AAU

አዲስ አበባ ዩኒቨርሲቲ

Addis Ababa University

(Since 1950)

ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Effects of Pressure Change on Water Quality: Case of Akaki Phase 3B Water Supply Distribution System in Addis Ababa, Ethiopia

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 & Environmental Engineering (Water Supply & Environmental Engineering Stream)

By: Yetnayet Tsegaye

Advisor: - Dr.Agizew Nigussie

Addis Ababa, Ethiopia
December, 2017

DEDICATE

I dedicate this thesis manuscript to my father Tsegaye Lemma. Dear dead, you wished to see my completion but that wasn’t Gods permission. I always keep working hard to succeed all your good wishes. I love you.

ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Effects of Pressure Change on Water Quality: Case of Akaki Phase 3B Water Supply Distribution System in Addis Ababa, Ethiopia

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 & Environmental Engineering (Water Supply & Environmental Engineering Stream)

By

Yetnayet Tsegaye

Approved by Board of Examiners:

Dr: - Agizew Nigussie		
Advisor	Signature	Date
Internal Examiner	Signature	Date
External Examiner	Signature	Date
Chairman	Signature	Date

ACKNOWLEDGEMENT

First, I owe many thanks to the Almighty God for His protection and guidance in all aspects of my life.

Next, I would like to express my sincere thanks to my supervisor Dr. Agizew Nigussie (Addis Ababa Institute of Technology, School of Civil and Environmental Engineering) for his invaluable and tireless efforts in supporting, and supervising me on this M.Sc. study. Thank you for your timely supervising and sharing your valuable professional experiences to equip me in the areas of the effect of pressure on the water quality.

I am also Indebted to and give special thanks to family for their encouragement and motivation during my study.

I would like to give my deep appreciation to AAWSAs staffs; for those supported me in giving necessary information and documents, organizing field visits for data collection and arranging laboratory for conducting laboratory tests. My special thanks goes to ; Ato Fekadu Zeleke, Water case manager; Ato Zeleke Teferi, Water Quality Control case manager; Ato Solomen Tadese, Laboratory Technician; Ato Zelalem ,Water distribution system supervision staff; Ato Abebe ,Main line construction and maintenance staff.

I also thanks my teachers, friends and relatives who in one way or another supported me during the hole length of my M.Sc study.

ABSTRACT

Water network performance is defined as the ability to deliver a required quantity of water under sufficient pressure and an acceptable level of quality. Systems that have big transmission line may have problem on changes of pressure in the distribution system. Because pressure rate changes have been too much between water treatment plant and dead ends in distribution system. This study is to analyze the effects pressure changes on the water quality in water distribution systems for case study of Akaki phase 3B water supply system was assessed. Efforts were also made to identify the relationship between the residual chlorine and pressure in the distribution of the water supply network systems using Water Cad software /tool to model water distribution system.

Accordingly, simulation results show that, for maximum and minimum pressures were used as base to evaluate the hydraulic performance; and simulation results for minimum residual chlorine were used as base to assess water quality transformation in distribution system. Pressure has a direct relationship with residual chlorine. Pressure change has been causing to increase chlorine consumption more. The result of pressure changes among others can decrease in chlorine content and hydraulic factors, because the systems may be extensive and thus possess complex networks and hangs in quantity cause pressure of flow. Hence hydraulic changes (pressure) could affect of water quality in the water distribution system.

Keywords: *Pressure Changes; Residual Chlorine; Water Quality; water distribution systems*

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ABBREVIATIONS

AAWSA	Addis Ababa Water and Sewerage Authority
Asl	Above Sea Level
DCI	Ductile Iron
EPS	Extended Period Simulation
GIS	Geographical Information System
GS	Galvanized Steel Pipe
PRV	pressure-reducing (pressure-regulating) valve
PVC	Polyvinyl chloride
RMS	Root Mean Square Error
WDS	water distribution system

CHAPTER ONE

1.0 INTRODUCTION

1.1 Background

Addis Ababa is the capital city of Ethiopia. Geographically the city is located 9^{o}01'29" to north and 38^{o}44'48" to east. It is the largest city in Ethiopia, with a population of 3,107,423 [1]... The city is located at the center of Ethiopia with an area of 540 km2 of which 18.174 km2 is rural and its altitude ranges from 2000m - 2800 m above sea level (asl).

Addis Ababa has subtropical highland climate. The city possesses a complex mix of highland climate zones, with temperature differences of up to 10 °C, depending on elevation and prevailing wind patterns. According to National Meteorological Agency of Ethiopia, mean minimum and maximum temperature of the day varies from season to season; however, the variation is not too large. Mean minimum temperature varies from 7°C to 11°C and mean maximum varies from 21°Cto 25°C. For month of December mean minimum temperature is 7°C and mean maximum temperature is 23°C. For month of May mean minimum temperature is 11°C and mean maximum temperature is 25°C. From June to mid-September is main rainy season for city of Addis Ababa. Mid-November to January is a season for occasional rain.

Addis Ababa was established as the capital city of Ethiopia in 1886 and has grown to become the largest urban and commercial center in the country. During its early years, the principal sources of water were the numerous springs located at the foot of Entoto Mountain and hand dug wells located in the lower areas. The larger springs were tapped and fed into a number of small tanks for local distribution [9].

Continued growth necessitated the construction, in 1938, of a plant at the foot of Entoto to treat water from a number of springs and the nearby Kechene River, and in 1944 the original Gefersa dam located North West of the city was constructed [9].The *Gefersa* Dam was raised and a treatment plant built in 1960, while many of the springs were taken out of service because their quality was deteriorated. In 1966, the raw water storage capacity in the *Gefersa* watershed was increased with the construction of another small dam north of the existing dam. This dam was also assumed to assist as a sediment trap. At this time primary source of Addis Ababa's water supply relied on the *Gefersa* facilities. The supply from *Gefersa* was transmitted via twin 400mm pipelines to nine service reservoirs for distribution [9].

The next major phase of expansion of the water supply facilities commenced in 1970 with the commissioning of the *Legedadi* Dam and treatment plant, which was located on the Akaki River east of Addis Ababa.

To transfer and distribute these additional water additional reservoirs, pumping stations and pipelines were constructed in the eastern and northern areas of the city. These facilities came to be known as the Stage I Water Supply Project.

The third huge water source of Addis Ababa has been explored from Ground Water source; which is the Akaki well field is situated southeast of Akaki town and about 22 km south of Addis Ababa. The well field covers an area of about 16 km2.

A total of 35 wells are drilled within this area: 25 production wells, four monitoring wells, four wells for water supply to Akaki, one well for isotope sampling, and one deep test well. The Akaki well field was put into operation in 2001. The capacity of the wells is 347 l/s. Further development of the water supply facilities was pursued during the l980s under the Stage II Water Supply Project [9]. The First phase included expansion at the *Legedadi* treatment plant, construction of a new transmission pipeline into the city, the rehabilitation of the *Gefersa* treatment plant and the construction or upgrading of several reservoirs and pumping stations throughout the city. The second phase included extensive primary and secondary pipeline installations and improvements to the distribution network. The capacity of the supply facilities, of 150,000 and 30,000 m3/day respectively for Legedadi and Gefersa were projected to be adequate to serve the need of Addis Ababa up to 1992.

Planning for a Stage III water supply program commenced in the early 1980’s, when a reconnaissance study was undertaken of all potential water supply sources located within a 50km radius [9]. In 1991, feasibility studies and preliminary designs were completed for the development of a number of sources to serve the city to the year 2020 [9].

The delay in the implementation of Water Supply Stage Ill-A (WSS Ill-A) project called for an emergency program to fast-track the development of two water supply projects. These are part of the Akaki well field and the Dire dam, which are completed and both are under service. In addition, a program of spring rehabilitation and bore hole drilling has provided improved water supplies to outlying areas of the city not yet serviced by the distribution network Additional Programs are underway to reduce leakage losses and to improve operational efficiencies.

Addis Ababa City Administration is at present supplied with surface water from the Legadadi, Dire and Gefersa reservoirs, groundwater pumped from Akaki well field located to the south of Addis Ababa and other wells and springs located within the city. The current total daily production is estimated to be 301,597 m3/day. Out of the total production 65% is from the surface water sources and the remaining 35% is from ground water.

This thesis is focused on the *Akaki phase 3B subsystem* of its effects of change of pressure on water quality of water distribution system by modeling.

1.2 Existing Water Supply Sources

The 25 boreholes were drilled in 2010 for Akaki phase 3B water supply. The distribution network has 25 sources but in this study represented by one reservoir as a source (CT2 Reservoir).
This is located close to the well field, near Gelan condominium house, at an elevation of 2070 m a.s.l. The water will be chlorinated at CT-2 pumping station and pumped via DN800mm to Hana Mariam pumping station (5.92km), and then to Fana pumping station (3.60km) and then to K4 Tank (4.5 km), it is distributed toAlem Bank Village tank and Army Hospital tank, and they distributed to the consumers by gravity to Alem Bank village, kereniyo and other central parts of water deficit areas.

The network consists of more than one type of pipe such as (PVC – DCI – GSP…), there are about 5 main tanks. The largest One is 10000 m3 water storage tanks (k4), the distribution network of the Akaki phase 3B water supply which was commissioned in the year 2013. This source is located in Akaki kality sub city. Currently, the supply system is from the source which has a capacity of discharging 70,000m3/day. The supply system is both Gravity and pressurized system.

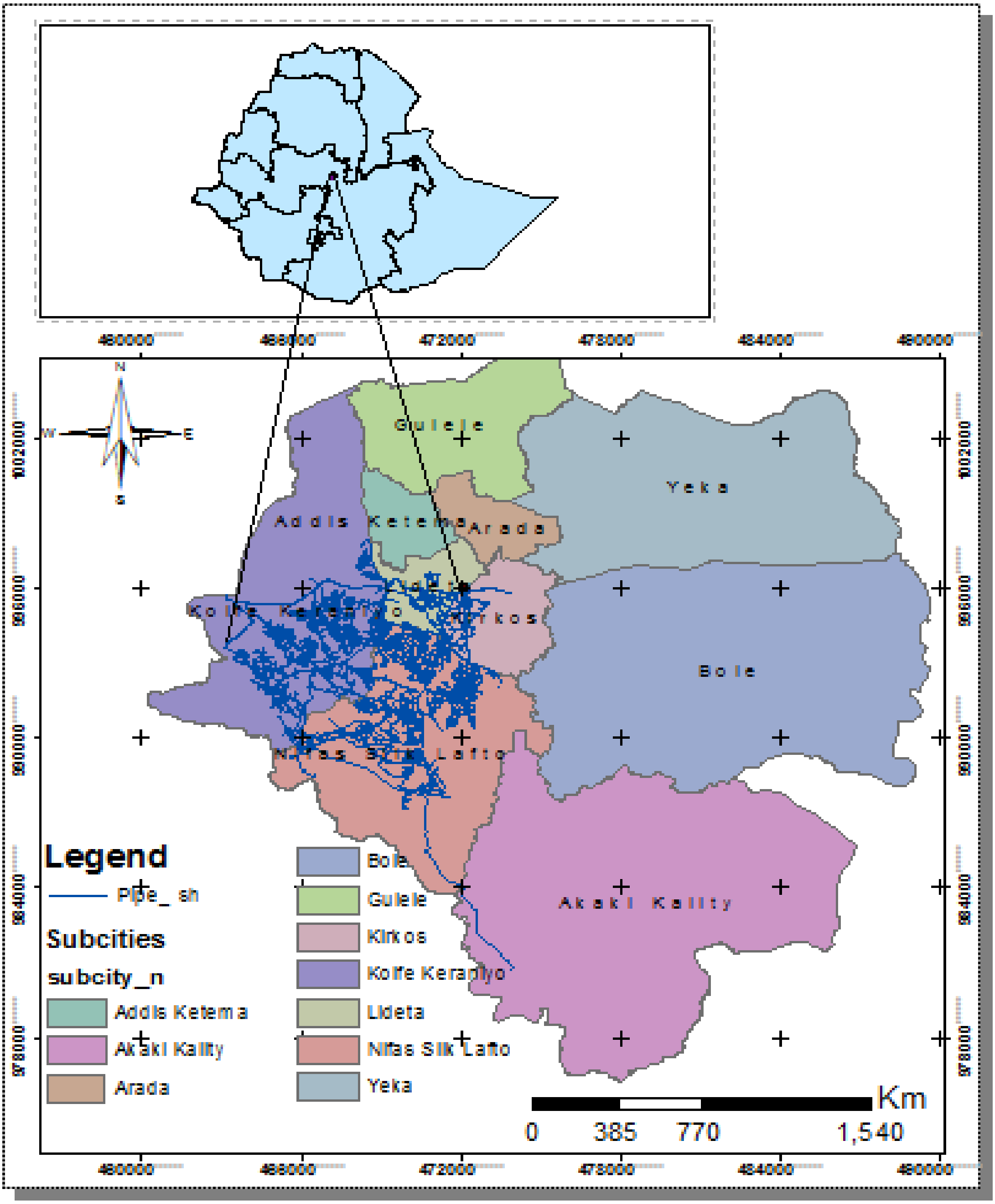

Figure 0-1: Location of study Area

1.3 History of piped water in Addis Ababa city

The early use of piped water supply in city of Addis Ababa is back to the beginning of twenty century. It was a new advent introduced 15 years after the establishment as the capital city [2]. Since then, the use of clean water which was readily available near the home of dwellers became the common practice in day to day life even though the traditional method of water collection is still continued.

In 1900, the possibility of conveying water from a distance was realized when the city was supplied processed water for the first time from Kebena river. Masonry ducts were constructed and laid along the course of Kebena river at the top of Entoto; where water was collected in small dams from where the water carried down to the desired destinations in the city [2].

In 1924, a major turn took place, considering the shortage of water the city had faced previously, when as many water pipes as possible were imported to undertake a revised water supply project. Accordingly, the water from springs near Entoto was collected in one reservoir and distributed by means of 3-inches pipes to different part of the city.

The uneven distribution of the supply and the growing demand led to the decision of takingFundamental action as a result of which the feasibility study of the first Gefersa dam was launched in 1940 and completed during the succeeding year. Gefersa dam introduced the construction project of the first dam in the country.

The new turning point and the beginning of a big leap forward in the history of Addis Ababa water supply started when Gefersa supply system was commissioned in 1943. However, the increase in demand which followed the growth of urban population had put pressure on the management and had necessitated the immediate and subsequent feasibility study for new project. To this effectConstruction of the Legedadi dam located 30km east of the capital was launched and finalized in 1970. Subsequently, in 1970 the Legedadi treatment plant construction was completed and began a service in the same year [2].

Since 1985, expansion works were undertaken to increase production capacity of Legedadi treatment plant. Dire earth dam was built in a crash program in 1995 to augment the production of the Legedadi dam. Along with dire, construction of Akaki ground water wells was also underway to boost the city's water production abruptly [3].

1.4 Statement of the problem

Water supply system is one of the infrastructures, where developing countries are working hard to expand. Still so many people both in rural and urban areas are suffering with the provision of adequate potable water supply and sanitation.

In many developing countries the existing situation of water supply system shows that does not meet the users demand. Similarly Addis Ababa city shows the same status. The degree of insufficient is both on the quality and quantity of water. And it is becoming the adverse effect on urban development and public health and poor sanitation [2].

City of Addis Ababa is the largest political, social and economic capital of Ethiopia as well as center of International Organizations. Currently the water supply sources for the city are reservoirs found at Gefersa, Legadadi sites and Akaki well fields, springs and wells located in various part of the city. According to the official bullet of Addis Ababa Water and Sewerage Authority (AAWSA) which is entitled.

Presently Addis Ababa faces a serious deficit in the water supply due to Increased population and expanded economic actually in and around the subsystems.

Systems that have large transmission and distribution lines may have problem on changes of pressure in the distribution system. For the reason that the increase in water residence is dependent on the difference between the production and consumption rates, high residence time in pipes and storage duration in water tanks some of the problems.

Therefore, Addis Ababa Town topography has great elevation variation from service reservoir to end of customer use, very long and complex networks of the distribution system and also water production is improved in the last few years (111 liter per capita per day), still there is high water shortage. There are instants in which water shortage is accelerated by undesirable pressure within distribution system. High leakage and pipe failure (due unmaintained maximum pressure) as well as provision of insufficient supply (due to unmaintained minimum pressure) are situations which circulate water shortage within distribution system. Therefore, this study was conducted to investigate the current pressure situation of the Akaki phase 3B water supply distribution system of Addis Ababa city.

1.5 Objectives

1.5.1 General Objective

To analyze the effect of pressure change on water quality in the case of Akaki phase 3B water supply Distribution system.

1.5.2 Specific Objectives

- ✓ To assess water pressure of distribution systems
- ✓ To evaluate the free residual chlorine level in the distribution system
- ✓ To recommend improvement measures

1.6 Research question

The main research questions that were effort to be addressed are:

I. Does the water pressure of Akaki phase 3B water supply distribution system fit the guideline set in the standard that of the Ethiopian recommended guideline?
II. Is there appropriate disinfectant at the water supply Service?
III. What relationship does exist between the pressure and residual chlorine concentration in the water supply gravity distribution system of the water supply network system?

1.7 Thesis organization

Chapter 1: The back ground of the Addis Ababa water supply distribution system a, statements of the problem, general objective, specific objectives, and the research question.

Chapter 2: it includes literature review and conceptual frame on present some literatures written by other authors which have similar research issues with this research paper the review of the pressure effect on water quality. as well as the review of Water CAD, and describes in brief about the importance GIS in water distribution system.

Chapter 3: methods and material used narrative of the study area in detailed and samples location points of study area were explained.

Chapter 4: it includes the result and discussion of the research in detailed.

Chapter 5: It includes the conclusion and recommendation of the research.

CHAPTER TWO

2.0 LETURATURE REVIEW

2.1 Network Hydraulics

In networks of interconnected hydraulic elements, every element is influenced by each of its neighbors; the entire system is interrelated in such a way that the condition of one element must be consistent with the condition of all other elements. Two basic equations that govern in Water CAD modeling network of these interconnections [4]

- Conservation of mass or continuity principle.
- Conservation of energy or energy principle.

2.1.1 Conservation of Mass

Several formulations of conservation of mass and energy can be written for water Distribution system under steady conditions [5]. Here the pipe flow equation formulation is summarized. For a junction that connects two or more pipes, conservation of mass is written as:

$$\Sigma Qi-U=0 \quad \ldots\ldots 2.1$$

Where, Qi = inflow to node in i-th pipe (L3/T)

U = water used at node (L3/T)

The term for accumulation of water at nodes is required to describe stored and withdrawn water from tanks, while extended period simulation is regarded [5].

$$\Sigma\ Qi-U-ds/dt=0 \quad \ldots\ldots 2.2$$

Where, (ds/dt) = changes in storage (L3/T).

2.1.2 Conservation of Energy

The Energy equation is known as Bernoulli's equation. It consists the pressure head, elevation head, and velocity head. There may be also energy added to the system (such as by a pump), and energy removed from the system (due to friction). The changes in energy are referred to as head gains and head losses. In hydraulics, energy is converted to energy per unit weight of water, "head".

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

$Z_1+P_1/\gamma+v_1^2/2g+hp=Z_2+P_2/\gamma+v_2^2/2g+hl+hm$..2.3

Where, Z = elevation (L)

P = Pressure (M/L/T2)

γ = fluid specific weight (M/L2/T2)

V = velocity (L/T)

g = gravitational acceleration constant (L/T2)

hp = head added at pump (L)

hL = head loss in pipes (L)s

hm = head loss due to minor losses (L)

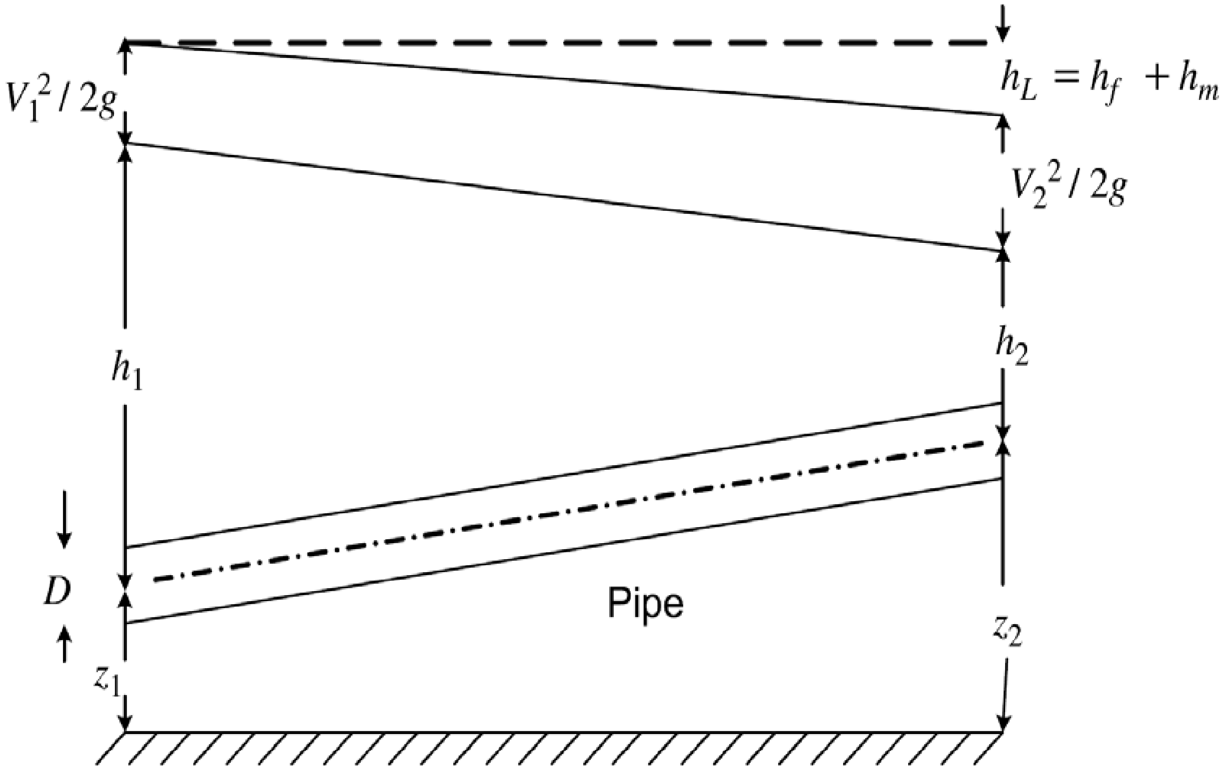

Figure 2.1. Definition sketch.

Fig. 2.1 Forms of energy in water pipes

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.

2.2 Hydraulic Performance of Water Supply Distribution system

The science of hydraulics is as old as civilization itself. For centuries, engineers have succeeded in making water flow from one place to another with as few hitches as possible. When problems do occur, they are usually related to the hydraulics involved in pipe flow. Liquids in motion produce forces and pressure whenever the velocity, flow direction, or elevation changes.
Knowing pipe pressure and flow at certain points along the pipe's path can help determine pipe size and capacity. It also can help in determining what pipe material would work best in given situations. Further, having an understanding of hydraulics can help system managers decide if pressure reducers or pumps are necessary to transport water in an efficient manner.

Most small water and wastewater systems do not have an engineer on staff, or even one they can consult, who might be able to answer simple or basic questions about hydraulics. And the cost of engineering advice can quickly add up. Because many small systems are on tight budgets, understanding hydraulics can save money.

Performance of a water distribution network can be defined as its ability to deliver a required quantity of water under sufficient pressure and an acceptable level of quality during different normal and abnormal operational situations.

Water distribution network performance can be accessed from different points of view including water quality parameters (e.g., residual chlorine, water age, etc.).

In the big distribution systems, water needs to travel a large distance with a long water residence time. The problem could affect water quality. This may be due to low pressure, big and multiple reservoir storages, insufficient disinfection in the system, leaking, fracture and loosening of joints, and so on. As a result, the problems of quantity are basic agents in the decay of water quality in distribution systems. One of the parameter that could reason to decrease water quality in distribution system is pressure change in network. Many researchers have been carried out about affects pressure change in the water quality in distribution systems.

2.2.1 Water Pressure

Water distribution networks must maintain adequate water pressure throughout the network to ensure continuity in service and for fire suppression. Low water pressure can result in flow reductions and high water pressure can cause leaks and damage to system components.

Water pressures vary by communities; however, water pressure is typically maintained between 25 and 75 psi. Pressure is controlled in the network by pumps and the elevation of reservoirs and tanks. Aside from pipe leaks and water consumption, pressure is lost in the system due to pipe friction.

A systems analysis can be performed to ensure that a specific network meets pressure range requirements under normal operating conditions and when the system is stressed by component failure or changes to operations. WATR CAD can be used to calculate pressures at all nodes in the system.

2.3 Water Quality Modeling

Water quality models first perform a hydraulic analysis then provide the resulting flow distribution to the water quality module to transport a constituent through the systems. Chlorine is the most popular water treatment disinfectant in municipal water distribution system. Chlorine is an oxidizing agent and it decays with time. Therefore, a minimum level of chlorine residual must be maintained in the distribution system to preserve both chemical and microbial quality of treated water [6]. While chlorine travels through the distribution system, it reacts with different materials inside the pipe. Reactions are impacted by the surrounding conditions due to the availability of reacting substances.

Reactants are present in the bulk water and may also occur at high concentrations on the surface of the pipe. Bulk reactions predominate in relatively less turbulent water. On the other hand, wall or surface reactions predominate in turbulent flows. Water quality transport mainly occurs by advective transport in which the constituent (chlorine) moves with water in the direction of flow with the magnitude of the main velocity component. In other words, advection transport is principles of conservation of mass coupled with reaction kinetics carrying constituent (chlorine) along with the flow of water [7].Thus, in addition to direct reaction parameters i.e., bulk reaction coefficient and wall reaction coefficient hydraulic parameters i.e., pipe diameter and roughness, and flow play important roles in determining the chlorine concentrations at all junction nodes in the systems.

2.3.1 Advective Transport in Pipe

According to [8], advective transport within a pipe can be represented with the next equation.

$$\frac{\partial ci}{\partial t} = \frac{\partial ci}{\partial Ai} * \frac{\partial ci}{\partial x} + (Ci) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.4$$

Where, Ci = concentration in pipe i (M/L)

Qi = flow rate in pipe i (L3/T)

Ai = cross-sectional area of pipe i (L2)

θ(Ci) = reaction term (M/L3/T)

The above equation shows concentration within a pipe i as a function of distance along its length (x) and time (t).The advective transport equation is a function of discharge divided by cross-sectional area.

2.3.2 Mixing at pipe junctions

At junctions receiving inflow from two or more pipes, the mixing of fluid is taken to be complete and instantaneous. Thus the concentration of a substance in water leaving the junction is simply the flow weighted sum of the concentrations from the inflowing pipes. The nodal mixing equation which is used by water quality simulation combines concentration from individual pipes and defines the boundary conditions for each pipe.

$Cout = (\Sigma QCi,ni+Ui)/\Sigma Q$………………………..………………………….....................2.5

Where, Coutj = concentration leaving the junction node j (M/L3)

OUTj = set of pipes leaving node j

INj = set of entering node j

Qi = flow rate entering the junction node from pipe i (L3/T)

Ci,ni = concentration entering junction node from pipe i (M/L3)

Uj = concentration source at junction node j (M/ L3)

2.3.3 Mixing in Tanks

It is convenient to assume that the contents of storage facilities (tanks and reservoirs) are completely mixed. This is a reasonable assumption for many tanks operating under filland draw conditions providing that sufficient momentum flux is imparted to the inflow [9]. Under completely mixed conditions the concentration throughout the tank is a blend of the current contents and that of any entering water. At the same time, the internal concentration could be changing due to reactions accordingly;

Equation 2.6 applies when a tank is filling.

$Dc/dt = Q/V\ (Ci,np(t) - cK) + (cK)$……………………………………………………………2.6

Where, Ck = concentration within tank or reservoir k (M/L3)

Qi = flow entering the tank or reservoir from pipe i (L3/T)

Vk = volume in tank or reservoir k (L3)

θ(Ck)= reaction term (M/L3/T)

The tank mixing equation accounts for blending and any reactions that occur within the tank volume during the hydraulic time step. During a hydraulic step in which draining occurs, terms can be dropped and the equation simplified;

$Dc/dt\ K = (ck)$……………………………………………………………………………......2.7

2.3.4 Chemical Reaction Terms

Chemical reaction terms are present in Equations 2.5 and 2.6. Concentrations within pipes, storage tanks, and reservoirs are a function of these reaction terms. After water leaves the treatment plant and enters the distribution system, it is subject to many complex physical and chemical processes. Three chemical processes that are frequently modeled, however, are bulk fluid reactions, reactions that occur on a surface (typically the pipe wall), and formation reactions involving a limiting reactant. First, an expression for bulk fluid reactions is presented, and then a reaction expression that incorporates both bulk and pipe wall.

2.3.4.1 Bulk Reactions

Bulk fluid reactions occur within the fluid volume and are a function of constituent Concentrations, reaction rate and reaction order, and concentrations of the formation products.

A comprehensive expression is given by;

$(C) = \pm K.Cn$……………………………………………………………………………2.8

Where, θ(C) = reaction term (M/L3/T)

K = reaction rate coefficient [(L3/M)n-1/T]

C = concentration (M/L3)

n = reaction rate order constant

The decay reactions frequently used to model chemical process that occur in water distribution system are: zero-, first-, and second-order. Using the generalized expression in Equation 2.8, these reactions can be modeled by letting n to equal 0, 1, or 2 and subsequently performing a regression analysis to experimentally determine the rate coefficient.. The most frequently used reaction model is the first order decay model. This is given by;

$Ct = Co*e-kt$……………………………………………………………………………2.9

Where, Ct = concentration at time t (M/L3)

Co= initial concentration (at time zero)

K = reaction rate (1/T)

The time it takes for the concentration of a substance to decrease to 50 percent of its original concentration is termed as half-life. For instance, the half-life of radon is approximately 3.8 days, and the half-life of chlorine can vary from hours to many days.

The relationship between the decay rate, k, and half-life is obtained by solving Equation 2.9 for the time t when Ct/Co is equal to a value of 0.5.

$T= -(0.693)/T$………………………………………………………………………2.10

2.3.4.2 Pipe Wall Reactions

While flowing through pipes, dissolved substances can be transported to the pipe wall and react with materials, such as corrosion products or biofilm that are on or close to the wall. The amount of wall area available for reaction and the rate of mass transfer between the bulk fluid and the wall also will influence the overall rate of this reaction.

The surface area per unit volume, which for a pipe equals 2 divided by the radius, determines a mass transfer coefficient, the value of which depends on the molecular diffusivity of the reactive Species and on the Reynolds number of the flow. For first-order kinetics, the rate of a pipe wall reaction can be expressed as:

$$r=\frac{2Kf*Kw*C}{R(kw+Kf)}\text{ ……………………………………………………………….……...…………}2.11$$

Where, kw = wall reaction rate constant (L/T), kf = mass transfer coefficient (L/T), and R =pipe radius (L). If a first-order reaction with rate constant kb also is occurring in the bulk flow, then an overall rate constant k (T-1) that incorporates both the bulk and wall reactions can be written as;

$$K=\frac{Kb+2Kf*Kw*C}{R\ (kw+Kf)}\text{ ……….…………………………………………………….......}2.12$$

2.4Effects of pressure changes on water quality in distribution system

One of the parameter that could reason to decrease water quality in distribution system is pressure change in network. Numerous researchers have been carried out about affects pressure change in the water quality in distribution systems. Changes in pressure could cause leakage in distribution systems, and that changes in pressure could lead to a problem of drinking water quality.

Changes of pressures and their effects on water quality. Various pressures and changing the velocity could increase turbidity and corrosion in distribution systems. The important factors that affect water quality is design network. Designers usually prefer to be over designed; this idea could be the reason why detention time in the distribution system is longer than desirable [10].

Decay of chlorine (Kw, Kb), velocity and pipe material. [1] it is necessary for the pressure in the distribution systems to be high. Ideally, pumps are required for this but will shackle the distribution systems. Also that the quality of water in the distribution systems decays over time. In addition, those components of distribution systems with laminar flow will develop deposits that increase corrosion, leakage and breakage and systems with large transmission lines usually have problem with pressure and that customers must utilize pumps to increase pressure to a minimum of 20 - 30 psi. Pump construction within the distribution system itself gives rise to negative pressures, especially when the system increases consumption. [10]

High residence time, high pressure and low pressure systems and distribution net work will cause a decline in the water quality in the gravity distribution system. Declared that systems with high pressure will have problem of pipe fracture in the distribution system [10].

one of the important parameters related to bacteria re-growth is hydraulic agents. [7] Changes in bacteria growth patterns in the distribution system depend on changes in hydraulics. [6], changes in pressure within the distribution system are important to water quality as well as leakage, the effects of pressure changes with different flow rates, making systematic measurements; also pressure changes decrease water quality and leakage in distribution systems.

Yet so, in large distribution networks and systems, hydraulic changes are high, there is so much pressure along the gravity distribution lines and this might cause fractures or cracks in pipes even at the dead end areas of the system.

2.5 Water Distribution Simulation Steady-state models

Simulation refers to the process of imitating the behavior of one system through the functions another. In our case, the term simulation refers to the process of using a mathematical representation or real system, called a model [15]. Simulation can be used to predict system responses to under a wide range of conditions without disrupting the actual system, and solutions can be evaluated before time, money, and materials are invested in a real-world project. There are two most basic types of simulations that a model may perform, depending on what the modeler is trying to observe or predict. These are:

- Steady state simulation.
- Extended period simulation (EPS).

2.5.1 Steady State Simulation

It computes the state of the system (flows, pressures, pump operating attributes, valve position, and so on) assuming that hydraulic demands and boundary conditions do not change with respect to time..A steady-state simulation provides information regarding the equilibrium flows, Pressures and other variables defining the state of the network for a unique set of Hydraulic demands and boundary conditions. Steady-state models are generally used to analyze specific worst-case conditions such as peak demand times, fire protection usage, and system component failures in which the effects of time are not particularly significant.

2.5.2 Extended Period Simulation

Extended period simulation tracks a system over time, and it is a serious of linked steady state run. The need to run extended period simulation is because the system operations change Over time.

- ✓ Demands vary over the course of the day.
- ✓ Pumps and wells go on and off.
- ✓ Valves open and close.
- ✓ Tanks fill and draw.

- ✓ **Simulation Duration:** An extended-period simulation can be run for any length of time,depending on the purpose of the analysis. The most common simulation duration istypically a multiple of 24 hours, because the most recognizable pattern for demands andoperations is a daily one.
- ✓ **Hydraulic Time Step:** An important decision when running an extended-period simulation is the selection of the hydraulic time step. The time step is the length of time for one steady-state portion of an EPS, and it should be selected such that changes in system hydraulics from one increment to the next are gradual. A time step, too large May cause abrupt hydraulic changes to occur, making it difficult for the model to give good results. Using an EPS model we can simulate based on the peak, minimum and average day demands.

2.6 Water Distribution Modeling

2.6.1 General

A model is a tool that can be used to determine the likely response of a system to a given set of stimuli without having to actually impose those stimuli on the system [6].Today, water distribution modeling is a critical part of designing and operating water distribution systems that are capable of serving communities reliably, efficiently, and safely, both now and in the future [6].With today's technology and expedient software packages, we are able to model a system relatively quickly. This saves us from the repetitive iterations that determine the flows and pressures.

2.6.2 Water CAD

Water CAD is an easy-to-use hydraulic and water quality modeling application for water distribution systems. Utilities, municipalities, and engineering firms trust Water CAD as a reliable, resource saving, decision support tool for their water infrastructure.

From fire flow and constituent concentration analyses, to energy cost management and pump modeling, Water CAD helps engineers and utilities analyze, design, and optimize water distribution systems.

The built-in water quality features help Water CAD users perform constituent, water age, tank mixing, and source trace analysis to develop comprehensive Chlorination schedules simulate emergency contamination events, visualize zones of influence for different water sources, and improve turbidity, taste, and odor by Identifying water blending problems in the system.

Water CAD provides sensitive access to the tools needed to model complex hydraulic situations. Some of the key features allow us to:

- Perform steady state and extended period simulations.
- Analyze multiple time-variable demands at any junction node.
- Quickly identify operating inefficiencies in the system.

- Perform hydraulically equivalent network skeletonization including data scrubbing, branch trimming, and series and parallel pipe removal.
- Efficiently manage large data sets and different “what if” situations with database query and edit tools.

2.6.3 Assembling a Model

A water distribution model is a mathematical description of a real-world system. Before building a model, it is necessary to gather information describing the network (Sources of data used in constructing models). Model skeletonization is the process of simplifying the real system for model representation, and it involves making decisions about the level of detail to be included. Below are several considerations for water distribution modeling that should be weighed while assembling our model components and developing your schematics.

- Potential large water consumption;
- Important loops;
- Large diameter pipes;
- Pumps, towers, tanks; and
- Topography.

2.6.4 GIS Integration with Hydraulic Models

This integration of the hydraulic model and the GIS leads to the following benefits: [8]

- Time-savings in constructing models
- Ability to integrate disparate land use, demographic, and monitoring data using GIS analysis tools to more accurately predict future system demands.
- Visual, map-based quality control of model inputs.
- Map-based display and analysis of model outputs in combination with other GIS layers.

In addition to being used for map-making; a GIS can be used to perform system analysis, Answering questions about: [8].

- Location (using proximity, buffer, or overlay analysis)
- Condition
- Temporal and spatial patterns (trends)
- What-if scenarios (in modeling)

Within a GIS, features (objects on a map) are not simply points and lines; they have attributes (information about the feature) associated with them. In a water distribution system, facilities such as pipes, tanks, and pumps are features possessing attributes [8]. The GIS also was used to assign demands to modeling nodes. (Nodes associated with pumps and tanks were temporarily removed so they would not have demands assigned.) [8]. Geocoding was used to geolocate customer meters to the Topologically Integrated Geographic Encoding and Referencing street centerline file. During geocoding, the GIS matched the address of the customer meter with address ranges in the street centerline file.

Demands were then assigned based on the proximity of the customer meter to a model node. Aggregate demands were computed and then applied to the node [8]. A GIS professional can use a GIS to create a model more efficiently, more accurately, and more Cost effectively than an engineer creating a model input file from scratch inside a traditional Modeling environment. Consider the following: [8].

2.7 Water Distribution Network Building and Model Setup

The approach to building the model is to first sketch out the system practically on existing topographic maps. The concept of a network is fundamental to a water distribution model. The network contains all of the various components of the system, and defines how those elements are interconnected. Networks are comprised of nodes, which represent features at specific locations within the system, and links, which define relationships between nodes.

Water distribution models have many types of nodal elements, including junction nodes where pipes connect, storage tank and reservoir nodes, pump nodes, and control valve nodes. Models use link elements to describe the pipes connecting these nodes. In addition, elements such as valves and pumps are sometimes classified as links rather than nodes.

Intelligent use of element labeling can make it much easier for users to query tabular displays of model data with filtering and sorting commands.

Rather than starting pipe labeling at a random node, it is best to start from the water source and number outward along each pipeline. In addition, just as pipe elements were not laid randomly, a pipe-labeling scheme should be developed to reflect that.

2.7.1 Reservoirs

The term reservoir has a specific meaning with regard to water distribution modeling that may differ slightly from the use of the word in normal water distribution construction and operation.

A reservoir represents a boundary node in a model thatcan supply or accept water with such a large capacity that the hydraulic grade of the reservoir is unaffected and remains constant. It is an infinite source, which means that it can theoretically handle any in flow or outflow rate, for any length of time, without running dry or overflowing. In reality, there is no such thing as a true infinite source.
For modeling purposes, however, there are situations where inflows and out-flows have little or no effect on the hydraulic grade at a node.

Reservoirs are used to model any source of water where the hydraulic grade is controlled by factors other than the water usage rate. Lakes, groundwater wells, and clear wells at water treatment plants are often represented as reservoirs in water distribution models. For modeling purposes, a municipal system that purchases water from a bulk water vendor may model the connection to the vendor’s supply as a reservoir.

For a reservoir, the two pieces of information required are the hydraulic grade line
(water surface elevation) and the water quality. By model definition, storage is not a concern for reservoirs, so no volumetric storage data is needed.

2.7.2 Tanks

A storage tank is a boundary node, but unlike a reservoir, the hydraulic grade line of a tank fluctuates according to the inflow and outflow of water. Tanks have a finite storage volume, and it is possible to completely fill or completely exhaust that storage (although most real systems are designed and operated to avoid such occurrences).

For steady-state runs, the tank is viewed as a known hydraulic grade elevation, and themodel calculates how fast water is flowing into or out of the tank given that HGL.

Given the same HGL setting, the tank is hydraulically identical to a reservoir for a steady-state run. In extended-period simulation (EPS) models, the water level in the tank is allowed to vary over time.
Regardless of the shape of the tank, several elevations are important for modeling purposes. The maximum elevation represents the highest fill level of the tank. The overflow elevation, the elevation at which the tank begins to overflow, is slightly higher than the maximum elevation. Similarly, the minimum elevation is the lowest waterlevel in the tank should ever be. A base or reference elevation is a datum from whichtank levels are measured, use fig.7 the levels of these tank elevation.Source: [7]

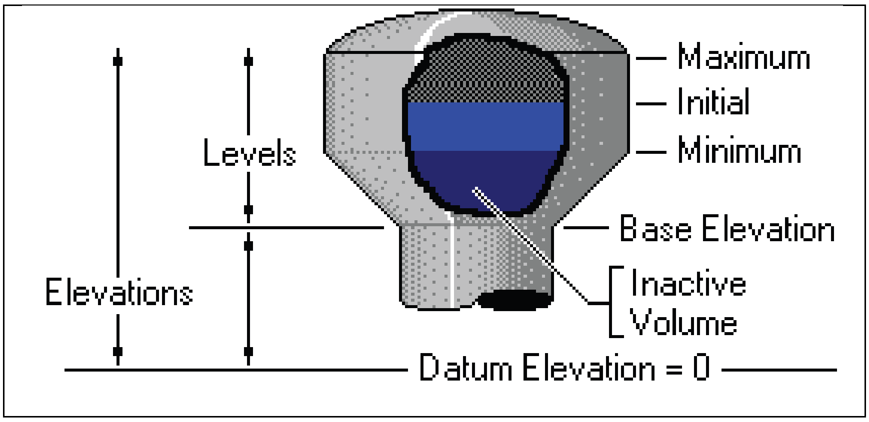

Fig2.2 Diagram illustrating the different tank elevations for modeling

2.7.3 Junctions

As the term implies, one of the primary uses of a junction node is to provide a location for two or more pipes to meet. The other is to provide a location to withdraw water demand from the system or inject inflows (sometimes referred to as negative demands) into the system. Junction nodes typically do not directly relate to real-world distribution components, since pipes are usually joined with fittings, and flows are extracted from the system at any number of customer connections along a pipe.

2.7.4 Pipes

A pipe conveys flow from one junction node to another in a network. In practice, pipelines may have various fittings, such as elbows, to handle abrupt changes indirection, or isolation valves to close off flow through a particular section of pipe.

For modeling purposes, individual segments of pipe and associated fittings can all be combined into a single pipe element. A model pipe should have the same characteristics (size, material, etc.) throughout its length.

- **Length**

The length assigned to a pipe should represent the full distance that water flows from one node to the next, not necessarily the straight-line distance between the end nodes of the pipe.

- **Scaled versus Schematic**

Most simulation software enables the user to indicate either a scaled length or a user defined length for pipes. Scaled lengths are automatically determined by the software, or scaled from the alignment along an electronic background map. User-defined lengths, applied when scaled electronic maps are not available, require the user manually enter pipe lengths. Even in some scaled models, there may be areas where there are simply too many nodes in close proximity to work with them easily at the model scale (such as at a pump station). In these cases, the modeler may want selectively depict that portion of the system schematically.

- **Diameter**

A pipe’s nominal diameter refers to its common name, such as a 16-in. (400-mm) pipe. The pipe’s internal diameter, the distance from one inner wall of the pipe to the opposite wall, may differ from the nominal diameter because of manufacturing standards. Most new pipes have internal diameters that are actually larger than the nominal diameters.

2.7.5 Pumps

A pump is an element that adds energy to the system in the form of an increased hydraulic grade. Since water flows “downhill” (that is, from higher energy to lower energy), pumps are used to boost the head at desired locations to overcome piping head losses and physical elevation differences.

CHAPTER THREE

3.0 METHODOLOGY, PROCEDURES AND MATERIALS USED

3.1 Source of Data

3.1.1 Data Collection

The most important step in any research study is data Collection. In building the model the distribution network, the secondary data were first gathered regarding all the distribution system parameters. Particular field measurements and visits to the Akaki phase 3B Subsystem are conducted with AAWSA technical team.

3.1.2 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 information such as:

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

For this study system maps were collected from Addis Ababa Water and Sewerage Authority (AAWSA).

3.1.3 Topographic Maps

We use Topographic map a sets of lines called Contours to indicate elevations of the ground surface. Contour lines represent a contiguous set of points that are at the same elevation and can be thought of as the outline of a horizontal “slice” of the ground surface. By the superimposing a topographic map on a map of the network model, it is possible to interpolate the ground elevations at a junction nodes and other location throughout the system.

3.2 Reaction-rate data

Dynamic models require reaction rate data (bulk and wall coefficients) to model chlorine decay. A first-order rate constant for chlorine decay in the bulk flow (bulk coefficient) was estimated by performing a bottle test in the laboratory.

Samples (by volume 200ml) were collected from two entrance locations. One set of samples were taken from CT2 site treatment plant (just prior to entering distribution system) and three sets of samples were taken from K4 reservoir site.

3.3 Modeling the Existing Distribution System

To analyze and improve the existing water distribution system, a model was developed Utilizing Water CADV8i software (Water CAD for Auto CAD 2007 software).

Water CAD is selected for this study because of the following reason;

- It is aided with good quality of manual.
- Its integration with other external software’s, like Auto CAD and Microsoft excel.
- It requires less effort and shorter time to build a model than others do.

3.3.1 Working Methodology

The approaches adopted for each of the system components to execute the model are described below:

- All the existing water distribution system and other related available data have been collected.
- Missed data for modeling of the system have been generated.

- The existing water distribution layout has been made using Water CAD for Auto CAD 2007 software tools, model representation.
- All the existing and generated data have been entered into the built model.
- The model has been simulated for single period and extended period.

3.4 Model Representation

The Concept of network is fundamental to a water distribution model. The network contains all of the various components of the system, and defines how these elements are interconnected. Networks are comprised of nodes, which represent features at specific locations within the system, and links which define relationships between nodes. Both primary and secondary data are collected and analyzed using the following Methodologies.

Physical observation of scheme facilities and the condition of the sources of the water supply scheme.

- Determination of chlorine dosage and feeding rate.
- Observing water tank level from staff gauge and Pressure meter reading.

3.5 Random Sampling location of Residual chlorine measurement

Samples must be taken from locations that are representative of the water source, treatment plant, storage facilities, distribution network, points at which water is delivered to the consumer, and points of use. In selecting sampling points, each locality should be considered individually; however, the following general criteria are usually applicable: Sampling points should be selected such that the samples taken are representative of the different sources from which water is obtained by the public or enters the system.

- These points should include those that yield samples representative of the conditions at the most unfavorable sources or places in the supply system, particularly points of possible contamination such as unprotected sources, loops, reservoirs, low-pressure zones, ends of the system, etc.

- Sampling points should be uniformly distributed throughout a piped distribution system, taking population distribution into account; the number of sampling points should be proportional to the number of links or branches.
- The points chosen should generally yield samples that are representative of the system as a whole and of its main components.
- Sampling points should be located in such a way that water can be sampled from reserve tanks and reservoirs, etc.
- In systems with more than one water source, the locations of the sampling points should take account of the number of inhabitants served by each source.
- There should be at least one sampling point directly after the clean-water outlet from each treatment plant.

Sampling regimes using variable or random sites have the advantage of being more likely to detect local problems but are less useful for analyzing changes over time. Samples are taking from locations that are representative the water reservoirs, treatment plant, storage facilities distribution network points at which water is delivered to the consumers' points at use. In selecting sampling each locality is considered. However the following criteria are usually applicable.

- These points include those that yield samples representative of the conditions at the most unfavorable sources or places in the supply system particularly points of possible Contamination such as unprotected Sources or raw water, treatment plant, reservoirs, low-pressure zone, ends of the system etc…
- Sampling points should be uniformly distributed throughout a piped distribution system, considering population distribution; the number of sampling points should be proportional to the number of links or branches.
- There should be at least one sampling point directly after the clean water outlet from each treatment plant [14].

3.6 Sampling of Pressure and residual chlorine

The analysis of the effects of pressure changes on the residual chlorine in water distribution systems are carried out by installing pressure gauge at selected sampling location such as, public functions, Customer tap points, distribution water reservoir, pump house and other sampling points. in order to assess the relationship the pressure and residual chlorine concentration in the system.

The measured pressure and chlorine residual collected at different locations are used for calibration and validation of WaterCad water quality model.

Pressure, tank level and residual chlorine data is take at both peak hourly demand and minimum hourly demand.

3.7 The variation of pressure with residual chlorine

The study of the effects of pressure changes on the residual chlorine in water distribution systems are carried out by install pressure gauge at selected sampling location such as, public fountains, customer tap point, distribution water reservoir, pump house, and other sampling points.

In order to assess the relationship between the pressure and residual chlorine concentration in the system, the measured pressures and chlorine residual collected at different locations were used for calibration and validation of water quality model.

3.8 Water Demand

The consumption or use of water also known as water demand is the driving force behind the hydraulic dynamics occurring in water distribution systems, anywhere that water can leaves the system represents a point of consumption, including a customer’s faucet, a leaky main or an open hydrant[14].

Three questions related to water Consumption must be answered when building a hydraulic model:

1. How much water is being used?
2. Where are the points of consumption located?
3. How does the usage change a function of time?

The above questions are for each of the three basic demand types illustrated below should be addressed.

1. Customer demand is the water required to meet the non-emergency needs of users in the system. This demand type typically represents the metered portion the total water consumption.
2. Unaccounted –for water(UFW) is the portion of total consumption that is “lost” due to system leakage, theft, unmetered services, or other causes.
3. Fire flow demand is a computed system capacity requirement of ensuring adequate protection is provided during fire emergencies.

3.8.1 Base line Demand

Determining base line demands to which a variety of peaking factors and demand multipliers can be applied, or to which a new land developments and customers can be added. Base line demands typically include both customer demands and unaccounted for water. Usually the average day demand in the current year the base line from which other demand distributions are built.

To assign base demand to each supply node, it is necessary to know the houses around each supply node.

Some of demand events frequently considered:

- Maximum day demand : the average rate of use of the maximum usage day (past, present or future)
- Average day demand : the average rate of day demand for an average day (past, present or future)
- Peak hour demand : the average rate of usage during the maximum hour of usage (past ,present or future)
- Maximum day of record: the highest average rate of demand for the historical record.

3.8.2 The varying demands

Water usage in municipal water distribution system is inherently unsteady due to continuously varying demands. In order for an extended period simulation to accurately the dynamics of real system, these demand fluctuations must be incorporated in to the model.

Demand pattern is a function relating water use to time of day. A diurnal curve is a type of pattern that describes changes in demand over the course of daily cycle.

The temporal variations in water usage for municipal water systems typically follow a 24-hour cycle called a diurnal demand pattern. However system flow experiences changes not only on a daily bases, but also weekly and annually.

3.8.3 Using GIS for Demand Allocation

When working with high-quality GIS data, the modeler can much more precisely allocate demands to nodes. Nodal demands can be loaded using several GIS-related methodologies, ranging from a simple inverse-pipe-diameter allocation model to a comprehensive polygon overlay.

3.8.3.1 Meter Aggregation

Meter aggregation is the technique of assigning all meters within a service polygon to a specified demand node. Service polygons define the service area for each of the demand junctions.

Meter aggregation is a polygon-to-point allocation technique because the service areas are contained in a GIS polygon layer and the demand junctions are contained in a point layer. The demands associated with each of the service-area polygons are assigned to the respective demand node points. In this study demand was located by meter Aggregation method.

Bottle test

A bottle test allows the bulk reactions to be unglued from other processes that affect water quality, and thus the bulk reaction can be evaluated exclusively as a function of time. The parameter of bulk reaction used to express the rate of the reaction occurring within the bulk fluid is called the bulk reaction coefficients can be determined using a simple experimental procedure called a bottle test. In addition, a bottle test allows for the evaluation of the impact of transport time on water quality and for an experimental determination of the parameters necessary to model this process accurately. Determining the length of the bottle test and the frequency of sampling is the first and most critical decision [8].

3.9 Calibration

Calibration was conducted by comparing the results of the actual element of the model with results generated by the model of the same locations.

To calibrate the model, some junctions accessible and available for pressure and chlorine data measurement are selected and flow measurements are taken using pressure gage for pressure and reagent for chlorine measuring instrument. Using the information provided by the AAWSA, the model was calibrated to meet the general calibration criteria for Pressure and chlorine.

Pressures are measured right through the water distribution system using pressure gauge installed at selected location to check the level of service and to collect data for use in model calibration. Therefore, matching only pressures is not adequate. In addition, ideally the model should be calibrated over an extended period, such as a time range for the maximum of one day. Successfully calibration of indicates that the network components are operating correctly, demand is appropriately allocated, and the Reservoir levels are correct.

Besides, the correct calibration of the pressure indicates that the pipe roughness Coefficients are set at the appropriate levels, Similar to actual conditions.

3.10 Method of data analyses

The result of the experimental data was used to analyzing by using application of software such as MS EXCEL, Water CAD and GIS software's.

CHAPTER FOUR

4.0 DATA ANALYSIS AND INTERPRETATION

4.1 General information about model inputs

4.1.1 Water Source

Most well-liked water supply network modeling software packages frequently model a water distribution system as a collection of links connected to nodes. The links represent pipes, pumps, and control valves. The nodes represent junctions, tanks, and reservoirs.

Accordingly, a complete water network modeling effort requires a combination of multiple raw data that can be processed and inputted in to either nodes or links but collected in diverse way. Typical data required include the probable maximum daily water production from available sources. In fact, quantification of produced and supplied water to a network system is the basic and an essential step to establish the entire modeling process.

AWSSA's official records indicate that currently city of Addis Ababa is obtaining treated Water from both Ground and Surface sources. Akaki Well fields along with various drilled wells (including Akaki phase 3b) at various part of the city are major ground sources. Legedadi and Gefersa are the major surface sources. Natural springs are also contributing significant quantity.

At subsystem level Akaki phase 3B sub system obtaining supply mainly from soft loan wells they are about 24 wells. Currently the soft loans produce an average of 70000 m3 water per day. In addition water produced from wells located in different parts of the city is directly inputted to the subsystem.

According to [3], AAWSA is currently supplying mean per capita demand of 111 liter per day at city level. This figure includes water loss in the system. Based on average customer billing records from September 2015 to February 2016 was able to account and “unaccounted-for” water are 99904.5m3/day. “Unaccounted-for” water is water that is leaves the system through illegal connections or leaks in the pipe line or it associates with unreadable meters. This Unaccounted for water rate represents approximately 39 % of the total system production.

Average production of water per day from September 2015- up to February 2016 was 99904.5m3/day. Presently from main distribution Akaki phase 3B produces 70000.00 m3/day quantity of water. Daily and production of deep wells are 29904.5m3/day.

4.2 Model Representation

The Concept of network is fundamental to a water distribution model. The network contains all of the various components of the system, and defines how these elements are interconnected. Network is comprised of nodes, which represent features at specific locations within the system, and links which define relationships between nodes.

Figure 4.1 illustrates layout of Akaki phase 3B water supply system. The sketch was extracted from Addis Ababa city water distribution system map and represented in the model according to available drawing options.

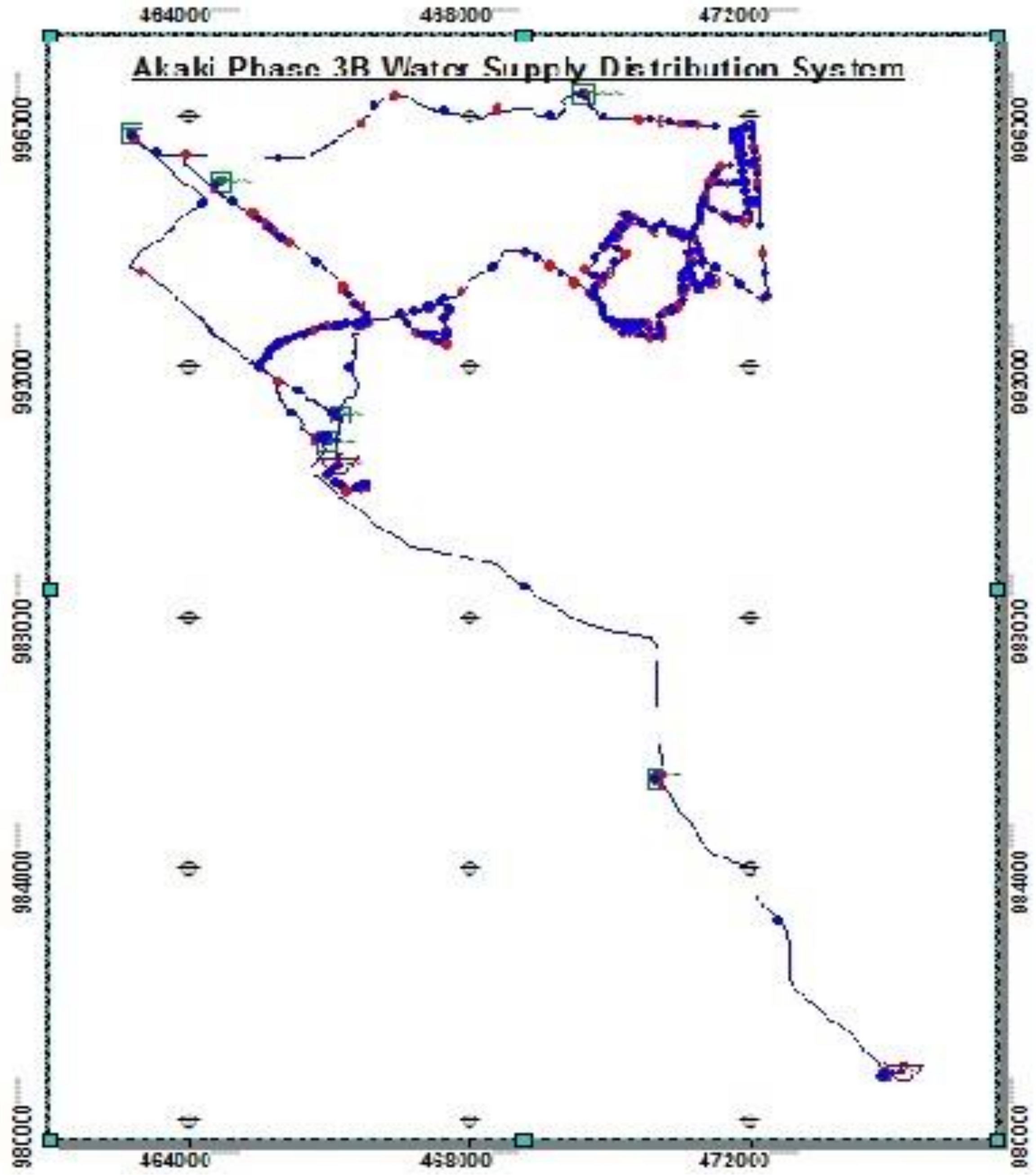

Figure 4- 1: Layout of Akaki phase 3B water supply Distribution system

4.3 Water Demand

Determining demand is not straight forward like collecting data on the physical characteristics of the system. Billing and production records are collected directly from AAWSA but are not directly interred into the model. According to [3], AAWSA is currently supplying mean per capita demand of 111 liter per day at city level. This figure includes water loss in the system. Based on average customer billing records from September 2015 to February 2016 was able to account and "unaccounted-for" water are 99904.5 m3/day.

Presently from main distribution Akaki phase 3B produces 70000.00 m3/day quantity of water. Daily and production of deep wells are 29904.5m3/day.

Using GIS for Demand Allocation

When working with high-quality GIS data, the modeler can much more precisely allocate demands to nodes. Nodal demands can be loaded using several GIS-related methodologies, ranging from a simple inverse-pipe-diameter allocation model to a comprehensive polygon overlay.

4.3.1 Meter Aggregation

Meter aggregation is the technique of assigning all meters within a service polygon to a specified demand node. Service polygons define the service area for each of the demand junctions. Meter aggregation is a polygon-to-point allocation technique because the service areas are contained in a GIS polygon layer and the demand junctions are contained in a point layer. The demands associated with each of the service-area polygons are assigned to the respective demand node points. In this study demand was located by meter Aggregation method.

In assigning nodal demand, nodes that representing each kebele were branded by overlapping water distribution system map with Addis Ababa Administrative city map. The city map was preferably used since it contains sub cities and kebeles boundaries. Accordingly the supplied kebeles were identified from Five sub city; Lideta , Nifas silk,Addis Ketema, Arada and Kolfe Keraniyo.

Using monthly billing meter of kebeles for the Average of 6 month from September 2015 to February 2016 and nodes representing each kebele with consideration of leakage (39 %.) water demand at each node was allocated.

4.4 Bulk Coefficient

Literatures recommend bulk coefficient has to be determined by performing bottle test in laboratory [11].Since the test was entirely focused on changes observed in residual chlorine at time due to reaction within the bulk water, collecting sample from all part of the distribution system was not required. Only one main entrance locations were selected. in advance of conducting the test, it was necessary to make certain arrangements. The first was to define the length of experiment. Maximum of 24 hour experimental period was selected. Then equipments desired for the experiment were identified and collected. The composed equipments were: Sampling kit for sample handling, sampling bottles (200 ml), and Test tube.

Accordingly, samples were collected random sampling technique from the Water Distribution system .Tests for bulk coefficient determinations were conducted in three test periods. Basically three test periods were preferred in order to avoid possible error and make sure that the result of each test is virtually the same. Three test samples were collected for each test period and measurements were taken early from collection time. Then samples were brought to laboratory and stored in complete darkness with temperature held constant that is by keeping the sample in the dark area being protected from direct sun light.

After all measurements have been taken and the experiment is over, the data will describe the constituent concentration for each of the samples as a function of time. The data can then be graphed. The constituent concentrations are charted along the y-axis (the dependent variable), and the time is charted along the x-axis (the dependent variable). At the finished of the lab test, the natural logarithms of the ratio of measured chlorine to initial chlorine (Ct/Co) value were plotted alongside time. The rate constant was the slope of the straight line as shown in the following figures.

Based on test results detailed in Appendix H, fig 4.2, fig4.3 and fig 4.4 were plotted using the ratio of Concentration at any time (Ct) to initial Concentration(Co) as ordinate and time as abscissa.

The best fitted line draw through charted result where the slope of line is the bulk reaction coefficient. Hence, for first bottle test, second bottle test and 3rd bottle test of the slope of line is - 0.021, -0.017, and -0.019, respectively.

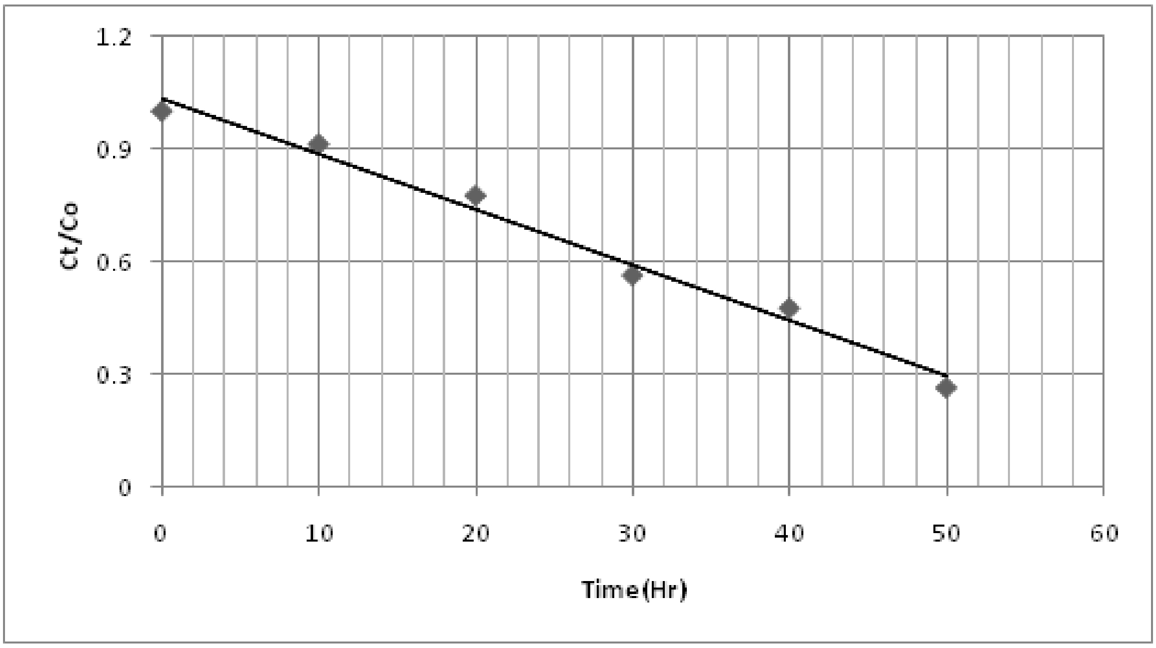

Figure 4- 2:Plot of residual chlorine for test 1

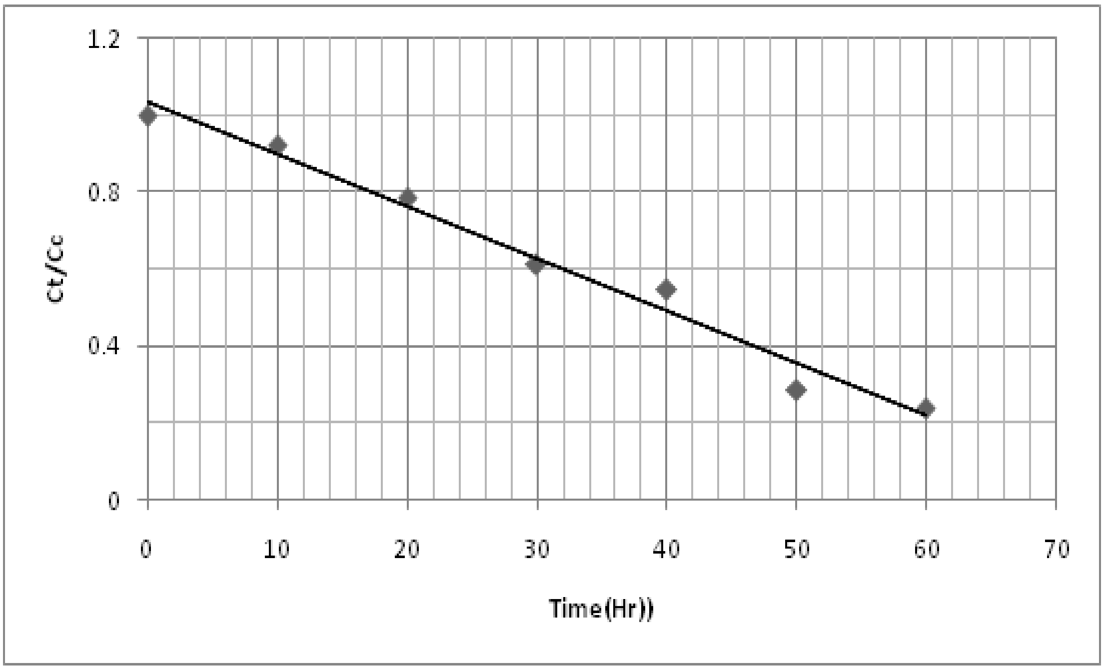

Figure 4- 3: Plot of residual chlorine for test 2

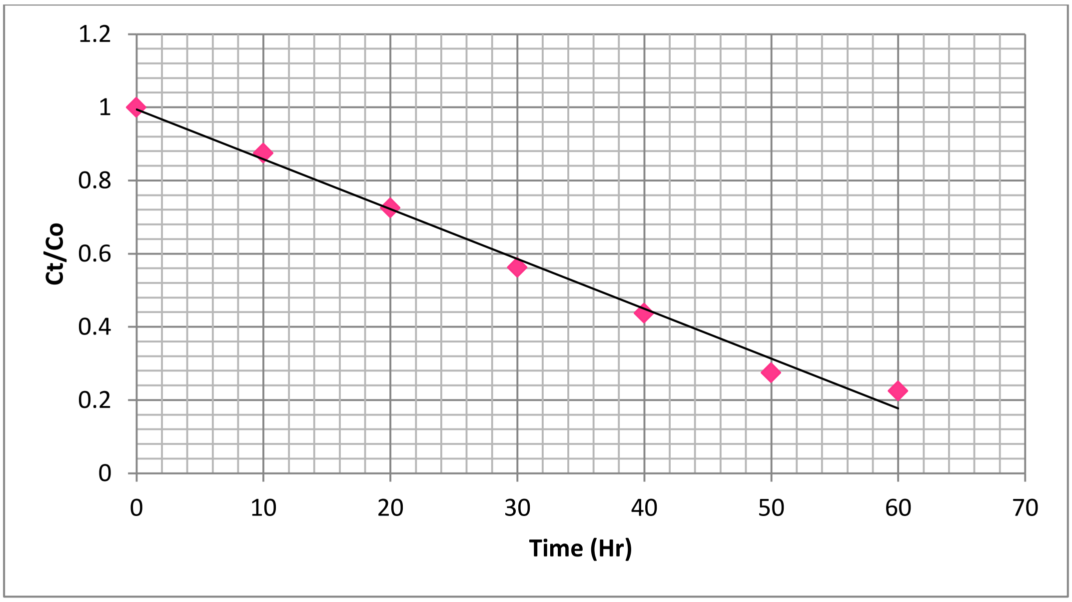

Figure 4- 4: Plot of residual chlorine for test 3

4.5 Model Calibration and Validation

The importance of a model is purely evident if a model result precisely imitates observed Field values. Thus, to have assurance on model result it needs to calibrate and validate a Model. Calibration is an iterative method of parameter estimation and refinement, as a result of comparing simulated and observed values of interest. Model validation is in reality an extension of the calibration process. Its purpose is to assure that the calibrated model properly assesses all the variables and conditions which can affect model results, and demonstrate the ability to predict field observations for periods separate from the calibration.

An effort to perform hydraulic and water quality model calibration and validation for this case study is presented as follows.

4.5.1 Model Performance Evaluation Criteria

There are many ways to evaluate on the performance of model calibration. The evaluations were made by calculating the squared relative difference between observed and simulated pressure for each test. The evaluation criteria used was statistical method using correlation coefficient (R^2) and Root Mean Square Error (RMS) and graphical method.

$$R^2 = \frac{(\sum(x-\bar{x})(y-\bar{y}))}{\sqrt{\sum(x-\bar{x})^2}\sum(y-\bar{y})^2}$$

Where R^2 = Correlation Coefficient, x and y are measured and simulated values respectively, $\bar{x}$ and $\bar{y}$ are average values of measured and simulated, respectively.

4.5.2 Calibration and Validation using Time-Series Data

According to [8] a vital step in calibrating and validating an extended-period simulation is to compare time-series field data to model results. If the field data and model results are acceptably close, the model is calibrated. If significant variations exist, Adjustments can be made to various model parameters in order to improve the match. Ideally, one set of data should be available for calibration, and another set of data should be available to validate that the model is properly calibrated. Hydraulic measurements, water quality data, and tracer data are frequently used in combination in the extended period simulation (EPS) Calibration and validation process.

4.5.2.1 Acceptable Levels of Calibration

Each application of a model is unique, and thus it is impossible to derive a single set of guidelines to evaluate calibration. The guidelines presented below give some numerical Guidelines for calibration accuracy; however, they are in no way meant to be definitive. A range of values is given for most of the guidelines to reflect the differences among water Systems and the needs of model users. The higher numbers generally correspond to larger, more complicated systems, and the lower end of the range is more relevant to smaller, simpler systems. The words “to the accuracy of elevation and pressure data” mean that the model should be as good as the field data. If the Hydraulic Grade Line (HGL) is known to within 8 ft (2.5 m), then the model should agree with field data to within the same tolerance. It is important to remember that these guidelines need to be tempered by site-specific considerations and an understanding of the intended use of the model [8].

4.5.2.2 Master planning for smaller systems [24-in. (600-mm) pipe and smaller]:

The model should accurately predict hydraulic grade line (HGL) to within 5–10 ft (1.5–3 m) (depending on size of system) at calibration data points during fire flow tests and to the accuracy of the elevation and pressure data during normal demands. It should also reproduce tank water level fluctuations to within 3–6 ft (1–2 m) for EPS runs and match treatment plant/pump station/ well flows to within 10–20 percent.

4.5.2.3 Master planning for larger systems [24-in. (600-mm) and larger]:

The model should accurately predict HGL to within 5–10 ft (1.5–3 m) during times of peak velocities and to the accuracy of the elevation and pressure data during normal demands. It should also reproduce tank water level fluctuations to within 3 to 6 ft (1–2 m) for EPS runs and match treatment plant/ well/pump station flows to within 10–20 percent.

4.5.2.4 Disinfectant models:

The model should reproduce the pattern of observed disinfectant concentrations over the time samples were taken to an average error of roughly 0.1 to 0.2mg/l.

4.5.3Hydraulic Calibration and Validation

Hydraulic behavior refers to flow conditions in pipes, valves and pumps, and Pressure/head levels at junctions and tanks. Accordingly the hydraulic model calibration Parameters that are typically set and adjusted include pipe roughness factors, minor Losses, demands at nodes, the position of isolation valves (closed or open), control valve Settings, pump curves, and demand patterns. When initially establishing and adjusting these parameters, care should be taken to keep the values for the parameters within Reasonable bounds.

Model validation is in actuality an addition of the calibration process. It is used to give surety that the calibrated model properly assesses all the variables and conditions, which can affect model results, and display the ability to forecast field observations different data set. The hydraulic model calibration parameters that are typically locate and adjusted include pipe roughness factors. The change in these parameters affect head losses, demands at node, pressure and residual; chlorine. The result shows that when the Hazen-Williams roughness coefficient increases the value of pressure increases and head losses decreases.

4.5.3.1Tank level time series calibration and validation

Two locations of data were collected for hydraulic model calibration and validation effort. The first location of data were used for model calibration purpose and detailed in Appendix J and L. While the second locations of data were used for model validation purpose and detailed in Appendix Mand N. Figure 4.5 and Figure4.6show plots of observed vs. computed values along with minimum and maximum difference between them.

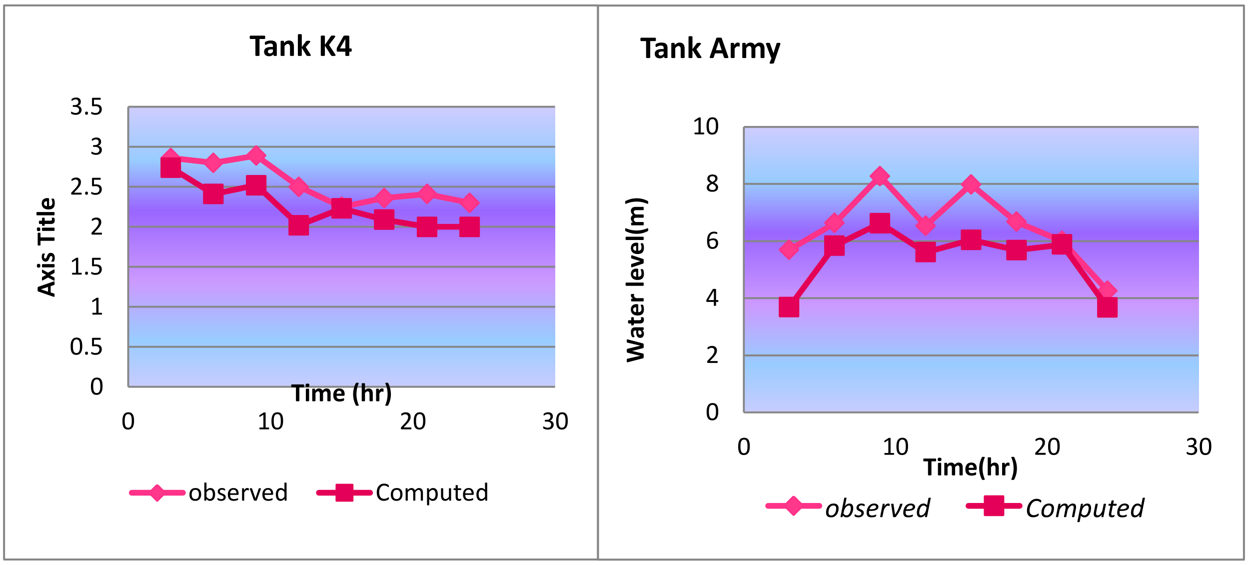

Figure 4- 5: Tank level Calibration using time series data

With regard to up to standard level of model calibration, there are different views. According to [8] the true test of model calibration is that the end user (for example, the pipe design engineer or chief system operator) of the model results considers comfortable using the model to support in decision making. To that end, calibration should be continued until the cost of performing additional calibration exceeds the value of the extra calibration work. Each application of a model is unique, and thus it is impossible to derive a single set of guidelines to evaluate calibration. The guidelines presented below give some numerical guidelines for calibration accuracy; however, they are in no way meant to be definitive [8].

- The model should accurately reproduce tank water level fluctuations to within 3–6 ft(1–2 m) for EPS runs and match treatment plant/pump station/ well flows to within10–20 percent.
- The model should reproduce the pattern of observed disinfectant concentrations over the time. Samples were taken to an average error of roughly 0.1 to 0.2 mg/l, depending on the complexity of the system accordingly, model performance level was evaluated based on the above guide lines and presented as follows.

From data sets collected for calibration effort (16 data sets from tanks water level, 83.68% &84.79 %of computed tank water level are found in the range of the guide line (1-2m).

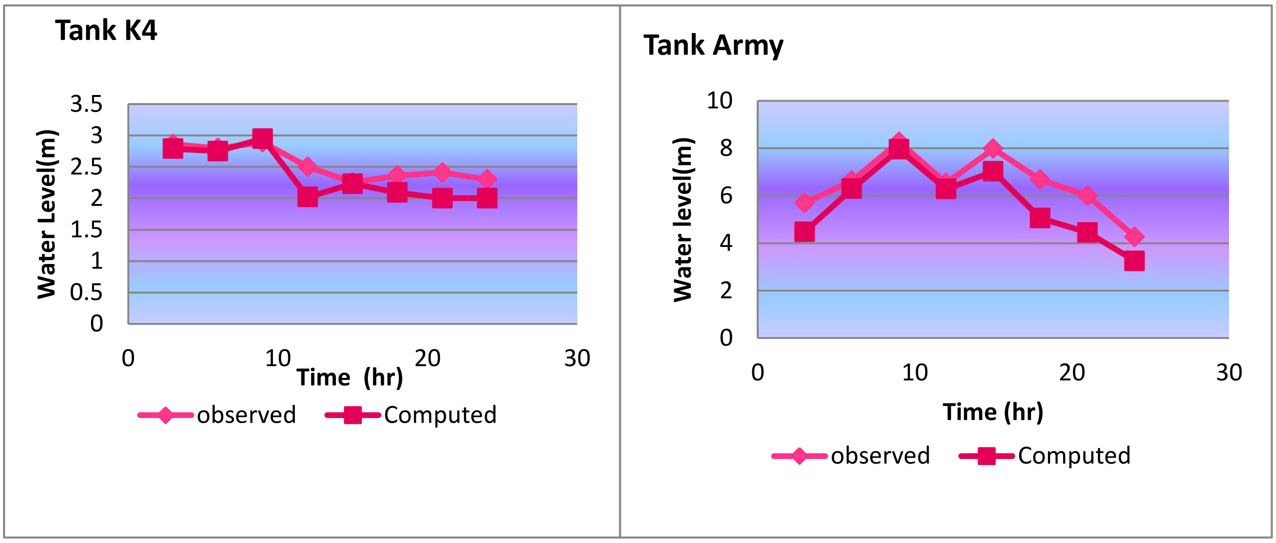

Figure 4- 6: Tank level Validation

From data sets collected for validation effort (16data sets from tanks water level), 90.95% &94.33 % of computed tank water level are in the range of theguide line (1-2m).

4.5.3.2Pressure calibration using time series along pipe networks

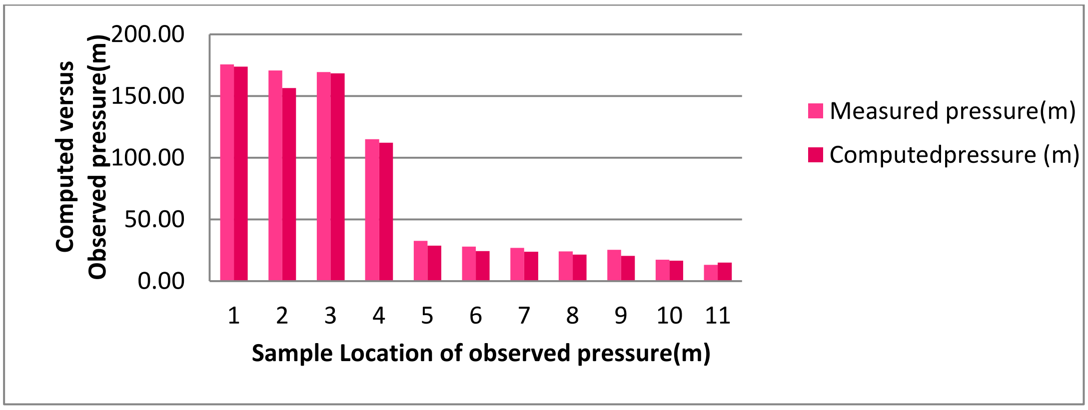

Figure 4- 7: Observed versus Compute Pressure during calibration

The calibration of pressures were completed both graphical and statistical method. Figure 4-7 shows the graphical representation of measured and computed pressure at different locations and time series.

Figure 4-8 shows that the statistical correlation plots of observed versus computed pressure during calibration process. The results show that R^2 value of 99.64 %. This implies that the computed pressures are within the acceptable limit recommended by [8].

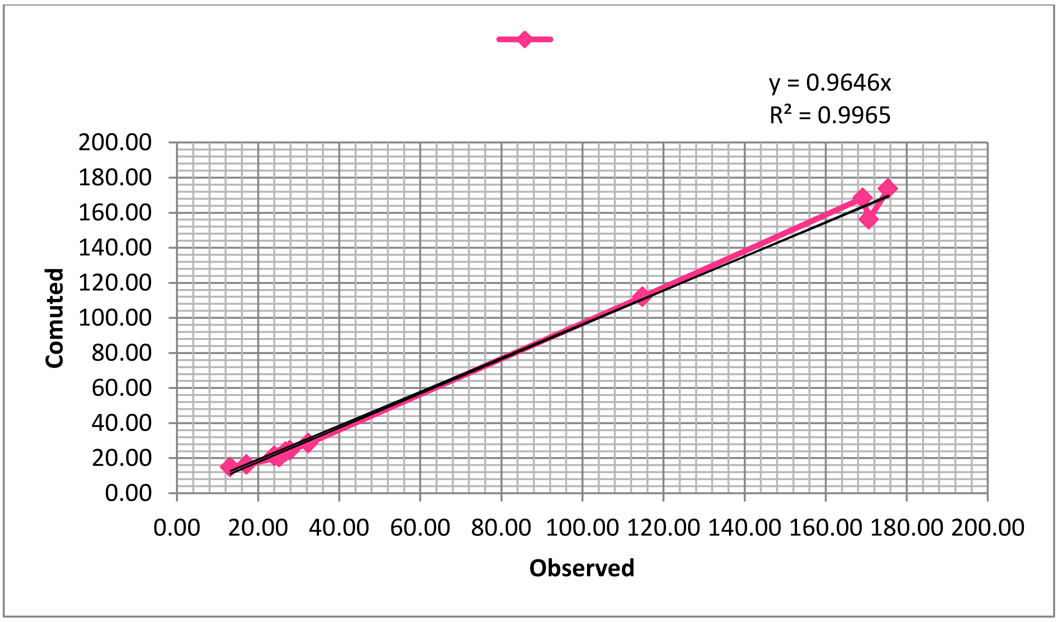

Figure 4- 8:Correlated plot of computed versus observed pressure during calibration

4.5.3.3Pressure validation using time series along pipe networks

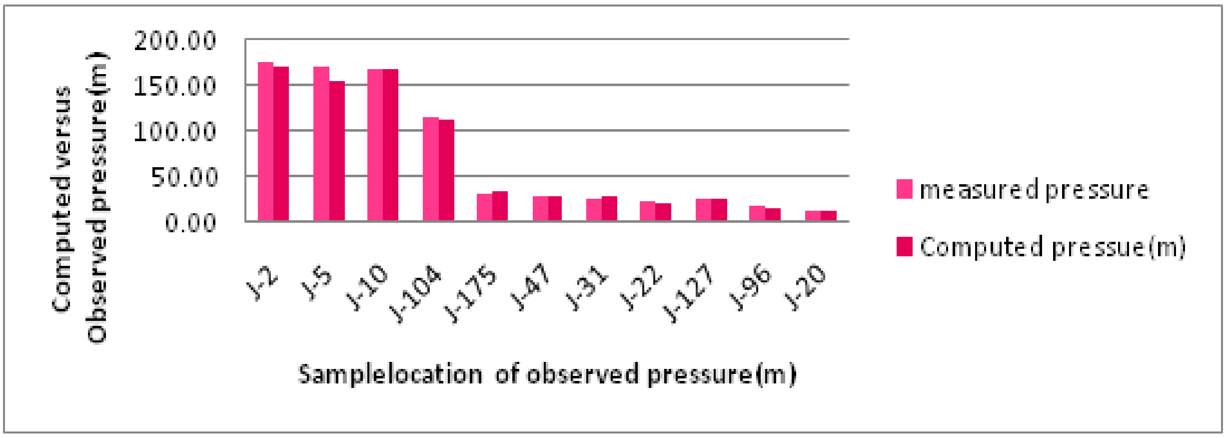

Figure 4- 9: Observed and Computed Pressure during validation

The validation of pressures were completed both graphical and statistical method. Figure 4-9 shows the graphical representation of measured and computed pressure at different locations and time series.

Figure 4-10 shows that the statistical correlations plot of observed versus computed pressure during calibration process. The results show that R^2 value of99.96 %. This implies that the computed pressures are within the acceptable limit recommended by [8].

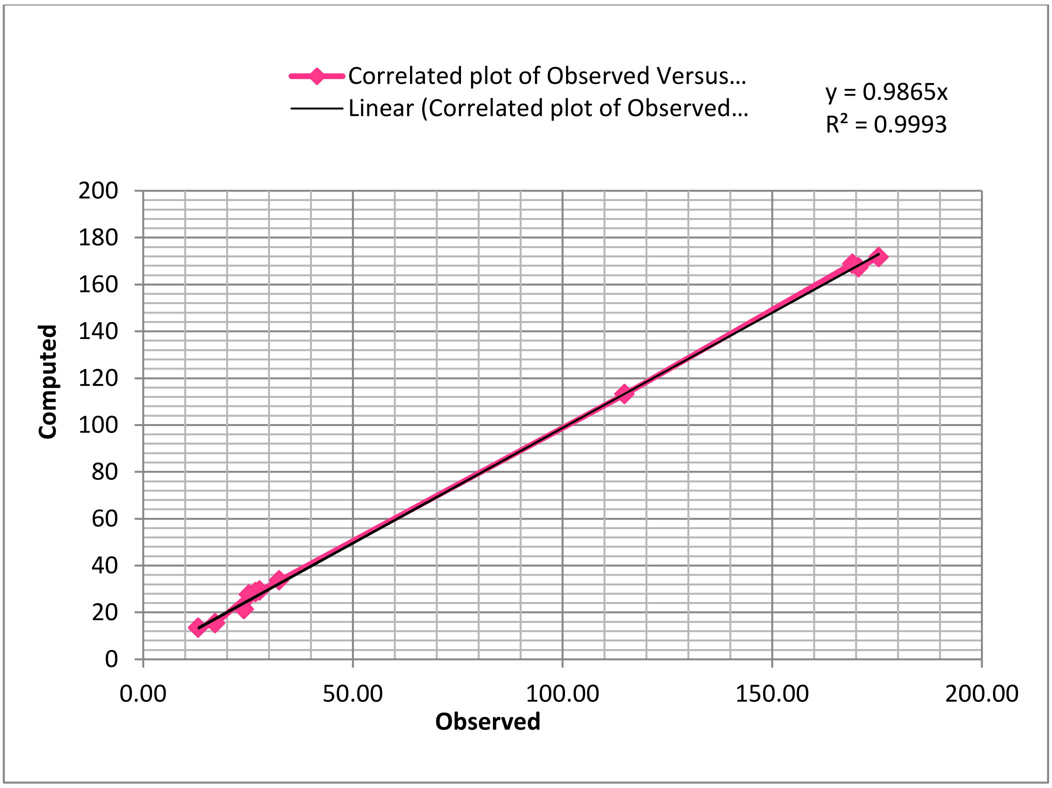

Figure 4- 10: correlated plot of observed versus computed pressure during validation

4.6 Water Quality Model Calibration

The sampling results were used to make available considerate of chlorine decay first order kinetics reaction of the Akaki Phase 3B water supply system from the bottle test data discussed previously to develop a calibrated chlorine model of the system. Therefore, water quality models that may possibly require certain degree of calibration use the following boundaries:

The following parameters are usedby water quality models that may require some degree of calibration:

- ✓ Initial Conditions: Defines the water quality parameter (concentration) at allLocations in the distribution system at the start of the simulation.
- ✓ Reaction Coefficients: Describes how water quality may vary over time due to chemical,
- ✓ Source Quality: Defines the water quality characteristics of the water source overthe time period being simulated.

4.6.1Residual Chlorine Calibration and Validation

Chlorine disinfectant models, the model should reproduce the pattern of observed disinfectant concentrations over the time when the samples were taken to an average error of roughly 0.1 to 0.2mg/l[8]. Consequential to hydraulic model calibration and validation, water quality model calibration has to be performed independently. To this effort data sets were collected from diverse part of water supply distribution networks. Table 4-1 and Table 4-2 depict an attempt to water quality model calibration as guide line recommends to have an average error of roughly 0.1 to 0.2 mg/l [8] The residual chlorine was calibrated by adjusting Hazen Williams's roughness coefficient. Initially the value of Hazen Williams's roughness coefficient in the system were 140 for PVC, 100 for galvanized iron pipe, and 110 for DCI pipe. The corresponding value of residual chlorine decreases in the pipe networks from distribution reservoir to point of use. This implies that the wall reactions materials of pipe increases.

During calibration Hazen Williams's roughness coefficient were adjusted to 150 for PVC pipes, 120 for GIP, and 130 for DCI. These results in slightly increase in the residual chlorine in the system. This implies that the wall material reaction decreases.

Table 4- 1: Summary of first data arrangement for residual chlorine calibration

Calibration statistics for residual chlorine`						Location of samples points		
S.no	Time (hour)	Nodes ID	Observe residual chlorine (mg/l)	Computed residual chlorine (mg/l)	Difference	x(m)	y(m)	elevation(m)
1	6:00	CT 2	0.85	0.8	0.05	473926.24	980811.74	2067
2	7:00	Hana	0.8	0.69	0.11	470621.33	985423.05	2221
3	8:00	Fana	0.71	0.64	0.07	465909.3	990865.67	2349
4	9:00	K4	0.42	0.54	-0.12	463,186.39	995,674.63	2500
5	10:00	Alb	0.36	0.42	-0.06	464,372.93	994,864.02	2400
6	11:00	Army	0.3	0.54	-0.24	469,606.60	996,326.28	2371
7	12:00	J-31	0.21	0.14	0.07	471865.38	995563.58	2337.6
8	13:00	J-15	0.42	0.31	0.11	463,935.49	995,397.23	2456.89
				RMS	0.112			

Table 4- 2: Summary of second data arrangement for residual validation

Validation Statistics for residual chlorine`						Location of samples points		
S.no	Time (hour)	Nodes ID	Observe residual chlorine (mg/l)	Computed residual chlorine (mg/l)	Difference	x(m)	y(m)	Elevation (m)
1	6:00	CT 2	0.85	0.8	0.05	473926.24	980811.74	2067
2	7:00	Hana	0.8	0.69	0.11	470621.33	985423.05	2221
3	8:00	Fana	0.71	0.55	0.16	465909.3	990865.67	2349
4	9:00	K4	0.42	0.62	-0.2	463186.39	995674.63	2500
5	10:00	Alb	0.36	0.3	0.06	464,372.93	994864.02	2400
6	11:00	Army	0.3	0.38	-0.08	469606.60	996326.28	2371
7	12:00	J-31	0.21	0.14	0.07	471865.38	995563.58	2337.56
8	13:00	J-15	0.42	0.34	0.08	463935.49	995397.23	2456.89
				RMS	0.102			

Table 4-1 and Table 4-2 show that the calibration and validation values of residual chlorine in the truck mains are respectively an average RMS error of 0.112 mg/l and 0.102mg/l.

4.7. Simulation Results

Simulation can be used to predict system responses to under a wide range of conditions without disrupting the actual system, and solutions can be evaluated before time, money and materials are invested in a real-world project.

There are two most basic types of simulations that a model may perform, depending on what the modeler is trying to observe or predict. These are:

- Steady state simulation.
- Extended period simulation (EPS).

Single period and extended period simulation were subsequently executed. It was needed to runSingle period simulation at the beginning of the simulation as to observe the model under snap shot situation. In line with this, running single period simulation was helpful while performingpreliminary model calibration. However, it should not be used for network evaluation as waterdistribution system is likely to experience variations. Hence, only extended period simulation wasexclusively used for entire model calibration and model evaluation effort. Demand patterns used inSimulating extended period simulation is presented in (Appendix A).An extendedperiod simulation can be run for any length of time, depending on the purpose of the analysis. The most common simulation duration is typically a multiple of 24 hours, because the most recognizable pattern for demands and operations is a daily one.

When modeling emergencies or disruptions that occur over the short term, however, it may be desirable to model only a few hours into the future to predict immediate changes in tank level and system pressures.

For water quality applications, it may be more appropriate to model duration of several days in order for quality levels to stabilize. Even with established daily patterns, simulation duration of a week or more.

4.7.1 Residual chlorine

Chlorine is a chemical that is used to disinfect water prior to it being discharged into the distribution system. It is used to ensure the water quality is maintained from the water source to the point of consumption. When chlorine is supply into the water, it reacts with any iron, manganese, or hydrogen sulphide that may be present. If there is any chlorine residual left, it will then react with organic materials, including bacteria. In order to ensure that water is sufficiently treated through the whole distribution system, an excess of chlorine is usually added. This amount is usually attuned to make sure there is enough chlorine presented to completely react with all organics present. The chlorine will decrease in concentration with distance from the source, until it reaches the point where the chlorine level can become ineffective as a disinfectant. Bacterial growth will occur in distribution systems when very low levels of chlorine are encountered. Therefore, it is important to make sure there is enough chlorine to well disinfect even at the far ends of the distribution system. Chlorination can kill many pathogenic, disease-causing microorganisms such as E-coli, but others, like Cryptosporidium and Giardia, are very resistant to chlorine and require other measures to properly remove them.

The World Health Organization guidance level for drinking water supply recommends a minimum free Chlorine residual of 0.2mg/L and maximum residual chlorine 0.5mg/L [13] in the distribution systems of any water supply.

Studies have shown that when residual chlorine levels drop under recommendations, several water quality problems can occur. With regard to public health, bacteria and selected viruses called bacteriophage are able to multiply in water that was not properly disinfected. Moreover, depending on the species, could potentially cause waterborne diseases. It is important to note that, although chlorination has been the most common method of disinfection for over many years. While recommendations only state minimum residual chlorine levels, it is important that a careful balance be maintained in drinking water. There needs to be enough chlorine to make sure everything is properly disinfected. However, as shown the laboratory and model simulation results of the study area were high loss of chlorine in the system.

Water quality simulation requires a series of runs to understand the movement of water and water quality transformation in the system. Specific simulations included in this study are residual chlorine modeling. There is a variation of water quality in distribution system from hour to hour of a particular day. This hourly variation of water quality is mainly related to demand patterns.

4.7.2. System Pressure

Water distribution networks must maintain adequate water pressure throughout the network to ensure continuity in service and for fire suppression. Low water pressure can result in flow reductions and high water pressure can cause leaks and damage to system components.

Water pressures vary by communities; however, water pressure is typically maintained between 25 and 75 psi. Pressure is controlled in the network by pumps and the elevation of reservoirs and tanks. Aside from pipe leaks and water consumption, pressure is lost in the system due to pipe friction.

Pressure in water distribution system has to be maintained optimum; as to efficiently make waterAvailable to each demand category including high withdrawal period and as to reduce leakage as well as pipe breakage across the system. The former one is frequentlyachieved in setting minimum pressure to be maintained at each junction. The later one is achieved differently in setting maximum allowable pressure to be maintained in the system.

According to [12] the minimum design nodal pressures are prescribed to discharge design flows onto the properties. It is based on population served, types of dwellings inthe area, and firefighting requirements. The general consideration is that the water should reach up to the upper stories of low-rise buildings in sufficient quality and pressure, considering firefightingrequirements. In case of high-rise buildings, booster pumps are installed in the water supply systemto cater for the pressure head requirements. With these considerations, various codes recommendminimum ranging from 8 m to 20 m for residential areas.

Similarly, [12] recommend;

1. Minimum pressures at peak hour demand: sufficient to serve the highest supply point in thenetwork. Typically a mains pressure of not less than 15 to 20 m would be required to servebuildings up to three storeys high. Higher pressures may be necessary in some areas wherethere are significant numbers of dwellings exceeding three-storey height; but high risebuildings are normally required to have their own boosted supply.

2. Maximum static pressures during low demand periods: typically at night, should be as low asPracticable to minimize leakage. For flat areas a maximum static pressure in the range 30 to45 m is desirable.

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 flow is summarized in Table 4.3 and detailed in AppendixB.

Table 4- 3: Distribution of pressure at peak hour flow

Pressure(m)	Nodes (number)	Percentage (%)
>70	23	10.75
60—70	17	7.95
50---60	7	3.27
40---50	14	6.54
30---40	41	19.16
20---30	65	30.37
15—20	19	8.88
<15	28	13.08

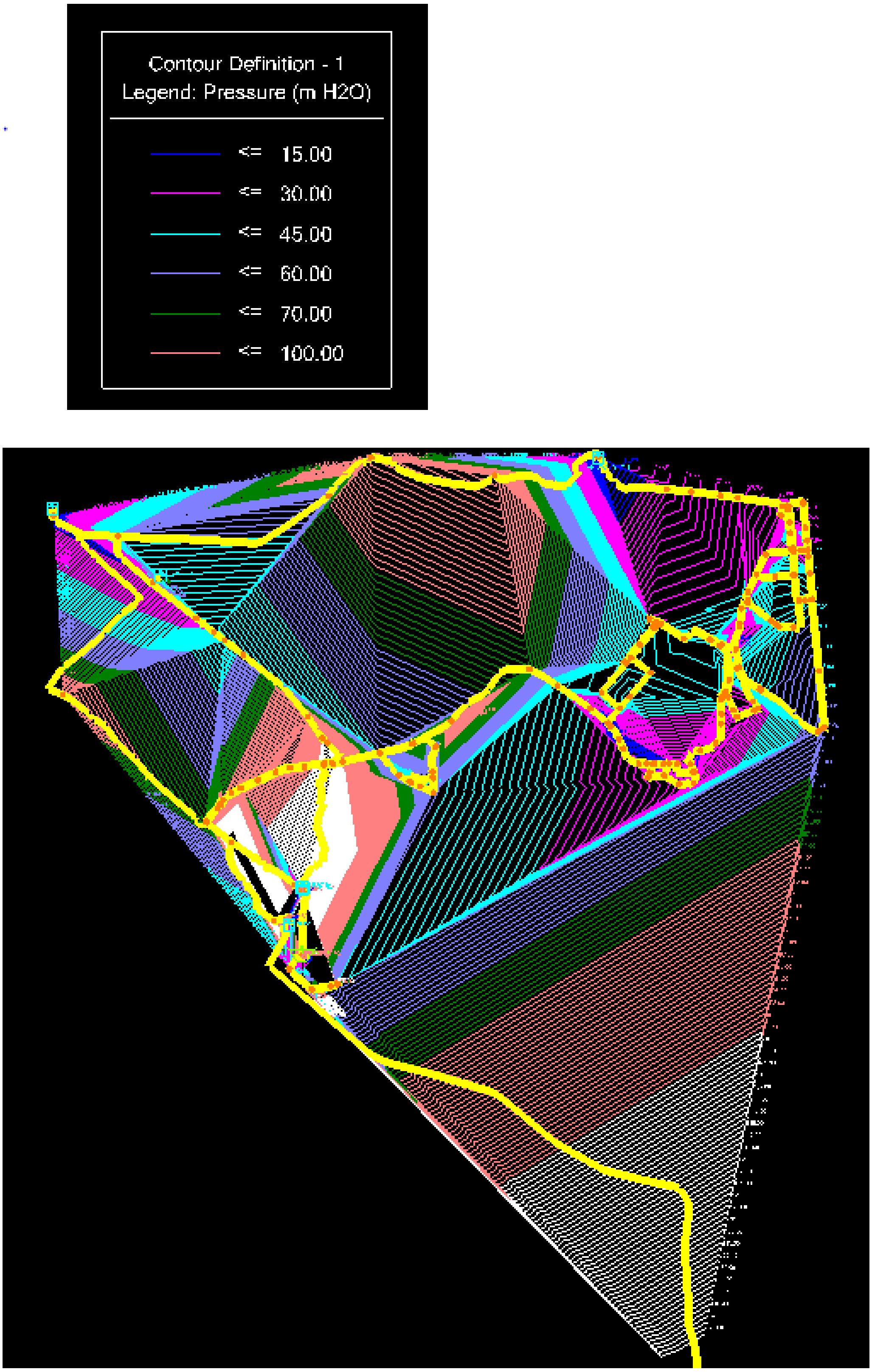

Figure 4- 11: Distribution of pressure at peak hour flow

As indicated in the above table 4.3, 13.08 % of nodes are failed to satisfy minimumpressure requirement during peak hour flow. And 10.75 % of nodes exceed maximum acceptable pressure of 70m. While 76.17 % of nodes are in the permissible pressure rangeof minimum 15m and maximum 70m. On the opposing minimum pressures are alsoobserved mainly nodes situated near to tanks. In few instances nodes positioned in flatarea of network are susceptible to low pressure. Whereas majority of nodes located inrelative perfect loop region receive optimum pressure which doesn’t violate minimum ormaximum allowable pressure range (Appendix-C).

Table 4- 4: Pressure distribution during low flow

Pressure(m)	Nodes (number)	Percentage
>70	38	17.75
60—70	9	4.21
50---60	6	2.8
40---50	25	11.68
30---40	42	19.63
20---30	58	27.1
15—20	11	5.14
<15	25	11.68

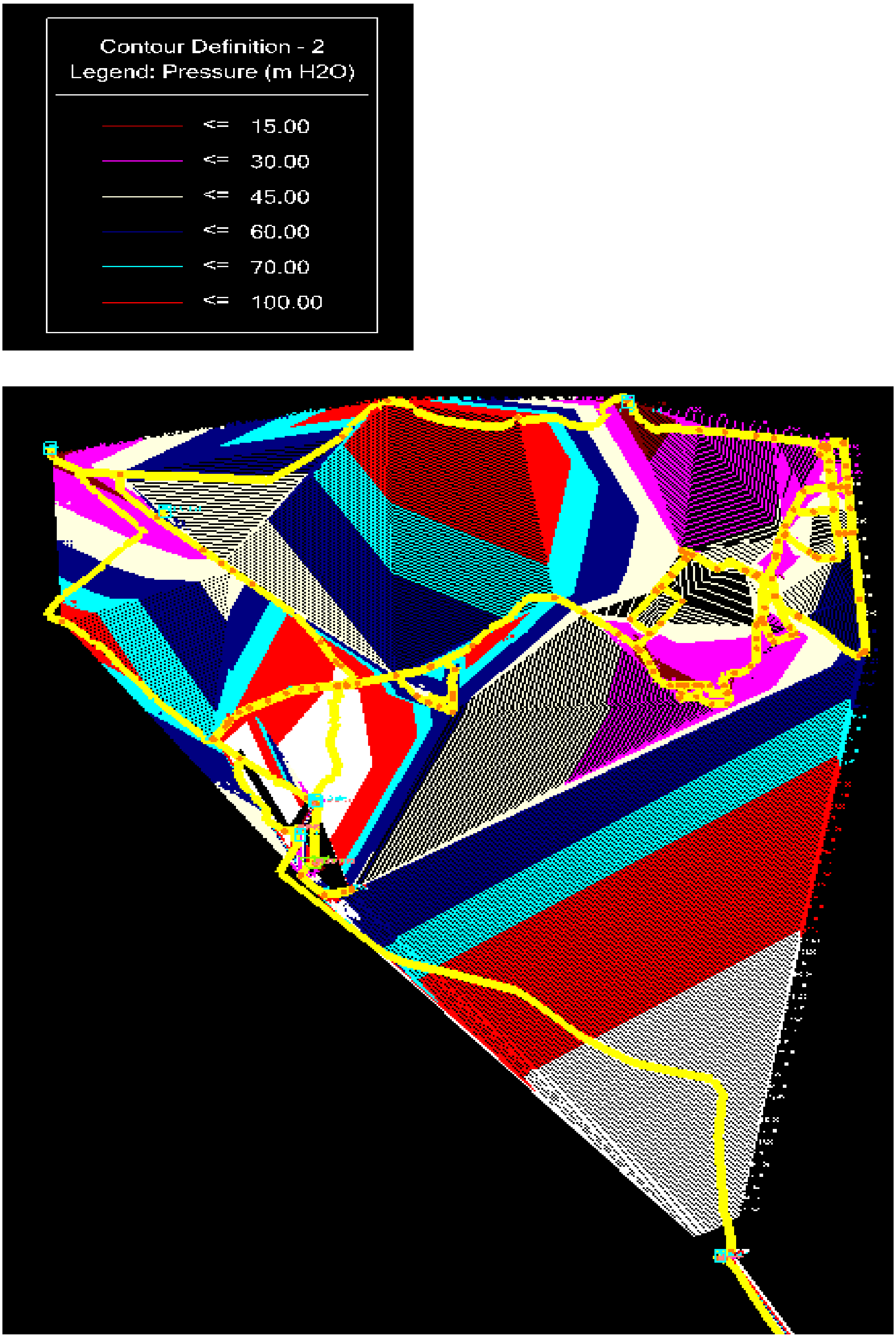

Figure 4- 12 Pressure distribution during low flow

During low flow typically at mid-night distribution system of case study is noticeable by excessivepressure. As portrayed in table 4.4 and detailed in appendixC. 17.75 % of nodes are liable to extremelyhigh pressure. This figure is relatively high.Minimum pressures are also observed during low consumption period. 11.68 % of nodes are received water with low pressure. Only 70. 56% of nodes are obtainingwater of optimum pressure.

While 29.44 %, of nodes are failed to satisfy allowable pressure range.For city of Addis Ababa, AWWSA is using an operating pressure which ranges from 60m to 80m.

However, there was no defined maximum and minimum pressure ranges set by the office. Therefore, literature based recommendation for optimum operating pressure was used to assess system hydraulic performance.The result of pressure changes among others can be water aging decrease in chlorine content and hydraulic factors, because the systems may be broad and thus hold complex networks.

4.7.3 Establishing a New Pressure Zone

The decision to create a new pressure zone may be triggered by

- Construction of a new isolated system
- Customers moving into an area with an elevation that is too high or too low to beadequately served from the existing pressure zone
- The utility wanting better control over an area

Choosing the boundaries for the pressure zone is done manually before beginning tomodel the system. When laying out pressure zones, the elevation]ns of the highest andlowest customers to be served are observed. If customers are less than Approximately 37m apart vertically, then most likely a single pressure zone can serve them. If the elevationdifference is significantly greater, more pressure zones are needed. In general, theelevation of the lowest and highest customers in the service area and the limits of therange of acceptable pressures are used to determine the HGL in a pressure zone.

Equations 4.1 and 4.2provide some useful guidelines for Selecting a HGL [27].

$HGLmin > (Elevation\ of\ highest\ customer) + Cf\ Pmin$............................4.1

$HGLmax < (Elevation\ of\ lowest\ customer) + Cf\ Pmax$............................4.2

Where ;HGLmin = minimum HGL (m)

HGLmax = maximum HGL (m

Pmin = minimum acceptable pressure (kPa)

Pmax = maximum acceptable pressure (kPa)

Cf = unit conversion factor (2.31 English, 0.102 SI)

The first criterion (Equation 4.1) ensures that the highest customer will have at least minimum pressure, while the second (Equation 4.2) ensures that the lowest customer will not experience excessive pressures. In flat terrain, there will usually be a band of possible HGL values that meet both criteria. In hilly terrain, however, because the elevations of the highest and lowest customers are very different, it may be impossible to find an HGL that satisfies both inequalities. Usually, this much difference means that the proposed pressure zone should actually be two (or more) pressure zones, or the lowest customers will have pressures in excess of *P max*.

4.8. Relationship between the Rate of Residual Chlorine and Pressure

The residual chlorine concentration in the distribution system decreases from the distribution reservoir to dead end of the pipe network. Hydraulic changes were causing others for chlorine consumption effectively. Figures 4.13 shows relationship between pressure changes and residual chlorine. Hydraulic changes (pressure) have a direct relationship with rate of the residual chlorine. Based on the analysis of the water distribution system of Akaki phase 3B as shown in result figures 4.13Appendix I the systems have large transmission line.

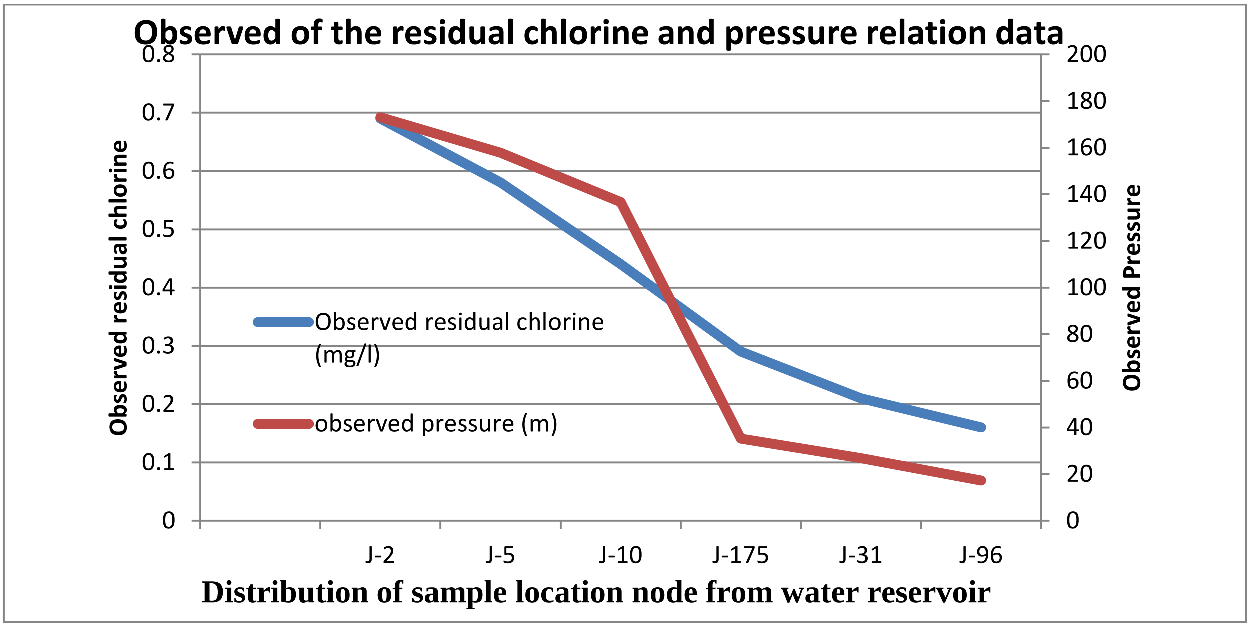

Figure 4.13: The relationship between observed residual chlorine and pressure of Akaki phase 3B, Addis Ababa

The distance travelled and residence times have been causing to increase chlorine consumption and hydraulic change (pressure) in water distribution system in Akaki Phase 3B; In Figure 4-13 indicates the below and above the permissible limit for both residual chlorine and pressure. Pressure has a direct relationship with residual chlorine. Hydraulic change (pressure) has been causing to increase chlorine consumption more.

4.9 Effects of pressure changes on water quality in distribution system

Flow rate variation happens between water reservoir and dead ends in distribution system. The major cause for the dissimilarity of the flow rate is the change of pressure along the distribution system that may be caused by the extent of the transmission line within the whole water supply network. Thus the pressure on the downstream distribution system will be lesser than the upstream water pressure. High residence time and high or low pressure along the distribution system will negatively affects the quality of water within the supply network.

In the cases of the transmission system lines with big transmission system line,such as the water network system in Akaki Phase 3B water is discharged with high Pressure however the long pathway within the network results a drop in water pressure and causes a high residence time in the system. Due to the long distance in the system of Akaki Phase 3B, it might be impossible to maintain the high pressure. Correspondingly low pressure could cause a high retention time in the system that may deteriorate quality of water in the distribution systems. The relationship between water pressure and residual chlorine is presented above in Figure 4-13, the results indicates that water pressure and the concentration of residual chlorine are directly related.

Changes in quantity of water also cause pressure fluctuations and will reduce the flow rate (speed) of water. The long transmission line, wide network and reduced flow rate consequences deterioration of water quality within the distribution system due to high water age. The increase in water age within the system reduces the residual chlorine concentration. The residual chlorine may not be well enough to effectively disinfect over the whole required area of coverage and Chlorine consumption will be high. The results of this indicate that decreasing the residual chlorine when pressure was decreasing. A similar finding was found in a study done in west of Ahwaz in Iran, which showed a direct relationship between pressure and residual chlorine. This might be due to big transmission line of the system which may cause problem on changes of pressure in the distribution system. Change in pressure and lack of speed in the transmission network could be responsible for high residence time. Which inturn decreases the level of residual chlorine (17).

CHAPTER FIVE

5. CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

The Effect of Change of Pressure and water quality on Residual Chlorine in the water supply system of Akaki phase 3B from point of Reservoir to point of distribution investigated through Experiment and Simulation Using Water Cad Software.

Pressure based hydraulic performance evaluation indicated that acceptable minimum and maximum pressures have not been met. During peak hour flow, parts of the distribution system take delivery of water with low pressure and under some circumstances danger of obtaining no water is observed because of the pressure in the distribution system is below the permissible minimum requirement. In line with this, about one tenth of the distribution system is prone to undesirable pressures which exceed maximum allowable pressure.

Along with this, hydraulic modeling results exposed the existence of operational Problems. Observed pressures which exceed maximum allowable pressure even during peak hour flow and observed pressures which is lower than minimum allowable pressure during low flow hours clearly proved the existence of problems. While generated excessive negative pressures clearly proved the existence of operational problems.

In General the simulated hydraulic result indicated that the current hydraulic performance of Akaki phase 3B supply system is not satisfactory.

Disinfection modeling result showed that the distribution system be deficient in the ability to fully distribute microbiologically safe water due to absence of minimum allowable residual chlorine. Based on simulation result, part of distribution system is liable to risk of health problem because of the absence of residual chlorine in water at least once per day.

The simulation results from Water Cad Software have indicated that the effect of pressure on residual chlorine concentration in the water supply networks systems is directly related with the distribution system of the networks.

So, the systems would have low pressure in the downstream distribution system. High residence time, high pressure and low pressure systems and distribution network will cause a decline in the water quality in the distribution system and the distance travelled and residence times have been causing to increase bacterial growth, chlorine consumption and pressure change in water distribution system in Akaki phase 3B.Hence pressure changes could affect of water quality in the water distribution system.

5.2. Recommendations

To improve the current situation of Akaki Phase 3B water supply system, both design and operational modifications are necessary. From the study undertaken and modeling end result the following sets of recommendations are drawn:

1. To permanently modify the hydraulic performance of the sub system, the design needs to be reviewed and pressure zones which serve customers situated in nearly equivalent elevation has to be established. If customers are less than approximately (37 m) [3] apart vertically, then most likely a single pressure zone can serve them. If the elevation difference is significantly greater, more pressure zones are needed.
2. Uses of pressure sustaining valves are recommended as to control the occurrences of Minimum pressures. These valves start closing if the upstream pressure falls below the present value as to guarantee allowable minimum pressure for isolated parts of network.
3. Modification in current flushing program which only targets storage tank cleaning per year is necessary. Periodic flushing of system elements associated with long water age may also minimize water quality degradation by removal of pipe scales and sediment associated with disinfectant consumption.
4. Installing new pipes as to eliminating dead-ends and letting water to route in the system is crucial. Such an effort donates reduction in water age. An attempt to eliminating dead ends of case study by installing new pipes showed significant reduction of water age.
5. Booster disinfection station has to be established to maintain life guarantee minimum residual chlorine across the distribution system. If implementation of booster disinfection is not possible, local disinfection mechanisms at home level such as use of household chemicals prior to consumption has to be promoted particularly in the rainy season.
6. To make sure minimum residual chlorine concentration regularly at the customers that are located at the furthest place from the treatment plant or distribution of service reservoir.

REFERRENCES

1. Central Statistical Agency of Ethiopia (2008). *Summary and Statistical Report of the 2007 Population and Housing Census*, Population size by Age and Sex, Addis Ababa, Ethiopia,United Nations Population Fund.
2. Addis Ababa Water and Sewerage Authority (2001). Water for Life, Special Edition Vol. II No.1, Addis Ababa, Ethiopia.
3. Addis Ababa Water and Sewerage Authority (2012). AAWSA a brief profile, Addis Ababa, Ethiopia.
4. K. Michael Johnson, Don D. Ratnayaka, Malcom J. Brandt. (2009). Twort's Water Supply,, 6th edition, Oxford, UK, Elsevier Ltd.
5. Boulos, P. F., Lansey, K. E., and Karney, B. W. (2004).Comprehensive water distribution systems analysis handbook for engineers and planners. MWH Soft, Inc.,Pasadena California.
6. Swamee, Probate K. (Prabhata Kumar), 1940, Design of water supply pipe networks /Prabhata K. Swamee, Ashok K. Sharma. Water distribution systems handbook/Larry W. Mays, ed.
7. Munavalli, G. R., and Mohan Kumar, M. S. (2003). “Water quality parameter estimation in a steady state distribution system.” Journal of Water Resources Planning and Management, ASCE, 129(2), 124-134.
8. Thomas M. Walski, Donald V. Chase, Dragan A. Savic, Walter Grayman, Stephen Beckwith, Edmundo Koelle (2003), Advanced Water Distribution Modeling and Management, U.S., Haestad press.
9. Rossman, L.A., Clark, R.M., and Grayman, W.M. (1994). “Modeling lorine iduals in drinking-water distribution systems”, Jour. Env. Eng., Vol. 120, No. 4, 803-820.

10. SHAMSAEI H. ET AL. (2013). Disadvantage Pressure changes on the Decline of Water Quality in Water Distribution Systems. Scientific Research, 97-105.
11. Vasconcelos, J. J., Rossman, L. A., Grayman, W. M., Boulos, P. F., and Clark, R. M.(1996). Characterization and Modeling of Chlorine Decay in Distribution Systems. AWWA Research Foundation, Denver, Colorado.

12. Swamee, Probate K. (Prabhata Kumar), 1940, Design of water supply pipe networks /Prabhata K. Swamee, Ashok K. Sharma. Water distribution systems handbook/Larry W. Mays, ed.
13. www.safe water.0rg_PDFS_communitywatertestkit. (n.d.). Safe Drink Water Foundation. Published.
14. Jeffrey A. Gilbert, P.E. (2012) “Practical Design of Water Distribution Systems”. www.PDHonline.org.
15. Bentley Water CAD/GEMs (2008). “Water Distribution Design and Modeling, Full Version V8i”.

16. Thomas M. Walski, Donald V. Chase, Dragan A. Savic, Walter Grayman, Stephen Beckwith, EdmundoKoelle, *Managing Editor:* Adam Strafaci, 2003, HAESTAD METHODS Advanced Waterdistribution Modeling And Management - Haestad Methods.
17. Shamsaei H.*, Jaafar O., Ahmad Basri N. (2012) ,Disadvantage Pressure Changes on the Decline of Water Quality in Water Distribution Systems.

APPENDICES

Appendix A: Demand pattern

Time period	Multiplier
1	0.9
2	0.9
3	1.2
4	1.2
5	1.3
6	1.4
7	1.42
8	1.48
9	1.45
10	1.45
11	1.4
12	1.36
13	1.1
14	1.1
15	0.75
16	0.75
17	0.55
18	0.55
19	0.45
20	0.45
21	0.47
22	0.57
23	0.9
24	0.9

Appendix B: Pressure at low flow

Node	X (m)	Y (m)	Pressure (m H2O)
J-2	473,926.47	980,811.63	172.77
J-1	473,899.21	980,712.02	4.78
J-4	470,674.92	985,438.11	6.73
J-3	470,694.32	985,417.02	7.09
J-5	470,621.33	985,423.05	157.11
J-6	470,767.34	985,510.18	155.95
J-156	469,977.46	993,503.08	35.59
J-155	470,221.84	993,787.99	32.64
J-153	469,939.11	993,902.48	32.61
J-154	469,641.25	993,551.97	32.44
J-157	469,858.97	993,366.31	34.46
J-152	469,917.75	993,919.89	31.91
J-146	469,990.52	993,965.13	31.88
J-142	470,610.16	994,188.45	35.31
J-150	470,251.48	994,327.39	30.08
J-144	470,273.74	994,310.34	33.37
J-174	470,987.17	992,956.35	26.25
J-148	470,207.79	994,418.48	29.43
J-151	470,087.43	994,123.99	31.34
J-145	470,105.61	994,106.93	31.90
J-147	470,211.49	994,427.35	29.37
J-143	470,339.42	994,362.97	32.76
J-149	470,278.79	994,385.48	28.99
J-213	466,523.67	990,108.88	45.88
J-138	471,058.11	994,123.90	37.13
J-139	471,037.74	994,105.64	36.73
J-137	471,046.21	994,101.16	36.76
J-140	470,931.51	994,167.61	35.74
J-175	471,049.96	993,183.01	28.53
J-141	470,915.54	994,177.98	35.51
J-135	471,052.78	993,471.14	32.90
J-176	471,048.22	993,298.79	30.91
J-136	471,055.42	993,493.57	32.84

J-212	466,522.09	990,119.62	44.61
J-134	471,053.81	993,299.47	32.05
J-211	466,439.68	990,087.48	44.37
J-210	466,217.68	990,048.33	42.99
J-170	470,727.03	992,480.64	16.97
J-169	470,553.99	992,469.07	18.94
J-166	470,379.76	992,569.74	24.36
J-133	470,733.90	992,616.45	25.66
J-171	470,733.62	992,607.90	13.62
J-172	470,689.13	992,709.85	13.45
J-132	470,713.49	992,611.94	24.58
J-173	470,802.95	992,867.39	13.35
J-167	470,190.46	992,548.91	21.12
J-123	467,870.85	993,218.09	84.85
J-131	470,532.23	992,711.01	15.99
J-168	470,210.54	992,608.80	14.43
J-87	471,197.32	993,343.48	26.27
J-81	471,424.34	993,445.40	27.88
J-88	471,131.05	993,499.85	27.04
J-91	471,145.51	993,700.66	30.61
J-65	472,197.19	993,092.60	52.15
J-82	471,492.24	993,366.63	24.72
J-89	471,102.54	993,552.00	27.30
J-90	471,108.33	993,562.51	27.47
J-83	471,441.33	993,342.63	24.28
J-80	471,239.74	993,842.06	36.79
J-69	471,259.89	993,850.11	37.01
J-68	471,266.67	993,852.83	37.15
J-66	471,524.34	993,580.17	43.67
J-84	471,247.72	993,267.34	24.05
J-209	465,952.94	990,287.70	26.65
J-85	471,242.66	993,265.68	23.97
J-86	471,238.02	993,263.42	23.98
J-66	471,505.19	993,591.54	42.77
J-67	471,508.03	993,595.35	42.70

J-208	465,958.01	990,294.12	26.14
J-165	470,433.59	992,710.13	11.59
J-64	472,260.06	993,140.25	49.45
J-130	470,369.23	992,713.28	10.90
J-63	472,274.29	993,140.32	48.74
J-162	470,081.47	992,714.09	11.34
J-163	470,175.17	992,711.74	9.49
J-164	470,243.99	992,710.44	9.01
J-129	470,265.81	992,712.88	9.74
J-116	467,649.89	992,612.11	53.37
J-62	472,206.85	993,809.77	50.82
J-117	467,652.16	992,554.54	49.67
J-118	467,652.76	992,540.34	49.02
J-161	469,897.29	992,888.89	6.85
J-124	468,785.77	993,827.94	65.71
J-128	469,900.91	992,890.35	6.90
J-74	471,292.63	994,545.53	34.71
J-125	468,795.71	993,826.42	65.30
J-78	471,733.08	994,333.50	42.45
J-77	471,701.39	994,356.84	42.38
J-119	467,655.16	992,369.71	46.82
J-76	471,638.93	994,403.42	41.67
J-79	471,906.80	994,322.00	44.77
J-115	467,683.79	992,906.45	53.08
J-160	469,848.51	992,999.25	1.99
J-114	467,685.90	992,914.20	53.34
J-120	467,500.45	992,442.75	42.02
J-75	471,362.78	994,616.23	31.44
J-182	471,371.95	994,640.50	30.53
J-121	467,392.95	992,488.62	40.07
J-122	467,370.49	992,500.34	39.81
J-159	469,751.34	993,151.28	-4.54
J-207	466,126.40	990,440.35	1.74
J-113	467,419.41	992,952.25	68.28
J-111	467,404.36	992,943.16	69.33

J-110	467,399.40	992,946.59	70.56
J-73	471,274.07	994,370.64	20.05
J-158	469,714.76	993,189.32	-5.81
J-127	469,508.87	993,362.38	20.25
J-205	466,124.92	991,192.88	140.08
J-206	466,108.86	990,460.85	142.18
J-72	471,272.43	994,361.81	19.27
J-70	471,225.58	994,061.28	11.46
J-123	467,251.81	992,575.56	38.71
J-71	471,267.49	994,337.29	17.41
J-181	471,369.32	994,739.57	26.48
J-60	472,132.52	994,649.23	43.80
J-61	472,116.92	994,649.27	43.3
J-59	472,132.56	994,655.26	38.72
J-126	469,123.12	993,630.26	35.23
J-58	472,039.30	994,644.31	39.04
J-179	471,688.68	994,920.89	38.15
J-57	471,972.36	994,639.35	40.77
J-56	471,971.71	994,652.33	40.20
J-112	467,019.43	992,836.23	45.82
J-202	466,359.53	992,710.82	99.83
J-201	466,131.70	992,683.76	82.48
J-109	466,585.72	992,731.89	102.24
J-178	471,903.16	994,943.45	37.60
J-107	466,455.88	992,725.65	108.89
J-204	466,132.51	992,674.29	81.68
J-108	466,466.58	992,705.51	108.48
J-55	471,915.70	994,943.35	37.23
J-105	466,363.07	992,714.06	111.83
J-54	471,915.30	994,949.35	37.11
J-104	466,371.57	992,717.39	111.62
J-106	466,369.02	992,694.88	110.14
J-197	465,387.03	992,424.33	76.16
J-196	465,380.19	992,421.19	75.39
J-18	468,379.14	996,117.71	135.98

J-180	471,467.25	994,944.95	20.29
J-53	472,100.62	994,964.10	35.66
J-188	464,984.60	992,019.11	75.70
J-189	465,129.53	992,229.95	74.54
J-191	465,144.07	992,224.23	74.46
J-190	465,134.17	992,228.00	74.37
J-195	465,175.23	992,282.62	71.82
J-200	465,961.47	992,664.82	72.35
J-192	465,111.10	992,183.68	71.11
J-198	465,586.55	992,514.68	70.47
J-194	465,584.66	992,518.07	70.38
J-193	465,267.98	992,371.33	69.53
J-103	466,403.23	992,959.43	104.82
J-177	471,574.42	995,185.29	22.08
J-199	465,792.40	992,611.39	68.03
J-35	471,864.84	995,437.67	35.94
J-33	471,883.84	995,235.04	29.74
J-36	471,899.07	995,224.12	29.27
J-102	466,347.62	993,011.33	102.30
J-38	471,907.41	995,218.33	29.13
J-37	471,906.83	995,225.07	28.75
J-32	471,897.87	995,236.43	28.89
J-39	471,906.11	995,233.12	28.59
J-34	471,883.06	995,255.35	28.81
J-31	471,895.75	995,257.38	28.64
J-203	464,969.85	992,030.36	65.35
J-48	471,987.85	995,229.92	27.17
J-52	472,082.39	995,158.13	25.89
J-49	472,056.30	995,231.12	23.68
J-47	472,048.07	995,500.01	24.85
J-45	472,027.38	995,787.58	26.65
J-44	472,019.29	995,788.18	26.65
J-43	472,021.49	995,809.34	26.66
J-46	472,027.10	995,797.39	26.40
J-50	472,075.65	995,237.93	22.35

J-51	472,075.84	995,232.42	22.33
J-42	471,854.76	995,752.21	25.26
J-41	471,845.45	995,749.77	24.93
J-40	471,848.24	995,738.01	25.04
J-31	471,865.38	995,563.58	24.78
J-48	472,063.51	995,348.95	19.39
J-101	466,173.35	993,267.92	90.70
J-30	471,839.38	995,712.41	22.74
J-24	471,242.73	995,856.34	28.64
J-9	465,941.84	990,889.07	10.05
J-12	465,271.09	991,780.59	169.76
J-29	471,843.83	995,712.99	19.64
J-8	465,946.31	990,881.73	8.07
J-23	471,044.13	995,881.44	24.94
J-11	465,772.18	990,857.45	169.17
J-7	465,917.59	990,863.42	7.13
J-27	471,752.39	995,645.60	22.81
J-26	471,763.48	995,643.13	22.65
J-28	471,832.09	995,756.27	19.39
J-25	471,773.29	995,752.98	21.92
J-10	465,909.30	990,865.67	167.34
J-22	470,699.80	995,926.73	19.65
J-21	470,410.70	995,954.59	19.98
J-17	466,923.30	996,331.82	115.46
J-185	466,022.75	991,281.20	77.00
J-20	469,607.18	996,339.87	13.52
J-19	469,598.73	996,338.70	66.98
J-187	466,064.02	991,231.49	65.16
J-100	465,409.19	993,996.83	51.12
J-186	466,112.21	991,211.99	40.33
J-16	466,421.88	995,876.48	75.26
J-99	465,190.71	994,168.64	37.27
J-98	465,070.14	994,264.91	30.69
J-184	466,078.14	991,254.19	20.83
J-97	464,859.98	994,430.81	24.21

J-183	466,088.87	991,230.61	20.44
J-96	464,350.10	994,848.99	16.07
J-13	463,283.05	993,524.74	75.65
J-95	464,359.97	994,860.95	20.24
J-15	463,935.49	995,397.23	44.92
J-14	463,176.50	995,641.57	5.51

Figure B.1 Pressure Map of Nodes for low flow, EPS Analysis

Appendix C: Pressure at Peak flow

Node	X (m)	Y (m)	Pressure (m H2O)
J-2	473,926.47	980,811.63	149.73
J-1	473,899.21	980,712.02	4.18
J-4	470,674.92	985,438.11	-5.97
J-3	470,694.32	985,417.02	4.51
J-5	470,621.33	985,423.05	134.29
J-6	470,767.34	985,510.18	133.36
J-156	469,977.46	993,503.08	31.24
J-155	470,221.84	993,787.99	28.66
J-153	469,939.11	993,902.48	28.64
J-154	469,641.25	993,551.97	28.49
J-157	469,858.97	993,366.31	30.25
J-152	469,917.75	993,919.89	28.02
J-146	469,990.52	993,965.13	28.00
J-142	470,610.16	994,188.45	31.00
J-150	470,251.48	994,327.39	26.425
J-144	470,273.74	994,310.34	29.30
J-174	470,987.17	992,956.35	23.07
J-148	470,207.79	994,418.48	25.85
J-151	470,087.43	994,123.99	27.53
J-145	470,105.61	994,106.93	28.01
J-147	470,211.49	994,427.35	25.80
J-143	470,339.42	994,362.97	28.77
J-149	470,278.79	994,385.48	25.47
J-213	466,523.67	990,108.88	40.15
J-138	471,058.11	994,123.90	32.60
J-139	471,037.74	994,105.64	32.24
J-137	471,046.21	994,101.16	32.27
J-140	470,931.51	994,167.61	31.37
J-175	471,049.96	993,183.01	25.07
J-141	470,915.54	994,177.98	31.17
J-135	471,052.78	993,471.14	28.89
J-176	471,048.22	993,298.79	27.15

J-136	471,055.42	993,493.57	28.83
J-212	466,522.09	990,119.62	39.03
J-134	471,053.81	993,299.47	28.14
J-211	466,439.68	990,087.48	38.82
J-210	466,217.68	990,048.33	37.62
J-170	470,727.03	992,480.64	14.95
J-169	470,553.99	992,469.07	16.67
J-166	470,379.76	992,569.74	21.40
J-133	470,733.90	992,616.45	22.56
J-171	470,733.62	992,607.90	12.01
J-172	470,689.13	992,709.85	11.87
J-132	470,713.49	992,611.94	21.61
J-173	470,802.95	992,867.39	11.78
J-167	470,190.46	992,548.91	18.57
J-123	467,870.85	993,218.09	74.27
J-131	470,532.23	992,711.01	14.09
J-168	470,210.54	992,608.80	12.71
J-87	471,197.32	993,343.48	23.10
J-81	471,424.34	993,445.40	24.50
J-88	471,131.05	993,499.85	23.77
J-91	471,145.51	993,700.66	26.89
J-65	472,197.19	993,092.60	45.75
J-82	471,492.24	993,366.63	21.74
J-89	471,102.54	993,552.00	24.00
J-90	471,108.33	993,562.51	24.14
J-83	471,441.33	993,342.63	21.36
J-80	471,239.74	993,842.06	32.30
J-69	471,259.89	993,850.11	32.50
J-68	471,266.67	993,852.83	32.62
J-66	471,524.34	993,580.17	38.32
J-84	471,247.72	993,267.34	21.16
J-209	465,952.94	990,287.70	23.32
J-85	471,242.66	993,265.68	21.09
J-86	471,238.02	993,263.42	21.10
J-66	471,505.19	993,591.54	37.54

J-67	471,508.03	993,595.35	37.48
J-208	465,958.01	990,294.12	22.87
J-165	470,433.59	992,710.13	10.23
J-64	472,260.06	993,140.25	43.38
J-130	470,369.23	992,713.28	9.63
J-63	472,274.29	993,140.32	42.76
J-162	470,081.47	992,714.09	10.01
J-163	470,175.17	992,711.74	8.40
J-164	470,243.99	992,710.44	7.97
J-129	470,265.81	992,712.88	8.62
J-116	467,649.89	992,612.11	46.71
J-62	472,206.85	993,809.77	44.59
J-117	467,652.16	992,554.54	43.47
J-118	467,652.76	992,540.34	42.90
J-161	469,897.29	992,888.89	6.08
J-124	468,785.77	993,827.94	57.51
J-128	469,900.91	992,890.35	6.13
J-74	471,292.63	994,545.53	30.49
J-125	468,795.71	993,826.42	57.15
J-78	471,733.08	994,333.50	37.26
J-77	471,701.39	994,356.84	37.20
J-119	467,655.16	992,369.71	40.98
J-76	471,638.93	994,403.42	36.58
J-79	471,906.80	994,322.00	39.30
J-115	467,683.79	992,906.45	46.45
J-160	469,848.51	992,999.25	1.83
J-114	467,685.90	992,914.20	46.68
J-120	467,500.45	992,442.75	36.78
J-75	471,362.78	994,616.23	27.62
J-182	471,371.95	994,640.50	26.83
J-121	467,392.95	992,488.62	35.07
J-122	467,370.49	992,500.34	34.85
J-159	469,751.34	993,151.28	-3.88
J-207	466,126.40	990,440.35	1.49
J-113	467,419.41	992,952.25	59.76

J-111	467,404.36	992,943.16	60.67
J-110	467,399.40	992,946.59	61.75
J-73	471,274.07	994,370.64	17.66
J-158	469,714.76	993,189.32	-4.99
J-127	469,508.87	993,362.38	17.78
J-205	466,124.92	991,192.88	122.05
J-206	466,108.86	990,460.85	123.91
J-72	471,272.43	994,361.81	16.97
J-70	471,225.58	994,061.28	10.14
J-123	467,251.81	992,575.56	33.88
J-71	471,267.49	994,337.29	15.34
J-181	471,369.32	994,739.57	23.29
J-60	472,132.52	994,649.23	38.45
J-61	472,116.92	994,649.27	38.01
J-59	472,132.56	994,655.26	34.00
J-126	469,123.12	993,630.26	30.87
J-58	472,039.30	994,644.31	34.28
J-179	471,688.68	994,920.89	33.51
J-57	471,972.36	994,639.35	35.79
J-56	471,971.71	994,652.33	35.29
J-112	467,019.43	992,836.23	40.10
J-202	466,359.53	992,710.82	87.29
J-201	466,131.70	992,683.76	72.02
J-109	466,585.72	992,731.89	89.45
J-178	471,903.16	994,943.45	33.02
J-107	466,455.88	992,725.65	95.27
J-204	466,132.51	992,674.29	71.32
J-108	466,466.58	992,705.51	94.91
J-55	471,915.70	994,943.35	32.70
J-105	466,363.07	992,714.06	97.85
J-54	471,915.30	994,949.35	32.59
J-104	466,371.57	992,717.39	97.67
J-106	466,369.02	992,694.88	96.36
J-197	465,387.03	992,424.33	66.45
J-196	465,380.19	992,421.19	65.78

J-18	468,379.14	996,117.71	113.61
J-180	471,467.25	994,944.95	17.87
J-53	472,100.62	994,964.10	31.33
J-188	464,984.60	992,019.11	66.05
J-189	465,129.53	992,229.95	65.03
J-191	465,144.07	992,224.23	64.97
J-190	465,134.17	992,228.00	64.89
J-195	465,175.23	992,282.62	62.66
J-200	465,961.47	992,664.82	63.14
J-192	465,111.10	992,183.68	62.04
J-198	465,586.55	992,514.68	61.47
J-194	465,584.66	992,518.07	61.40
J-193	465,267.98	992,371.33	60.65
J-103	466,403.23	992,959.43	91.76
J-177	471,574.42	995,185.29	19.44
J-199	465,792.40	992,611.39	59.34
J-35	471,864.84	995,437.67	31.57
J-33	471,883.84	995,235.04	26.15
J-36	471,899.07	995,224.12	25.73
J-102	466,347.62	993,011.33	89.57
J-38	471,907.41	995,218.33	25.61
J-37	471,906.83	995,225.07	25.28
J-32	471,897.87	995,236.43	25.40
J-39	471,906.11	995,233.12	25.13
J-34	471,883.06	995,255.35	25.33
J-31	471,895.75	995,257.38	25.18
J-203	464,969.85	992,030.36	56.99
J-48	471,987.85	995,229.92	23.89
J-52	472,082.39	995,158.13	22.78
J-49	472,056.30	995,231.12	20.84
J-47	472,048.07	995,500.01	21.86
J-45	472,027.38	995,787.58	23.44
J-44	472,019.29	995,788.18	23.44
J-43	472,021.49	995,809.34	23.45
J-46	472,027.10	995,797.39	23.22

J-50	472,075.65	995,237.93	19.68
J-51	472,075.84	995,232.42	19.66
J-42	471,854.76	995,752.21	22.22
J-41	471,845.45	995,749.77	21.93
J-40	471,848.24	995,738.01	22.03
J-31	471,865.38	995,563.58	21.80
J-48	472,063.51	995,348.95	17.09
J-101	466,173.35	993,267.92	79.44
J-30	471,839.38	995,712.41	20.02
J-24	471,242.73	995,856.34	25.18
J-9	465,941.84	990,889.07	8.98
J-12	465,271.09	991,780.59	149.69
J-29	471,843.83	995,712.99	17.31
J-8	465,946.31	990,881.73	7.25
J-23	471,044.13	995,881.44	21.94
J-11	465,772.18	990,857.45	149.12
J-7	465,917.59	990,863.42	6.43
J-27	471,752.39	995,645.60	20.08
J-26	471,763.48	995,643.13	19.94
J-28	471,832.09	995,756.27	17.09
J-25	471,773.29	995,752.98	19.30
J-10	465,909.30	990,865.67	147.52
J-22	470,699.80	995,926.73	17.32
J-21	470,410.70	995,954.59	17.61
J-17	466,923.30	996,331.82	97.735
J-185	466,022.75	991,281.20	67.05
J-20	469,607.18	996,339.87	11.96
J-19	469,598.73	996,338.70	51.11
J-187	466,064.02	991,231.49	56.68
J-100	465,409.19	993,996.83	44.92
J-186	466,112.21	991,211.99	34.78
J-16	466,421.88	995,876.48	63.54
J-99	465,190.71	994,168.64	32.83
J-98	465,070.14	994,264.91	27.09
J-184	466,078.14	991,254.19	17.94

J-97	464,859.98	994,430.81	21.45
J-183	466,088.87	991,230.61	17.56
J-96	464,350.10	994,848.99	14.39
J-13	463,283.05	993,524.74	67.47
J-95	464,359.97	994,860.95	20.27
J-15	463,935.49	995,397.23	40.52
J-14	463,176.50	995,641.57	6.22

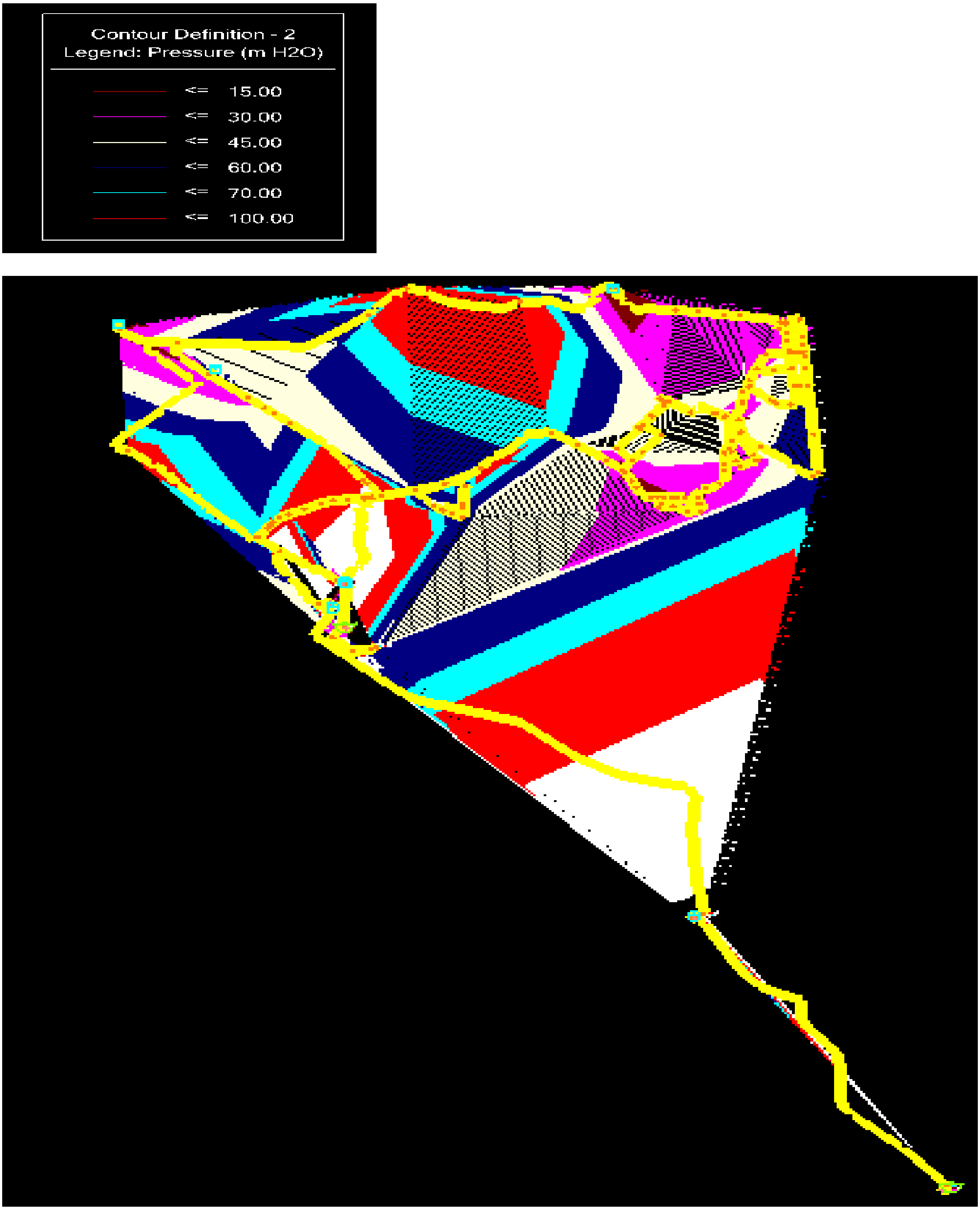

Figure C.1 Pressure Map of Nodes for Peak hour, EPS Analysis

Appendix D: First data arrangement for pressure calibration and time series with pressure networks

Item	Sample taken point	Location			Measured time	Computed pressure (m)	Observed pressure (m)
		X (m)	Y (m)	Z (m)			
1	J-2	473,926.24	980,811.74	2,067.00	6:00	173.67	175.47
2	J-5	470,621.33	985,423.05	2,221.00	7:00	156.24	170.69
3	J-10	465,909.30	990,865.67	2,349.00	8:00	168.33	169.19
4	J-104	466,371.57	992,717.39	2,316.25	9:00	112.03	114.89
5	J-175	471,049.96	993,183.01	2,260.12	9:30	28.53	32.49
6	J-47	472,048.07	995,500.01	2,333.98	10:00	24.35	27.84
7	J-31	471,865.38	995,563.58	2,337.56	10:30	23.69	26.87
8	J-22	470,699.80	995,926.66	2,353.12	11:00	21.27	24.08
9	J-127	469,508.87	993,362.38	2,301.16	11:30	20.47	25.28
10	J-96	464,350.10	994,848.99	2,430.00	12:00	16.37	17.24
11	J-20	469,607.18	996,339.87	2,365.07	12:30	14.96	13.21

$$R^2 = \frac{(x-\bar{x})(y-\bar{y})}{\sqrt{\Sigma(x-\bar{x})^2\Sigma(y-\bar{y})^2}} = 99.84\%$$

Appendix E: second data arrangement for pressure calibration and time series with pressure networks

Item	Sample taken point	Location			Measured time	Computed pressure (m)	Observed pressure (m)
		X (m)	Y (m)	Z (m)			
1	J-2	473,926.24	980,811.74	2,067.00	18:00	173.24	175.87
2	J-5	470,621.33	985,423.05	2,221.00	19:00	156.7	157.45
3	J-10	465,909.30	990,865.67	2,349.00	20:00	168.33	169.01
4	J-104	466,37157	992,717.39	2,316.25	21:00:00	111.71	112.57
5	J-175	471,049.96	993,183.01	2,260.12	22:00:00	27.90	32.84
6	J-47	472,048.07	995,500.01	2,333.98	23:00	25.20	26.27
7	J-31	471,865.38	995,563.58	2,337.56	23:30	24.28	25.16
8	J-22	470,699.80	995,926.66	2,353.12	24:00:00	20.00	20.54
9	J-127	469,508.87	993,362.38	2,301.16	24:30:00	20.08	20.96
10	J-96	464,350.10	994,848.99	2,430.00	25:00:00	16.37	17.24
11	J-20	469,607.18	996,339.87	2,365.07	1:30	13.43	14.18

$$R^2 = \frac{(x-\bar{x})(y-\bar{y})}{\sqrt{\sum(x-\bar{x})^2\sum(y-\bar{y})^2}} = 99.98\%$$

Appendix F: First data arrangement for pressure validation and time series with pressure networks

Item	Sample taken point	Location			Measured time	Computed pressure (m)	Observed pressure (m)
		X (m)	Y (m)	Z (m)			
1	J-2	473,926.24	980,811.74	2,067.00	6:00	171.69	175.47
2	J-5	470,621.33	985,423.05	2,221.00	7:00	167.25	170.69
3	J-10	465,909.30	990,865.67	2,349.00	8:00	168.82	169.19
4	J-104	466,371.57	992,717.39	2,316.25	9:00	113.16	114.89
5	J-175	471,049.96	993,183.01	2,260.12	9:30	33.64	32.49
6	J-47	472,048.07	995,500.01	2,333.98	10:00	29.3	27.84
7	J-31	471,865.38	995,563.58	2,337.56	10:30	28.54	26.87
8	J-22	470,699.80	995,926.66	2,353.12	11:00	21.29	24.08
9	J-127	469,508.87	993,362.38	2,301.16	11:30	27.615	25.28
10	J-96	464,350.10	994,848.99	2,430.00	12:00	15.34	17.24
11	J-20	469,607.18	996,339.87	2,365.07	12:30	13.4	13.21

$$R^2 = \frac{(x-\bar{x})(y-\bar{y})}{\sqrt{\sum(x-\bar{x})^2 \sum(y-\bar{y})^2}} = 99.96\%$$

Appendix G: second data arrangement for pressure validation and time series with pressure networks

Item	Sample taken point	Location			Measured time	Computed pressure (m)	Observed pressure (m)
		X (m)	Y (m)	Z (m)			
1	J-2	473,926.24	980,811.74	2,067.00	18:00	171.26	175.87
2	J-5	470,621.33	985,423.05	2,221.00	19:00	156.92	157.45
3	J-10	465,909.30	990,865.67	2,349.00	20:00	167.44	169.01
4	J-104	466,371.57	992,717.39	2,316.25	21:00:00	113.19	112.57
5	J-175	471,049.96	993,183.01	2,260.12	22:00:00	34.32	32.84
6	J-47	472,048.07	995,500.01	2,333.98	23:00	29.13	26.27
7	J-31	471,865.38	995,563.58	2,337.56	23:30	27.87	25.16
8	J-22	470,699.80	995,926.66	2,353.12	0:00	21.05	20.54
9	J-127	469,508.87	993,362.38	2,301.16	24:30:00	27.68	20.96
10	J-96	464,350.10	994,848.99	2,430.00	1:00	16.91	17.24
11	J-20	469,607.18	996,339.87	2,365.07	1:30	13.63	14.18

$$R^2 = \frac{(x-\bar{x})(y-\bar{y})}{\sqrt{\sum(x-\bar{x})^2 \sum(y-\bar{y})^2}} = 99.94\%$$

Appendix H: Bulk coefficients data tests

	data test-1	
Date	5-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
8:00AM	CT2	0.82
8:00AM	K4 Tank	0.43
8:00AM	J-31	0.21
8:00AM	Alb tank	0.34
Date	6-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
10:00AM	CT2	0.8
10:00AM	K4 Tank	0.42
10:00AM	J-31	0.2
10:00AM	Alb tank	0.32
Date	9-Jun-16	Result
12:00 AM	CT2	0.8
12:00 AM	K4 Tank	0.4
12:00 AM	J-31	0.29
12:00 AM	Alb tank	0.33
Date	11-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
2:00 PM	CT2	0.8
2:00 PM	K4 Tank	0.42
2:00 PM	J-31	0.21
2:00 PM	Alb tank	0.35
Date	11-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
4:00PM	CT2	0.81
4:00PM	K4 Tank	0.48
4:00PM	J-31	0.03
4:00PM	Alb tank	0.32
Date	12-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
6:00PM	CT2	0.8

6:00PM	K4 Tank	0.38
6:00PM	J-31	0.17
6:00PM	Alb tank	0.28
Date	13-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
8:00PM	CT2	0.8
8:00PM	K4 Tank	0.41
8:00PM	J-31	0.2
8:00PM	Alb tank	0.28
	data test-2	
Date	15-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
8:00AM	CT2 Tank	0.8
8:00AM	K4 Tank	0.43
8:00AM	J-31	0.26
8:00AM	Alb tank	0.31
Date	17-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
10:00AM	CT2 Tank	0.8
10:00AM	K4 Tank	0.44
10:00AM	J-31	0.21
10:00AM	Alb tank	0.34
Date	18-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
12:00AM	CT2 Tank	0.8
12:00AM	K4 Tank	0.45
12:00AM	J-31	0.2
12:00AM	Alb tank	0.41
Date	19-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
2:00PM	CT2 Tank	0.8
2:00PM	K4 Tank	0.44
2:00PM	J-31	0.15
2:00PM	Alb tank	0.36
Date	20-Jun-16	Result

Time	Sample	Residual Chlorine(mg/l)
4:00PM	CT2 Tank	0.8
4:00PM	K4 Tank	0.48
4:00PM	J-31	0.2
4:00PM	Alb tank	0.34
Date	21-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
6:00PM	CT2 Tank	0.8
6:00PM	K4 Tank	0.45
6:00PM	J-31	0.09
6:00PM	Alb tank	0.28
Date	22-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
8:00PM	CT2 Tank	0.8
8:00PM	K4 Tank	0.41
8:00PM	J-31	0.04
8:00PM	Alb tank	0.3
	data test-3	
Date	23-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
8:00AM	CT2 Tank	0.8
8:00AM	K4 Tank	0.39
8:00AM	J-31	0.2
8:00AM	Alb tank	0.32
Date	24-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
10:00AM	CT2 Tank	0.8
10:00AM	K4 Tank	0.36
10:00AM	J-31	0.19
10:00AM	Alb tank	0.3
Date	25-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
12:00AM	CT2 Tank	0.8
12:00AM	K4 Tank	0.45
12:00AM	J-31	0.23

12:00AM	Alb tank	0.37
Date	26-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
2:00PM	CT2 Tank	0.8
2:00PM	K4 Tank	0.43
2:00PM	J-31	0.18
2:00PM	Alb tank	0.39
Date	27-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
4:00PM	CT2 Tank	0.8
4:00PM	K4 Tank	0.48
4:00PM	J-31	0.12
4:00PM	Alb tank	0.33
Date	28-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
6:00PM	CT2 Tank	0.8
6:00PM	K4 Tank	0.4
6:00PM	J-31	0.06
6:00PM	Alb tank	0.32
Date	29-Jun-16	Result
Time	Sample	Residual Chlorine(mg/l)
8:00PM	CT2 Tank	0.8
8:00PM	K4 Tank	0.41
8:00PM	J-31	0.15
8:00PM	Alb tank	0.29

Appendix I: Observed of the residual chlorine and pressure relationship data

S. no	Time (hour)	Sample point node	Observed residual chlorine (mg/l)	observed pressure (m)	Computed pressure(m)	GPS sample location		
						x(m)	y(m)	elevation(m)
1	6:00	J-2	0.8	175.47	171.69	473,926.24	980,811.74	2,067.00
2	7:00	J-5	0.67	170.69	167.25	470,621.33	985,423.05	2,221.00
3	8:00	J-10	0.59	169.19	168.82	465,909.30	990,865.67	2,349.00
5	9:00	J-175	0.51	32.49	33.64	471049.96	993183.01	2260.12
6	10:00	J-31	0.24	26.87	28.54	471865.38	995563.58	2337.56
7	11:00	J96	0.38	17.24	16.91	464350.1	994848.99	2430

Appendix J: tank level data arrangement calibration of K4 tank

Time in (hours)	computed tank level in (m)	Observed tank level in (m)	difference (m)
3	2.86	2.74	-0.12
6	2.8	2.41	-0.39
9	2.89	2.52	-0.37
12	2.5	2.02	-0.48
15	2.25	2.23	-0.02
18	2.36	2.09	-0.27
21	2.41	2	-0.41
24	2.3	2	-0.3

$$R^2 = \frac{(x-\bar{x})(y-\bar{y})}{\sqrt{\Sigma(x-\bar{x})^2 \sum(y-\bar{y})^2}} = 83.68\%$$

Where=X and Y sample means of average array1 and array2 respectively and R^2 correlation mean square percent

Appendix L: tank level data arrangement calibration of Army tank

Time in (hours)	computed tank level in (m)	Observed tank level in (m)	difference (m)
3	3.69	5.69	-2
6	5.84	6.63	-0.79
9	6.62	8.27	-1.65
12	5.61	6.53	-0.92
15	6.04	7.98	-1.94
18	5.68	6.67	-0.99
21	5.88	6	-0.12
24	3.67	4.26	-0.59

$$R^2 = \frac{(x-\bar{x})(y-\bar{y})}{\sqrt{\Sigma(x-\bar{x})^2 \sum (y-\bar{y})^2}} = 84.79\%$$

Where=X and Y sample means of average array1 and array2 respectively and R^2 correlation mean square percent

Appendix M: tank level second data arrangement validation of K4 tank

Time in (hours)	computed tank level in (m)	Observed tank level in (m)	difference (m)
3	2.86	2.79	0.07
6	2.8	2.75	0.05
9	2.89	2.95	-0.06
12	2.5	2.02	0.48
15	2.25	2.23	0.02
18	2.36	2.09	0.27
21	2.41	2	0.41
24	2.3	2	0.3

$$R^2 = \frac{(x-\bar{x})(y-\bar{y})}{\sqrt{\Sigma(x-\bar{x})^2\sum(y-\bar{y})^2}} = 90.95\%$$

Where=X and Y sample means of average array1 and array2 respectively and R^2 correlation mean square percent

Appendix N: tank level second data arrangement validation of Army tank

Time in (hours)	**computed tank level in (m)**	**Observed tank level in (m)**	**difference (m)**
3	4.48	5.69	1.21
6	6.29	6.63	0.34
9	7.95	8.27	0.32
12	6.28	6.53	0.25
15	7.02	7.98	0.96
18	5.05	6.67	1.62
21	4.45	6	1.55
24	3.25	4.26	1.01

$$R^2 = \frac{(x-\bar{x})(y-\bar{y})}{\sqrt{\Sigma(x-\bar{x})^2 \sum(y-\bar{y})^2}} = 94.33\%$$

Where=X and Y sample means of average array1 and array2 respectively and R^2 correlation mean square percent