relationship in the following form **(Lambert 2001; Thornton 2003; UKWIR 2003; Thornton and Lambert 2005):**

$$L = kP^n \quad (2.10)$$

Where, L= the leakage,

P = the average pressure of the network, and

k and n = parameters to be calibrated.

The exponent n ranges from 0.5 to 2.5 or even higher depending on the type of leakage and pipe material **(Lambert 2001; Thornton and Lambert 2005).** For leaks from joints and fittings, bigger values of n (>1) are usually obtained. For leaks from holes in pipe, n usually has smaller values. Regarding to the pipe material, plastic pipes have bigger n values than metal pipes **(Lambert 2001).**Obviously, water leakage will be sensitive to water pressure if n>1.

The standard IWA **(International Water Association)** terminology for assessing water losses can be abbreviated as follows:

- **System Input Value** is the annual volume input to the water supply/system.
- **Authorized Consumption** is the annual volume of metered and /or non-metered water taken by registered customers, the water supplier and others that are implicitly or explicitly authorized to do so.
- **Non-Revenue Water (NRW)** is the difference between system in put volume and Billed Authorized Consumption.
- **Water Losses** is the difference between system Input Volume and Authorized Consumption and consists of Apparent Losses and Real Losses.
- **Apparent Losses** consist of Unauthorized Consumption and all types of meter inaccuracies.
- **Real Losses** are the annual volumes lost through all types of leaks, bursts and overflows on mains, service reservoirs and service connections, up to the point of the customer meter.

IWA Water Loss Task Force developed equations for UARL (Unavoidable Annual Real Losses). UARL is used in calculating the Infrastructure Leakage Index (ILI), specifically designed to compare technical real losses management between Utilities.

The UARL formula, based on clearly auditable assumptions, was first published in **Lambert et al (1999)** as shown in Equation (2.11):

UARL (liters/day) = (18 x Lm + 0.8 x Nc + 25 x Lp) x P ……………………… **(2.11)**

Where, Lm = mains length (km),

Nc= No of service connections (main to property line)

Lp = total length of service connections (property line to customer meter) in km

P = average operating pressure (meters).

The most important performance indicator related to water losses Infrastructure Leakage Index – ILI **(Alegre et al., 2006):**

$$ILI = \frac{CARL}{UARL}$$

World Bank Institute Banding System to broadly classify ILI performance, and identify appropriate priority actions for individual systems according to the following table below.

Table 2-2 Sub-Division of World Bank Institute Bands (2010)

Low and Middle Income Countries	**High Income Countries**	**Bands**	**General description of Real Loss Management Performance Categories** (WBI Band limits for ILI for Low and Middle Income Countries are double those for High Income Countries)
ILI range	**ILI range**		
Less than 3	< 1.5	**A1**	Further loss reduction may be uneconomic unless there are shortages; careful analysis needed to identify cost-effective improvement.
3 to < 4	1.5 to <2	**A2**	
4 to < 6	2 to<3	**B1**	Potential for marked improvements; consider pressure management, better active leakage control practices, and better network maintenance.
6 to <8	3 to <4	**B2**	
8 to < 12	4 to <6	**C1**	Poor leakage record; tolerable only if water is plentiful and cheap; even then, analyze level and nature of leakage and intensify leakage reduction efforts.
12 to < 16	6 to <8	**C2**	
16 to < 24	8 to <12	**D1**	Very inefficient use of resources; leakage reduction programs imperative and high priority.
24 or more	12 or more	**D2**	

2.11 Relationship between pressure and water Demand

Pressure variation in a distribution network is caused, amongst others, by changes of demand of the users. There is an excess pressure build up in a network when demand drops especially during the night. However, slow down this unduly excessive pressure in order to avoid the bursting of pipes or reduce the amount of leakage.

There are a number of methods for reducing pressure in the system, including variable speed pump controllers and break pressure tanks. However, the most common and cost effective is the automatic pressure reducing valve (P RV).

P RVs are instruments that are installed at strategic points in the network to reduce or maintain network pressure at a set level. The valve maintains the pre-set downstream pressure regardless of the upstream pressure or flow rate fluctuations. P RV should be downstream of the meter so that turbulence from the valve does not affect the accuracy off the meter. It should be noted that in recent years advanced pressure management techniques have been introduced such as flow and time modulation and critical control point pressure management which reduce even further the pressure in the network achieving corresponding reduction in leakage and new breaks.

An indirect benefit of pressure management is the extension of the life of the assets due to lower operating pressures.

2.12 Review of related works

The other research reviewed was conducted by Shimeles Kabeto. He conducted a research titled "water Supply Coverage and Water Loss in Distribution systems with Modeling (The Case Study of Addis Ababa)". His intention was to assess the supply coverage and explore the water loss in city water supply distribution system. The researcher attempted to quantify the average water supply per person at city level and determine water loss as leakage at the city level and at the sub system level. In his findings, he found the average water supply coverage of the city as 86.59 liter/person/day and water loss at city level and sub-city level as 39% and 37.56% respectively.

Saleamlak Muluken also conducted a research titled "Hydraulic Modeling and Improvement of Addis Ababa Water Supply System (The Case of Bole Bulbula)". His intension was to assess supply and demand gap analysis, hydraulic and water quality modeling, water loss in water distribution system. The researcher attempted to analyze evaluating hydraulic performance of the distribution system as well as analyzing issues of supply network and water quality in the distribution system in selected sub-city. In his findings, he found the Domestic water supply coverage of the sub-city by family connection (57%) and average base line demands (58 l/c/d) and 8.33% of the system's total distribution hydraulic performance improved by resizing existing distribution mains pipes with respect to low pressure and Excessive rate of unaccounted for water is estimated (34.46%). The researcher used model to evaluate alternative scenario to improve system performance.

Within this context, the present research is similar to reviewed research in data collection aspects. But the current study is different in method of data analysis, tools used for analysis and its scope. It includes hydraulic performance evaluation using modeling, demand gap analysis and physical water loss at the same time, which was not studied in the pervious, the place in which my research is conducted.

CHAPTER THREE

3. Research Methodology

3.1 Description of the Study Area

3.1.1 General

Ginchi town is found in Dendi woreda, West Shewa Zone, Oromia Regional state of Ethiopia. Geographically, the town is locating between **404362.74E to 408005.87E** and **998126.30 N to 997663.10 N UTM**. The town is located on the main road of **Addis Ababa to Ambo** at a distance of 78 **km** in west of Addis. It is one of the largest towns in Dendi woreda administrative. The present population of the town is estimated to be **53,600** with an annual growth rate of **4.1 %**(Oromia Regional Finance and Economic Development Bureau). The town is located at center of Dendi woreda with an area of **6.5 km^2** and its altitude ranges from **2231 to 2380 m** above sea level.

The town possesses a complex mix of medium land climate zones, with average temperature **18^oc**, depending on elevation and prevailing wind patterns. According to National Meteorological Agency of Ethiopia, mean minimum and maximum temperature of the day varies from **15^oc to $27^{o\,c}$**. From June to mid-September is the main rainy season for the town of Ginich. The hottest and driest months are usually April and May. The shortest rainfall during March to mid-April, characterized by relatively cool nights and warm days.

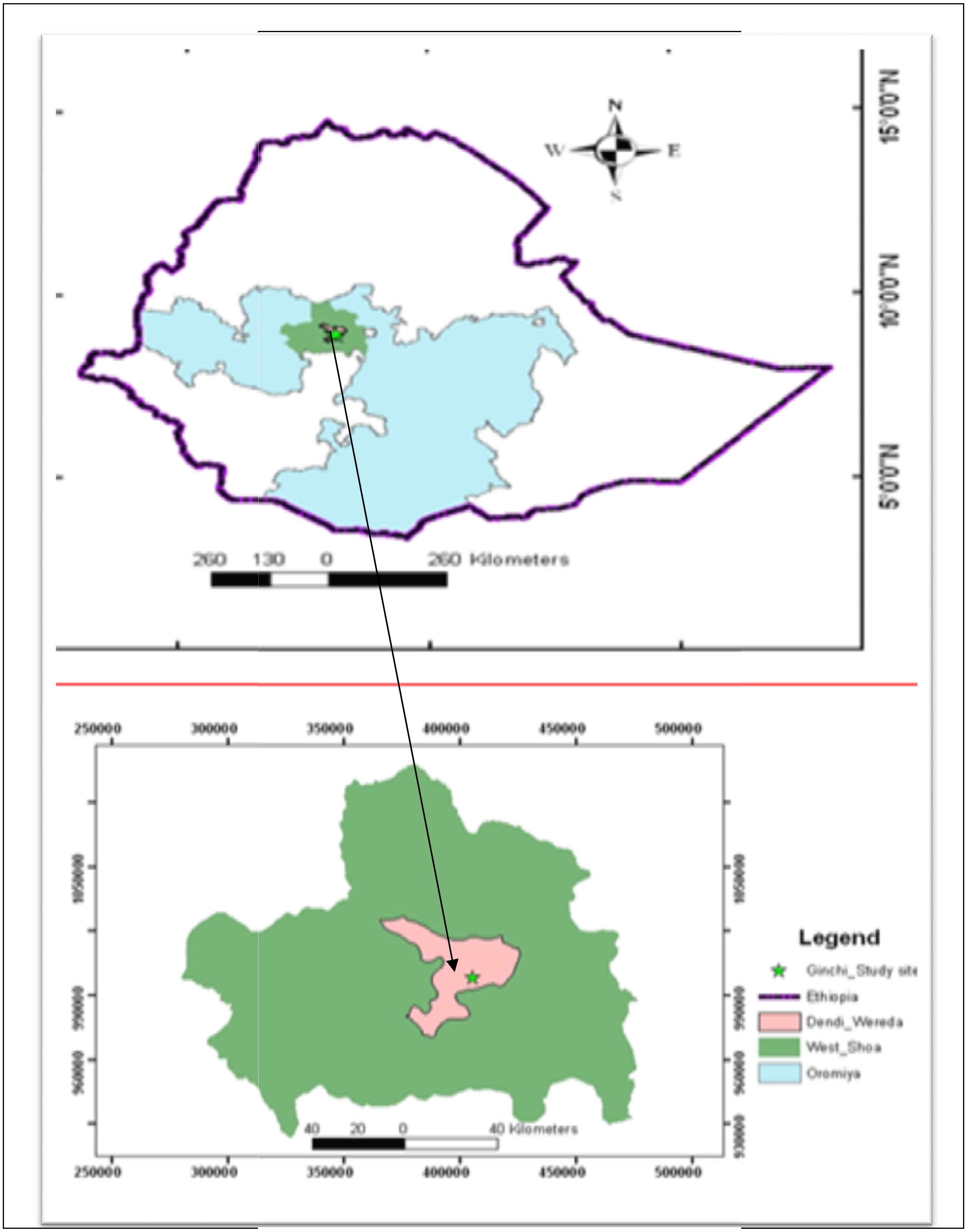

Figure 3 -1 : Location of study Area

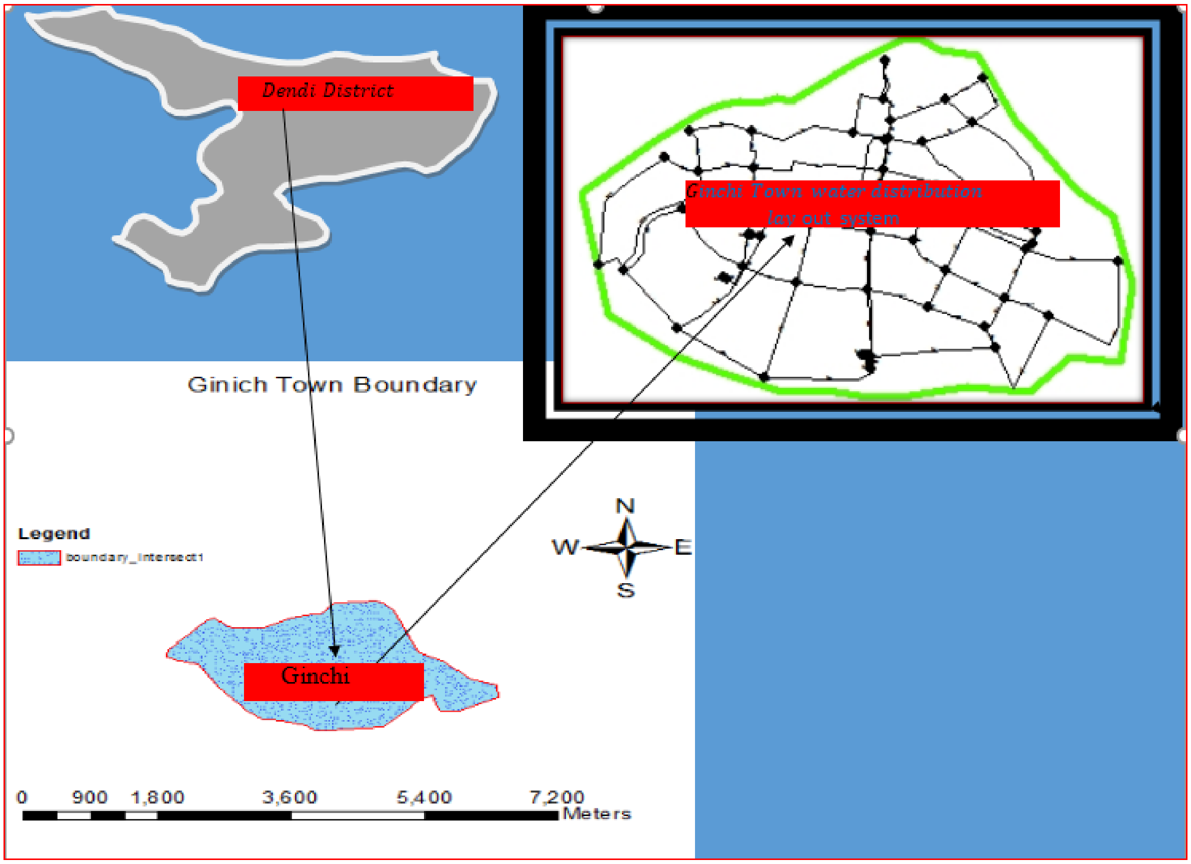

Figure 3-2: Ginchi Town Boundary and Water Distribution System lay out map

3.1.2 Existing Water Supply of the Town

Ginchi Town Water supply and Sewerage Enterprise is a public institution in the town which is responsible for potable water supplies.

The existing water supply scheme of Ginchi Town was first constructed in 1979 through drilling of one borehole. From the period of started up to 2009 the town have been drilled 4 boreholes during these period. Now the system consisted of four boreholes, two service reservoir, and pipe network systems and 30 water points constructed. The current population of the town is male **31,400** and female **22,200** total **53,600**(CSA, 2007 and 2011 Town's Structural plan study figures are used as a base). From these numbers of population, total customers **2472**, **2286** domestic, **156** non domestic and **30** public tap (standpipe) have their own service line connection from the GTWSSE. Currently the town gets its water supply from ground water source. The ground water source is from near the town wells situated **3km** in the west south of the town and the spring's source from Warka Gara Large Gravity

Springs which is situated **16.5 km** in the west north of Ginchi town. But the population of the town increased from time to time and town also enlarged considerably. Hence, presently the residents of the town facing critical water shortage. According to the design documents and Ginchi Town Water Supply and Sewerage Enterprise report in **2017** the yields from four boreholes as shown in table 3-1.

Table 3-1 Current production from bore hole and gravity spring

Source	**Bore hole Capacity (l/s)**	**Pumping hr.**	**Total production liter per day**
BH-1	3.6	less than 22 hr.	285,120
BH-2	4.0	less than 22 hr.	316,800
BH-3	9.8	less than 22 hr.	776,160
BH-4	7.9	less than 22 hr.	625,680
Gravity Spring	2.6	24 hr.	224,640
Total production	**27.9**	**l/day**	**2,228,400**
		m3/day	**2,228.4**
		m3/year	**813,366**

Source :(Ginchi Town Water Supply and Sewerage Enterprise and Design Documents)

Warka Gara large Gravity Spring has a potential of more than **33.3l/s (Design Document of Warka Gara Large Gravity Water Supply)**. But only **16.2l/s** was diverted for the community living in the surrounding **8** kebeles. The spring project was designed both for the rural community and Ginchi town water supply. Since the pipe connection is direct supply to the town water supply distribution system, the Town Water Supply and Sewerage Enterprise do not know the total production amount of water for the town population.

3.2 Description of Study Design (Research Process)

The design approaches adopted for each of the system components to perform the model are described as figure below:

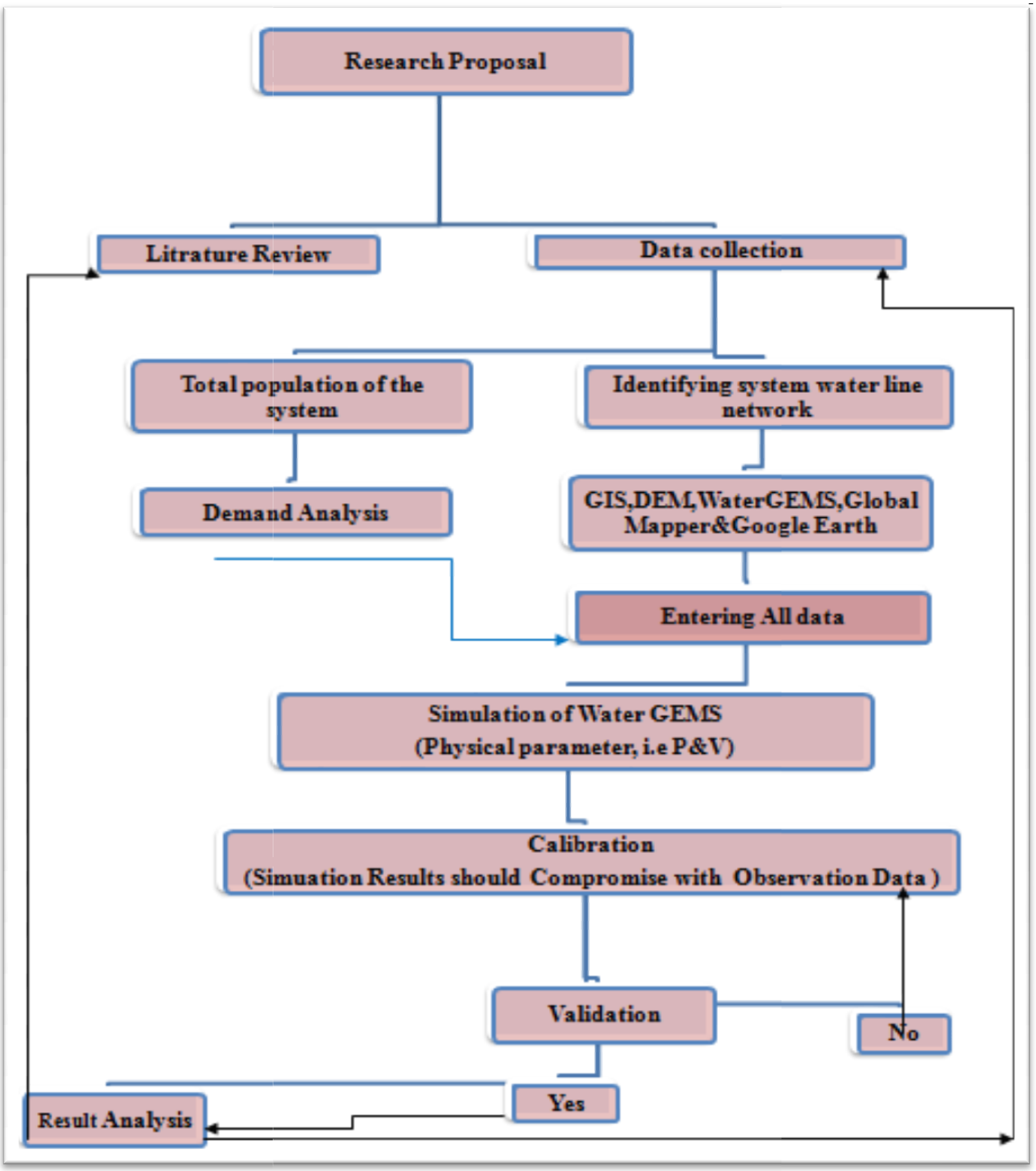

Figure 3-3: Shows Research Flow Chart

3.3 Methods of Data Collection

3.3.1 Secondary Data Collection

To generate water distribution model, many potential sources of data is required. The following sections discuss some of the most commonly used resources, including:

- Hydraulic water system schematic
- All water mains, laterals and water facilities
- Water production and water usage records **(2008-2017)**
- General plan and land use information.
- Ground Elevation contour line.

All the above data have been taken from Ginchi Town water supply and Sewerage Enterprise, or have been carried out by the researcher to make the modeling of the system is able.

3.3.2 Primary Data Collection

- Pressure measurements taken at different time and location to perform calibration and validation.
- Ground Elevation using GPS.
- Field measurements of storage reservoirs dimensions.

3.3.3 Source of Data to be Collected

3.3.3.1 System Maps

System maps are typically the most useful documents for gaining an overall understanding of a water distribution system because they illustrate a wide variety of valuable system characteristics. System maps may include such information as

- Pipe alignment, connectivity, material, diameter, and so on.
- The locations of other system components, such as tanks and valves.
- Pressure zone boundaries
- Elevations
- Miscellaneous notes or references for tank characteristics
- Background information, such as the locations of roadways, streams, plan
- Other utilities.

3.3.3.2 Topographic Map

A topographic map uses sets of line called contours to indicate elevations of the ground surface. By superimposing a topographic map on a map of the network model, it is possible to interpolate the ground elevations at junction nodes and other locations throughout the system. Of course, the available topographic maps cannot provide the level of precision needed, other sources of elevation data need to be considered.

Topographic maps are also available in the form of Digital Elevation Models (DEMs), which can be used to electronically interpolate elevations. The results of the DEM are only as accurate as the underlying topographic data on which they are used; thus, it is possible to calculate elevations to a large display precision but with no additional accuracy.

3.3.3.3 Utility Corporate Data Base

Model data will be derived from water utility corporate data bases. Specially data such as water consumption and water billing records will be obtained from utility corporate databases. GWSSEs official records for water consumption and water billing data will be used in this research to undertake water balance analysis and subsequently to quantify losses.

3.3.3.4 As-Built Drawing

As built drawings can be especially helpful in areas where a fine level of precision is required for pipe lengths, fittings types and locations, elevations, and so forth.

As–built drawings can also provide reliable descriptions of other system components such as storage tanks and pumping stations. There may be a complete set of drawings for single tank, or the tank plans could be included as part of a larger construction project.

3.3.3.5 Electronic Map and Records

Many water distribution utilities have some form of electronic representation of their systems in formats that may vary from a non-graphical database, to a graphics only Computer Aided Drafting (CAD) drawing, to a geographic Information System (GIS) that combines graphics and data. These sources of data can be quite helpful in expediting the process of model construction. Even so, care needs to be taken to ensure that the network topology is correct, because a simple typographic error in a monographic network can be difficult to detect.

3.3.3.6 Geographic Information Systems

GIS can be used for tasks such as proximity analysis (identifying customers within a certain distance of a particular node), overlay analysis (determining all junctions that are completely within a particular zoning area), network analysis (identifying all households impacted by a water–main break), and visualization (displaying and communicating master plans graphically). With a hydraulic model that links closely to a GIS, the benefits can extend well beyond just the process of building the model and can include skeletonization, demand generalization, and numerous other operations.

3.4 Data Preparation

3.4.1 Population Projection

Population projection has a paramount importance since it is the most important variable in all types of development planning at both macro and micro levels.

Simple mathematical function (Geometric increase method) is adopted to project the population of Ginchi town for the coming **20** years, up to year **2037**. The population of Ginchi town adopted from Ginchi Town Water Supply and Sewerage Enterprise **(2017)** which is estimated to be **53,600(CSA, 2007)**has been used as a base to project the population size of the planning period. The following formula has been adopted for the population projection.

$$Pn = Po*(1+K)^{n} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.1)}$$

Where:

Po= initial Population

Pn= Population at n decades or Years

n = decade or years

K= Percentage (geometric) increase

3.4.2 Water Production and Consumption

There are some years of records of water production from boreholes according to the information obtained from the Ginchi town utility operator. The utility operator has explained that the production and customer water meters are functional and records are taken regularly. According to, the information gathered from the GTWSSE, water is currently supplied to the town consumers through a total of **2,444** private connections (domestic, institutional, commercial and industrial connections) and **30** public fountains (standpipe) with four faucets out of which 21 are operational. These customers within the entire area have been considered to estimate water supply coverage, water loss, and demand and gap analysis purpose.

3.4.4 Water Demand Estimation

Water demand estimation is one of the basic inputs to select source of water supply and to find the amount of water required to fill the gap between supply and demand of the distribution systems**.**

The current estimation of water use by entire population required to review a set of various data. Water Production and Consumption data along with present and future population data were repeatedly reviewed. Accordingly, the two mentioned data were primarily used for water demand analysis. In this study the demand is estimated in considering the current per capita demand calculation:

$$\text{Per Capita Consumption (l/c/d)} = \frac{\text{Annual Consumption (m3)}}{\text{(Population Number)} * 365\text{day}} * 1000\ \text{L/m3} \quad \ldots\ldots \textbf{(3.2)}$$

3.4.5 Fire Fighting Water Demand

Fire demand is the quantity of water needed to extinguish fire which depends up on population, centers of buildings density of buildings and their resistance to fire.

There are many formulas can be used to estimate the amount of water needed for fire. The most popular formula in the world is **National Board of Fire underwriters (NBFU)** for communities less than 200,000 in habitants.

$$\mathbf{QF = 231.6\sqrt{P}(1-0.01\sqrt{P})} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \textbf{(3.3)}$$

Where, QF = fire demand (m3/hr)

P = Population in 1000`s

Note: this formula is used for sizing reservoir taking the community as whole and should not be used for distribution system pipes.

The number of expected fires that may occur simultaneously is calculated using the following formula:

Number of fire hydrants need for town = $\frac{\sqrt{P}}{3}$**……………. (Eq---3.4)**

Where, P = the population in thousands.

3.4.6 Unaccounted for Water

3.4.6.1 Total Water Loss Calculation

Unaccounted-for water includes water losses in the water supply system, illegal connections, firefighting, overflow from reservoir and improper metering. It is also sometimes called "non-revenue-water" which also includes water for which bills have not been paid. The amount is usually expressed in percentage of the sum of the domestic demand, public demand, and the industrial demand covered from the water supply system.

NRW is calculated as the amount of water produced by production water quantity minus the metered customer use divided by the amount of water produced multiply by 100.

$$\text{NRW(Unacconted for Water)}\% = \left(\frac{\text{Water produced} - \text{Water consumed}}{\text{Water production}}\right) * 100 \quad ....(3.5).$$

3.4.6.1.1 Water Loss Per Number of Connection

The water loss per connection computed by using this expression :

$$\text{Water loss} = \frac{\text{Annual total loss} \ * 1000}{(\text{Number of connection} * 365)} \quad ……………………………(3.6)$$

3.4.6.1.2 Water Loss as Per Length of Pipes

One way to indicate the loss is expresing water loss as per kilo meter of main pipe.The total water loss per length of pipe expressed :

$$\text{Water loss} = \frac{\text{Annual loss}}{(\text{Length in km} * 365\text{day})} \quad ………………………(3.7)$$

3.4.6.2 Real and Apparent Loss Calculation

According to IWA (International Water Association) Water Loss Task Force, water loss have two main componets: Apparent Losses and Real Losses.

Apparent losses is water that is consumed but is not properly measured,accounted or paid for . These losses cost utilities revenue and distort data on customer consumption pattern.

Real losses are the physical losses of water distribution system, including leakage and storage over flows.

These losses increase the water utility`s production costs and stress water resources since they represent water that is extracted and treated but never reaches beneficial use.

Water losses and unbilled authorized consumption makes the Non–Revenue Water (NRW). The main task for every water utility is to reduce NRW.

IWA Water Loss Task Force and are widely accepted by water utilities suggested the following equation for determination of :

Unavoidable Annual Real Losses (UARL)=(18Lm/Nc + 0.8+ 25Lp/Nc)*P ----(Eq. 3.8)

Where:**UARL**=Unavoidable Annual Real Loss in liters pre day **(l/service connection /day).**

Lm = Length of pipelines (km).

Np = Number of service connections (main to property line).

Lp = Total length of service connections pipes from main pipe to water meter(km). i.e property line to customer meter.

P = Average Pressure in the disrtibution network(m).i.e estimate from weighted average ground level and average pressure at zone inlets (reservior).

The most important performance indicator related to water losses is infrastructure leakage Index-ILI(Alegre et al.,2006):

ILI = CARL/UARL …………………………………………..(3.9)

Both CARL(Current Annual Real Losses) and UARL(Unavoidable Annual Real Loss) shall be expressed in same units and therefore ILI is dimensionless. The IWA work group

suggested that ILI should be around 1.0 for the system with very low water losses and could go above for high leaking systems.

3.5 Nodal Demand Calculation

The water demand allocation is based on the number of customer of domestic, public and industrial use of each junction by using the current per capital demand. The base demand is analyzed by considering 24 hours' demand pattern.

Base demand = Population * Per Capital demand per day (**64 l/day**) **…… (3.10)**

PHD = Population *per Capital demand per day * PHF (1.8)

MHD = Population *per Capital demand per day *MDF (1.2)

LHD = Population *per Capital demand per day *LHDF (0.30)

3.6 Modeling Tools

Water Distribution network mathematical models have become increasingly accepted within the water industry as a mechanism for simulating the behavior of water distribution systems**.** Their purpose is to support the decision–making processes in various utility management applications including planning, design, operation and improvements of water distribution systems.

The most popular and powerful models that simulates the behavior of water distribution systems in the real world**: Water CAD, Water GEMs, Epanet, WaterNetGen and GIS** are available on website**.**

For this study **Water GEMs** modeling software was preferred as analyzing tools due to its multi-species modeling capabilities and its integration with GIS software **(Alaeddinne Eljamassi, Rola Ahmead Abeaid, 2013).**

Water GEMs is an open-structured, Public domain hydraulic and water quality model developed by Bentley and is used world wide.

3.6.1 Hydraulic Modeling Analysis

Water GEMS is one of computer model program developed by **Bentley** to evaluate the hydraulics analysis and water quality analysis in any distribution system networks. You can use water GEMS to perform a variety of function, including steady state and extended period simulations of pressure networks with pumps, Pipes, nodes, control valves, reservoirs and storage tanks.

Water GEMS models simulates the pipe flows, node pressures, velocities and head loss in the entire system. After running a system analysis, the results can be viewed on color coded network maps, contour plots, and time series graphs and data tables. The analyses are performed for both the existing state of the systems and for the planned system expansions in future.

Hydraulic models are often used to validate the design of new or rehabilitated pipelines. They are also used to verify the system capacity or to analyses the effect of modified infrastructure within the entire water distribution system or its sub system.

Models have become an essential tool for the management of water distribution systems around the world. There are numerous purposes for using a computer model to simulate the flow conditions within a system. A model can be employed to:

- Ensure adequate quantity and quality service of the potable water resource to the community.
- Evaluate planning and design alternatives
- Assess system performance
- Verify operating strategies for better management of the water infrastructure system.
- Perform vulnerability studies to assess risks that may be presented and affect the water supply.

3.6.2 Hydraulic Model Capability

Bentley Water GEMS V8*i* provides modeling capabilities, so that you can model and optimize practically any distribution system aspect, including the following operations:

- Perform a steady-state analysis for a snapshot view of the system, or perform an extended-period simulation to see how the system behaves over time.
- Use any common friction method: Hazen-Williams, Darcy-Weisbach, or Manning's methods.
- Control pressure and flow completely by using flexible valve configurations.
- Control pumps, pipes, and valves based on any pressure junction or tank in the distribution system.
- Perform automated fire flow analysis for any set of elements and zones in the network.
- Calibrate your model manually, or use the Darwin Calibrator.
- Computer system head curves.

3.6.3 Model Representation

A hydraulic model consists of two types of elements: links and nodes. Depending on the model, the major components such as pipes, junctions, pumps, tanks, hydrants, and valves are represented by either a link or a node. Other components such as customer points rupture disks, or orifices may also be available. The Model is a valuable asset to the user and is the result of substantial effort in data collection, entry and quality control. One of the keys to maintaining the hydraulic model`s value is making sure the model is updated with appropriate changes in infrastructure components, system demands or operating parameters. To maintain confidence in the results of the model, data within the model should best represent the current system configuration.

Table 3-2: Common Network Modeling Elements

Element	**Type**	**Primary Modeling Purpose**
Reservoir	Node	Provides water to the system
Tank	Node	Stores excess water within the system and releases that water at times of high usage
Junction	Node	Removes (demand) or adds (inflow) water from/to the system
Pipe	link	Conveys water from one node to another
Pump	Node or link	Raises the hydraulic grade to overcome elevation differences and friction losses.
Control Valve	Node or link	Controls flow or pressure in the system based on specified criteria.

3.6.4 Model Calibration and Verification

Model calibration and verification based on the observed data is required in order to establish the necessary level of accuracy of the hydraulic model. It is also necessary in order to develop trust in the ability of the model to represent what is currently happening in the system and what can happen under future scenarios. The hydraulic model is typically calibrated and verified based on the flow and pressure measurements throughout the system. We can provide such monitoring services.

Once a water distribution model has been developed, model calibration and verification effort data were collected from field visits. Because of the complexity of variation and combination the achievements in approximating the computed model to measured values have been limited. Even if the tendencies were getting better and better, making minor adjustment to the input data then the model accurately simulated the pressure rate in the system.

Calibration of water distribution model is a complicated task. There are many uncertain parameters that need to be adjusted to reduce the discrepancy between the model predictions and field observation of junction HGL and pipe discharges. Pipe roughness coefficients are often considered for calibration. However, there are many other parameters that are uncertain and affect junction HGL and pipe flow rate. To minimize errors in model parameters and eliminate the compensation error of calibration parameters. You should consider calibrating all the model parameters, such as junction demand, operation status of pipes and valves, and pipe roughness coefficients.

Calibrating water distribution network models relies upon field measurement data, such as junction pressures and pipe flows rates.

In addittion , a typical network representation of water network may include hundreds of link and nodes.Ideally , during the water distribution model calibration process, the roughness coefficient is adjusted for each link and demand is adjusted for eachnode.However,only a small percentage of representative sample measurements can be made available for the use of modelcalibration , due to the limited financial and labor requrements for data collection.

Field observation data are measured and collected at different times of the day and at various locations on site, which may correspond to various demand loadings and boundary conditions. In order for the model simulation results to more closely represent observed data , simulation results must use the same demand loading and boundary conditions as observed data. Thus, the calibration process must be conducted under multiple demand loading and boundary conditions.

Figure 3-4: Shows Pressure measurment at faucet line at near of the main distribution line .

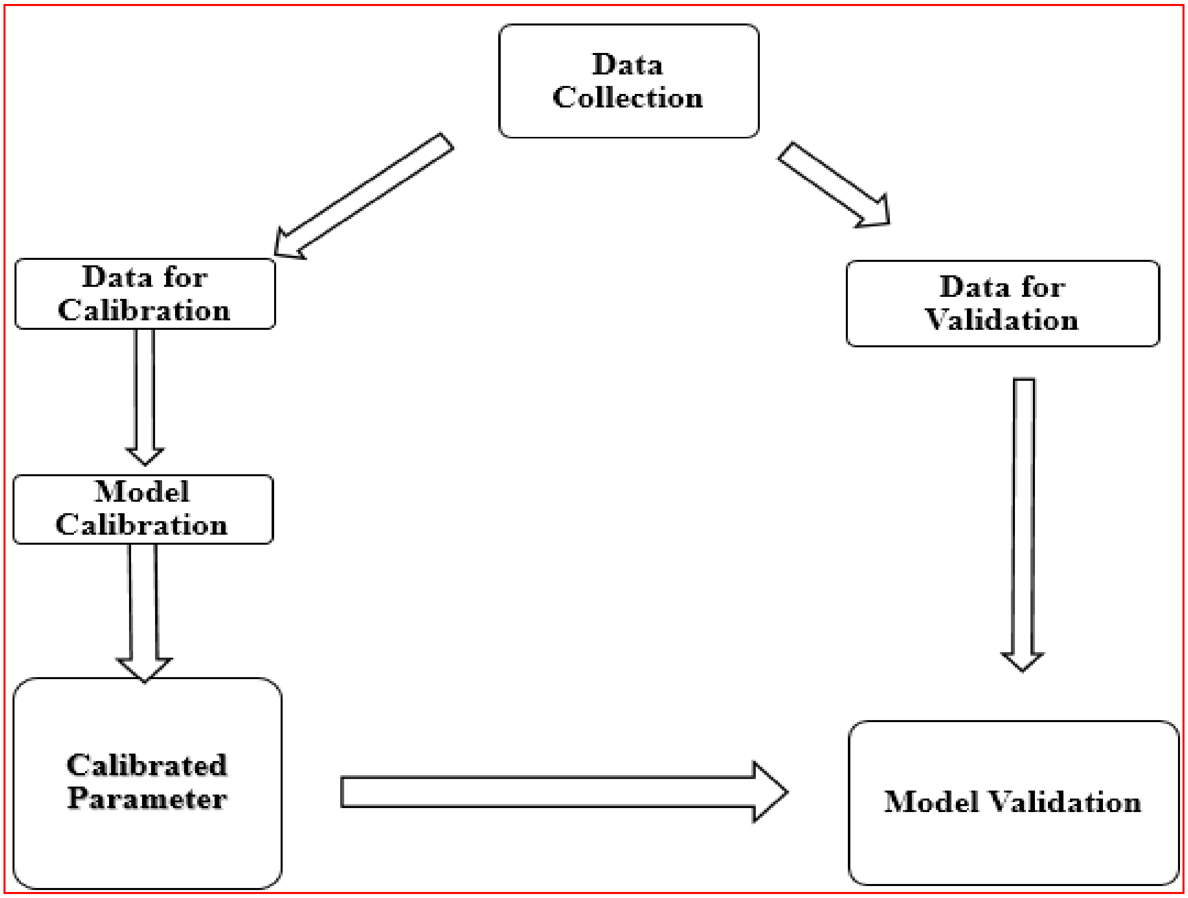

Figure 3-5: Shows that Calibration and Validation Procedure in the study

3.6.5 Data Analysis and Representation

The physical characteristics of the system were represented in the model by nodes and pipes (or elements). The nodes, joined together by pipes, represent: pipe junctions, changes in pipe diameter and the locations of system attributes such as valves and of large demands. The node and pipe data sets contain geographic coordinates, ground levels, basic demand information, internal diameter and friction coefficients, pump curves, service reservoir geometry and valve performance characteristics.

Water demand was allocated to the node nearest to its draw off point. Nodal demands were distributed based on population estimates served by the nodes, with considerations of leakage and pattern of use.

The analysis applied operational conditions to the network data such as diurnal demand patterns, times at which pumps start and stop or when valves were opened and closed. The analysis for a sequence of time steps, known as extended period simulation, each step representing a unique set of demand and operational conditions. An extended period analysis used the initial set of demand profiles, reservoir levels and network operational conditions to calculate demands, pressures and flows in the network over the first time period to determine the operational status of automated pumps and control valves and the net reservoir inflows / out flows and thereby, using the reservoir geometry, changes in reservoir levels.

The new reservoir level together with the diurnal demands and operational changes became the starting values for the second time step. The analysis was repeated for each subsequent time step. The results were displayed both graphically and in tabular form either for a single time step or a sequence of steps to illustrate the changing performance of the network and individual elements of the system over the period of the analysis.

The analysis was an iterative solution of a set of algorithms that simulate the hydraulic behavior of the flow of water through the piped network, solving the equations to specified tolerances by successive approximations subject to the following rules:

- The algebraic sum of the flows entering and leaving a node must be zero.
- In any closed loop in the system the algebraic sum of the pressure losses must be zero.
- The combined inputs to the system must equal the total of the nodal demands.

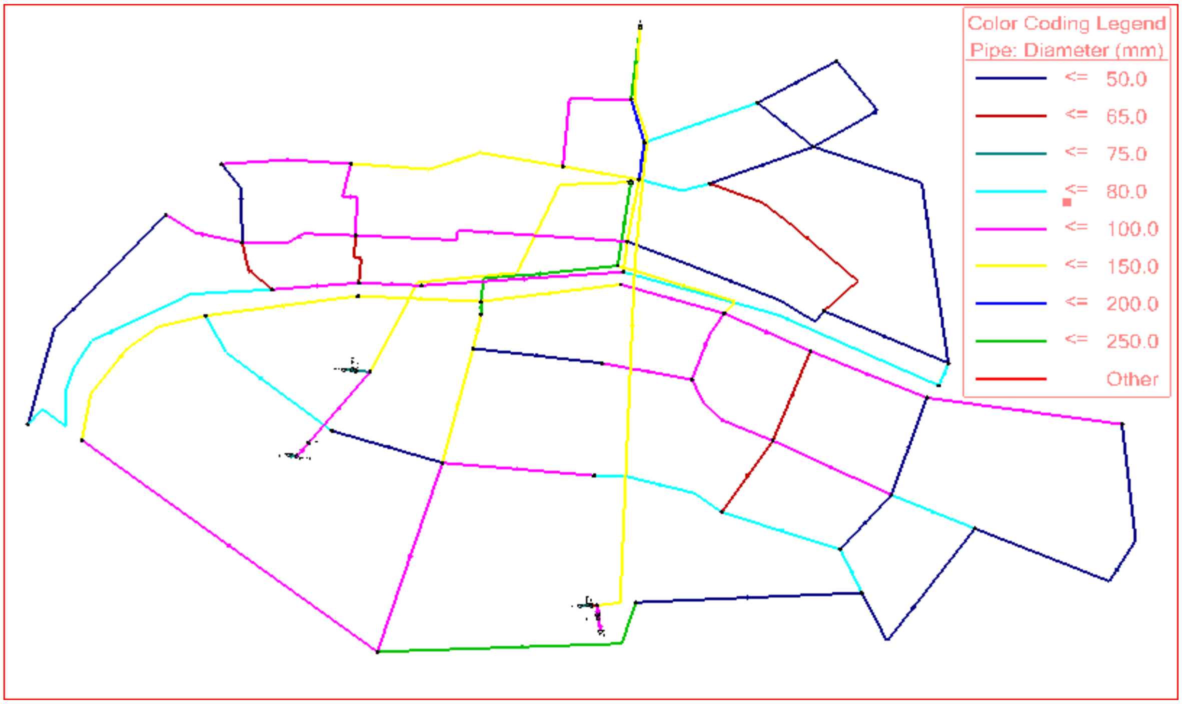

Figure 3-6: Ginchi Town Water Supply Distribution Model Representation

CHAPTER FOUR

4. Results and Discussion

4.1 Existing Water Supply Distribution Evaluation

Remodeling and analysis of Ginchi Town water supply distribution system as continuous supply systems are to verify the ability of the existing system to serve the town area in the future has been discussed depending on facts considering with the future water consumption availability of water, and other factors. Therefore, evaluating the entire distribution of the water supply system is important in order to identify the problematic areas and intervene accordingly.

4.1.1 Existing Domestic Water Supply

Based on information obtained from Ginchi Town water Supply and Sewerage Enterprise, nearly about **2444** of the households have got connection for piped water. But due to shortage of water and insufficient system, the service is much less which is evidenced by the fact that **100%** of the respondents in the household survey replied that there is a critical shortage of water. Comparing the gap between the current production capacity and the water demand, one can see that only the domestic demand is being met. From the comparison, this also falls short in the consecutive years.

As to the issue of additional connections, according to the survey low connection rate prevails due to water shortage in water supply system, population rise, and operation and maintenance capacity limitation. As a result, there are still applicants waiting for new connections.

Table 4-1: Town Water Connection and Population Served.

No.	Connection	2017	multiple	Population served
1	Private	2286	5	11430
2	Commercial	89	1	89
3	Governmental & NGO	66	1	66
4	Industrial	3	1	3
5	Public Tap	30	600	18000
		2474		**29588**

4.1.1.1 Population projection

Table 4.2 Town Population Projection (2018-2037)

Description	**Base Population 2017**	**2018**	**2023**	**2028**	**2033**	**2037**
Growth rate	4.60%	4.10%	3.90%	3.70%	3.60%	3.50%
Population	53,600	53,600	64,900	77,828	92,883	106,585

Source:(CSA, 2007 and The 2011 Town's Structural plan study figures are used as a base)

Growth rate adopted from Oromia Regional Finance and Economic Development Bureau and Population projected for 20 years of the project life time are shown in Table 4.2 under medium variant case.

4.1.1.2 Water Production and Consumption

Table 4-3 Production and Consumption History

S.N.	Year	Production (m3/year)	Consumption (m3/year)
1	2009	139,320	115635
2	2010	130,315	105,559
3	2011	122,400	102,816
4	2012	139,555	108,854
5	2013	136,980	116,433
6	2014	185,097	149,054
7	2015	255,534	164,045
8	2016	294,986	174,845
9	2017	302,585	178,186

Source: Ginchi Town Water Supply and Sewerage Enterprise Billed Consumption Data.

4.1.2 Analysis of Overall Water Demand Coverage

4.1.2.1 Water Demand Estimation

Table 4-4: Projected Demand Estimation

Item	Description	Unit	Years				
			2018	2023	2028	2033	2037
1	Population to be served	no.	53,600	64,900	77,828	92,883	106,585
2	Per capita demand	l/c/d	61.0	61.5	64.1	67.1	73.1

4.1.2.2 Domestic Demand Adjustment Factors

The average domestic demand shall be adjusted based on the socio-economic revelation and prevailing climatic conditions.

4.1.2.2.1 Socio-Economic Factor

The socioeconomic adjustment factor is determined based on the degree of the development of the particular town under study as the socio-economic conditions play great role on the amount of water consumption. The determination of the degree of the existing development and future potential of the town depends on good judgment. The following grouping and adjustment factor corresponding to each group is given below in table below.

Table 4-5: Socio Economic Factors

Group	Description	Factor
A	Towns enjoying high living standards and with high potential for development	1.10
B	Towns having a very high potential for development , but lower living standards at present	1.05
C	Towns under normal Ethiopian condition	1.00
D	Advanced rural towns	0.9

Accordingly, Ginchi town will fall under category B.

4.1.2.2.2 Climatic Factors

The water consumption is less in area where the average temperature is low and high where temperature is very high. According to this research, the mean annual temperature of Ginchi is around **18 ^{o}c**, hence the climatic factor of 1.0 is considered.

Table 4-6: Climatic Effects Factors

Mean Annual Temp. (^{o}c)	Description	Altitude	Factor
<10	Cool	>3,300	0.8
10-15	Cool temperate	2,300-3,300	0.9
15-20	temperate	1,500-2,300	1.0
20-25	Warm temperate	500-1,500	1.3
25 and above	Hot	<500	1.5

Source: Design Criteria Manual of MOWR (2006)

The adjusted per capita water demand according to the socioeconomic and climatic conditions is as shown in table below.

Table 4-7: Adjusted Average Water Demand (l/c/d)

Adjusted Factors		Years				
Socio-economic factor	Climate factor	2018	2023	2028	2033	2037

1.05	1.0	64.05	64.57	67.30	70.45	76.75

4.1.2.3 Fire Fighting Water Demand

Table 4-8: Fire fighting water demand of Ginchi Town.

Year	2018	2023	2028	2033	2037
population	53,600	64,900	77,828	92,883	106,585
Q(m3/hr)	1571.45	1715.47	1862.93	2016.94	2144.19
Q(m3/day)	37,714.8	41,171.28	44,710.32	48,406.56	51,460.56
No. hydrant	2	3	3	3	3

4.1.3 Current and Future Water Demand in the Study Area

Water demand is the driving force behind the hydraulic dynamics occurring in water distribution systems. Anywhere that water can leave the system represents a point of consumption, including a customer's faucet, a leaky main, or an open fire hydrant **(Jeffray, 2012).** Water demand estimation is one of the basic inputs to select source of water supply and to find the amount of water required to fill the gap between supply and demand of the subsystem.

Water distribution models are created not only to solve the problems of today, but also to prevent problems in the future. With almost any endeavor, the future holds a lot of uncertainty, and demand projection is no exception. Long range planning may include the analysis of a system for **20-year** time frames. When performing long term planning analyses, estimating future demands is an important factor influencing the quality of information provided by the model **(Walski et al., 2003).**

Table 4-9: Summary of Current and projected water demand in the study area.

Item	Description	Unit	Years				
			2018	2023	2028	2033	2037
1	Population to be served	no.	53600	64900	77828	92883	106585
2	Per capital demand	l/c/d	64	65	67	70	77
3	Average Domestic Demand(ADD)	m3/d	3430	4193	5238	6544	8180
3.1	Average Public Water Demand(5-7% of ADD)	m3/d	2058	2516	3143	3926	4908
3.2	Average Industrial Water Demand (30% of ADD)	m3/d	1029	1258	1571	1963	2454

3.3	Total Average Demand(TAD)	m3/d	6518	7966	9952	12433	15543
4	UFW (41%,28%,20%,15%, 10%)of TAD	m3/d	2672	2230	1990	1865	1554
5	Fire Fight water Demand	m3/d	37715	41171	44710	48407	51461
6	Total Average Day Water Demand(TADD)	m3/d	46905	51368	56653	62704	68558
6.1	Total Average Day Water Demand(TADD)	l/s	543	595	656	726	793
6.2	Max Day Factor		1.2	1.2	1.2	1.2	1.2
6.3	Max Day Demand	m3/d	56286	61641	67983	75245	82269
6.4	Max Day Demand	l/s	651	713	787	871	952
6.5	Peak Hour Factor		1.8	1.8	1.8	1.8	1.8
6.6	Peak Hour Demand	m3/d	101315	110954	122370	135442	148084
6.7	Peak Hour Demand	l/s	1173	1284	1416	1568	1714
		m3/day	2228.4				
6.8	Total production (current production)	l/s	25.8				
6.9	Total production from gravity springs	l/s	2.6				
	Total Production	**l/s**	**28.4**				

Water supply and demand gap between production and demand: **Supply–Domestic Demand** = **2228.4 –9,256 = -7,027.6** m3/day. The negative sign indicates that there is a shortage of water supply (source). So, additional water must be required in the system per day to meet the supply and demand gap.

4.1.4 Existing Water Supply System Gap Identified

The existing water supply to Ginchi distribution was **2,228.4 m3/day**. This water is supplying in the study area for a maximum of **22 hr**. To fill the gap additional **7,027.6 m3/day** amount of water source is required to fulfill the supply shortage and domestic demand of Ginchi town water supply system.

Table 4-10: Gap Identified

Indicators	unit	Recommended	current status	Gap
Per Capital demand	L/C/d	64	41.6	22.42
Production	m3/day	9256	2228.4	7, 027.6
Fire hydrant needed	no.	2	0	2
Water Coverage	%	100	65	35
water Supply continuity	Hr	24	12	12

4.1.5 Findings and Conclusions

From the current water supply analysis as per capital consumption of the town found to be **41.6 l/c/day**. This per capita consumption is lower while compared with the current requirement of domestic per capita demand which is **64 l/c/day.**

Based on the above evaluation of the town water coverage as per capita consumption. The level of coverage as per capita consumption is about **65%** which is relatively low coverage compared to the standard per capital demand.

4.2 Water loss Analysis

Water distribution systems often contain large amounts of unknown losses. The total water loss in the systems is **41.1** percent of the total volume of pumped. The result is a loss of product, including water and chemicals used to treat it; environmental damage, demand shortfalls, increased energy usage and unnecessary pump capacity expansions. It is clear that more control efforts need to be implemented on these systems to reduce losses and increase energy efficiencies. The water loss analysis has been made by using expression in terms of percentage of (UFW), loss per number of connections and loss per kilometer of main pipes. To compute the level of apparent loss due to limitation of data, the total loss is calculated as per level of connection and per the length of pipe and the real loss became the different between total loss and apparent loss.

4.2.1 Non Revenue (NRW) water loss Analysis

Nine-year production and consumption data of the study area used to compute the total loss as shown in the table 4-11 below by using this expression.

$$\text{NRW(Unacconted for Water)\%} = \frac{\text{Water produced} - \text{Water consumed}}{\text{Water production}} * 100$$

Table 4-11: Computed Non-revenues water loss

S.N.	Year	Production (m 3/year)	Consumption (m 3/year)	Loss(m3/year)	Loss (%)
1	2009	139,320	115635	23,685	17.0
2	2010	130,315	105,559	24,756	19.0
3	2011	122,400	102,816	19,584	16.0

4	2012	139,555	108,854	30,701	22.0
5	2013	136,980	116,433	20,547	15.0
6	2014	185,097	149,054	36,043	19.5
7	2015	255,534	164,045	91,489	35.8
8	2016	294,986	174,845	120,141	40.7
9	2017	302,585	178,186	124,399	41.1

In particular, in developing countries the rate of U FW is extremely high. The analysis of U FW rate shows the efficiency of the economic system and technical part of the water distribution network and its management. The main task for every water utility is to reduce N R W. The following physical losses are called technical and non-physical losses are administrative losses. The thesis concentrates mainly on the technical losses (**bursting, leakage)** due to **pressure**.

4.2.2 Water loss per number of connection

The total number of connection in the study area is **2,474** and using the above table the nine years computed annual total water loss **124,399m3/year** which the recent and the maximum annual total water loss value are used.

$$\textbf{Water loss} = \frac{\textbf{Annual total loss} * \textbf{1000}}{(\textbf{Number of connection} * \textbf{365})}$$

Water loss = (124,399m3*1000) /(2474*365**)** = **137.8 liter/connection/day.**

4.2.3 Water loss as per length of pipes

One way to indicate the loss is expresing water loss as per killo meter of main pipe. The total water loss per length of pipe expressed:

Table 4-12: Summary of pipe length.

Materials	**Diameter(mm)**	**Length (m)**
		Pipe Class: PN 10

HDPE &GS	50	2,732.23
	65	2,050.69
	80	3,643.88
	100	9,890.76
	150	4,473.24
	200	439.83
	250	1,511.81
Sub Total		24,742
Total Length (m)		24.742 km

Source: Ginchi Town Waters Supply Design Documents

Water loss = Annual loss / (Length in km*365day)

Water loss = 124,399m3/(24.742km*365) = **13.8m3/km/day.**

4.2.4 Real and Apparent Loss Analysis

Total water loss is the sum of real loss and apparent loss. IWA Water Loss Task Force are widely accepted by water utilities suggested the following equation for determination of Unavoidable Annual Real Losses **(UARL**).

(UARL) = **(18Lm/Nc + 0.8 + 25Lp/Nc)*P**

UARL = (18*24.742/2474 + 0.8 + 25*14.844/2474)*52 = **58.76 liter/connection /day.**

Annual Volume of UARL = 58.76 liter*365 = 21,447.4 liter/connection /year .

Total Annual UARL in all connection = 21,447.4 *2474 = 53,060.87 m3/year.

From the above equation it is clear that water losses are assumed to be directly proportional to length of pipes(mains and service connections) and average pressure in the distribution network.

The most important performance indicator related to water losses is infrastructure leakage Index-ILI**(Alegre et al.,2006)**: **ILI = CARL/UARL**

Both CARL(Current Annual Real Losses) and UARL (Unavoidable Annual Real Loss) shall be expressed in same units and therefore ILI is dimensionless. The IWA work group suggested that ILI should be around 1.0 for the system with very low water losses and could go above for high leaking systems.

Therefore,CARL of 2017 used from table **4-12** which is **124,399 m^3/year– 53,060.87m^3/Year = 71,338.13m^3/year.**

ILI = (**71,338.13m3/year)/(53,060.87m3/year**) = **1.35** . The result it shows according to world Bank Institue guide lines**(2005),**classified under Band **A** system. Further loss reduction may be uneconomical unless there are shortages.

Therefore, the real losses of the system were **(71,338.13/302,585) x 100 = 23.6%.** Understanding of Real Losses requires prority setting and decisions on whether to repair, replace, rehabilitate, or leave the assets as they are, while simultaneously implementing pressure management and improving the operation and maintenance programme.

The combined effect of the above strategies assists water utilities in reducing and sustaining their real loss component at an economic level.

4.3 Hydraulic Model Analysis

4.3.1 General information of model in put

4.3.1.1 Water Source

Ginchi Town Water Supply and Sewerage Enterprise official records indicate that currently the town is obtaining water from ground water. Ground water is the major sources. Natural gravity springs are also contributing significant quantity.

Currently the bore hole alone produces an average of **2,003.76 m 3/day**. In addition to this, water produced from large gravity spring is directly supply to the system. Presently four wells and one gravity spring are producing significant quantity of water. Daily Average production of water per day was **2, 003.76 m3/day**. Warka gara large gravity spring is contributing **224.64 m3/day**.

4.3.1.2 Pipes

Pipes are links that convey water from one point in the network to another. Since, pipes service as link to connect nodes. They are also the major physical component part of a water distribution system. Due to their importance, pipes are frequently referred as the central part of a water distribution network system.

Ginchi Town distribution system contains pipes of various diameters and various material types. For operational purpose the network system has been categorized in to transmission main and distribution lines. Transmission main lines enclose multiple connections along its length. Distribution lines are lines of smaller diameter and supply water directly to the end users. For this study transmission lines of diameter vary from **100 mm up to 250 mm** were included while distribution lines of lower diameters were concentrated to nearby nodes.

Table :4-14 Pipe distribution in diameter and material

Materials	Diameter(mm)	Length (m) Pipe Class: PN 10	%
HDPE	50	2,732.23	11.00
HDPE	65	2,050.69	8.28
HDPE	80	3,643.88	14.72
HDPE , GI	100	9,890.76	39.97
HDPE ,GI	150	4,473.24	18.07
GI	200	439.83	1.77
HDPE	250	1,511.81	6.11
Sub Total(m)		**24,742**	
Total Length (km)		**24.742**	

Source: Ginchi Town Waters Supply Design Documents

4.3.1.3 Pumps

Pumps are used to develop the necessary head (pressure) to distribute water to the consumer and storage reservoirs. Each productive borehole shall be equipped with a submersible pump. The pump will lift raw water from the borehole & fed to the respective zone reservoirs.

Pumps impart energy to water thereby raising its hydraulic head for conveying water to points of desire. Frequently, pumping is required to raise water to service reservoirs likely located on higher elevations. Hence, pumps are repeatedly situated nearby to water sources and treatment facilities. But in few cases, they are situated in distribution system to boost pressure.

Table: 4-15 Distribution of Pump in the system

S/No.	Site name	Head (m)	Q(l/s)	Year of Installation
1	Pump -1	313.8	4	1989
2	Pump -2	307.5	9	2014
3	Pump -3	234.9	3.6	1980
4	Pump -4	228.6	9	2014
Total			**25.6**	

4.3.1.4 Reservoirs

The service reservoir is designed to provide the storage capacity to meet fluctuations in demand, to provide reserve for fire-fighting use, other emergency situations during power outages or mechanical failure of supply units, and to equalize pressures in the distribution system. In general, important function of service reservoir is: -

- To balance the fluctuating demand from the distribution system.
- To meet peak demand.
- To provide a supply during failure or shut down of the treatment plant, pumps or trunks main leading to the reservoir.
- To eliminate the necessity of continuous pumping.
- To give a suitable pressure for the distribution system and reduce pressure fluctuations.
- To provide a reserve of water to meet fire and other emergency demands.

Table: 4-16 Distribution and Location of service reservoirs

S/No.	Name of Reservoirs	Capacity(m 3)	Base level(m)	Max. level(m)	Location Name
1	Tank-1	450	2379	2383	Kidanemirat church
2	Tank-2	300	2300	2304	01 Kebele

Both service reservoir is ground level type and one is located at higher local topographical condition permit and one is located at the centre of the town were the ground level reservoir is preferable.

4.3.1.5 Valves

In water distribution system regulation and control of either discharge or pressure is frequently achieved through use of valves. Different types of valves are available. Among them the widespread are: Pressure reducing valves, pressure sustaining valves, flow reducing valves check valves and general purpose valves. However, the most common type of valve in water distribution system is isolation valves that may be used, including gate valves (the most popular type) butterfly valves and plug valves.

4.3.2 Node Demand Analysis

The analysis of the water system under these conditions is performed using water GEMS software, which has developed by the Bentley, and represents a powerful; easy to use that helps in design and analyzes water distribution systems. This package of software enables the designer to model complex hydraulic situations. The input data needed to perform the analysis by the water GEMs Technique comprise the demand in the nodes, their elevations, demand patterns, valves with different types and their control; also if there are pumps in the system, they can be modeled depending on their operating curves.

To calculate the demand at different loading condition needs respective demand factors.

Table 4 -17: Ginchi Town Recommended Hourly Demand Factors

Time	values
00-01	0.3
01-02	0.3
02-03	0.3
03-04	0.3
04-05	0.3
05-06	0.7
06-07	1.2
07-08	1.8
08-09	1.6
09-10	1.5
10-11	1.5
11-12	1.4
12-13	1.4
13-14	1.4
14-15	1.3
15-16	1.4
16-17	1.5
17-18	1.5
18-19	1.2
19-20	1.1
20-21	1
21-22	0.6
22-23	0.4

23-24	0.3
Total	**24**

Source: Ginchi Town Water Supply Design document

From the above table, the LHF, MHF and PHF of Ginchi Town are **0.3**, **1.2** and **1.8** respectively. Therefore,

Base demand =Population* Per Capital demand per day **(64 l/day as computed above).**

= 53,600*64 = **39.7 l/s**

PHD = Population *per Capital demand per day * P H F (1.8)

= 53,600*64*1.8 = **71.47 l/s**

MHD = Population *per Capital demand per day *MDF (1.2)

= 53,600*64*1.2 **= 47.64 l/s**

LHD = Population *per Capital demand per day *LHDF (0.30)

= 53,600 *64*0.3 = **11.91 l/s**

4.3.3 Major Factors that affect water distribution system (Hydraulic performance)

4.3.3.1 Pressure

Pressure in municipal distribution system ranges from **15m -30m** in residential districts with structures of four stories or less and **40-50m** in commercial districts. Also, for fire hydrants the pressure should not be less than **15 m** of water. In general, for any node in the network the pressure should not be less than **25 m** of water. Moreover, the maximum pressure should be limited to **84 m** of water **(AWWA, 1999).**

4.3.3.1.2 Relationship between pressure and Leakage

Pressure management is one of the fundamental elements of a well- developed leakage management strategy. The rate of the leakage in water distribution networks is a function of the pressure applied by pumps or by gravity.

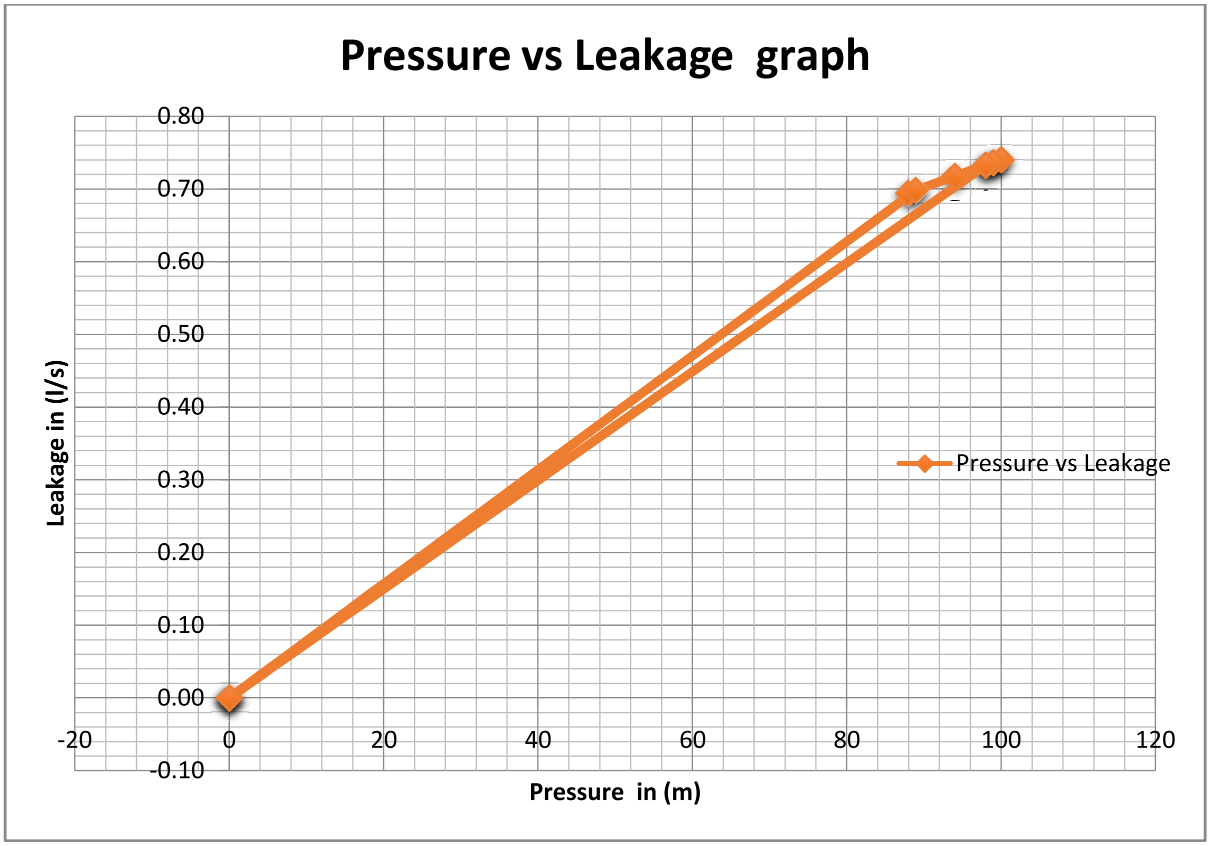

Figure 4 -1: Shows relationship between pressure (P) and Leakage Rate at J-24

There is a physical relationship between leakage flow rate and pressure and the frequency of new bursts is also a function of pressure:

- The higher or lower the pressure, the higher or lower the leakage. The relationship is complex, but water utility managers should initially assume a linear relationship (10% less pressure = 10% less leakage)**(B. Charalambous, D. Foufeas and N. Petroulias, 2014).**
- The lower the pressure the lower the number of new bursts. Water utility managers should initially assume that for 10% reduction in pressure 14% reduction in new breaks until a relationship is established based on actual data relevant to the specific distribution network.

Figure 4-2: Shows that bursting of pipe is a function of pressure

To assess the suitability of pressure management in a particular system, water utilities should first carry out a series of tasks, including:

- Identify potential zones, installation points and customer issues through a desktop study.
- Identify customer types and control limitations through demand analysis.
- Gather field measurements of flow and pressure (the latter usually at inlet, average zone point and critical node points).
- Model potential benefits using specialized models.
- Identify correct control valves and control devices.
- Model correct control regimes to provide desired results
- Analyze the costs and benefits.

Figure 4 -3: Typical installation of P RV.

4.3.2.2 Velocity

In water distribution system network velocity should not be lower than **0.6 m/s** to prevent sedimentation and should not be more than **3 m/s** to prevent erosion and head losses and the minimum velocity also should not be less than **0.05**m/s at service taps**.** Generally, commonly used values are **1-1.5 m/s (Walski et al., Thomas , Chase, Donald , Savic, Dragan and TAHAL , 2015,).**

4.3.2.3 Head Losses

In general, the optimum head losses in the distribution system ranges is **1- 4m/km**. the maximum head loss should not exceed **10 m/km.**

4.3.3 Diameter and Length of Pipes

Pipe lines which provide only domestic flow may be as small as 100 mm but should not exceed 400 m in length **(if dead–ended)** or 600 m if connected to the system at both ends. Pipe lines as small as **50-75 mm** is sometimes used in small communities with length not to exceed 100 m **(if dead –ended)** or 200 m if connected at both ends.

4.4 Model Result and Discussion

Once a model is built and running and the results are satisfactory, the system can be drafted for construction. It is a good idea to analyze the actual plans for inconsistencies in the model input. Significant changes from the model should warrant the model to be edited and tested with the accurate input from actual plans.

4.4.1 Pressure Distribution at Nodes

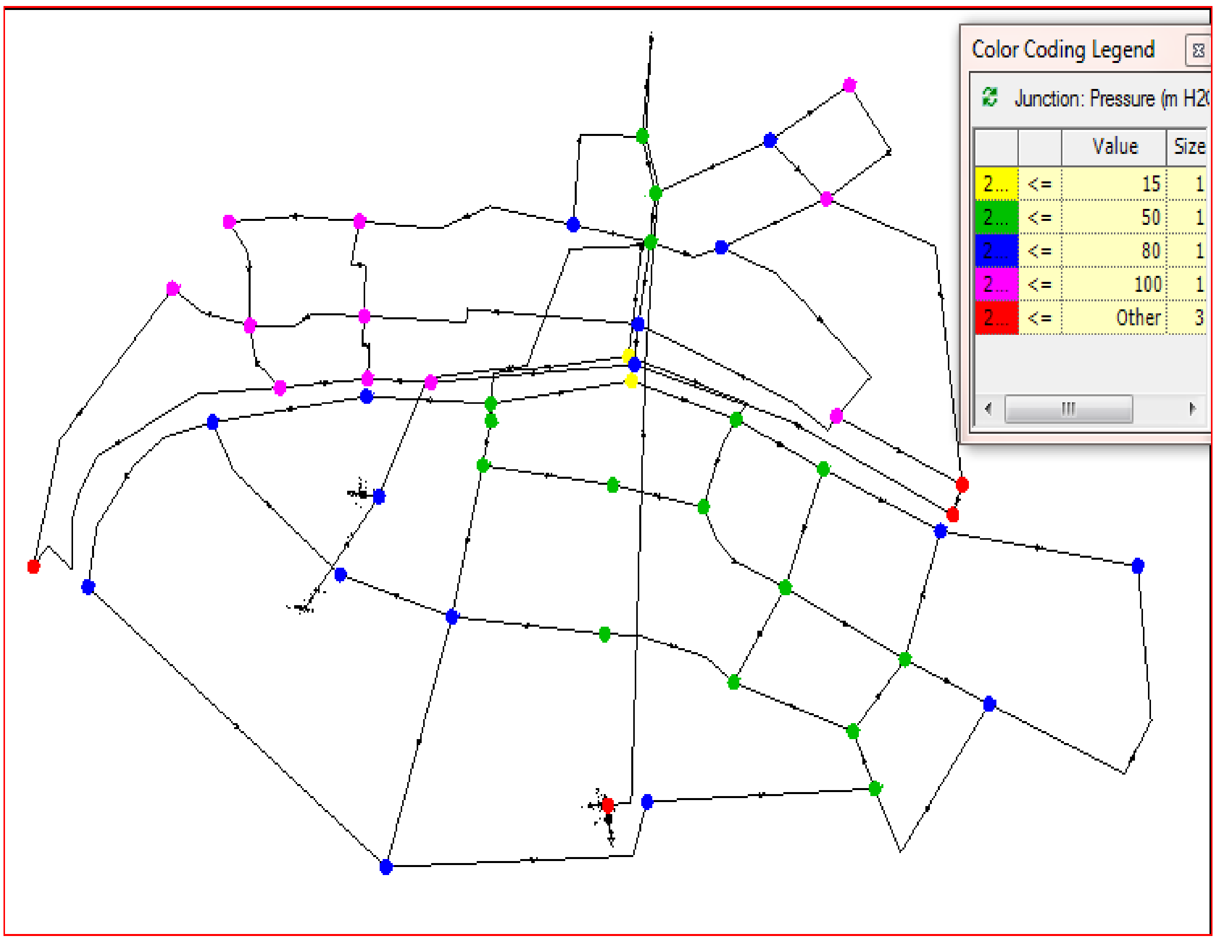

Figure 4-4: Pressure distribution at Junctions

4.5 Model Calibration and Validation

Calibration 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 hydraulic model is calibrated for two scenarios:

1. Steady state calibration:

Objective of the steady state calibration is to validate the assumed pipeline roughness coefficient(C-factors) in the hydraulic model and make modifications, where appropriate.

The water GEMs Model was calibrated by adjusting sensitive parameter such as Hazen Williams Coefficient. As the model gives automatically C value of GI Pipe 120 and for Pvc pipe 130 since, the system is old and should have roughness coefficient of pipe less than the model value. There for, this standard value of Hazen-Williams (C-value) used to adjust the model value until closed to the measured value.

Table 4-18: C-Value for pipe materials

S/N	Type of Material	C value for New pipe	C value for Existing pipe
1	PVC	130	100 - 110
2	Steel	110	90 - 110
3	DCI/GI	120	100 - 110

Source: Design Criteria Manual of MOWR (2006)

2. Extended period Simulation (EPS) Calibration:

A model calibrated for steady state scenario provides an instantaneous snapshot of a water distribution system. As steady state modeling does not involve time steps, the behavior of a water distribution system over time cannot be analyzed. The goal of EPS calibration is to estimate the accuracy with which the model simulates the field operations over a 24-hour period.

Five points of observed pressure data were selected at main and distributing networks as shown in the table below for calibration and validation.

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

4.5.1 Pressure Calibration and validation

ATSDR(2000) illustrated that an average pressure difference of **±15.2kpa (± 1.51 m)** with a maximum difference of **± 50.3 kpa (± 5.03 m)** represents a "**Good**" data set and an average pressure difference of **± 29.6 kpa (± 2.96 m)** with a maximum difference of **± 97.9 kpa (± 9.79 m)** represents a **"poor"** data set.

4.5.1.1 Sample location

Five representative samples measurement location have been selected for the calibration. It was difficult to take measurement at a direct connection to the water main nodes, due to size of pressure gauge available in the town, which is **15 mm** and **25 mm**. the size of water main in the study model integrates a size greater or equal to **100** mm as previously stated. The measurements were taken at a location other than the direct connection to the water mains, nearer to the supply main nodes at homes faucet. For the calibration, the head loss between the supply main nodes and the site where pressure is measured had been considered. The head loss included the elevation head and pipe friction loss between two corresponding locations. This is carried out to examine the levels of accuracy between the model and the actual physical network. Field test locations for this paper are identified through a process known as the sampling design problem which essentially defines the limiting calibration criteria that delineate the test location sample space **(Walski et al., 2003).** Test location sampling is done randomly and the following limiting criteria often used **(AWWA, 1999).**

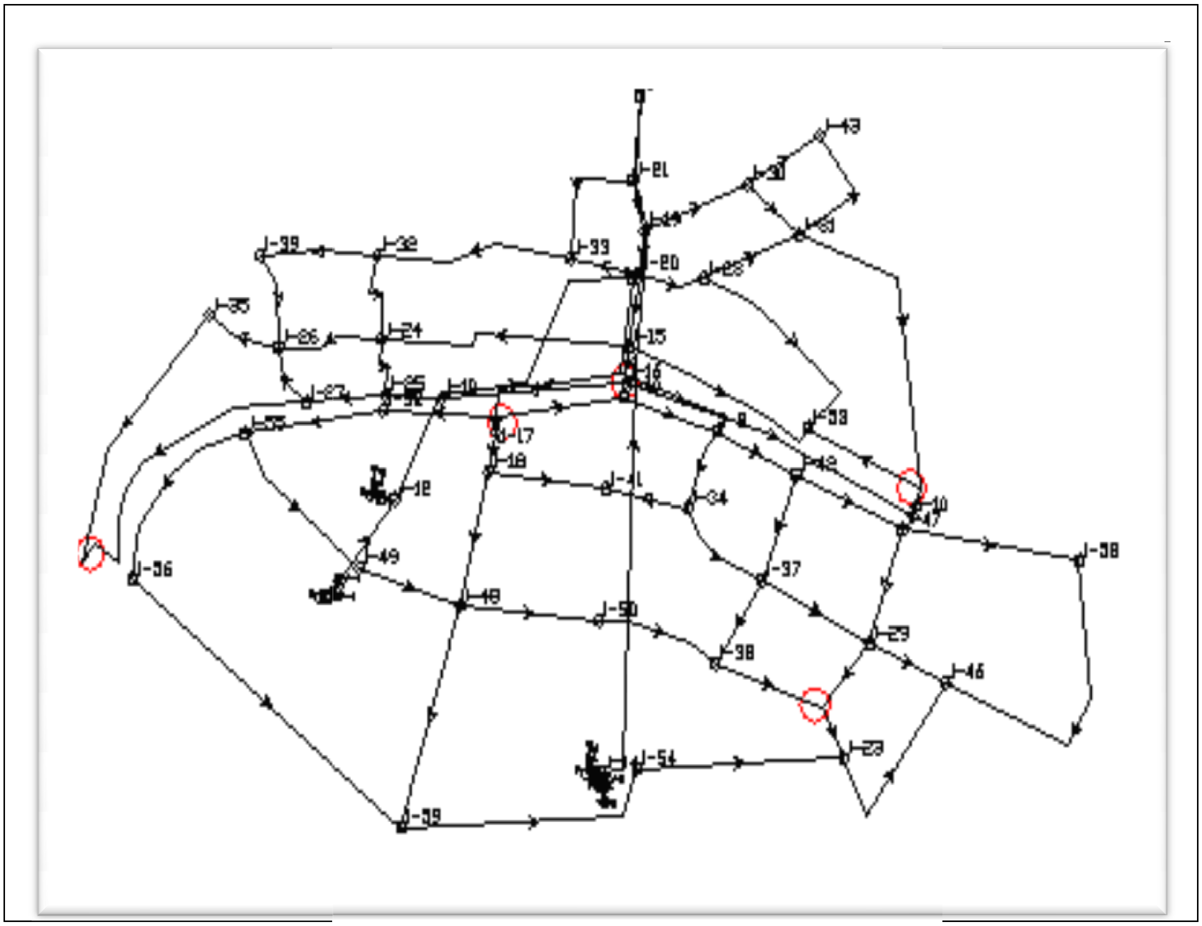

Figure 4-5: Sample locations in the distribution networks

4.5.1.2 Sampling Size

In general, internationally proposed guidelines stipulate that for a medium to highly detailed network model, the following limits should be adopted modeling based on **ECAC, 1999** sample calibration criteria for modeling pressure and flow criteria as mentioned below:

According to operational case criteria the sample size consideration has been taken:

Pressure:

1. Number of pressure reading **10% - 2%** of nodes and accuracy of pressure reading **± 2 psi (1.4 m)**
2. Number of flow reading in the pipe **2%** of pipes and accuracy of flow readings ± **5%.**

Junction:

- Total number of junctions in the network = **49** junctions
- The maximum acceptable sample **= 10 %** of all the junctions in the network.
- Hence, sample size = 0.1 x 49 = **4.9** which is approximately **5** junctions.

So, **5** node's points were tested in the network due to limited financial, material, resources for data collection and time constraints to measure all junctions five times each at the same time. It is difficult by only a single pressure gage on hand and those junctions sample location is also located at upstream, middle and downstream of the system to meet the sample location criteria. Due to above constraints **5** junctions have been observed for calibration and validation process.

Sites should be spread throughout the study area and should reflect a variety of situations of interest, such as transmission mains and local lines, areas served directly from a source ,and areas under the influence of tanks. In addition , sampling taps should be placed close to mains.

Table 4-19: Pressure Calibration at Junction based on measured pressure and simulated in the networks.

Time (hr)	Junction Points	Sample locations			Measured pressure(m)	Computed pressure(m)	Difference pressure error(m)
		x	y	z			
7:00AM	J-9	1360221.07	804881.6	2240	112.17	109	3.17
	J-16	1358762.65	805320.42	2286	66.28	63	3.28
	J-22	1359735.69	803983.34	2255	45.89	45	0.89
	J-51	1358122.62	805178.33	2256	50.99	47	3.99
	J-60	1356089.82	804584.51	2228	112.17	115	-2.83
9:00AM	J-9	1360221.07	804881.6	2240	101.97	106	-4.03
	J-16	1358762.65	805320.42	2286	61.18	60	1.18
	J-22	1359735.69	803983.34	2255	40.79	41	-0.21
	J-51	1358122.62	805178.33	2256	45.89	46	-0.11
	J-60	1356089.82	804584.51	2228	107.07	109	-1.93
12:00PM	J-9	1360221.07	804881.6	2240	101.97	105	-3.03
	J-16	1358762.65	805320.42	2286	61.18	60	1.18
	J-22	1359735.69	803983.34	2255	45.89	42	3.89
	J-51	1358122.62	805178.33	2256	40.79	45	-4.21
	J-60	1356089.82	804584.51	2228	112.17	110	2.17
	Average				**73.76**	**73.53**	**0.23**

As shown in the above table, measured values are within an average error of + **0.23 m** pressure simulated. Hence, the model is acceptable calibrated which is satisfied the setting pressure calibration and validation criteria under average level (average ± **1.5 m** to the maximum ± **5 m**).

4.5.2 Model performance Evaluation

There are many ways to evaluate the performance of model calibration. The evaluation was made by calculating the squared relative difference between observed and simulated pressure for each test.

Coefficient of determination (R^2) describes the degree of co linearity between simulated and measured data. The coefficient of determination, R^2, equation below, which ranges between 0 and 1, describes the proportion of the variance in the measured data, which is explained by the model, with higher values indicating less error variance. Typically, $\mathbf{R^2>0.5}$ is considered acceptable (Singh, 2004, Santhi, 2001).

The evaluation criteria used was statically method using correlation coefficient (R^2).

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

Where $\mathbf{R^2}$ is correlation coefficient, X and Y are measured and simulated values, X Mean and Y mean are average value of measured and simulated data respectively.

The calibration of pressures was done in statically method and **figure below** shows that the statistical correlation plots of observed versus computed pressure during calibration process. The result shows that $\mathbf{R^2}$= **0.9959.** This implies that the computed pressure is within the acceptable range. Hence, the model is well calibrated.

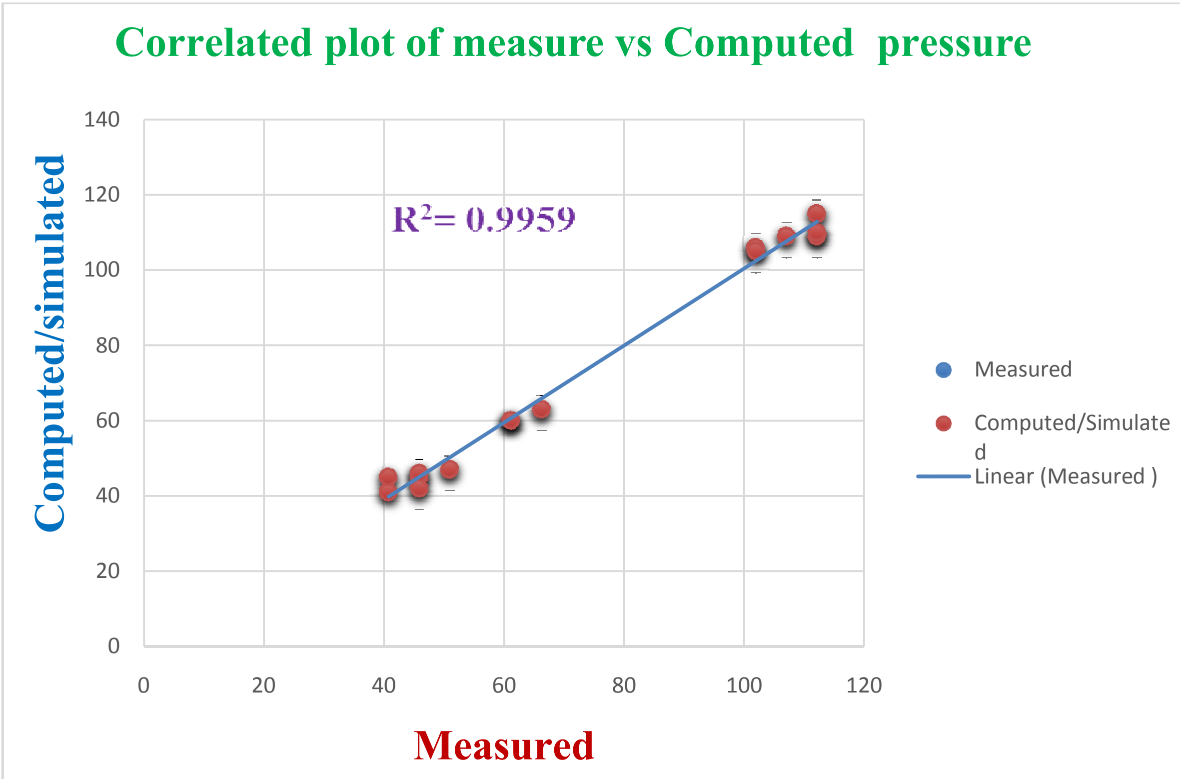

Figure 4-6: Shows correlated plot during peak hour demand for calibration

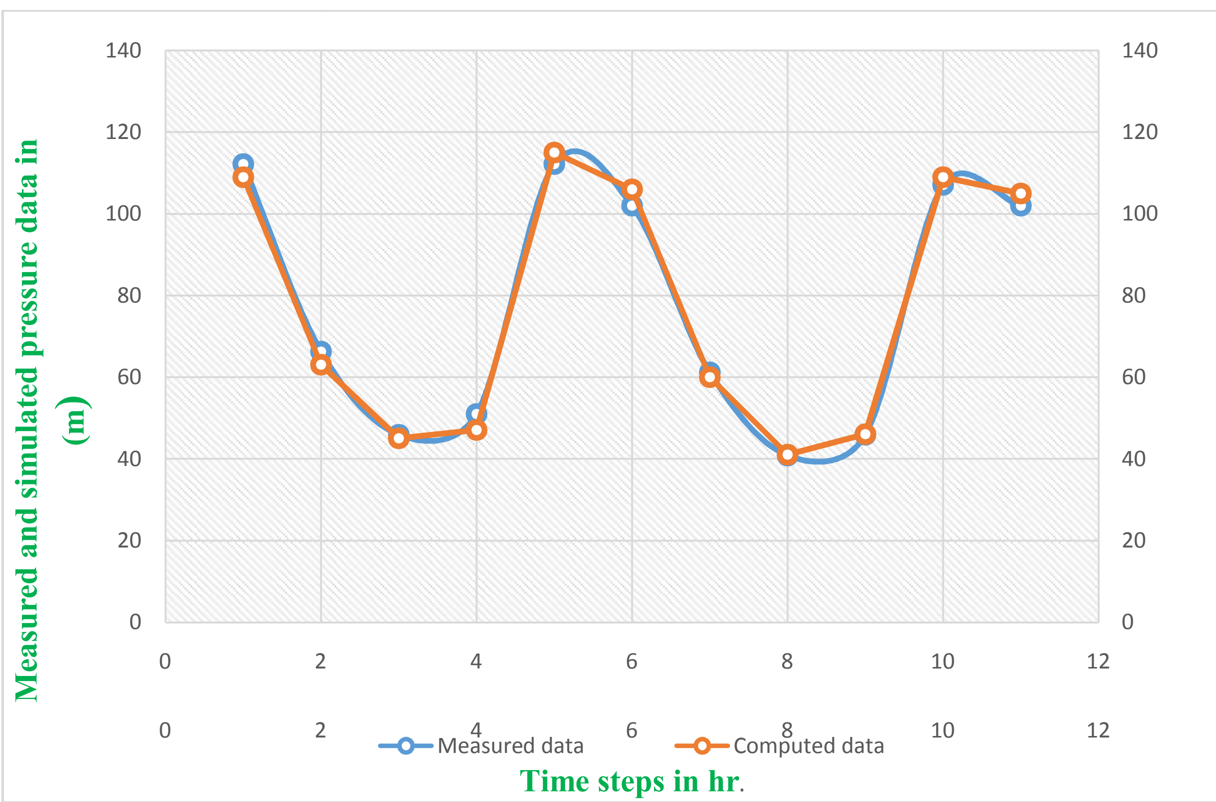

Figure 4-7: Shows that Pressure Calibration fitness test.

4.5.3 Sensitivity Analysis

Sensitivity Analysis Provides information on to what extent the hydraulic parameter (nodal pressures or flow velocity) change if a parameter is varied. The goal is to determine the sampling set with the lowest uncertainty in model parameters or predictions.

Table 4-20: Comparison of the measured and simulated pressure before and after calibration.

Nodes	Measured pressure(m)	Simulated pressure after calibration	Simulated Pressure before calibration	Av. Error before calibration	Av. Error After calibration
J-9	101.97	105	107	-1.6	-1.3
J-16	61.18	60	61	1.88	1.88
J-22	45.89	42	43	1.19	1.52
J-51	40.79	45	44	1.89	-0.11
J-60	112.17	110	114	3.53	0.86
			Total average error	**1.378**	**0.57**

In this study, the water distribution network pressure head at 5 nodes were measured hourly during a day. In the calibration process, the data of 5 nodes were used at test data. Comparison of results before and after calibration indicated that the average error of the test data **(J-9, J-16, J-22, J-51 & J-60)** was improved from **1.378** to **0.57**, which showed optimal performance of calibration model.

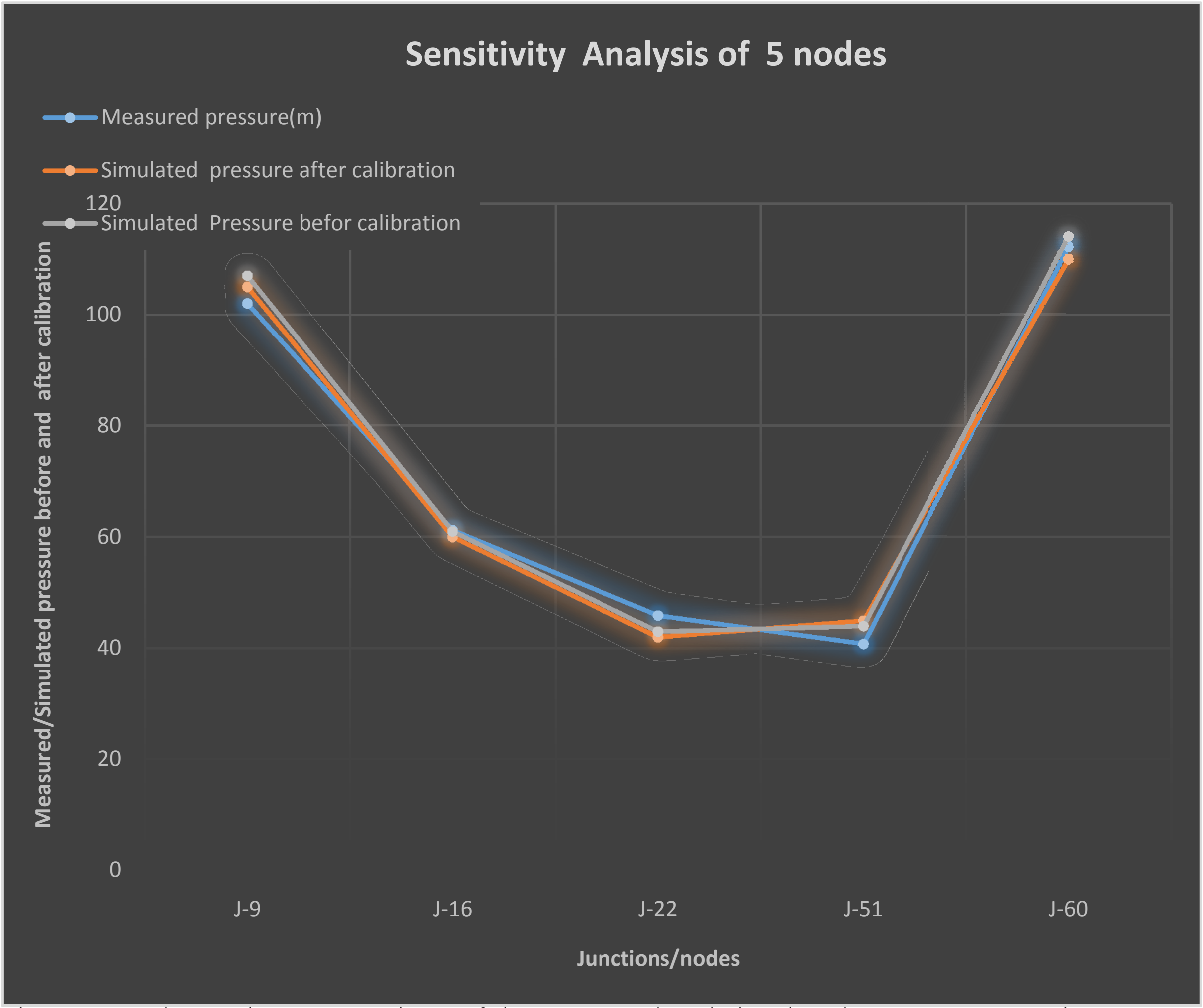

Figure: 4-8 shows that Comparison of the measured and simulated pressure curves in 5 nodes before and after calibration.

Table 4-21: Comparison of the measured and simulated pressure before and after calibration at J-60.

Time(hr.)	Measured Pressure	Simulated pressure after calibration	Simulated pressure before calibration	Error before calibration	Error After calibration
7:00	112.17	115	114	1.83	2.83
9:00	107.07	109	114	6.93	1.93
12:00	112.17	110	114	1.83	-2.17
			Average error	**3.53**	**0.86**

A comparison of results indicated that there is a significant difference between the measured and simulated data before calibration, but it is significantly close to the measured data after calibration.

Comparison of results before and after calibration indicates that the average error of the test data **(J-60)** was improved from **3.53 to 0.86**, which showed also optimal performance of calibration model.

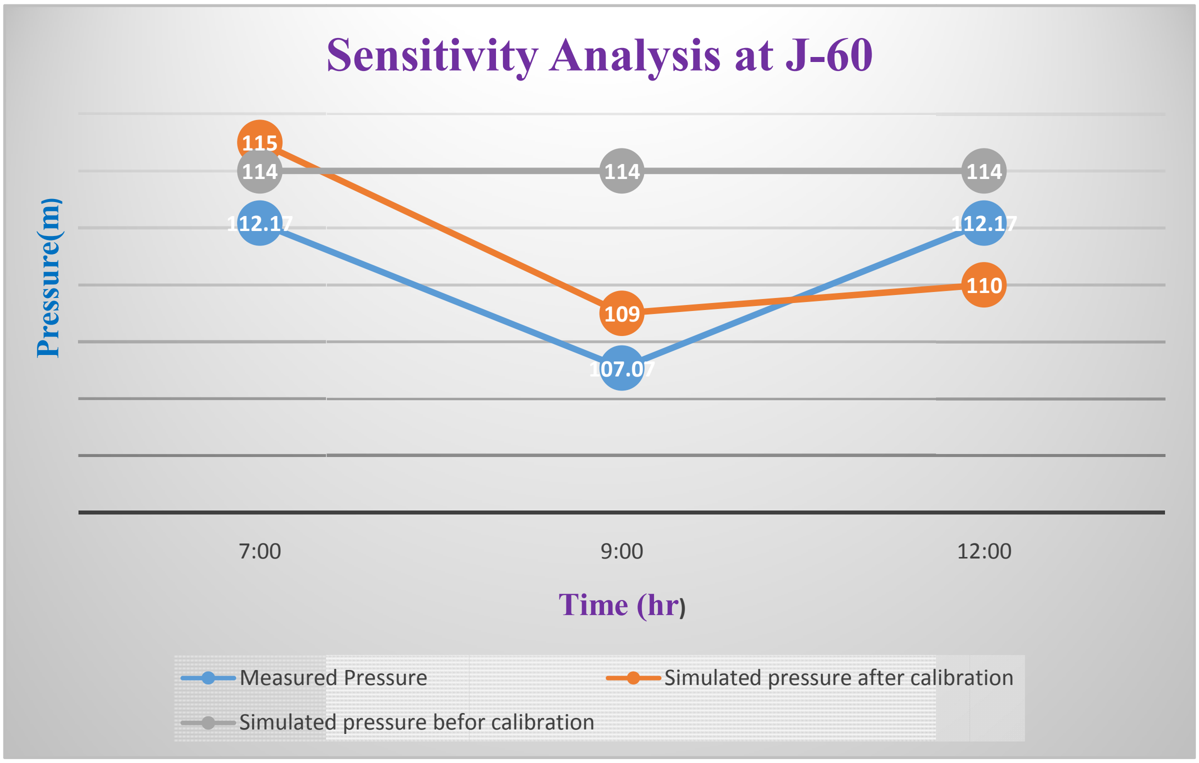

Figure 4-9: Shows that Comparison of the measured and simulated pressure curves in node J-60 before and after calibration.

4.5.3.1 HWC Sensitivity Analysis

In this study Hazen Williams's coefficients are the calibration parameter and measured data consist of nodal pressure heads. The defined objective function minimizes the difference between the measured and simulated values.

Historically, most attempts at model calibration have typically employed trial and error approach. Such an approach can prove to be extremely time consuming and frustrating when dealing with most typical water systems. The level of frustration will, of course, depend somewhat on the expertise of the modeler, the size of the system and the quantity and quality of the field data. Some of the frustration can be minimized by breaking complicated systems in to smaller parts and then calibrating the model parameters using an incremental approach.

Table 4-22: HW Coefficient Analysis at selected Node and Final adjustment of pipe roughness values.

Junctions	HWC	Av. Error before calibration	Av. Error After calibration	Error Diff.
J-9	130	-1.6	-1.3	-0.3
	110	-1.6	-1.3	-0.3
	100	-1.6	-1.3	-0.3
J-16	130	1.88	1.88	0
	110	1.88	1.88	0
	100	1.88	1.88	0
J-22	130	1.19	1.52	-0.33
	110	1.19	1.52	-0.33
	100	1.19	1.52	-0.33
J-51	130	1.89	-0.11	2
	110	1.89	-0.11	2
	100	1.89	0.22	1.67
J-60	130	3.53	0.86	2.67
	110	4.53	0.86	3.67
	100	4.53	0.86	3.67

4.5.4 Validation

Model validation is the steps that follows calibration and uses an independent observed data set to verify that the model is well calibrated. 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. Significant deviations indicate that further calibration is required **(USEPA, 2005).**

Table 4-23: Pressure Validation at Junction based on measured pressure and simulated in the networks.

Time (hr)	Junction Points	Sample locations x	y	z	Measured pressure(m)	Computed pressure(m)	Difference pressure error(m)
1:00AM	J-9	1360221.07	804881.6	2240	96.87	98	-1.13
	J-16	1358762.65	805320.42	2286	66.28	62	4.28
	J-22	1359735.69	803983.34	2255	45.89	46	-0.11
	J-51	1358122.62	805178.33	2256	45.89	46	-0.11
	J-60	1356089.82	804584.51	2228	117.27	120	-2.73
3:00AM	J-9	1360221.07	804881.6	2240	101.97	99	2.97
	J-16	1358762.65	805320.42	2286	61.18	63	-1.82
	J-22	1359735.69	803983.34	2255	45.89	48	-2.11
	J-51	1358122.62	805178.33	2256	45.89	47	-1.11
	J-60	1356089.82	804584.51	2228	117.27	121	-3.73
5:00AM	J-9	1360221.07	804881.6	2240	96.87	100	-3.13
	J-16	1358762.65	805320.42	2286	61.18	64	-2.82
	J-22	1359735.69	803983.34	2255	45.89	48	-2.11
	J-51	1358122.62	805178.33	2256	45.89	48	-2.11
	J-60	1356089.82	804584.51	2228	117.27	122	-4.73
					1112	**1,132**	**-20.5**
				Average	**74.10**	**75.47**	**-1.37**

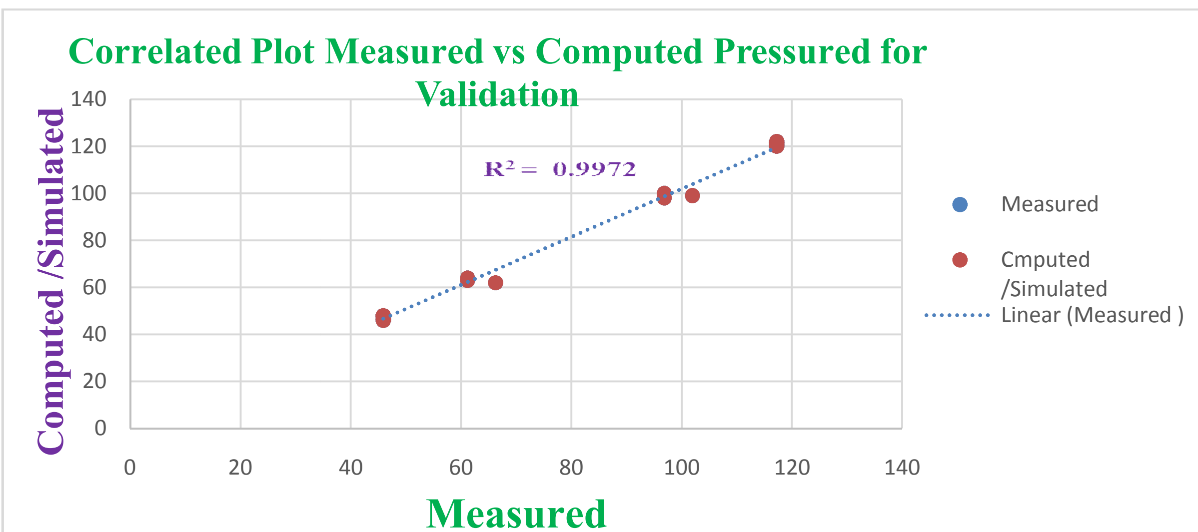

Figure 4-10: Correlated plot during minimum day demand for Validation

As depicted in figure 4-6 and 4-10; it explains the results of correlation value (R^2) for both peak hour and minimum day demand time was represent as **0.9959** and **0.9972** respectively. Therefore, the calibrated pressure value was validated within the recommended standard.

Table 4 -24 Peak Hour Day Demand Pressure Distribution at Junction

Label	X (m)	Y (m)	Elevation (m)	Hydraulic Grade (m)	Demand (L/s)	Pressure (m)
J-24	1,357,560.64	805,496.63	2,250.00	2,336.02	9.11	86
J-48	1,357,950.38	804,400.46	2,248.00	2,300.64	0.00	53
J-23	1,359,832.53	803,773.79	2,250.00	2,295.08	0.00	45
J-30	1,359,365.40	806,136.26	2,271.00	2,348.79	0.00	78
J-45	1,358,736.99	805,350.96	2,289.00	2,302.92	0.00	14
J-56	1,356,331.92	804,510.00	2,225.00	2,300.66	0.00	76
J-54	1,358,818.87	803,727.42	2,250.00	2,300.58	0.00	50
J-29	1,359,965.88	804,246.07	2,251.00	2,294.93	0.00	44
J-18	1,358,087.70	804,952.12	2,253.00	2,301.49	0.00	48
J-42	1,359,602.34	804,939.02	2,262.00	2,295.44	0.00	33
J-53	1,359,662.27	805,132.43	2,257.00	2,347.24	0.00	90
J-51	1,358,122.62	805,178.33	2,256.00	2,301.84	0.00	46
J-16	1,358,762.65	805,320.42	2,286.00	2,346.57	0.00	60
J-17	1,358,124.21	805,115.63	2,255.00	2,301.81	2.39	47
J-33	1,358,490.92	805,829.21	2,283.00	2,347.71	0.00	65
J-25	1,357,574.93	805,268.03	2,250.00	2,335.87	1.78	86
J-43	1,359,719.42	806,337.34	2,259.00	2,348.60	0.00	89
J-28	1,359,150.03	805,747.32	2,284.00	2,347.67	1.62	64
J-35	1,356,708.15	805,597.44	2,252.00	2,324.07	10.82	72
J-32	1,357,539.21	805,842.70	2,256.00	2,344.01	0.00	88
J-41	1,358,666.51	804,881.07	2,275.00	2,296.00	3.04	21
J-10	1,360,179.79	804,772.59	2,242.00	2,346.94	0.00	105
J-58	1,361,001.06	804,586.33	2,232.00	2,286.77	6.73	55
J-22	1,359,735.69	803,983.34	2,255.00	2,294.97	0.00	40
J-27	1,357,185.99	805,236.28	2,249.00	2,334.96	0.77	86
J-26	1,357,051.06	805,463.29	2,245.00	2,330.96	1.98	86
J-60	1,356,089.82	804,584.51	2,228.00	2,333.72	0.00	106
J-37	1,359,433.27	804,507.22	2,270.00	2,295.16	1.98	25
J-52	1,357,570.17	805,204.53	2,247.00	2,300.94	2.34	54
J-21	1,358,797.08	806,152.14	2,308.00	2,349.84	5.67	42
J-34	1,359,069.73	804,801.70	2,281.00	2,296.37	1.78	15
J-39	1,356,958.98	805,841.12	2,252.00	2,343.41	0.00	91
J-19	1,358,856.34	805,944.87	2,304.00	2,348.98	0.85	45
J-31	1,359,616.23	805,924.06	2,257.00	2,348.29	0.00	91
J-47	1,360,123.83	804,714.39	2,246.00	2,293.13	1.46	47
J-40	1,358,749.85	805,260.49	2,286.00	2,301.07	1.75	15
J-46	1,360,340.53	804,083.35	2,244.00	2,294.78	0.00	51
J-8	1,359,214.99	805,120.79	2,278.00	2,299.89	2.83	22

J-9	1,360,221.07	804,881.60	2,240.00	2,346.97	0.00	107
J-15	1,358,779.61	805,467.92	2,292.00	2,346.74	2.92	55
J-49	1,357,453.49	804,554.71	2,244.00	2,298.57	2.59	54
J-59	1,357,658.28	803,489.23	2,236.00	2,300.58	0.00	64
J-55	1,356,885.96	805,110.07	2,244.00	2,300.67	1.30	57
J-50	1,358,630.79	804,338.94	2,263.00	2,295.25	5.22	32
J-38	1,359,204.67	804,161.94	2,271.00	2,294.77	2.18	24
J-20	1,358,833.59	805,766.37	2,301.00	2,348.34	0.74	47
J-10	1,357,856.12	805,255.10	2,252.00	2,337.85	0.00	86

According to the result of the model: Maximum pressure is recorded **86 -107 m** of water in 14-nodes out of 49 nodes during peak hour day demand. This it shows **28.57%** of the nodes are subjected to maximum pressure. The rest of the nodes have normal pressure that **is 69.39%.** Only **2.04%** of nodes have minimum pressure which means less than **15 m** of water. Generally, reducing excess pressures in distribution system reduces flow rates from existing leaks and extends infrastructure working life.

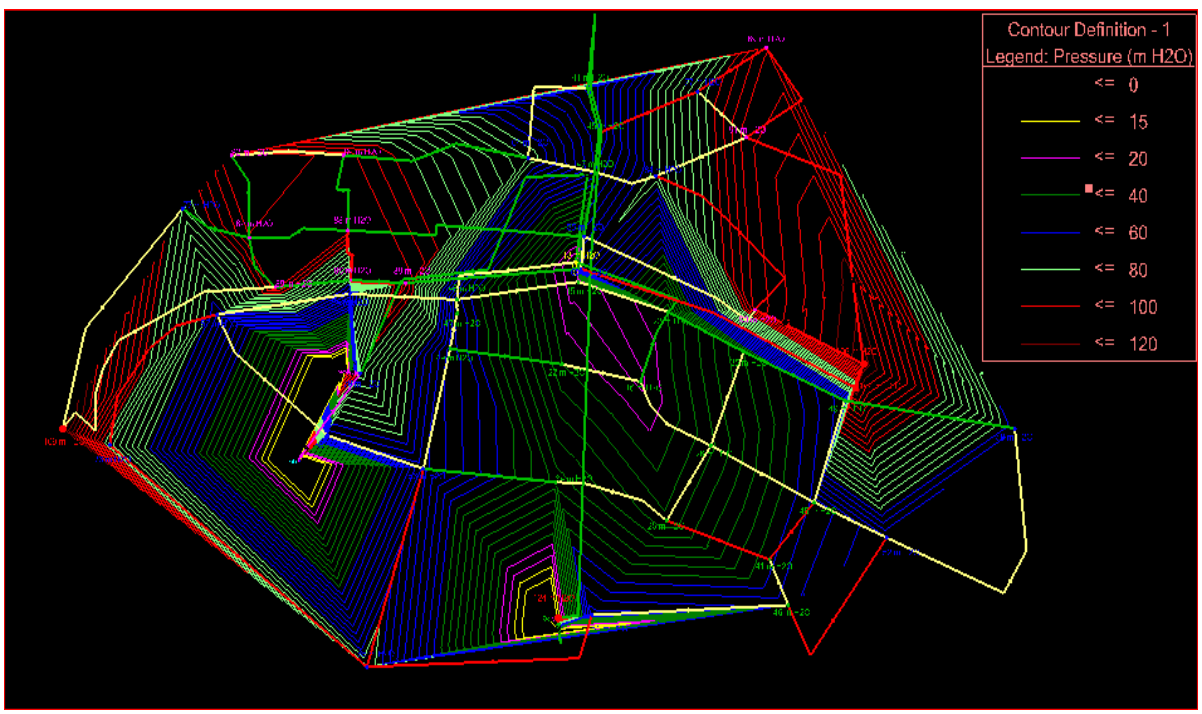

Figure 4-11: Pressure Distribution by Contour at Peak hour Day Demand (8:00 hr.)

The model shows that most of the system minimum pressures are less than **15 m** of water during the maximum day. The figure above shows the minimum pressures throughout the system on maximum day. The evaluation criteria state that pressures above **15 m** are desirable. Pressure range from **15 to 107 m** of water on the west side of the system and decrease to the east with minimum pressures ranging from **14 to 15 m**.

Figure 4-12: Pressure contour at Peak hour-2 Day Demand (15:00 hr.)

Table: 4 -25 Pressure Distribution during Minimum Day Demand

Label	X (m)	Y (m)	Elevation (m)	Hydraulic Grade (m)	Pressure (m)
J-24	1,357,560.64	805,496.63	2,250.00	2,350.36	100
J-48	1,357,950.38	804,400.46	2,248.00	2,303.78	56
J-23	1,359,832.53	803,773.79	2,250.00	2,303.57	53
J-30	1,359,365.40	806,136.26	2,271.00	2,350.81	80
J-45	1,358,736.99	805,350.96	2,289.00	2,303.84	15
J-56	1,356,331.92	804,510.00	2,225.00	2,303.78	79
J-54	1,358,818.87	803,727.42	2,250.00	2,303.78	54
J-29	1,359,965.88	804,246.07	2,251.00	2,303.56	52
J-18	1,358,087.70	804,952.12	2,253.00	2,303.81	51
J-42	1,359,602.34	804,939.02	2,262.00	2,303.58	41
J-53	1,359,662.27	805,132.43	2,257.00	2,350.76	94
J-51	1,358,122.62	805,178.33	2,256.00	2,303.82	48
J-16	1,358,762.65	805,320.42	2,286.00	2,350.72	65
J-17	1,358,124.21	805,115.63	2,255.00	2,303.82	49
J-33	1,358,490.92	805,829.21	2,283.00	2,350.77	68
J-25	1,357,574.93	805,268.03	2,250.00	2,350.36	100

J-43	1,359,719.42	806,337.34	2,259.00	2,350.80	92
J-28	1,359,150.03	805,747.32	2,284.00	2,350.77	67
J-35	1,356,708.15	805,597.44	2,252.00	2,349.93	98
J-32	1,357,539.21	805,842.70	2,256.00	2,350.64	94
J-41	1,358,666.51	804,881.07	2,275.00	2,303.60	29
J-10	1,360,179.79	804,772.59	2,242.00	2,350.72	108
J-58	1,361,001.06	804,586.33	2,232.00	2,303.27	71
J-22	1,359,735.69	803,983.34	2,255.00	2,303.56	48
J-27	1,357,185.99	805,236.28	2,249.00	2,350.33	101
J-26	1,357,051.06	805,463.29	2,245.00	2,350.18	105
J-60	1,356,089.82	804,584.51	2,228.00	2,350.28	122
J-37	1,359,433.27	804,507.22	2,270.00	2,303.57	33
J-52	1,357,570.17	805,204.53	2,247.00	2,303.79	57
J-21	1,358,797.08	806,152.14	2,308.00	2,350.84	43
J-34	1,359,069.73	804,801.70	2,281.00	2,303.61	23
J-39	1,356,958.98	805,841.12	2,252.00	2,350.62	98
J-19	1,358,856.34	805,944.87	2,304.00	2,350.81	47
J-31	1,359,616.23	805,924.06	2,257.00	2,350.80	94
J-47	1,360,123.83	804,714.39	2,246.00	2,303.50	57
J-40	1,358,749.85	805,260.49	2,286.00	2,303.80	18
J-46	1,360,340.53	804,083.35	2,244.00	2,303.56	59
J-8	1,359,214.99	805,120.79	2,278.00	2,303.74	26
J-9	1,360,221.07	804,881.60	2,240.00	2,350.72	110
J-15	1,358,779.61	805,467.92	2,292.00	2,350.73	59
J-49	1,357,453.49	804,554.71	2,244.00	2,303.71	60
J-59	1,357,658.28	803,489.23	2,236.00	2,303.78	68
J-55	1,356,885.96	805,110.07	2,244.00	2,303.78	60
J-50	1,358,630.79	804,338.94	2,263.00	2,303.58	40
J-38	1,359,204.67	804,161.94	2,271.00	2,303.56	32
J-20	1,358,833.59	805,766.37	2,301.00	2,350.79	50
J-11	1,357,856.12	805,255.10	2,252.00	2,350.45	98

Table 4-26: Pressure line (pumped line) at minimum and peak day demand

Label	X (m)	Y (m)	Elevation (m)	Hydraulic Grade (m)	Demand (L/s)	Pressure (m)
J-14	1,358,683.03	803,713.85	2,252.00	2,383.90	0.00	132
J-12	1,357,625.63	804,838.94	2,247.00	2,330.05	0.00	83

According to the result of the model: maximum pressure is recorded **92-122 m** of water in **15**-nodes out of **49** nodes during minimum day demand. This it shows **30.6%** of the nodes are subjected to maximum pressure. The rest of the nodes have normal pressure that **is 67.35%.** Only **one** node **(2.04%)** has minimum pressure which means **14 m** of water.

Generally, unless we manage the excess pressure in distribution system, the leakage also increased by **30%.**

The basis of this excess pressure in the distribution systems are:

- ☞ Topographic Constraint (elevation deference of 150 m) in the network system.
- ☞ Direct pumping from the borehole to the existing network system.
- ☞ Demand fluctuation at peak day and minimum day demand.
- ☞ Lack of enough Sustaining Reducing Valve in the existing network systems are the major one.

Therefore, minimizing excess (unnecessary) pressures in the water distribution system as well as removing transients. It can be implemented through suitable pressure zoning and DMA management. It should be borne in mind that simple and inexpensive pressure management activities can often lead to considerable reductions in Real Losses.

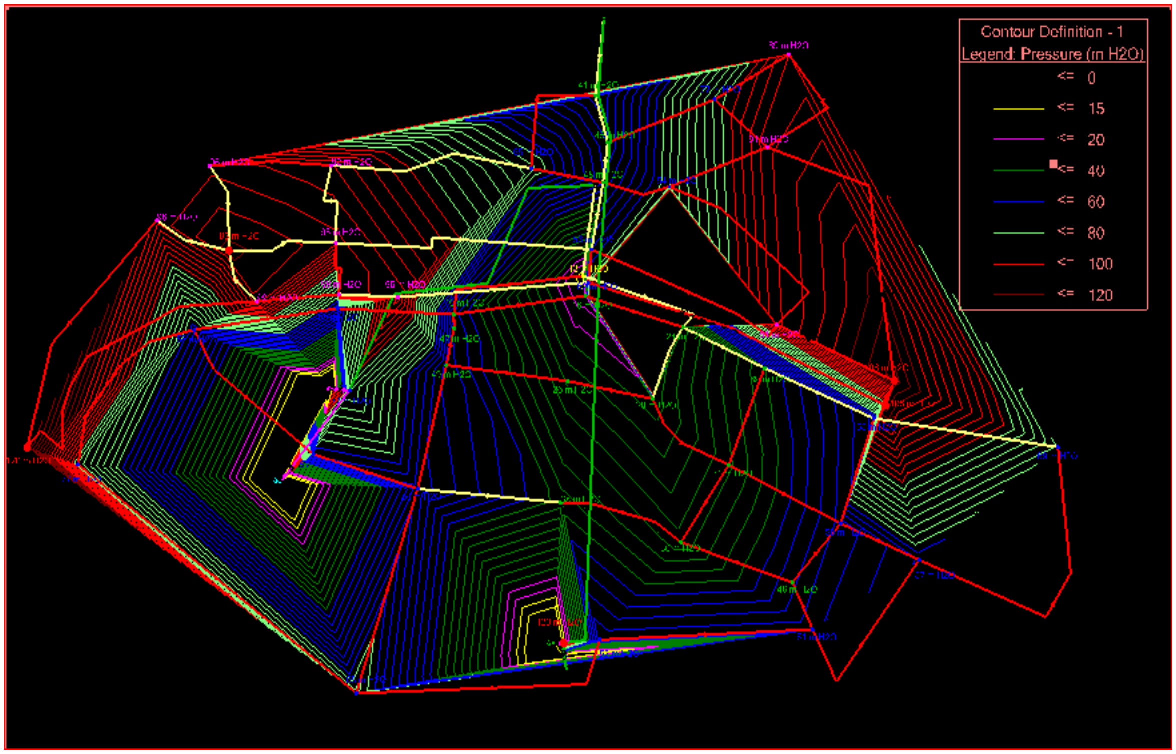

Figure 4 -13: Pressure distribution by contour at Minimum day demand at (1:00hr.)

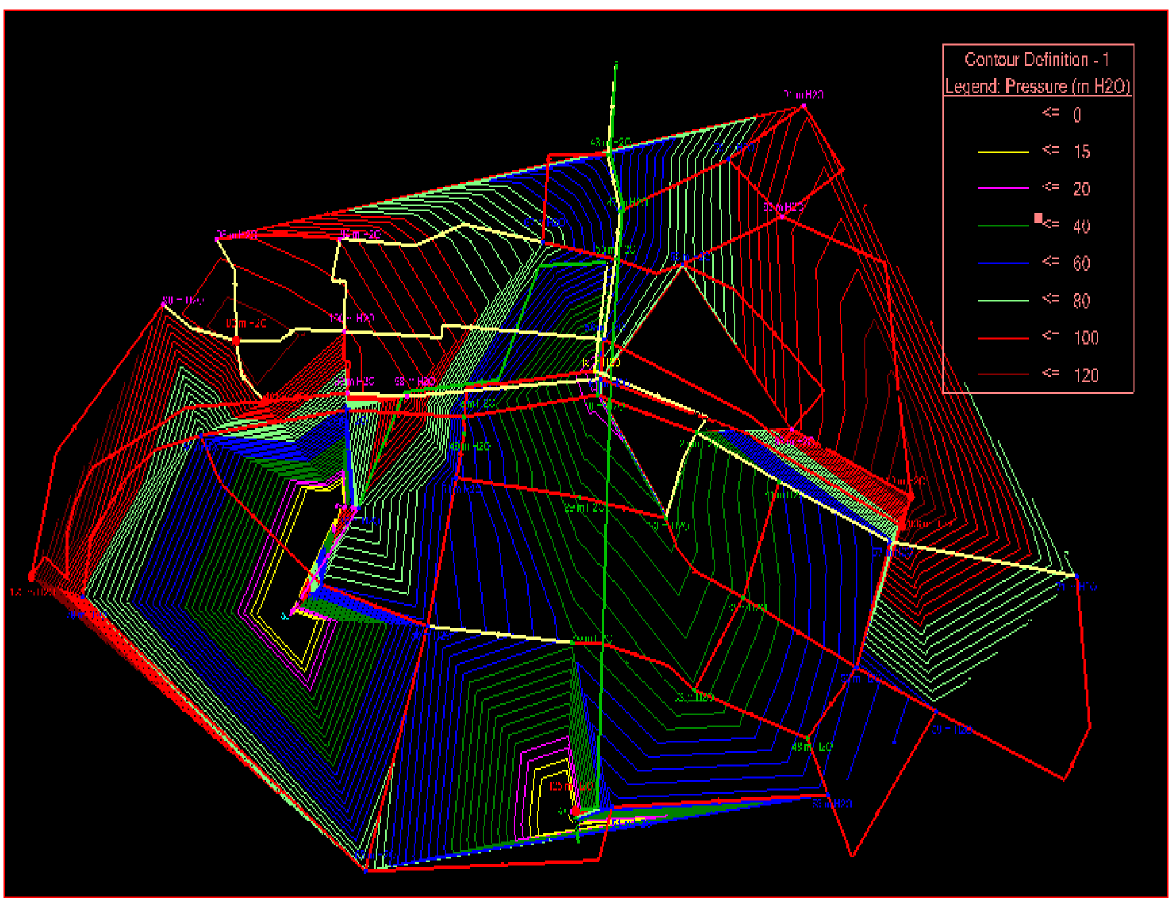

Figure 4-14: Pressure distribution by contour at (6:00 hr.)

4.5.4.2 Velocity, Head loss and Diameter of distribution and pressure line pipes in the system

Table: 4-27 Velocity, head loss and diameter of distribution pipes report at peak day demand.

Label	Start Node	Stop Node	Length (m)	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Head loss (Friction) (m)
P-17	T-1	J-21	354	250	PVC	130	36.25	0.739	0.84
P-18	J-21	J-19	215	200	PVC	130	26.56	0.846	0.86
P-19	J-19	J-20	180	200	PVC	130	25.08	0.798	0.64
P-20	J-20	J-15	303	150	PVC	130	14.53	0.822	1.6
P-24	J-15	J-16	148	150	PVC	110	5.39	0.305	0.17
P-25	T-2	J-45	407	250	PVC	130	35.59	0.725	0.94
P-26	J-20	J-33	348	150	PVC	130	8.15	0.461	0.63
P-27	J-21	J-33	605	100	PVC	130	4.02	0.512	2.13
P-28	J-33	J-32	972	150	PVC	130	12.17	0.689	3.7
P-29	J-32	J-24	413	100	PVC	130	10.08	1.284	7.99
P-30	J-24	J-26	522	100	PVC	130	6.94	0.884	5.06
P-32	J-32	J-39	582	100	PVC	130	2.08	0.265	0.61
P-33	J-39	J-26	407	50	PVC	130	2.08	1.062	12.44
P-34	J-26	J-35	375	100	PVC	130	9.81	1.25	6.89
P-35	J-60	J-35	1,221	50	PVC	130	1.00	0.512	9.65
P-36	J-27	J-26	278	65	PVC	130	2.77	0.834	4
P-37	J-25	J-27	206	100	PVC	130	4.55	0.579	0.91
P-39	J-25	J-24	258	65	PVC	130	-0.49	0.147	0.15
P-40	J-27	J-60	1,545	80	PVC	130	1.00	0.2	1.24
P-41	J-55	J-56	892	150	PVC	130	0.38	0.022	0.01
P-42	J-52	J-55	691	150	PVC	130	3.58	0.202	0.27
P-46	J-45	J-51	716	250	PVC	100	21.80	0.444	1.08
P-47	J-51	J-17	63	250	PVC	100	10.84	0.221	0.03
P-48	J-17	J-18	168	150	PVC	130	8.45	0.478	0.32
P-49	J-18	J-48	568	150	PVC	130	7.34	0.416	0.85
P-50	J-48	J-59	957	100	PVC	130	0.44	0.056	0.06
P-51	J-56	J-59	1,674	100	PVC	130	0.38	0.049	0.08
P-52	J-55	J-49	807	80	PVC	130	1.90	0.378	2.1
P-53	J-48	J-49	520	50	PVC	130	0.69	0.353	2.07
P-54	J-48	J-50	683	100	PVC	130	6.21	0.791	5.39
P-57	J-15	J-24	1,265	100	PVC	130	6.45	0.822	10.72
P-58	J-10	J-25	281	100	PVC	130	5.84	0.744	1.98

P-59	J-16	J-10	909	100	PVC	110	5.84	0.744	8.72
P-60	J-51	J-52	553	150	PVC	100	5.92	0.335	0.9
P-61	J-51	J-40	632	150	PVC	100	5.04	0.285	0.77
P-62	J-18	J-41	583	50	PVC	130	1.10	0.562	5.49
P-63	J-19	J-30	544	80	PVC	130	0.64	0.127	0.19
P-64	J-30	J-43	407	50	PVC	130	0.22	0.113	0.2
P-65	J-30	J-31	329	50	PVC	130	0.42	0.212	0.51
P-68	J-31	J-28	499	50	PVC	130	0.37	0.188	0.61
P-69	J-43	J-31	639	50	PVC	130	0.22	0.113	0.31
P-70	J-20	J-28	329	80	PVC	130	1.66	0.33	0.67
P-75	J-40	J-8	486	100	PVC	130	3.30	0.42	1.19
P-76	J-8	J-42	428	100	PVC	130	7.21	0.918	4.45
P-77	J-42	J-47	254	100	PVC	130	6.70	0.853	2.31
P-78	J-47	J-58	887	100	PVC	130	5.90	0.752	6.37
P-81	J-9	J-10	116	80	PVC	110	0.45	0.089	0.03
P-82	J-31	J-9	1,393	50	PVC	110	0.27	0.137	1.32
P-83	J-10	J-16	1,524	80	PVC	110	0.45	0.089	0.38
P-84	J-53	J-9	613	50	PVC	110	0.18	0.091	0.27
P-85	J-28	J-53	1,037	65	PVC	130	0.41	0.123	0.43
P-87	J-53	J-15	996	50	PVC	130	0.23	0.116	0.51
P-88	J-45	J-8	633	150	PVC	130	13.78	0.78	3.03
P-89	J-8	J-34	353	100	PVC	130	7.05	0.897	3.52
P-90	J-34	J-41	411	100	PVC	130	1.94	0.247	0.37
P-91	J-34	J-37	490	100	PVC	130	3.33	0.424	1.22
P-92	J-42	J-37	464	65	PVC	130	0.51	0.152	0.29
P-93	J-37	J-38	414	65	PVC	130	0.63	0.191	0.39
P-94	J-50	J-38	615	80	PVC	130	0.99	0.197	0.48
P-95	J-22	J-38	560	80	PVC	110	0.55	0.11	0.2
P-96	J-22	J-29	349	50	PVC	110	0.09	0.046	0.04
P-97	J-29	J-47	494	50	PVC	130	0.66	0.336	1.79
P-100	J-37	J-29	593	100	PVC	130	1.22	0.155	0.23
P-101	J-23	J-22	231	80	PVC	110	0.64	0.128	0.11
P-102	J-23	J-46	930	50	PVC	130	0.18	0.091	0.3
P-103	J-46	J-58	1,448	50	PVC	130	0.83	0.422	8.01
P-104	J-29	J-46	408	80	PVC	130	0.65	0.129	0.15
P-105	J-54	J-23	1,015	50	PVC	130	0.82	0.417	5.5
P-108	J-59	J-54	1,306	250	PVC	130	0.82	0.017	0

Table: 4-28: Velocity, head loss and diameter of pressure line (pumped line) pipe report at peak day demand.

Label	Start Node	Stop Node	Length (m)	Diameter (mm)	Material	Hazen - Williams C	Flow (L/s)	Velocity (m/s)	Head loss (Friction) (m)
P-1	R-1	PMP-1	62	100	Galvanized iron	110	11.69	1.489	2.17
P-3	R-2	PMP-2	55	75	Galvanized iron	120	7.08	1.603	2.58
P-8	J-14	T-1	2,858	150	Galvanized iron	110	18.77	1.062	33.06
P-9	R-4	PMP-4	37	75	Galvanized iron	120	12.24	2.771	4.85
P-11	R-3	PMP-3	37	75	Galvanized iron	110	9.57	2.167	3.56
P-16	J-12	T-2	1,715	150	Galvanized iron	110	21.81	1.234	26.2
P-111	PMP-1	CV-1	20	100	Galvanized iron	110	11.69	1.489	0.7
P-112	CV-1	J-14	48	100	Galvanized iron	120	11.69	1.488	1.4
P-113	CV-2	J-14	48	57	Galvanized iron	120	7.08	2.776	8.63
P-114	PMP-2	CV-2	51	75	Galvanized iron	120	7.08	1.603	2.41
P-115	CV-3	J-12	64	75	Galvanized iron	120	9.57	2.167	5.3
P-116	PMP-3	CV-3	18	75	Galvanized iron	110	9.57	2.167	1.75
P-117	PMP-4	CV-4	83	100	Galvanized iron	120	12.24	1.559	2.67
P-118	CV-4	J-12	441	100	Galvanized iron	120	12.24	1.559	14.17

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. Velocity distribution is also varying with demand pattern changes. Based on the model result in table 4-21, **4.82%** of the pipes, do not satisfy the minimum velocity at service taps (the velocities are below the standard). The water supply distribution system network velocity during peak hour demand is summarized in the figure 4-9.

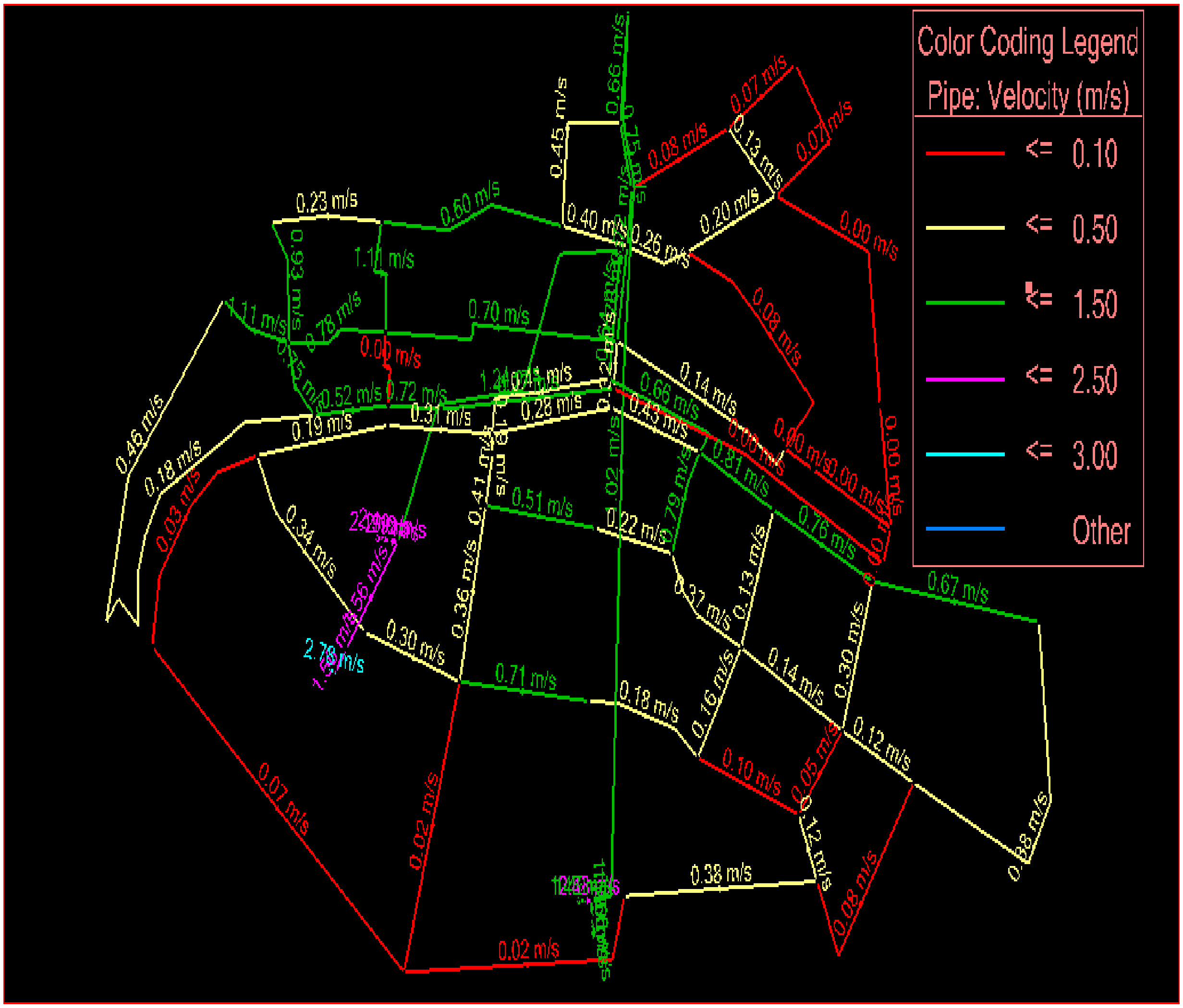

Figure 4-15: Velocity distribution in the pipes at peak hour day demand

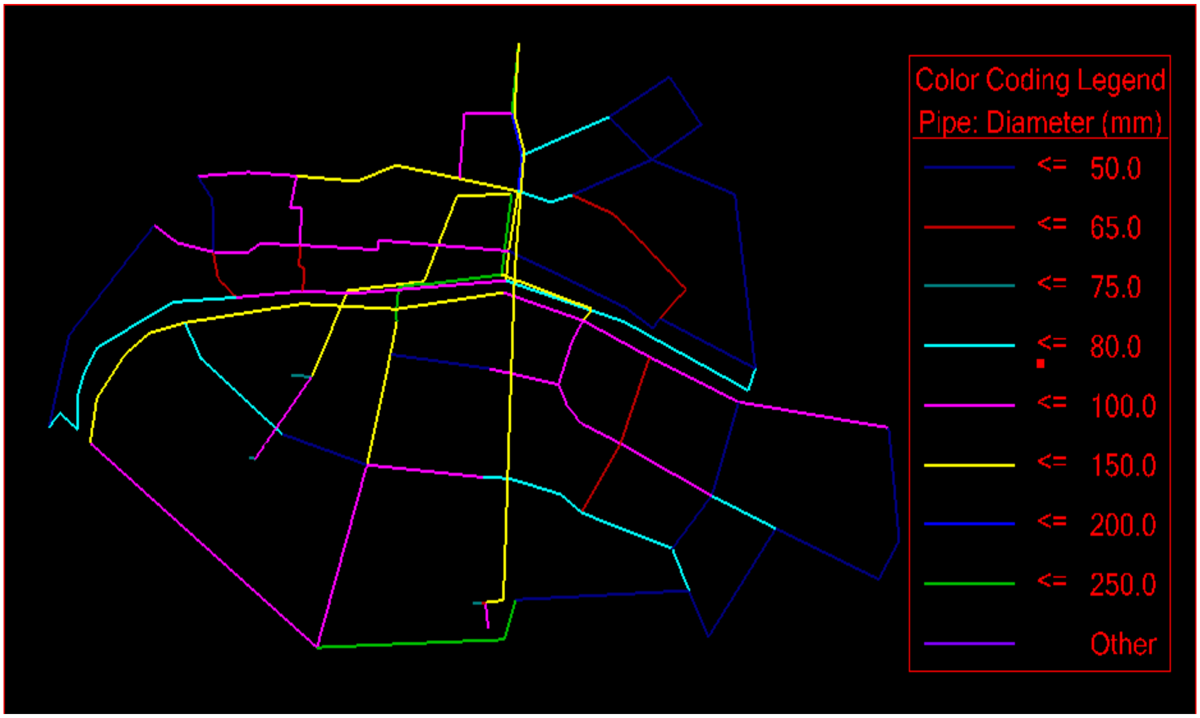

Figure 4 -16: Pipe Diameter in distribution system

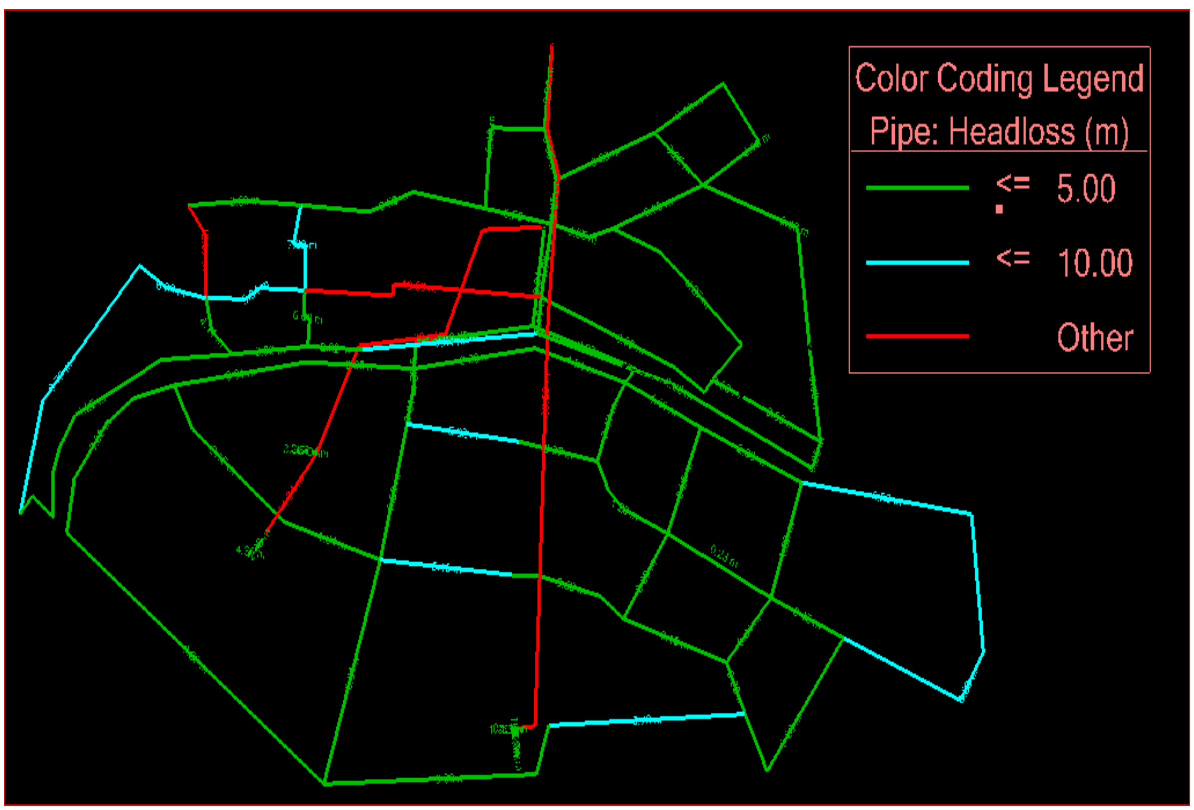

Figure 4-17: head losses in the pipe at peak hour day demand.

Table 4-29: Maximum head losses in the distribution systems

Distribution lines		Pressure Lines		Head loss (m/km)
Pipe number	Maximum head losses (m)	Pipe number	Maximum head losses (m)	
P-33	12.44	P-8	33.06	11.56
P-57	10.72	P-16	26.2	15.28
P-35	9.65	P-118	14.17	32.13
P-59	8.72			

In order to improve the flow rate and minor loss in the pressure line (pumped line) system, pipes **8, 16** and **118** were redesigned by changing pipe layout into straight pipe lay out. This reduces the minor losses in the fittings and the unnecessary turns and the only losses remaining would be the friction loss along the pipe for that section. This eliminates the minor losses slightly in the fittings and friction losses along the length of pipe.

4.6 Performance Evaluations

In this section is made the analysis of the data related to the performance evaluation of the distribution systems based on velocity and pressure (minimum and maximum).

4.6.1 Performance Assessment for Velocity

Based on permissible velocities stated by **Walski et al., Thomas , Chase, Donald , Savic, Dragan and TAHAL, 2015**, about **4.82%** of the distribution network systems pipes have a low performance that is less than **0.05 m/s,** all of transmission lines network systems **(10.84%),** like pipe-1, 9, 11,111, 112,113, 115, 116,117 and 118 have very good performance **(1.5 - 2.78 m/s),** about **30.12%** of distribution network system pipes have good performance **(i.e. 0.6- 1.5 m/s**) and about **54.22%** of distribution pipe systems have satisfactory performance between **0.05 and 0.6 m/s.**

Based on model output, the performance assessment related to minimum and maximum velocities, which means that hydraulic problems associated to velocity are caused by low velocities. On the other hand, the performance assessment related to maximum velocity shows that all the network systems have an average performance very close to optimum values, means that systems do not have problems related to high velocity.

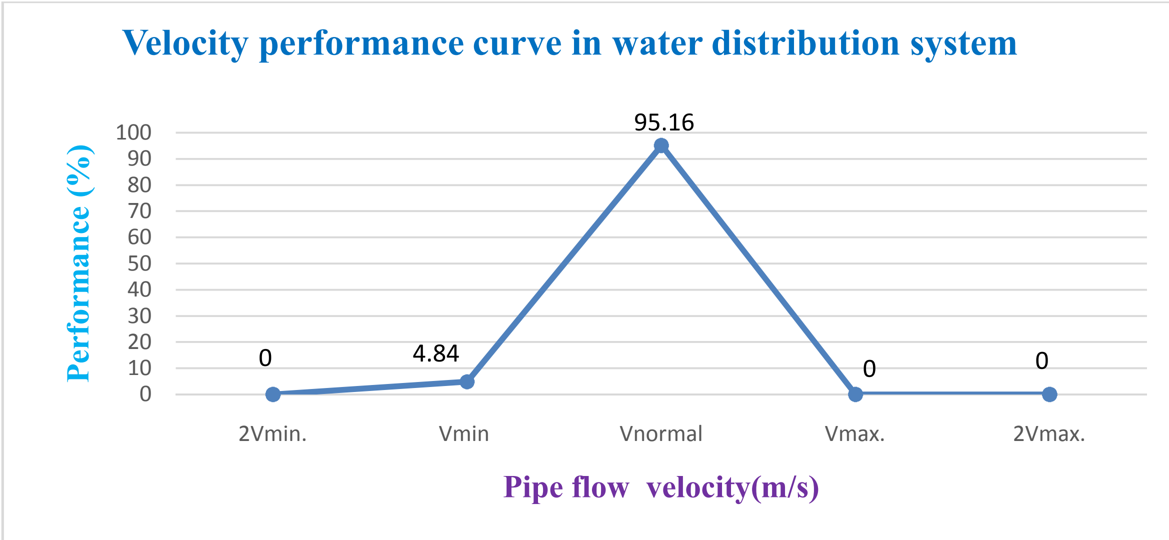

Figure 4-18: Velocity performance curve in distribution system during Peak day demand.

4.6.2 Performance Assessment for Pressure

Based on the model output results related to performance evaluation of pressure analysis also shows that some distribution systems (J-45, J-40, J-24, J-32, J-43, J-53, J-14, J-25, J-10, J-11, J-27, J-26, J-60, J-39, J-31 and J-9) have a bad performance (both high and low) which indicates to conclude that some distribution network systems have hydraulic problems. Similarly, to velocity, the maximum pressure performance evaluation should be complemented with the minimum and maximum pressure assessment to allow the identification of the hydraulic problems.

According to the results shown in table **4-18,** the nodes **(67.35%)** have a performance of **20m -80 m** pressure, with the exception of junction **J-45 and J - 40** that has low performance associated with minimum pressures and junction(nodes) J-24, J-53, J-14, J-25, J-10, J-27, J-26, J-60, J-39, J-31, J-11, J-43, J-32 and J-9 have a high performance associated with maximum pressure during Peak -1 Day Demand **(8:00 - 9:00).**

In contrary, according to model output result in table **4-19**, some of the junctions **(51%)** have a low performance (minimum /-ve pressure) during Peak-2 Day demand **(15:00-21:00).**

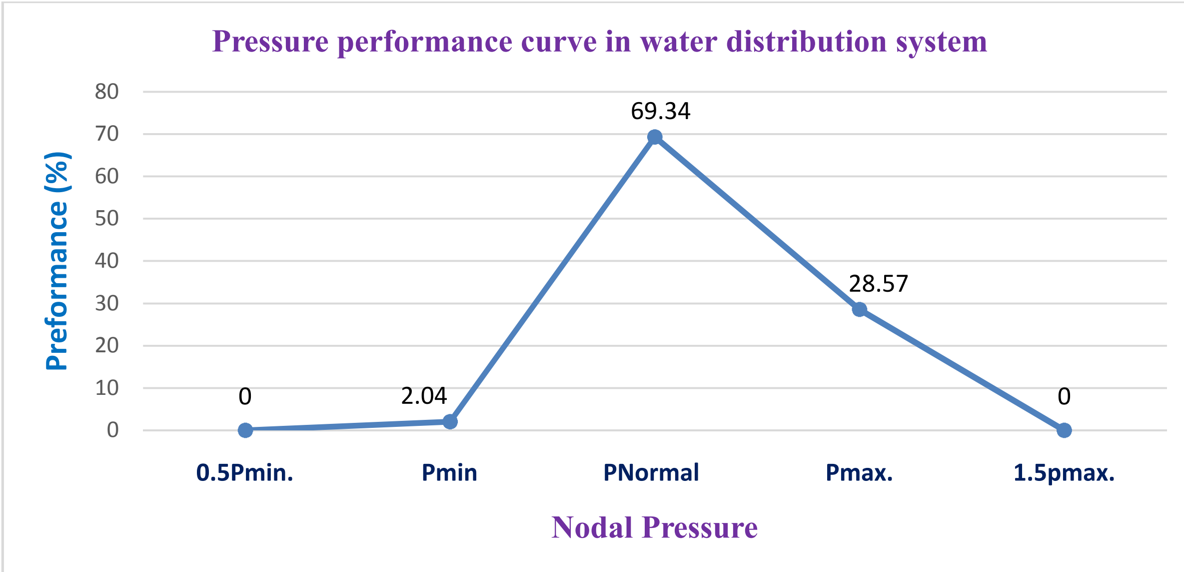

Figure 4-19: Pressure performance Curve in water distribution system during Peak day demand.

4.6.3 Performance Assessment for head loss

The head loss in the distribution system is about **77.1 %** of the Pipe length of the network systems have head losses below **5m/km and about 18.07% of pipes length have 5-10 m/km.** There are also some pipes in the network systems (i.e. **4.82%** of length) with head losses in the highest class **(above 10 m/km)(e.g. pipes 8,16,33,118).**

4.7 Identified Problems and Its improvements

4.7.1 Identified problems

4.7.1.1 Water Demand and Supply

Water supply

Ginchi Town obtains its water from four bore wells located near the town and Warka Gara large gravity spring. The current pumping rates of well range from a low **285.12 m^3/day** to a maximum of 776.2 **m^3/day**. The total capacity of gravity spring and bore hole wells with all pumps operating simultaneously is **2,228.4** m^3/day.

Per capita per person

Based on a service population of **53,600**, the measured per capital demands for 2018 were:

- Current water losses = **41** %.
- Current per capital per person is = **24.53 l/c/d.**

These per capital demands are much lower than the standard demands **(50 -70l/c/d)** set by ministry of water, Irrigation and electricity of Ethiopia. The present domestic per capita consumption in Ginchi town is low showing that the demand is suppressed due to inadequacy of the existing water supply. Therefore, it is clear that the current per capita consumption is far below the recommended rate for towns in the water sector development program, **50 - 70 l/c/d.**

4.7.1.2 Service Reservoir Capacity

Supply and storage must meet current daily demands and future anticipated demands for 20 years in the future. In order to provide for security of supplies above the need for balancing purposes it is recommended that the minimum total reservoir storage capacity be in the range of **30%** to **50%** of the average daily demand **(Harry Hickey, 2008).**

The study area has two **300m3** reservior and one 150 m3 which have a total **750 m3** capacity. To check this capacity is enough or not the total average day demand as per computed above is **9256 m3/day** and the level of existing storage computed as :-

- 0.3 x 9256 = **2,776.8 m3/day.**
- 0.5 x 9256 = **4,628 m3/day**.

Therefore, the storage capacity of existing reservoir has not been on the recommended range and so far the scheme is not safe regarding to storage capacity and it was one of the major problems of the day to day intermittent water supply distribution in the town.

4.7.1.3 Pressure

Pressure in water distribution system has to be maintained optimum as to efficiently make water available to each demand category including at instances of firefighting and as to reduce leakage as well as pipe burst across the system.

AWWA recommends normal static pressure of **400-500** kpa**(40.79 m -50.99 m)** for**:**

- Supplies ordinary uses in building up to 10 stories.
- Supply sprinkler system in buildings up to 5 stories.
- Will provide a useful fire flow without pumper trucks.
- Will provide a relatively large margin of safety to offset sudden high demand or closure of part of the system.

However, there was no defined maximum and minimum pressure ranges set by the town water utility.

According to AWWA design criteria standard for water distribution systems, the proposed water distribution system shall be analyzed for the following three conditions:

1. For the peak hour demand flow analysis, the pressure at each node shall be a minimum of **40 psi (28 m**) and maximum of **120 psi (84 m).**
2. For maximum day demand plus fire flow analysis, fire flow should be selected for the worst–case scenario (typical the hydrant furthest from the connections, at the highest system elevation) and as directed by western`s staff. The pressure at each node shall be a minimum of **20 psi (14 m)** and the maximum velocity in the pipe lines shall be **7.5 ft/sec (2.29 m/s).**
3. For minimum hour demands analysis, the maximum velocity in the pipe lines shall be **5 ft/sec (1.52 m/s)** and the maximum pressure at each node shall be **120 psi (84 m).**

Therefore, literature based recommendation for optimum operating pressure was used to assess system hydraulic performance. With regard to current simulation, result for pressure at peak-1 and peak -2-day demand is summarized in Table below figure below in detail.

Table 4 -30: Distribution of actual Node pressure at **Peak-1 Day Demand** (**8:00 hr**).

Pressure (m)	Number of Nodes	Percentage (%)
>84	14	28
70-80	4	8
60-70	4	8
50-60	8	16
40-50	10	20
30-40	3	6
<28	7	14
Total	**49**	**100%**

As depicted in table above and figure below **14%** of nodes are below the minimum desirable pressures (**28 m)** and **28%** of nodes are above the maximum desirable pressures **(84 m)** during peak hour demand. While **58%** of nodes are in the permissible pressure ranges (minimum **28 m** and maximum **84 m** pressure). Estimating pressure distribution at the peak hour demand is the governing parameters for the purposes of design and improving the existing water distribution network next to minimum consumption hour demand type. Demand is peak especially at morning (peak-1) and early evening (peak-2) for domestic water consumption or residential use. Therefore, from the above table pressure which is less than minimum pressure and greater than the maximum pressure must be improved for the purpose of leakage and bursting rate reduction in the study area.

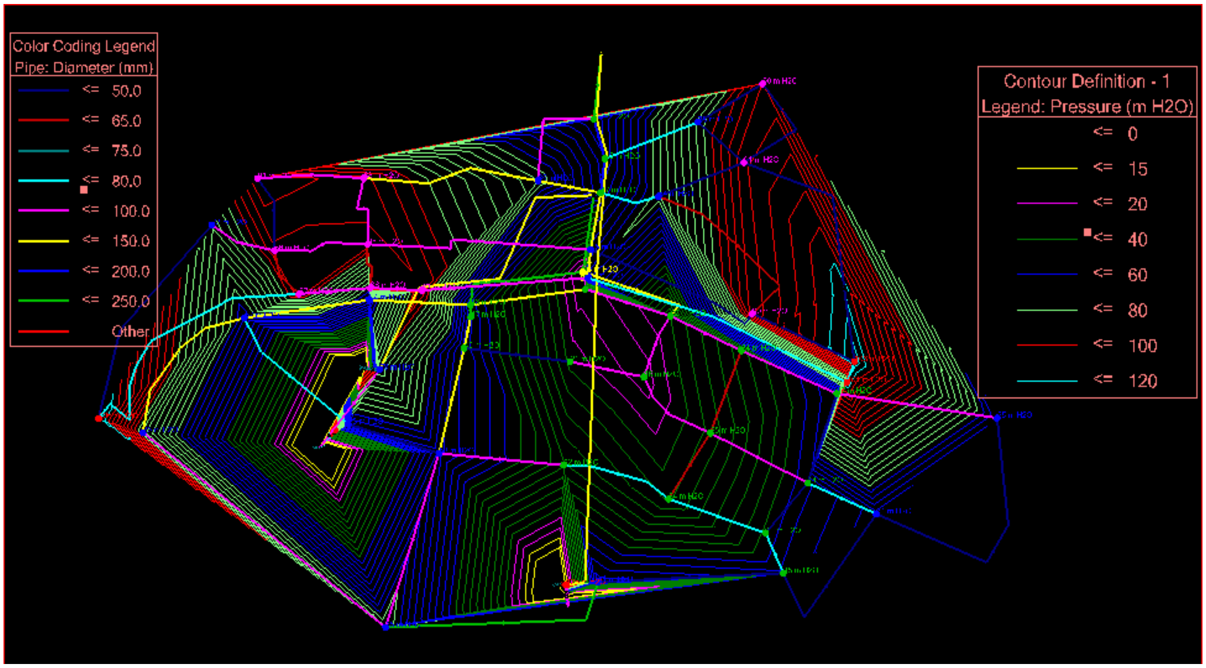

Figure 4-20: shows pressure contour map at peak-1 Day Demand hour.

Table 4-31: Distribution of actual Node pressure at peak-2 Day Demand (15:0021:00).

Pressure (m)	Number of Nodes	Percentage (%)	Remark
>84	1	2.04	Pressure line junction
70 - 80	2	4.08	
60 - 70	2	4.08	Pressure line junction
50 - 60	7	14.28	
40 - 50	6	12.24	
30 - 40	2	4.08	
12 - 30	7	14.28	
< 0	22	44.89	
Total	**49**	**100%**	

During low flow typically at afternoon **(15:00 PM)** up to evening **(21:00 PM)** the distribution system of case study is marked by very low pressure. As shown in Table above **44.89%** of nodes are liable to extremely very low pressure **(-ve pressure).** Therefore, from the above table results pressure which is less than minimum pressure must be improved for the purpose of to demand and water quality problems in the study area.

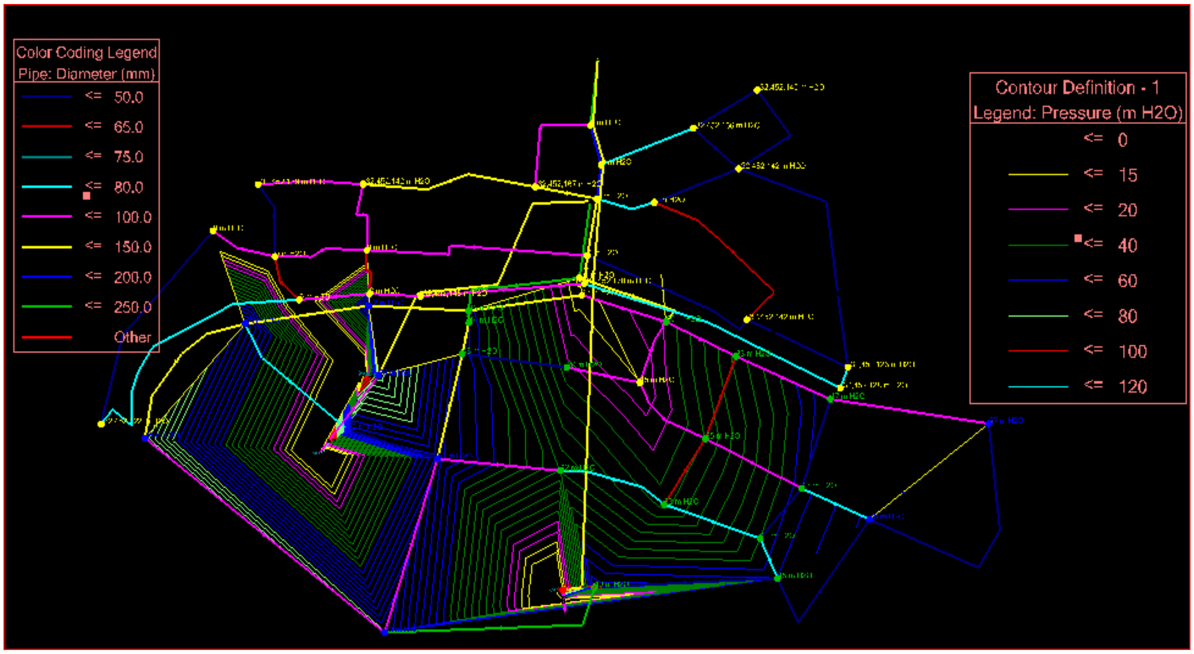

Figure 4-21: Shows pressure contour map at peak-2 Day Demand hour (18:00 PM)

As shown in the above figure all nodes are susceptible to lower pressure (negative pressure) due to shortage of supply. There are some reasons that are why the negative pressure is occurred in the water supply distribution system, but, in this case of study the negative pressure was the result of demands that are greater than the design demand: negative Q demand and insufficient tank capacity in the distribution system.

Table 4-32: Distribution of actual Node pressure during Minimum Day Demand (1:00)

Pressure (m)	Number of Nodes	Percentage (%)
>84	15	30.6
70-80	4	8.2
60-70	6	12.2
50-60	10	20.4
40-50	7	14.3
29-40	3	6.1
<28	4	8.2
Total	**49**	**100 %**

As shown in Table the above and figure below **30.6%** of nodes are liable to extremely high pressure. This figure is relatively high. Also minimum pressure **(8.2%)** is observed during low consumption period. Only **61.2%** of nodes are getting optimum pressure at low consumption hour.

Node pressure at minimum consumption hour, is very important rather than others two peak, because, leakage and bursting of pipes are deter rioted during this low consumption hour. Due to the above reasons the existing water supply distribution systems are designed by taking the base parameters. For this study, improving the existing water supply system has been done at both peak and minimum consumption hour level, because minimum and maximum pressures are found in both cases.

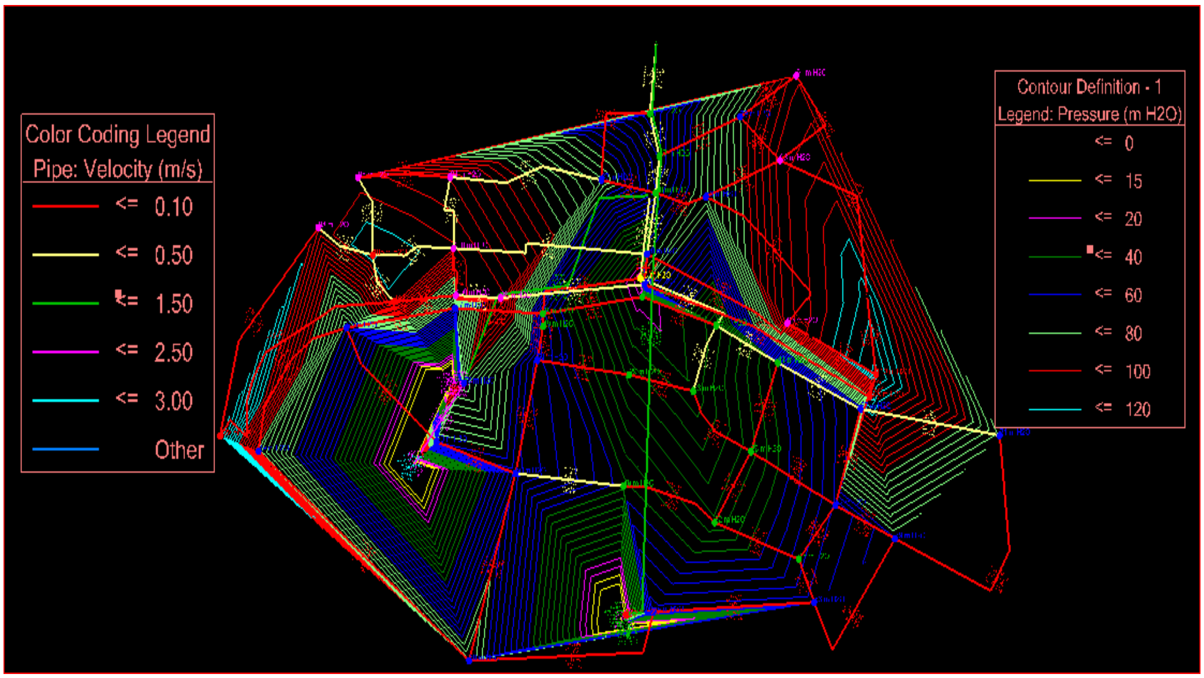

Figure 4-22: Shows pressure contour map at minimum day demand hour.

4.7.1.4 Velocity

Velocity of water flow in the pipe is also one of the important parameter in hydraulic performance modeling evaluation in water supply distribution and transmission line. Velocity in water distribution system is also varying with demand pattern changes. At the peak hour demand the values are different as compare to the minimum consumption hour. The water supply distribution system network velocity during peak hour-1 demand is summarized in the table below.

Table 4-33: Distribution of actual Pipe Velocity at Peak Day-1 Demand.

Velocity (m/s)	Number of Pipes	Percentage (%)	Remark
≥ 2.5	2	2.4	
1.5 - 2.5	7	8.4	
0.5 - 1.5	29	34.9	
0.1- 0.5	36	43.4	
<0.1	9	14.81	
Total	**83**	**100%**	

The model shows that most of the system peak velocities are less than **2.8m/s** during the peak day. Table above shows the peak velocities through the system during peak Day-1 Demands. The evaluation criteria states that maximum velocities should not exceed **3m/s**. The few locations where peak velocities exceed **2.5m/s** are near the wells (supply sources). The existing system piping evaluation includes an analysis of velocity and head loss under peak demand conditions. The findings of this piping evaluation are described below.

Figure 4-23: Velocity distribution in the pipe during peak day hour.

As shown on the above figure comparing the minimum velocities to the standard water design manual, it was discovered that about **53%** pipes in the distribution system below the standard requirement **(0.6 m/s)** during peak hour**.**

4.7.1.5 Relationship between pressure and leakage Rate

Table 4–34: Due to Excess Pressure the amount of water lost by leakage

Label	Actual Pressure (m)	Excess Pressure	n	k(l/s/m$^{1/2}$)	Actual Leakage Rate (L=KP^n(l/s))
J-24	100	20	0.5	0.074	0.33
J-53	94	14	0.5	0.074	0.28
J-25	100	20	0.5	0.074	0.33
J-43	92	12	0.5	0.074	0.26
J-35	98	18	0.5	0.074	0.31
J-32	94	14	0.5	0.074	0.28
J-10	108	28	0.5	0.074	0.39
J-27	101	21	0.5	0.074	0.34
J-26	105	25	0.5	0.074	0.37
J-60	122	42	0.5	0.074	0.48
J-39	98	18	0.5	0.074	0.31
J-31	94	14	0.5	0.074	0.28
J-9	110	30	0.5	0.074	0.41
J-10	98	18	0.5	0.074	0.31
				(l/s)	**4.68**
		Total		**m3/day**	**404.03**

High pressure during low demand conditions can cause pipe bursting, leakage and large amount of water losses through the distribution networks. An increase in operating pressure will result in an increase in system leakage, which follows a linear relationship, i.e., a **1%** increase in system pressure results in a **1%** increase in system leakage. It can be expected that a pressure increase would also result in an increase in the number of main breaks.

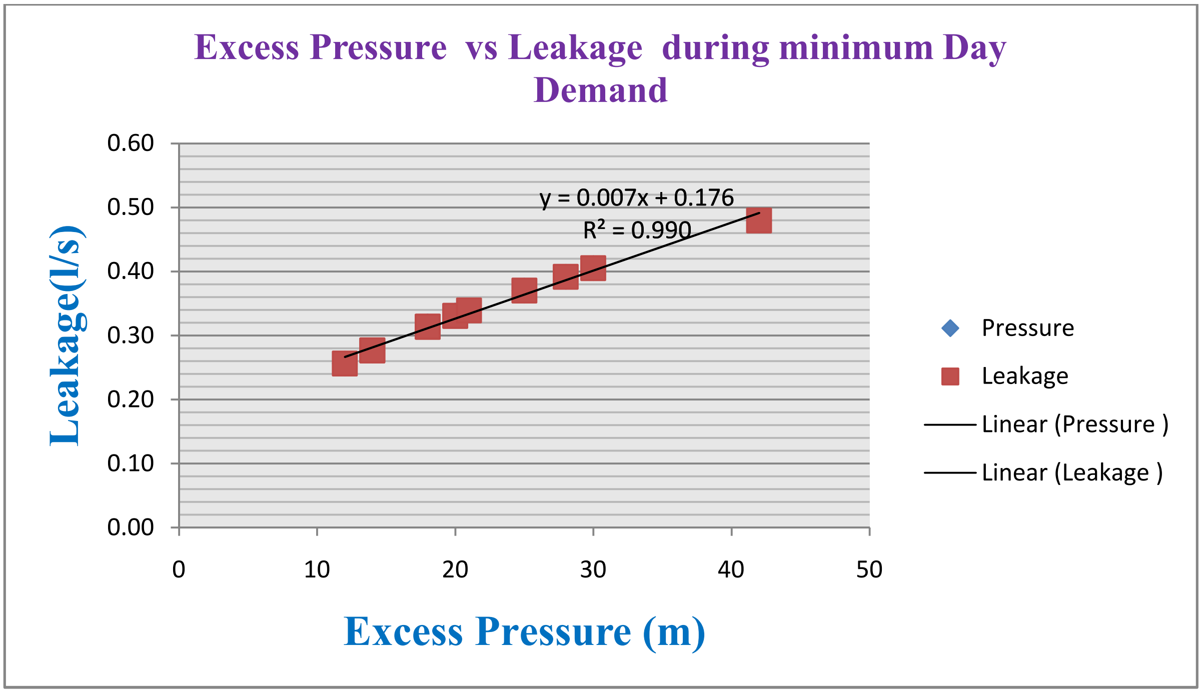

Figure 4-24: Relationship between Pressure and Leakage rate during minimum day demand.

Therefore, when dealing with high pressures, P R Vs should be used to reduce and regulate pressure in the system.

4.7.2 Existing System Design Improvement and Proposed Design Improvement System

4.7.2.1 Service Storage Capacity

The existing storage tank capacity currently serves the town are not adequate. For these reason, the actual storage capacity required for the coming 20 years have been computed and constructed in the town should be carefully analyzed and recommended.

Therefore, the storage volume required for Ginchi Town water supply distribution system were calculated using the following, generally accepted, formula from the **(MMCDA, 2014).**

Storage Volume = A+ B+ C

Where, A = Fire storage

B= Equalization (peaking) Storage (25% of average day demand)

C= Emergency storage (25% of (A+B)) for water main breaks.

Storage requirements for existing conditions (2018):

A. Fire storage: a fire flow of **64**m3/day for 2 hours was used based on single residential land use. This requires a storage volume of **10.7m^3**.

B. Peaking Storage: Calculated at **25%** of average day demand. Based on the 2018 average day demand of **9256m3/day** for the entire water system, the peaking storage volume requirement is **2,314m^3**.

C. Emergency Storage: Calculated at 25% (fire storage + peaking Storage). This requires a volume of **581.2 m^3.**

Therefore, Totally A+ B +C = **2905 m^3 storage** volume is required for Ginchi Town water supply distribution. The existing **750 m^3** reservoirs can be incorporated to the system and an additional **2,155** m^3 new reservoirs should be constructed to deliver adequate water in the distribution networks.

4.7.2.2 Pressure Zones

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. A differential of less than **20 m** not require pressure zone. But, more than **25 m** differential generally will require a pressure zone **(Jeffrey and Gilbert, 2012).** In areas of even large grade differentials, such as hill or mountain communities. Several consecutive pressure zones may be needed. Therefore, Ginchi town water supplies distribution systems whether requires or not the following equations can assist in determining the HG Ls for the pressure zones:

- ☞ HGL min= Highest Elevation +(2.31*minimum working pressure) =2347+2.31*15 m = **2381.65**
- ☞ H G L max = Lowest Elevation + (2.31*maximum working pressure) =2230 + 2.31*84 m = **2424.04**

Since, the difference of H G L in the distribution networks system is **42.39** m. Therefore, more pressure zones may be required.

4.7.2.2.1 Improved Pressure Zones Distribution System

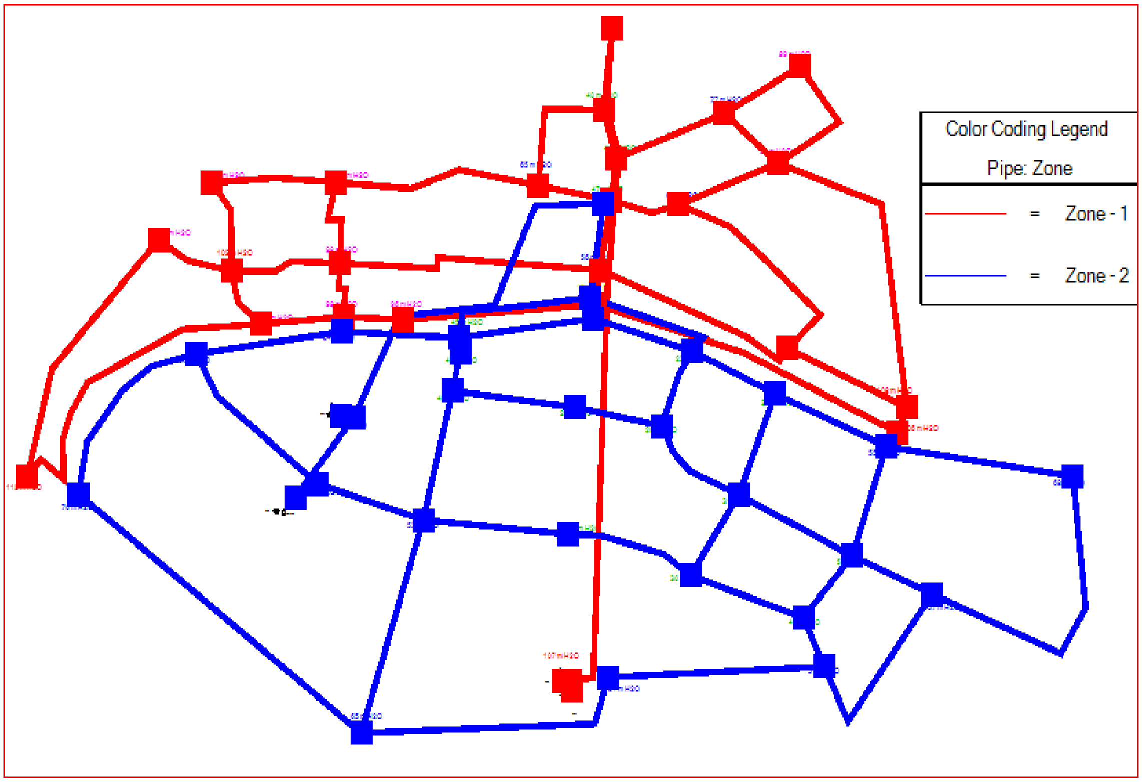

Figure 4-25: Shows all Pressure zone boundaries

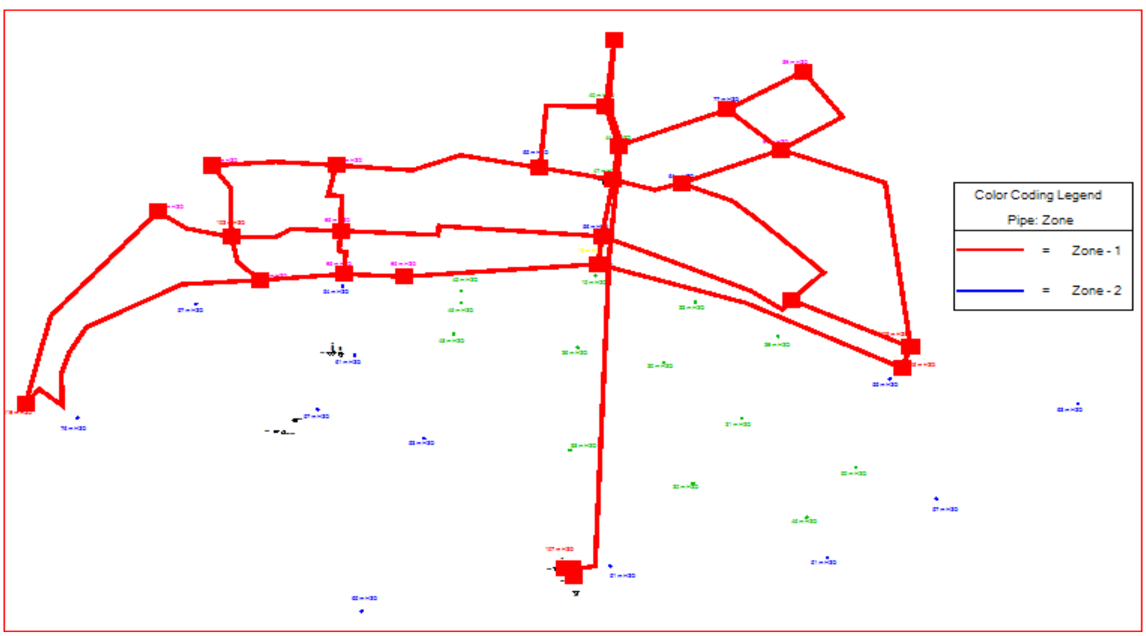

Figure 4-26: pressure zone -1 boundary

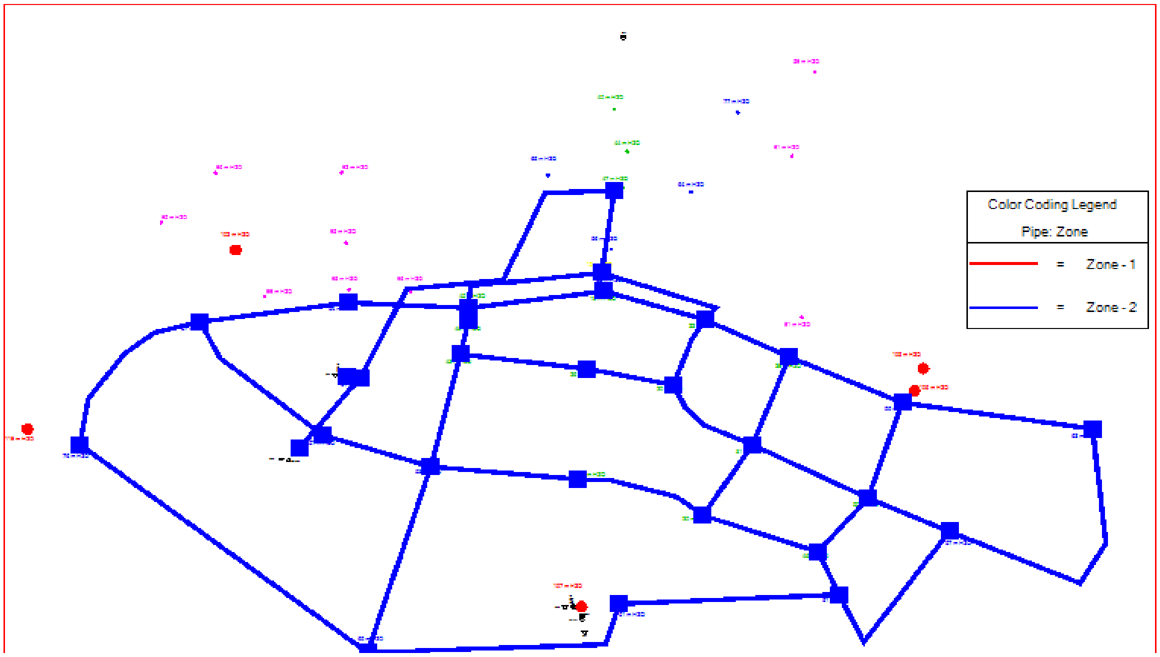

Figure 4-27 : Pressure zone – 2 boundary

4.7.2.3 Excess pressure and Its Improvements

Excess pressure has a great effect on leakage and bursting, reduction of excessive pressure to the desired values and feasible suggestions has been made by installing pressure reducing valves (PRVs) at links which has maximum pressure during minimum consumption hour. There is excess pressure which is greater than the permissible pressure **(84m),**see table 4-35.

Table 4 -35: Excess pressure locations in the distribution system at minimum consumption hour.

S/No.	Label	x(m)	y(m)	Elevation (m)	H GL(m)	Max. Pressure (m)
1	J-24	1,357,560.64	805,496.63	2,250.00	2,347.69	100
2	J-53	1,359,662.27	805,132.43	2,257.00	2,348.08	94
3	J-14	1,358,645.07	803,713.47	2,252.00	2,360.37	142
4	J-25	1,357,574.93	805,268.03	2,250.00	2,347.69	100
5	J-43	1,359,719.42	806,337.34	2,259.00	2,348.13	92
6	J-35	1,356,708.15	805,597.44	2,252.00	2,347.26	98
7	J-32	1,357,539.21	805,842.70	2,256.00	2,347.97	94
8	J-10	1,360,179.79	804,772.59	2,242.00	2,348.05	108
9	J-27	1,357,185.99	805,236.28	2,249.00	2,347.66	101
10	J-26	1,357,051.06	805,463.29	2,245.00	2,347.51	105
11	J-60	1,356,089.82	804,584.51	2,228.00	2,347.61	122
12	J-39	1,356,958.98	805,841.12	2,252.00	2,347.94	98
13	J-31	1,359,616.23	805,924.06	2,257.00	2,348.13	94
14	J-9	1,360,221.07	804,881.60	2,240.00	2,348.05	110
15	J-11	1,357,856.12	805,255.10	2,252.00	2,347.77	98

Due to the above excess pressure, the distribution systems are modified and pressure reducing valves are added to the systems. P RVs are instruments that are installed at strategic points in the network to reduce or maintain network pressure at set level. 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 as can be seen in figure 4-26. 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:

- It is relatively simple to install as it requires only a P R V.
- Cost is relatively low as it involves no electric equipment.
- Maintenance and operation is relatively simple.

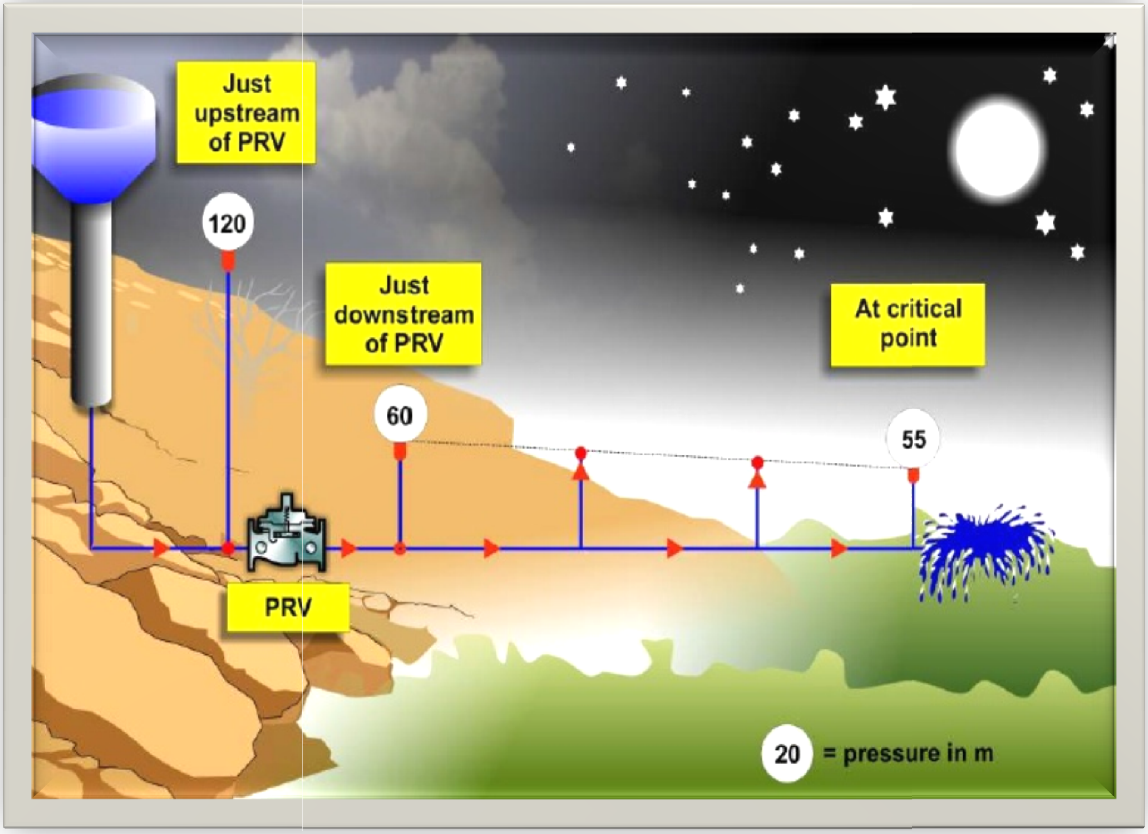

Figure 4-28: Typical pressure reducing valve installation and pressure management adopted from IWA-Press-Paper-0309.

Table 4-36: Pressure reducer valves (P RVs) locations in the improved distribution system

Label	Y (m)	X (m)	Elevation (m)	Diameter (Valve) (mm)	Hydraulic Grade Setting (Initial) (m)	Pressure Setting (Initial) (m)	Hydraulic Grade (From) (m)	Hydraulic Grade (To) (m)	Pressure (From) (m)	Pressure (To) (m)
PRV-1	805,270.50	1,360,164.38	2,244.80	50	2,324.93	80	2,347.19	2,346.42	102	101
PRV-2	805,068.98	1,359,807.59	2,252.60	50	2,312.70	60	2,346.47	2,346.42	94	94
PRV-3	806,061.03	1,359,170.41	2,283.68	80	2,363.81	80	2,347.44	2,347.44	64	64
PRV-4	805,821.70	1,357,783.29	2,262.80	150	2,342.93	80	2,338.00	2,338.00	75	75
PRV-5	805,489.57	1,357,705.14	2,254.80	100	2,314.90	60	2,344.87	2,314.95	90	60
PRV-6	805,264.90	1,357,670.95	2,250.68	100	2,310.78	60	2,345.36	2,310.83	94	60
PRV-7	805,157.44	1,357,226.49	2,245.49	150	2,305.59	60	2,299.87	2,299.87	54	54

Table 4 -37: Improved pressure in the distribution system at minimum consumption hour.

Label	X (m)	Y (m)	Elevation (m)	Hydraulic Grade (m)	Pressure (m)
J-24	1,357,560.64	805,496.63	2,250.00	2,311.69	62
J-25	1,357,574.93	805,268.03	2,250.00	2,311.46	61
J-35	1,356,708.15	805,597.44	2,252.00	2,311.19	59
J-32	1,357,539.21	805,842.70	2,256.00	2,312.80	57
J-10	1,360,179.79	804,772.59	2,242.00	2,302.72	61
J-27	1,357,185.99	805,236.28	2,249.00	2,311.46	62
J-26	1,357,051.06	805,463.29	2,245.00	2,311.45	66
J-60	1,356,089.82	804,584.51	2,228.00	2,311.43	83
J-39	1,356,958.98	805,841.12	2,252.00	2,312.74	61
J-9	1,360,221.07	804,881.60	2,240.00	2,302.72	63
J-11	1,357,856.12	805,255.10	2,252.00	2,331.76	80

As computed in table **(4-35)** all junctions except **J-53**, **J-14**, **J-43** and **J-31** are found under desirable allowable maximum pressure **(84 m)** in the system during minimum consumption hour demand.

Table 4-38: Evaluation of pressure effects on leakage in the study area

Label	Actual Pressure (m)	Excess pressure before	Improved pressure (m)	Excess pressure after PRV Installation	n	$k(l/s/m^{1/2})$	Leakage amount before ($L=KP^n$(l/s))	Leakage amount after ($L=KP^n$(l/s))	Save Leakage amount (l/s)
J-24	97	17	62	0	0.5	0.074	0.305	0.00	0.305
J-53	91	11	91	11	0.5	0.074	0.245	0.25	0.000
J-14	108	28	100	20	0.5	0.074	0.392	0.33	0.061
J-25	97	17	61	0	0.5	0.074	0.305	0.00	0.305
J-43	89	9	89	9	0.5	0.074	0.222	0.22	0.000
J-35	95	15	59	0	0.5	0.074	0.287	0.00	0.287
J-32	92	12	57	0	0.5	0.074	0.256	0.00	0.256
J-10	106	26	61	0	0.5	0.074	0.377	0.00	0.377
J-27	98	18	62	0	0.5	0.074	0.314	0.00	0.314
J-26	102	22	66	0	0.5	0.074	0.347	0.00	0.347
J-60	119	39	83	3	0.5	0.074	0.462	0.13	0.334
J-39	96	16	61	0	0.5	0.074	0.296	0.00	0.296
J-31	91	11	91	11	0.5	0.074	0.245	0.25	0.000
J-9	108	28	63	0	0.5	0.074	0.392	0.00	0.392
J-11	96	16	80	0	0.5	0.074	0.296	0.00	0.296
						Total(l/s)	**4.74**	**1.17**	**3.57**

4.7.2.4 Water Supply Source and Distribution System Improvements

To overcome the town water supply shortage in the future substantially it is important to investigate sustainable potential source of potable water. Therefore, the potential source of Warka Gara Large Gravity Spring is now more than **33.3 l/s** and only **16.2 l/s** of this has been diverted for communities living in **6** rural kebeles and for **2** kebeles of Ginchi Town**.** Due to this reason upgrading the discharge capacity of the spring by using effective and efficient operational management is the simplest way rather than searching additional Ground water potential to supply sustainable water supply distribution to the town. Therefore, by increasing the capacity of the springs from **2.6 to 10 l/s** and also changing direct supply system in to service reservoir, can satisfy the existing demand of Ginchi Town Water Supply throughout **24 hr.** with optimum pressure.

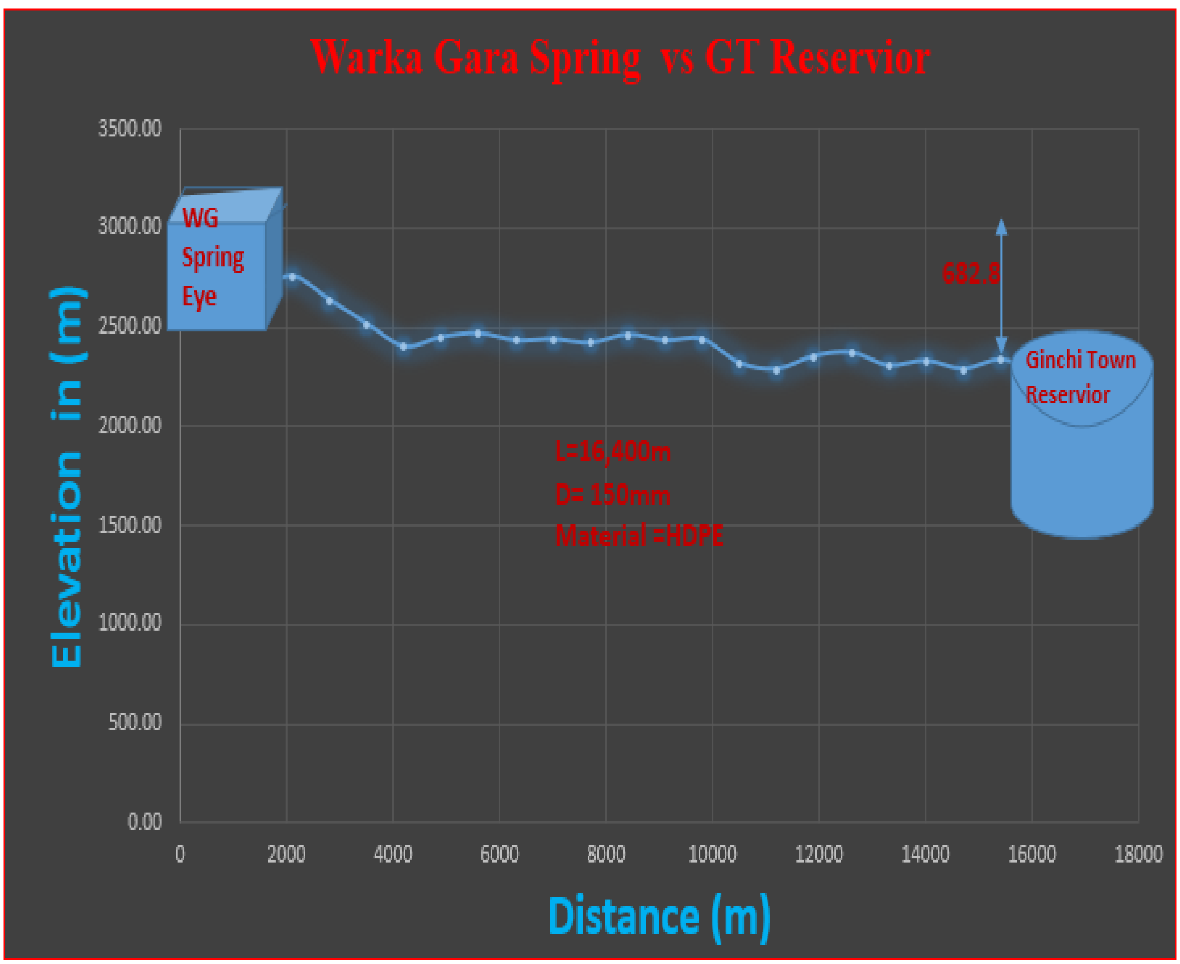

Figure 4-29 : Profile of the new proposed gravity spring pipe line

The results of the improved systems are illustrated in table belows :

Table: 4-39 Improved System Pressure at Minimum Consumption Hour.

Label	X (m)	Y (m)	Elevation (m)	Hydraulic Grade (m)	Demand (L/s)	Pressure (m)	Remark
J-24	1,357,560.64	805,496.63	2,250.00	2,346.41	0.339	96	
J-48	1,357,950.38	804,400.46	2,248.00	2,345.55	0.873	97	
J-23	1,359,832.53	803,773.79	2,250.00	2,345.16	0.198	95	
J-30	1,359,365.40	806,136.26	2,271.00	2,347.18	0.162	76	
J-45	1,358,736.99	805,350.96	2,289.00	2,345.84	0.417	57	
J-56	1,356,331.92	804,510.00	2,225.00	2,345.51	0.612	120	
J-54	1,358,818.87	803,727.42	2,250.00	2,345.29	0.645	95	
J-29	1,359,965.88	804,246.07	2,251.00	2,345.23	0.381	94	
J-18	1,358,087.70	804,952.12	2,253.00	2,345.73	0.417	93	
J-42	1,359,602.34	804,939.02	2,262.00	2,345.41	0.228	83	
J-53	1,359,662.27	805,132.43	2,257.00	2,345.92	0.894	89	
J-51	1,358,122.62	805,178.33	2,256.00	2,345.81	0.051	90	
J-16	1,358,762.65	805,320.42	2,286.00	2,345.94	0.252	60	
J-17	1,358,124.21	805,115.63	2,255.00	2,345.81	0.102	91	
J-33	1,358,490.92	805,829.21	2,283.00	2,347.25	0.306	64	
J-14	1,358,645.07	803,713.47	2,252.00	2,393.51	0	141	Pumped Line
J-12	1,357,625.63	804,838.94	2,247.00	2,497.00	0	249	Pumped Line
J-25	1,357,574.93	805,268.03	2,250.00	2,346.01	0.09	96	
J-43	1,359,719.42	806,337.34	2,259.00	2,346.83	0.243	88	
J-28	1,359,150.03	805,747.32	2,284.00	2,346.96	0.27	63	
J-35	1,356,708.15	805,597.44	2,252.00	2,346.27	0.27	94	
J-32	1,357,539.21	805,842.70	2,256.00	2,346.95	0.225	91	
J-41	1,358,666.51	804,881.07	2,275.00	2,345.47	0.441	70	
J-10	1,360,179.79	804,772.59	2,242.00	2,345.92	0.666	104	
J-58	1,361,001.06	804,586.33	2,232.00	2,345.20	0.666	113	
J-22	1,359,735.69	803,983.34	2,255.00	2,345.16	0.321	90	
J-27	1,357,185.99	805,236.28	2,249.00	2,346.02	0.102	97	
J-26	1,357,051.06	805,463.29	2,245.00	2,346.29	0.264	101	
J-60	1,356,089.82	804,584.51	2,228.00	2,346.02	0.168	118	
J-37	1,359,433.27	804,507.22	2,270.00	2,345.31	0.408	75	
J-52	1,357,570.17	805,204.53	2,247.00	2,345.68	0.312	98	
J-21	1,358,797.08	806,152.14	2,308.00	2,347.75	0.042	40	
J-34	1,359,069.73	804,801.70	2,281.00	2,345.48	0.336	64	

J-39	1,356,958.98	805,841.12	2,252.00	2,346.91	0.081	95	
J-19	1,358,856.34	805,944.87	2,304.00	2,347.48	0.135	43	
J-31	1,359,616.23	805,924.06	2,257.00	2,346.80	0.339	90	
J-47	1,360,123.83	804,714.39	2,246.00	2,345.32	0.396	99	
J-40	1,358,749.85	805,260.49	2,286.00	2,345.92	0.15	60	
J-46	1,360,340.53	804,083.35	2,244.00	2,345.20	0.243	101	
J-8	1,359,214.99	805,120.79	2,278.00	2,345.73	0.147	68	
J-9	1,360,221.07	804,881.60	2,240.00	2,345.93	0.141	106	
J-15	1,358,779.61	805,467.92	2,292.00	2,346.38	0.138	54	
J-49	1,357,453.49	804,554.71	2,244.00	2,345.30	0.828	101	
J-59	1,357,658.28	803,489.23	2,236.00	2,345.30	0.894	109	
J-55	1,356,885.96	805,110.07	2,244.00	2,345.56	0.483	101	
J-50	1,358,630.79	804,338.94	2,263.00	2,345.30	0.723	82	
J-38	1,359,204.67	804,161.94	2,271.00	2,345.18	0.579	74	
J-20	1,358,833.59	805,766.37	2,301.00	2,347.27	0.15	46	
J-11	1,357,856.12	805,255.10	2,252.00	2,345.89	0.132	94	
J-61	1,358,676.63	805,344.08	2,286.20	2,345.84	0	60	
J-62	1,358,671.81	805,312.77	2,282.59	2,345.85	0	63	

Table: 4-40 Improved System Pressure at Peak Day Demand hour.

Label	X (m)	Y (m)	Elevation (m)	Hydraulic Grade (m)	Demand (L/s)	Pressure (m)	Remark
J-24	1,357,560.64	805,496.63	2,250.00	2,315.11	1.808	65	
J-48	1,357,950.38	804,400.46	2,248.00	2,296.25	4.656	48	
J-23	1,359,832.53	803,773.79	2,250.00	2,287.63	1.056	38	
J-30	1,359,365.40	806,136.26	2,271.00	2,331.72	0.864	61	
J-45	1,358,736.99	805,350.96	2,289.00	2,302.77	2.224	14	
J-56	1,356,331.92	804,510.00	2,225.00	2,295.36	3.264	70	
J-54	1,358,818.87	803,727.42	2,250.00	2,290.65	3.44	41	
J-29	1,359,965.88	804,246.07	2,251.00	2,289.22	2.032	38	
J-18	1,358,087.70	804,952.12	2,253.00	2,300.41	2.224	47	
J-42	1,359,602.34	804,939.02	2,262.00	2,293.28	1.216	31	
J-53	1,359,662.27	805,132.43	2,257.00	2,304.43	4.768	47	
J-51	1,358,122.62	805,178.33	2,256.00	2,302.16	0.272	46	
J-16	1,358,762.65	805,320.42	2,286.00	2,305.05	1.344	19	
J-17	1,358,124.21	805,115.63	2,255.00	2,302.11	0.544	47	
J-33	1,358,490.92	805,829.21	2,283.00	2,333.28	1.632	50	
J-14	1,358,645.07	803,713.47	2,252.00	2,393.95	0	142	Pumped Line
J-12	1,357,625.63	804,838.94	2,247.00	2,325.23	0	78	Pumped Line
J-25	1,357,574.93	805,268.03	2,250.00	2,306.36	0.48	56	
J-43	1,359,719.42	806,337.34	2,259.00	2,324.01	1.296	65	
J-28	1,359,150.03	805,747.32	2,284.00	2,327.01	1.44	43	
J-35	1,356,708.15	805,597.44	2,252.00	2,312.13	1.44	60	
J-32	1,357,539.21	805,842.70	2,256.00	2,326.95	1.2	71	
J-41	1,358,666.51	804,881.07	2,275.00	2,294.51	2.352	19	
J-10	1,360,179.79	804,772.59	2,242.00	2,304.58	3.552	62	
J-58	1,361,001.06	804,586.33	2,232.00	2,288.58	3.552	56	
J-22	1,359,735.69	803,983.34	2,255.00	2,287.64	1.712	33	
J-27	1,357,185.99	805,236.28	2,249.00	2,306.71	0.544	58	
J-26	1,357,051.06	805,463.29	2,245.00	2,312.55	1.408	67	
J-60	1,356,089.82	804,584.51	2,228.00	2,306.67	0.896	79	

J-37	1,359,433.27	804,507.22	2,270.00	2,290.96	2.176	21	
J-52	1,357,570.17	805,204.53	2,247.00	2,299.33	1.664	52	
J-21	1,358,797.08	806,152.14	2,308.00	2,344.29	0.224	36	
J-34	1,359,069.73	804,801.70	2,281.00	2,294.66	1.792	14	
J-39	1,356,958.98	805,841.12	2,252.00	2,326.03	0.432	74	
J-19	1,358,856.34	805,944.87	2,304.00	2,338.31	0.72	34	
J-31	1,359,616.23	805,924.06	2,257.00	2,323.42	1.808	66	
J-47	1,360,123.83	804,714.39	2,246.00	2,291.15	2.112	45	
J-40	1,358,749.85	805,260.49	2,286.00	2,304.61	0.8	19	
J-46	1,360,340.53	804,083.35	2,244.00	2,288.50	1.296	44	
J-8	1,359,214.99	805,120.79	2,278.00	2,300.27	0.784	22	
J-9	1,360,221.07	804,881.60	2,240.00	2,304.60	0.752	64	
J-15	1,358,779.61	805,467.92	2,292.00	2,314.44	0.736	22	
J-49	1,357,453.49	804,554.71	2,244.00	2,290.83	4.416	47	
J-59	1,357,658.28	803,489.23	2,236.00	2,290.70	4.768	55	
J-55	1,356,885.96	805,110.07	2,244.00	2,296.52	2.576	52	
J-50	1,358,630.79	804,338.94	2,263.00	2,290.84	3.856	28	
J-38	1,359,204.67	804,161.94	2,271.00	2,288.18	3.088	17	
J-20	1,358,833.59	805,766.37	2,301.00	2,333.91	0.8	33	
J-11	1,357,856.12	805,255.10	2,252.00	2,303.86	0.704	52	
J-61	1,358,676.63	805,344.08	2,286.20	2,302.86	0	17	
J-62	1,358,671.81	805,312.77	2,282.59	2,303.04	0	20	

As shown in **Table 4-39** and **Table 4-40** about **62.7%** of nodes subjected to maximum pressure during minimum consumption hour demand. But during peak day demand all the nodes are found optimum working pressure.

Table: 4-41 Improved System Velocity at Minimum Consumption hour.

Velocity (m/s)	Number of pipe	Percentage (%)
≥ 3	1	1.12
3.0-2.0	11	12.36
2.0-1.0	12	13.48
1-0.5	12	13.48
0.5-0.1	32	35.95
≤ 0.1	20	22.47
Total	**89**	**100**

Table: 4-42 Improved System Velocity at Peak hour.

Velocity (m/s)	Number of pipe	Percentage (%)
≥ 3	1	1.12
3-2	11	12.36

2-1	18	20.22
1-0.5	23	25.84
0.5-0.1	30	33.7
≤0.1	6	6.74
Total	**89**	**100**

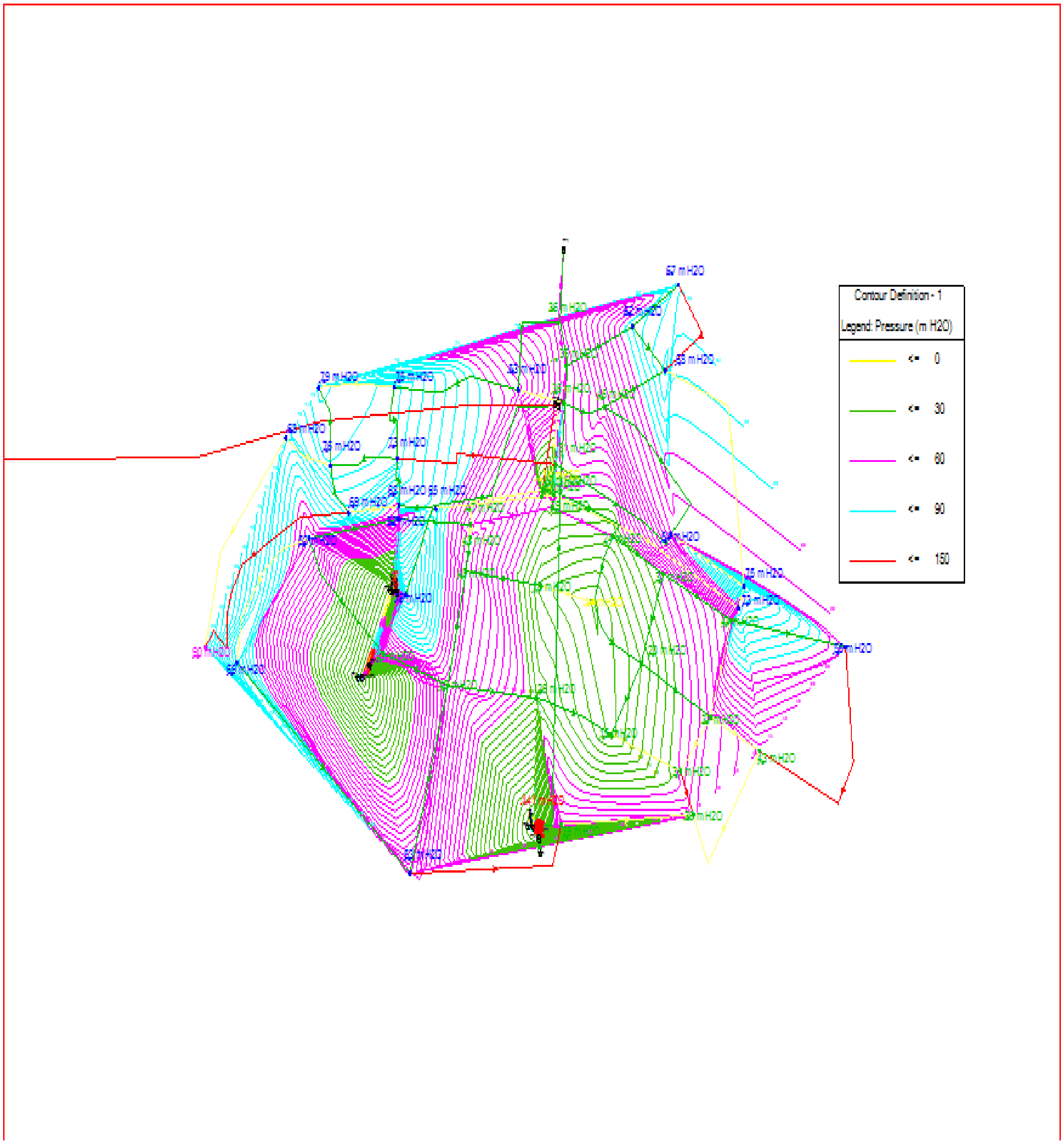

Figure 4-30: Shows improved system pressure contour map at Peak hour consumption

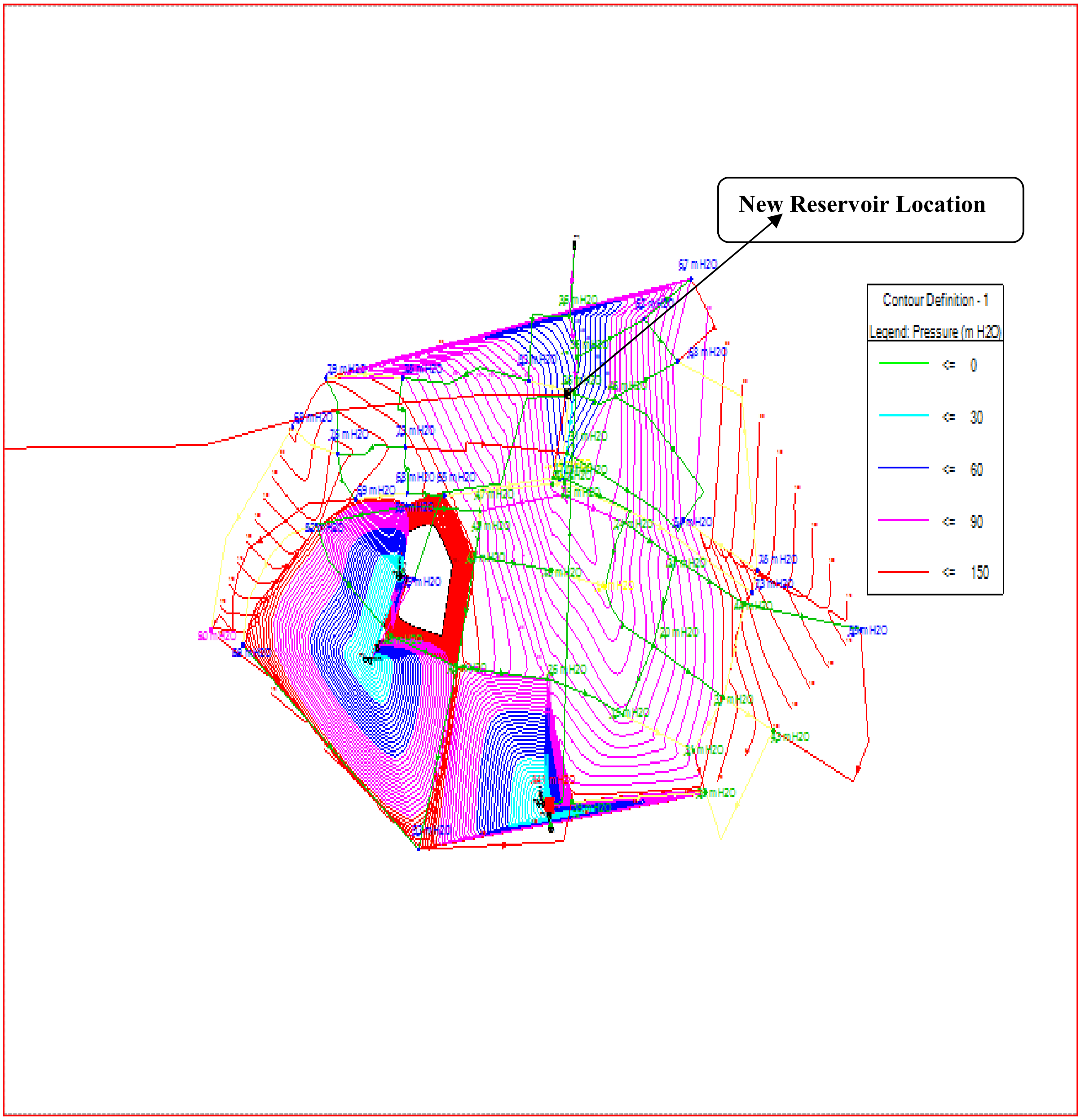

Figure 4-31: Shows Improved System pressure contour map at minimum hour consumption

Table 4-43: Shows Excess Pressure After Improved System During Minimum Day Consumption.

Label	X (m)	Y (m)	Elevation (m)	Hydraulic Grade (m)	Pressure (m)
J-24	1,357,560.64	805,496.63	2,250.00	2,346.41	96
J-48	1,357,950.38	804,400.46	2,248.00	2,345.55	97
J-23	1,359,832.53	803,773.79	2,250.00	2,345.16	95
J-56	1,356,331.92	804,510.00	2,225.00	2,345.51	120
J-54	1,358,818.87	803,727.42	2,250.00	2,345.29	95
J-29	1,359,965.88	804,246.07	2,251.00	2,345.23	94
J-18	1,358,087.70	804,952.12	2,253.00	2,345.73	93
J-42	1,359,602.34	804,939.02	2,262.00	2,345.41	83
J-53	1,359,662.27	805,132.43	2,257.00	2,345.92	89
J-51	1,358,122.62	805,178.33	2,256.00	2,345.81	90
J-17	1,358,124.21	805,115.63	2,255.00	2,345.81	91
J-25	1,357,574.93	805,268.03	2,250.00	2,346.01	96
J-43	1,359,719.42	806,337.34	2,259.00	2,346.83	88
J-35	1,356,708.15	805,597.44	2,252.00	2,346.27	94
J-32	1,357,539.21	805,842.70	2,256.00	2,346.95	91
J-10	1,360,179.79	804,772.59	2,242.00	2,345.92	104
J-58	1,361,001.06	804,586.33	2,232.00	2,345.20	113
J-22	1,359,735.69	803,983.34	2,255.00	2,345.16	90
J-27	1,357,185.99	805,236.28	2,249.00	2,346.02	97
J-26	1,357,051.06	805,463.29	2,245.00	2,346.29	101
J-60	1,356,089.82	804,584.51	2,228.00	2,346.02	118
J-52	1,357,570.17	805,204.53	2,247.00	2,345.68	98
J-39	1,356,958.98	805,841.12	2,252.00	2,346.91	95
J-31	1,359,616.23	805,924.06	2,257.00	2,346.80	90
J-47	1,360,123.83	804,714.39	2,246.00	2,345.32	99
J-46	1,360,340.53	804,083.35	2,244.00	2,345.20	101
J-9	1,360,221.07	804,881.60	2,240.00	2,345.93	106
J-49	1,357,453.49	804,554.71	2,244.00	2,345.30	101
J-59	1,357,658.28	803,489.23	2,236.00	2,345.30	109
J-55	1,356,885.96	805,110.07	2,244.00	2,345.56	101
J-50	1,358,630.79	804,338.94	2,263.00	2,345.30	82
J-11	1,357,856.12	805,255.10	2,252.00	2,345.89	94

Maximum pressure can affect systems losses in a number of ways**.** This type of pressure has a great effect on leakage; reduction of excessive pressure to the desired allowable value, feasible suggestions has been made by installing pressure reducer valves **(PRVs)** at links which has maximum pressure**, see Table 4-40**. During this minimum consumption hour there is excess pressure which is greater than the permissible pressure (**80m).**

Table 4-44: Shows Pressure Reducer Valves Location in the improved system

Label	Y (m)	X (m)	Elevation (m)	Diameter (Valve) (mm)	Pressure Setting (Initial) (m)	Hydraulic Grade (From) (m)	Hydraulic Grade (To) (m)	Pressure (From) (m)	Pressure (To) (m)
PRV-1	805,614.63	1,357,563.92	2,251.71	100	80	2,327.42	2,318.48	76	67
PRV-2	805,028.22	1,359,635.55	2,259.36	150	30	2,317.74	2,283.42	58	24
PRV-3	805,196.06	1,359,213.61	2,280.29	150	80	2,302.67	2,302.67	22	22
PRV-4	805,178.91	1,359,021.34	2,281.33	100	80	2,308.33	2,308.33	27	27
PRV-5	805,233.47	1,358,546.40	2,276.27	150	80	2,312.93	2,312.93	37	37
PRV-6	805,323.89	1,358,495.48	2,277.80	250	80	2,302.77	2,305.91	25	28
PRV-7	805,822.74	1,357,802.81	2,263.34	150	80	2,339.46	2,327.49	76	64
PRV-8	805,727.85	1,358,964.67	2,293.94	80	80	2,337.40	2,337.40	43	43
PRV-9	806,028.45	1,359,075.70	2,289.76	80	80	2,340.62	2,340.62	51	51
PRV-10	804,812.54	1,359,888.90	2,253.17	100	80	2,297.76	2,297.76	44	44
PRV-11	804,735.11	1,356,373.09	2,229.88	150	80	2,303.85	2,294.03	74	64
PRV-12	805,000.38	1,356,193.86	2,236.42	50	80	2,314.69	2,300.58	78	64
PRV-13	805,148.06	1,356,682.81	2,241.95	80	30	2,317.90	2,298.97	76	57
PRV-14	805,301.20	1,358,510.15	2,279.66	150	50	2,317.90	2,317.90	38	38
PRV-15	805,153.52	1,360,183.13	2,243.35	50	60	2,334.41	2,283.33	91	40

Table 4-45: Leakage Evaluation before and after improved system in the study area

Label	Actual Pressure (m)	Excess Pressure	Improved Pressure (m)	Excess Pressure after improved	n	k(l/s/m$^{1/2}$)	Actual Leakage L=KP^n(l/s)	Leakage after Improved L= KP^n(l/s)	Saved leakage (l/s)
J-24	96	16	95	15	0.5	0.074	0.30	0.29	0.009
J-48	97	17	68	0	0.5	0.074	0.31	0.00	0.305
J-23	95	15	66	0	0.5	0.074	0.29	0.00	0.287
J-56	120	40	90	10	0.5	0.074	0.47	0.23	0.234
J-54	95	15	65	0	0.5	0.074	0.29	0.00	0.287
J-29	94	14	67	0	0.5	0.074	0.28	0.00	0.277
J-18	93	13	63	0	0.5	0.074	0.27	0.00	0.267
J-42	83	3	57	0	0.5	0.074	0.13	0.00	0.128
J-53	89	9	76	0	0.5	0.074	0.22	0.00	0.222
J-51	90	10	60	0	0.5	0.074	0.23	0.00	0.234
J-17	91	11	61	0	0.5	0.074	0.25	0.00	0.245
J-25	96	16	95	15	0.5	0.074	0.30	0.29	0.009
J-43	88	8	72	0	0.5	0.074	0.21	0.00	0.209
J-35	94	14	93	13	0.5	0.074	0.28	0.27	0.010
J-32	91	11	89	9	0.5	0.074	0.25	0.22	0.023
J-10	104	24	88	8	0.5	0.074	0.36	0.21	0.153
J-58	113	33	86	6	0.5	0.074	0.43	0.18	0.244
J-22	90	10	61	0	0.5	0.074	0.23	0.00	0.234
J-27	97	17	96	16	0.5	0.074	0.31	0.30	0.009
J-26	101	21	100	20	0.5	0.074	0.34	0.33	0.008
J-60	118	38	88	8	0.5	0.074	0.46	0.21	0.247
J-52	98	18	69	0	0.5	0.074	0.31	0.00	0.314
J-39	95	15	93	13	0.5	0.074	0.29	0.27	0.020
J-31	90	10	74	0	0.5	0.074	0.23	0.00	0.234
J-47	99	19	73	0	0.5	0.074	0.32	0.00	0.323
J-46	101	21	74	0	0.5	0.074	0.34	0.00	0.339
J-9	106	26	90	10	0.5	0.074	0.38	0.23	0.143
J-49	101	21	72	0	0.5	0.074	0.34	0.00	0.339
J-59	109	29	79	0	0.5	0.074	0.40	0.00	0.399
J-55	101	21	72	0	0.5	0.074	0.34	0.00	0.339
J-50	82	2	53	0	0.5	0.074	0.10	0.00	0.105
J-11	94	14	93	13	0.5	0.074	0.28	0.27	0.010
Total							**9.50**	**3.29**	**6.207**

As shown on table 4-44 above **6.207** l/s of water is saved by reducing maximum pressure which is occurred during minimum day demand hour in the system by installing a pressure reducing valves.

CHAPTER FIVE

5. Conclusions

5.1 General Physical Characteristics of Infrastructures

This research work focused on the analysis of the technical parameters obtained by the hydraulic simulation using the integrated mathematical model developed. The research includes the technical performance assessment of the analyzed systems. The main conclusions can be drawn from this research work:

- Most of analyzed distribution network system about **53% of** pipes in the distribution system below the standard requirement **(0.6 m/s)** during peak hour**.** This is due to the pipe design being carried out for peak load conditions in the next 20-year time. This is one cause of sedimentation problem in the pipe.
- The second is that most of distribution network systems have pressure in excess, being evident the potential of savings of energy and water losses that these systems have. This is many times due to the need of supplying consumers at higher sections. On the other hand, all distribution network system **(44.89%)** does not satisfy the demand as minimum pressures **(i.e 15 m)** are not guaranteed in some parts of the system during peak-2-hour consumption **(15:00 PM-23:00PM).**
- Also there is, opposite to excess pressure **(negative pressure)** is that of suction, causing pipe to collapse. If a gate valve at the top end of a water-filled pipe is closed, water is not prevented from flowing out of the lower end, the resulting negative pressure can make even quite substantial pipes (especially plastic ones) collapse. Another negative consequence of negative pressure insides a pipe is that water containing pathogens may be sucked into the water supply through any small leaks in the pipe, for example at joints.

In general, it was concluded that the current water distribution network systems of Ginchi town categorized under satisfactory hydraulic performance situation and were not supply adequate water to various demand categories of the town. Particularly, the distribution system is not maintaining the minimum and maximum pressure and velocity.

- The existing system consisted of the boreholes, service reservoirs, and pipe system and water points. One borehole is pumped to the distribution network before it delivers to the service reservoir, while the rest borehole is directly pumped to the service reservoir. Direct type of distribution system has a problem of equitable water distribution system.

CHAPTER SIX

6. Recommendations

To improve the current conditions of water distribution system in the town, the following recommendations were drawn to Ginchi town existing water supply system:

Based on the findings:

- 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 developing new wells , and target the long term safe yield of **9,256m3/day** by carrying out detailed design and construction of a replacement for the two old wells.
- As computed additional **2,155m^3** service reservoir capacities should be constructed to deliver adequate water in the distribution networks at appropriate location.
- For **24hr.** Positive water pressure should be maintained in the system . A minimum residual pressure of **15m** under all operating conditions and at all locations (including at the system extremities) should be maintained.
- To improve the hydraulic performance of water distribution system , the system needs to have pressure zones to regulate pressure in locations where large elevation change differentials **(150m),**such as mountain communities , several consecutive pressure zones may be needed.
- Most of pipes are designed for positive pressures but may not withstand the same negative pressure and could collapse under extreme negative pressures. This collapse of the pipe could continue down the line until the elevation differential is no longer great enough to collapse it. Due to this reason air vacuum release valve allows air into the system to protect the infrastructure from damage.
- To avoid pipe deformation due to negative pressure , turn off the system`s gate valves , starting at the bottom and progressing up the hill while keeping the pipes pressured . If pipes need to be drained down, air must be allowed to enter the pipe at the higher part of the pipe while water is drained out at the lower end.
- To control maximum pressure ,water hammer and back water flow ; it was advised to installing a pressure reducing valves and necessary accessories in the distribution system. The valve is set to open only when there is large demand , such as during a fire.

- It was recommended that direct distribution from transmission main should be totally isolated from distribution main, and water should be distributed from the service reservior by gravity.
- Warka Gara Large Gravity Water supply Potential must be assessed and needs close discussion and arrangement to set substantial share of water source for Ginchi Town .

This research has generated several significant results. Pressure, head loss and velocity were modeled to provide deeper understanding of the current situation of Ginchi Town water distribution network system. Subsequently valuable suggestions were drawn based on findings to improve the situation. However, more comprehensive and detailed understanding was possible if plenty of time and resource were available. To have more complete understanding, future works are suggested to focus on:

- Impacts of various developments activities on performance of water distribution system.
- Performance of Warka Gara Large Gravity Springs as well as Ginchi town water supply system since all are interconnected system. Doing this provide more reliable water resource to the town.

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Appendixes

Appendix A

Base Demand Calculation

Junctions	Population Density(P/Km^2)	Area of Junctions (km^2)	Population of Junctions	Current Demand per capital (l/c/day)	Base demand at node (l/s)	At PHF	AT MDF	AT LDF
						1.8	1.2	0.3
J-24	8246	0.18	1525	64	1.13	2.03	1.4	0.34
J-48	8246	0.48	3926	64	2.91	5.24	3.5	0.87
J-23	8246	0.11	892	64	0.66	1.19	0.8	0.20
J-30	8246	0.09	728	64	0.54	0.97	0.6	0.16
J-45	8246	0.23	1878	64	1.39	2.50	1.7	0.42
J-56	8246	0.33	2752	64	2.04	3.67	2.4	0.61
J-54	8246	0.35	2898	64	2.15	3.86	2.6	0.64
J-29	8246	0.21	1721	64	1.27	2.29	1.5	0.38
J-18	8246	0.23	1883	64	1.39	2.51	1.7	0.42
J-42	8246	0.12	1026	64	0.76	1.37	0.9	0.23
J-53	8246	0.49	4030	64	2.98	5.37	3.6	0.90
J-51	8246	0.03	234	64	0.17	0.31	0.2	0.05
J-16	8246	0.14	1140	64	0.84	1.52	1.0	0.25
J-17	8246	0.06	464	64	0.34	0.62	0.4	0.10
J-33	8246	0.17	1382	64	1.02	1.84	1.2	0.31
J-25	8246	0.05	402	64	0.30	0.54	0.4	0.09
J-43	8246	0.06	465	64	0.34	0.62	0.4	0.10
J-28	8246	0.13	1097	64	0.81	1.46	1.0	0.24
J-35	8246	0.15	1222	64	0.90	1.63	1.1	0.27
J-32	8246	0.12	1013	64	0.75	1.35	0.9	0.23
J-41	8246	0.24	1989	64	1.47	2.65	1.8	0.44
J-10	8246	0.04	289	64	0.21	0.38	0.3	0.06
J-58	8246	0.36	2998	64	2.22	4.00	2.7	0.67
J-22	8246	0.18	1446	64	1.07	1.93	1.3	0.32
J-27	8246	0.06	461	64	0.34	0.61	0.4	0.10
J-26	8246	0.14	1181	64	0.88	1.58	1.1	0.26
J-60	8246	0.09	752	64	0.56	1.00	0.7	0.17
J-37	8246	0.22	1838	64	1.36	2.45	1.6	0.41
J-52	8246	0.17	1410	64	1.04	1.88	1.3	0.31
J-21	8246	0.02	194	64	0.14	0.26	0.2	0.04
J-34	8246	0.18	1509	64	1.12	2.01	1.3	0.34
J-39	8246	0.04	358	64	0.27	0.48	0.3	0.08
J-19	8246	0.07	613	64	0.45	0.82	0.5	0.14

J-31	8246	0.18	1520	64	1.13	2.03	1.4	0.34
J-47	8246	0.22	1785	64	1.32	2.38	1.6	0.40
J-40	8246	0.08	681	64	0.50	0.91	0.6	0.15
J-46	8246	0.13	1093	64	0.81	1.46	1.0	0.24
J-8	8246	0.08	656	64	0.49	0.87	0.6	0.15
J-9	8246	0.08	630	64	0.47	0.84	0.6	0.14
J-15	8246	0.08	624	64	0.46	0.83	0.6	0.14
J-49	8246	0.45	3730	64	2.76	4.97	3.3	0.83
J-59	8246	0.49	4017	64	2.98	5.36	3.6	0.89
J-55	8246	0.26	2167	64	1.61	2.89	1.9	0.48
J-50	8246	0.39	3248	64	2.41	4.33	2.9	0.72
J-38	8246	0.32	2611	64	1.93	3.48	2.3	0.58
J-20	8246	0.08	671	64	0.50	0.89	0.6	0.15
J-11	8246	0.07	591	64	0.44	0.79	0.5	0.13
		8.46	**69738**		**51.66**	**92.98**	**62.0**	**15.50**

Appendix B

Ginchi Water Supply Distribution System Junction Report at 8:00 AM

Label	X (m)	Y (m)	Elevation (m)	Hydraulic Grade (m)	Pressure (m)
J-24	1,357,560.64	805,496.63	2,250.00	2,336.86	86
J-48	1,357,950.38	804,400.46	2,248.00	2,301.14	53
J-23	1,359,832.53	803,773.79	2,250.00	2,295.39	45
J-30	1,359,365.40	806,136.26	2,271.00	2,349.16	78
J-45	1,358,736.99	805,350.96	2,289.00	2,302.96	14
J-56	1,356,331.92	804,510.00	2,225.00	2,301.33	76
J-54	1,358,818.87	803,727.42	2,250.00	2,301.12	51
J-29	1,359,965.88	804,246.07	2,251.00	2,295.26	44
J-18	1,358,087.70	804,952.12	2,253.00	2,301.94	49
J-42	1,359,602.34	804,939.02	2,262.00	2,295.75	34
J-53	1,359,662.27	805,132.43	2,257.00	2,347.78	91
J-51	1,358,122.62	805,178.33	2,256.00	2,302.26	46
J-16	1,358,762.65	805,320.42	2,286.00	2,346.69	61
J-17	1,358,124.21	805,115.63	2,255.00	2,302.25	47
J-33	1,358,490.92	805,829.21	2,283.00	2,348.01	65
J-25	1,357,574.93	805,268.03	2,250.00	2,336.86	87
J-43	1,359,719.42	806,337.34	2,259.00	2,349.06	90
J-28	1,359,150.03	805,747.32	2,284.00	2,348.05	64
J-35	1,356,708.15	805,597.44	2,252.00	2,324.93	73
J-32	1,357,539.21	805,842.70	2,256.00	2,344.47	88
J-41	1,358,666.51	804,881.07	2,275.00	2,296.31	21
J-10	1,360,179.79	804,772.59	2,242.00	2,346.69	104
J-58	1,361,001.06	804,586.33	2,232.00	2,287.08	55
J-22	1,359,735.69	803,983.34	2,255.00	2,295.30	40
J-27	1,357,185.99	805,236.28	2,249.00	2,335.93	87
J-26	1,357,051.06	805,463.29	2,245.00	2,331.82	87
J-60	1,356,089.82	804,584.51	2,228.00	2,334.68	106
J-37	1,359,433.27	804,507.22	2,270.00	2,295.49	25
J-52	1,357,570.17	805,204.53	2,247.00	2,301.66	55
J-21	1,358,797.08	806,152.14	2,308.00	2,350.11	42
J-34	1,359,069.73	804,801.70	2,281.00	2,296.68	16
J-39	1,356,958.98	805,841.12	2,252.00	2,343.88	92
J-19	1,358,856.34	805,944.87	2,304.00	2,349.25	45
J-31	1,359,616.23	805,924.06	2,257.00	2,348.90	92

J-47	1,360,123.83	804,714.39	2,246.00	2,293.44	47
J-40	1,358,749.85	805,260.49	2,286.00	2,301.70	16
J-46	1,360,340.53	804,083.35	2,244.00	2,295.11	51
J-8	1,359,214.99	805,120.79	2,278.00	2,300.16	22
J-9	1,360,221.07	804,881.60	2,240.00	2,346.69	106
J-15	1,358,779.61	805,467.92	2,292.00	2,346.86	55
J-49	1,357,453.49	804,554.71	2,244.00	2,299.19	55
J-59	1,357,658.28	803,489.23	2,236.00	2,301.12	65
J-55	1,356,885.96	805,110.07	2,244.00	2,301.34	57
J-50	1,358,630.79	804,338.94	2,263.00	2,295.68	33
J-38	1,359,204.67	804,161.94	2,271.00	2,295.15	24
J-20	1,358,833.59	805,766.37	2,301.00	2,348.60	48
J-11	1,357,856.12	805,255.10	2,252.00	2,339.18	87

Appendix C

Ginchi Water Supply Distribution System Junction Report at 15:00 PM

Label	X (m)	Y (m)	Elevation (m)	Hydraulic Grade (m)	Pressure (m)
J-24	1,357,560.64	805,496.63	2,250.00	2,250.00	0
J-48	1,357,950.38	804,400.46	2,248.00	2,299.92	52
J-23	1,359,832.53	803,773.79	2,250.00	2,296.77	47
J-30	1,359,365.40	806,136.26	2,271.00	-20,853,918.57	-20,814,179
J-45	1,358,736.99	805,350.96	2,289.00	2,300.92	12
J-56	1,356,331.92	804,510.00	2,225.00	2,300.02	75
J-54	1,358,818.87	803,727.42	2,250.00	2,299.91	50
J-29	1,359,965.88	804,246.07	2,251.00	2,296.70	46
J-18	1,358,087.70	804,952.12	2,253.00	2,300.36	47
J-42	1,359,602.34	804,939.02	2,262.00	2,296.97	35
J-53	1,359,662.27	805,132.43	2,257.00	-20,853,918.57	-20,814,165
J-51	1,358,122.62	805,178.33	2,256.00	2,300.53	44
J-16	1,358,762.65	805,320.42	2,286.00	-20,853,921.00	-20,814,198
J-17	1,358,124.21	805,115.63	2,255.00	2,300.52	45
J-33	1,358,490.92	805,829.21	2,283.00	-20,853,921.00	-20,814,193
J-25	1,357,574.93	805,268.03	2,250.00	2,250.00	0
J-43	1,359,719.42	806,337.34	2,259.00	-20,853,918.57	-20,814,168
J-28	1,359,150.03	805,747.32	2,284.00	2,284.00	0
J-35	1,356,708.15	805,597.44	2,252.00	2,252.00	0
J-32	1,357,539.21	805,842.70	2,256.00	-20,853,921.00	-20,814,167
J-41	1,358,666.51	804,881.07	2,275.00	2,297.28	22
J-10	1,360,179.79	804,772.59	2,242.00	-20,853,918.57	-20,814,151
J-58	1,361,001.06	804,586.33	2,232.00	2,292.22	60
J-22	1,359,735.69	803,983.34	2,255.00	2,296.73	42
J-27	1,357,185.99	805,236.28	2,249.00	2,249.00	0
J-26	1,357,051.06	805,463.29	2,245.00	2,245.00	0
J-60	1,356,089.82	804,584.51	2,228.00	-20,853,925.88	-20,814,144
J-37	1,359,433.27	804,507.22	2,270.00	2,296.83	27
J-52	1,357,570.17	805,204.53	2,247.00	2,300.20	53
J-21	1,358,797.08	806,152.14	2,308.00	2,308.00	0
J-34	1,359,069.73	804,801.70	2,281.00	2,297.48	16
J-39	1,356,958.98	805,841.12	2,252.00	-20,853,921.00	-20,814,163
J-19	1,358,856.34	805,944.87	2,304.00	2,304.00	0
J-31	1,359,616.23	805,924.06	2,257.00	-20,853,918.57	-20,814,165
J-47	1,360,123.83	804,714.39	2,246.00	2,295.71	50
J-40	1,358,749.85	805,260.49	2,286.00	2,300.23	14
J-46	1,360,340.53	804,083.35	2,244.00	2,296.62	53

J-8	1,359,214.99	805,120.79	2,278.00	2,299.38	21
J-9	1,360,221.07	804,881.60	2,240.00	-20,853,918.57	-20,814,148
J-15	1,358,779.61	805,467.92	2,292.00	2,292.00	0
J-49	1,357,453.49	804,554.71	2,244.00	2,298.85	55
J-59	1,357,658.28	803,489.23	2,236.00	2,299.91	64
J-55	1,356,885.96	805,110.07	2,244.00	2,300.03	56
J-50	1,358,630.79	804,338.94	2,263.00	2,296.93	34
J-38	1,359,204.67	804,161.94	2,271.00	2,296.64	26
J-20	1,358,833.59	805,766.37	2,301.00	2,301.00	0
J-11	1,357,856.12	805,255.10	2,252.00	-20,853,923.44	-20,814,165

Appendix D

Ginchi water supply Distribution Line Report during Peak Day Demand (8:00 AM)

Label	Length (Scaled) (m)	Start Node	Stop Node	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Head loss (Friction) (m)
P-17	354	T-1	J-21	250	PVC	130	36.252	0.739	0.84
P-18	216	J-21	J-19	200	PVC	130	26.6013	0.847	0.86
P-19	180	J-19	J-20	200	PVC	130	25.0844	0.798	0.64
P-20	303	J-20	J-15	150	PVC	130	14.7879	0.837	1.65
P-24	148	J-15	J-16	150	PVC	130	5.8677	0.332	0.15
P-25	406	T-2	J-45	250	PVC	130	35.586	0.725	0.94
P-26	348	J-20	J-33	150	PVC	130	7.8653	0.445	0.59
P-27	605	J-21	J-33	100	PVC	130	3.9807	0.507	2.09
P-28	972	J-33	J-32	150	PVC	130	11.8461	0.67	3.52
P-29	413	J-32	J-24	100	PVC	130	9.7989	1.248	7.58
P-30	522	J-24	J-26	100	PVC	130	6.9302	0.882	5.04
P-32	581	J-32	J-39	100	PVC	130	2.0472	0.261	0.59
P-33	407	J-39	J-26	50	PVC	130	2.0472	1.043	12.03
P-34	374	J-26	J-35	100	PVC	130	9.8082	1.249	6.89
P-35	1,221	J-60	J-35	50	PVC	130	1.0098	0.514	9.75
P-36	278	J-27	J-26	65	PVC	130	2.8108	0.847	4.11
P-37	390	J-25	J-27	100	PVC	130	4.5946	0.585	0.93
P-39	258	J-25	J-24	65	PVC	130	0.0104	0.003	0
P-40	1,545	J-27	J-60	80	PVC	130	1.0098	0.201	1.25
P-41	892	J-55	J-56	150	PVC	130	0.65	0.037	0.02
P-42	691	J-52	J-55	150	PVC	130	3.8678	0.219	0.31
P-46	716	J-45	J-51	250	PVC	130	22.3797	0.456	0.7
P-47	63	J-51	J-17	250	PVC	130	10.6263	0.216	0.02
P-48	168	J-17	J-18	150	PVC	130	8.2323	0.466	0.31
P-49	568	J-18	J-48	150	PVC	130	7.1141	0.403	0.8
P-50	957	J-48	J-59	100	PVC	130	0.1877	0.024	0.01
P-51	1,674	J-56	J-59	100	PVC	130	0.65	0.083	0.2
P-52	807	J-55	J-49	80	PVC	130	1.9218	0.382	2.15
P-53	520	J-48	J-49	50	PVC	130	0.6702	0.341	1.94
P-54	683	J-48	J-50	100	PVC	130	6.2562	0.797	5.46
P-57	1,265	J-15	J-24	100	PVC	130	6.229	0.793	10.03
P-58	281	J-10	J-25	100	PVC	130	6.387	0.813	2.34
P-59	909	J-16	J-10	100	PVC	130	6.387	0.813	7.55
P-60	553	J-51	J-52	150	PVC	130	6.2078	0.351	0.6
P-61	633	J-51	J-40	150	PVC	130	5.5456	0.314	0.56
P-62	583	J-18	J-41	50	PVC	130	1.1182	0.57	5.62

P-63	544	J-19	J-30	80	PVC	130	0.6709	0.133	0.21
P-64	407	J-30	J-43	50	PVC	130	0.2339	0.119	0.22
P-65	329	J-30	J-31	50	PVC	130	0.437	0.223	0.56
P-68	499	J-31	J-28	50	PVC	130	0.3547	0.181	0.57
P-69	639	J-43	J-31	50	PVC	130	0.2339	0.119	0.34
P-70	329	J-20	J-28	80	PVC	130	1.6932	0.337	0.69
P-75	486	J-40	J-8	100	PVC	130	3.7996	0.484	1.54
P-76	428	J-8	J-42	100	PVC	130	7.1761	0.914	4.41
P-77	568	J-42	J-47	100	PVC	130	6.6965	0.853	2.3
P-78	887	J-47	J-58	100	PVC	130	5.9025	0.752	6.36
P-81	117	J-9	J-10	80	PVC	130	0.5193	0.103	0.03
P-82	1,393	J-31	J-9	50	PVC	130	0.3162	0.161	1.29
P-83	1,524	J-10	J-16	80	PVC	130	0.5193	0.103	0.36
P-84	613	J-53	J-9	50	PVC	130	0.2031	0.103	0.25
P-85	1,036	J-28	J-53	65	PVC	130	0.4279	0.129	0.47
P-87	996	J-53	J-15	50	PVC	130	0.2248	0.114	0.49
P-88	633	J-45	J-8	150	PVC	130	13.2063	0.747	2.8
P-89	353	J-8	J-34	100	PVC	130	7.0038	0.892	3.47
P-90	411	J-34	J-41	100	PVC	130	1.9238	0.245	0.37
P-91	489	J-34	J-37	100	PVC	130	3.298	0.42	1.2
P-92	464	J-42	J-37	65	PVC	130	0.4796	0.145	0.26
P-93	414	J-37	J-38	65	PVC	130	0.584	0.176	0.33
P-94	615	J-50	J-38	80	PVC	130	1.0362	0.206	0.52
P-95	560	J-22	J-38	80	PVC	130	0.5578	0.111	0.15
P-96	349	J-22	J-29	50	PVC	130	0.1079	0.055	0.04
P-97	494	J-29	J-47	50	PVC	130	0.664	0.338	1.82
P-100	593	J-37	J-29	100	PVC	130	1.2137	0.155	0.23
P-101	231	J-23	J-22	80	PVC	130	0.6657	0.132	0.09
P-102	930	J-23	J-46	50	PVC	130	0.172	0.088	0.28
P-103	1,448	J-46	J-58	50	PVC	130	0.8295	0.422	8.03
P-104	408	J-29	J-46	80	PVC	130	0.6575	0.131	0.15
P-105	1,015	J-54	J-23	50	PVC	130	0.8377	0.427	5.73
P-108	1,306	J-59	J-54	250	PVC	130	0.8377	0.017	0

Pressure Line Report at peak Day Demand

Label	Length (Scaled) (m)	Start Node	Stop Node	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Head loss (Friction) (m)
P-1	62	R-1	PMP-1	100	Galvanized iron	120	11.7281	1.493	1.86
P-3	55	R-2	PMP-2	75	Galvanized iron	120	7.1221	1.612	2.61
P-8	2,858	J-14	T-1	150	Galvanized iron	120	18.85	1.067	28.35
P-9	37	R-4	PMP-4	75	Galvanized iron	120	12.2763	2.779	4.88
P-11	37	R-3	PMP-3	75	Galvanized iron	120	9.6583	2.186	3.08
P-16	1,715	J-12	T-2	150	Galvanized iron	120	21.9342	1.241	22.53
P-111	20	PMP-1	CV-1	100	Galvanized iron	120	11.7281	1.493	0.6
P-112	48	CV-1	J-14	100	Galvanized iron	120	11.7282	1.493	1.41
P-113	48	CV-2	J-14	57	Galvanized iron	120	7.1217	2.791	8.72
P-114	51	PMP-2	CV-2	75	Galvanized iron	120	7.1221	1.612	2.44
P-115	64	CV-3	J-12	75	Galvanized iron	120	9.6584	2.186	5.39
P-116	18	PMP-3	CV-3	75	Galvanized iron	120	9.6583	2.186	1.51
P-117	83	PMP-4	CV-4	100	Galvanized iron	120	12.2763	1.563	2.68
P-118	441	CV-4	J-12	100	Galvanized iron	120	12.2757	1.563	14.24