ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
ADDIS ABABA INSTITUTE OF TECHNOLOGY

WATER QUALITY MODELING OF WATER SUPPLY DISTRIBUTION SYSTEM
(The case of Alalbada-Erba Multi-Village Water Supply Scheme)

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.

(Major in Hydraulic Engineering)

Submitted by: - Asfaw Diro Meta

Advised by: - Dr. Agizew Nigussie

Addis Ababa
Ethiopia
November, 2016

Addis Ababa University
(Since 1950)

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Approved by Board of Examiners

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List of Symbols & Acronyms

MOWE	Ministry of Water and Energy
MTWSS	Mena Town Water Supply Scheme
PAs	Peasant association
WTP	Water treatment plant
NOM	Natural organic matter
TOC	Total organic carbon
TNTC	Too numerous to count
DBP	Disinfections by product
WDN	Water distribution network
SW	Southwest direction
NNE	North East direction
WSS	Water supply service
Jun	Junction
PF	Public fountain
HTP	House taps
DCI	Ductile Cast Iron
U-PVC	Univinayl polyethylene pipe
GSP	Galvanized steel pipe
RMS	Root mean square

Thesis Organization

The thesis is organized into the following six chapters.

Chapter-1: consists of the introduction of the paper, location of study area and statement of the problem, research objective, research questions and specific objectives

Chapter-2: it consists of literature review and conceptual framework, water quality relationship which is pertinent to the field.

Chapter-3: methodology, equipment's used, and scope of the study.

Chapter-4: consists of data analysis and Interpretation.

Chapter-5: consists of Conclusion and Recommendation.

Chapter-6: consists Reference materials.

Abstract

Water distribution systems are designed to fulfill all requirements of potable water needed for intended purposes. Initial system designs frequently consider any anticipated changes likely to happen. However, as time elapsed they slowly begin to fail to satisfy customers' requirements, both in quantity and quality. The main issue is to identify factors which bring those changes and propose viable solutions to improve the situation. To this effect modeling water supply distribution network system is very helpful. In this study, Alalbada-Erba multi-village Water distribution system was assessed as a case study.

EPANET software was used as tool to model water distribution system. The modeling effort included both hydraulic and water quality modeling. Simulation results for maximum and minimum pressures were used as base to evaluate the hydraulic performance and simulation result for water age and minimum residual chlorine were used as base to assess water quality transformation in the distribution system.

Modeling results showed violation of maximum and minimum pressure requirements. Along with this, water quality simulation results indicated water quality decline due to insufficient residual chlorine which is below WHO guideline. To retrieve the situation there is a need to intervene. Modifications in operation and design will improve the current situation of the case study of water distribution system.

Keywords: - ***Water distribution system, modeling, Hydraulic performance, Water quality, Maximum pressure, Minimum pressure, Water age, Residual chlorine*, *Alalbada-Erba, Ethiopia.***

CHAPTER 1:

Introduction

Water is considered necessary in all aspects of life. Water obtained from rivers, lakes, springs and wells has been used for drinking, washing, agriculture, and manufacturing. The quality of water is an essential concern for mankind since it is directly linked with human welfare; drinking water should have high quality so that it can be consumed without threat of immediate or long term adverse impacts to health. Such water is commonly called as “drinkable water”. In most developed countries, the water supplied to households and industry usually meets World Health Organization’s Drinking Water Quality Guidelines, even though only a small fraction is actually consumed by human or used in preparation of food. It is common that people who are most susceptible to water-borne diseases are those who use polluted drinking water sources. The report from UNICEF (2015) indicated that in the world 884 million people use unimproved drinking water sources in 2010, and it would be about 663 million people in 2015.

Chlorine disinfection is of great significance in maintaining drinking water quality related to pathogens and viruses throughout the water distribution system. Diseases like cholera and typhoid fever have virtually been eliminated in the developed countries since continuous chlorination practice started in the 20th century (Academic Journals of physical sciences Vol.7 (33), 2012).Drinking water filtration and the use of chlorine are the most significant public health advances of the last century. Along with control of microorganisms in the water distribution system, chlorine also removes the objectionable taste and odor from the water. Not amazingly, chlorine is the most widely used water disinfectant in the world. Water quality within water distribution system may vary with both location and time. Water quality models are used to predict the spatial and temporal variation of water quality throughout water system. Chlorine is an efficient and inexpensive disinfectant, and is widely used in drinking water works all over the world. Maintaining residual chlorine in the whole water distribution system can prevent water quality microbiological degradation. Chlorine is relatively unstable, may react with a variety of organic and inorganic compounds in bulk water or on pipe wall, and there are reactions of biofilm with chlorine.

All these result in consumption of residual chlorine in distribution system. It is very difficult to precisely predict the chlorine concentration in specific locations of the whole distribution system, because of the complexity of water distribution system, and chlorine reactions. Modeling the kinetics of residual chlorine decay in distribution system is necessary, in order to ensure delivery of high quality drinking water. The most widely used kinetic model describing chlorine decay in water distribution system is the first order decay model in which the chlorine concentration is assumed to decay exponentially (Academic Journals of physical sciences Vol.7 (33), 2012).The decay constant values vary with water quality, water temperature, flow rate, pipe diameter, pipe materials, the type of inner coating, and pipe roughness.

Ethiopia the millennium development agenda as the goals are in in line with its poverty reduction and development plans. Access to improved water is one of Millennium Development Goals. Drinking water quality is often the most important component for measuring access to improved water in terms of physical, chemical and bacteriological parameters (WHO, 2004). Use of community participation in ensuring safe water supply also carries great weight (Doria, 2010).

In this paper, the definition of water quality is limited to disinfectant residuals, namely to Chlorine residuals; water quality is indicated by the disinfectant concentration throughout the water distribution system. Chlorine is the most commonly used disinfectant. The decay of chlorine within a water distribution system can be attributed to its reaction with organic compounds in the source water and with biofilms on pipe surfaces. The rate of decay of chlorine is often described by a first or second order reaction.

Calibration of residual chlorine with in the network was also conducted after hydraulic calibration through determining bulk and wall reactions. After calibration, optimal design has been determined for the model through defining an objective function of minimum cost design satisfying constraints and considering future requirements.

1.1 Existing Water Supply Sources

The first motorized borehole was drilled in 1981E.C for Menna town and is still functional. But the yield of the well is very low and its recovery is also poor. So to fill $50m^3$ reservoir of the town the pump should have to be turned off many times to wait for recovery. Using this technique, it takes hours to quench the thirst of some of the dwellers. The functional shallow well that has pump is drilled for Hostel. Most of the hand pumps in the surrounding village are non-functional and the discharge of existing spring is not sufficient.

The water utility taken as a case study is supplying the multi-village along the way to Menna town. The distribution network is branched and grid types and it has one source. The length of the carrier and distribution networks is about 31 km with diameters from 25 mm to 250 mm of U-pvc (12.8% carrier lines with diameter from 150 mm to 300mm of DCI type and 82% of PVC pipes of distribution line with diameter from 38 mm to 150 mm) and the remaining is GI. The network consists of more than one type of pipe such as (PVC – DCI – GSP), one $500m^3$ water storage tank for Menna town $100m^3$ and $75m^3$ for Rural village, about 280 customer service house connection and more than 10 public fountains both along the gravity line of 31km and in the distribution network of the town which was commissioned in the year 2001E.C.

To alleviate the water shortage the Oromia Water Bureau has conducted study and design on Alalbada spring in April 1997 E.C (Design Report, 2005).This spring source is located in Bale Zone of Goba woreda and "Ogodo" Kebele. The spring source is a cluster of at least four-spring eyes. The spring is emerging from volcanic rocks activity and continuous effect of erosion. The area lies on tertiary trap series lava flow and it is part of highland Bale Mountain range that is formed by eruption of upper Oligocene to upper Miocene. From riverbank exposure it is observed that basalt of the area is fractured and jointed.

It covers the whole area and extends for more than 200m. So, it is expected to be the potential source of groundwater of the area. The springs that issue from the foots of the neighboring highlands prove the same thing. One of these springs is Ogodo spring. To reach this spring one has to first drive along the road of Menna–Goba for 28km, then abandoning the car here turn off right and go through the indigenous rain forest trees for about three km. The discharge of spring was not measured because the water spreads over a large area as it come out of the fractured basalt. But it is estimated to be about 58.4Lps

The elevation at spring is 1806m amsl; so it is possible to take the water to Menna town and surrounding kebeles (1305m amsl) by gravity.

1.2 Description of the Study Area

Menna town multi-village water supply scheme is located in Bale Zone south west of Robe. The general geographic location of this area is between 593262M Easting (UTM), and 708727M Northing (UTM) 556km far from Addis Ababa in south-east direction.

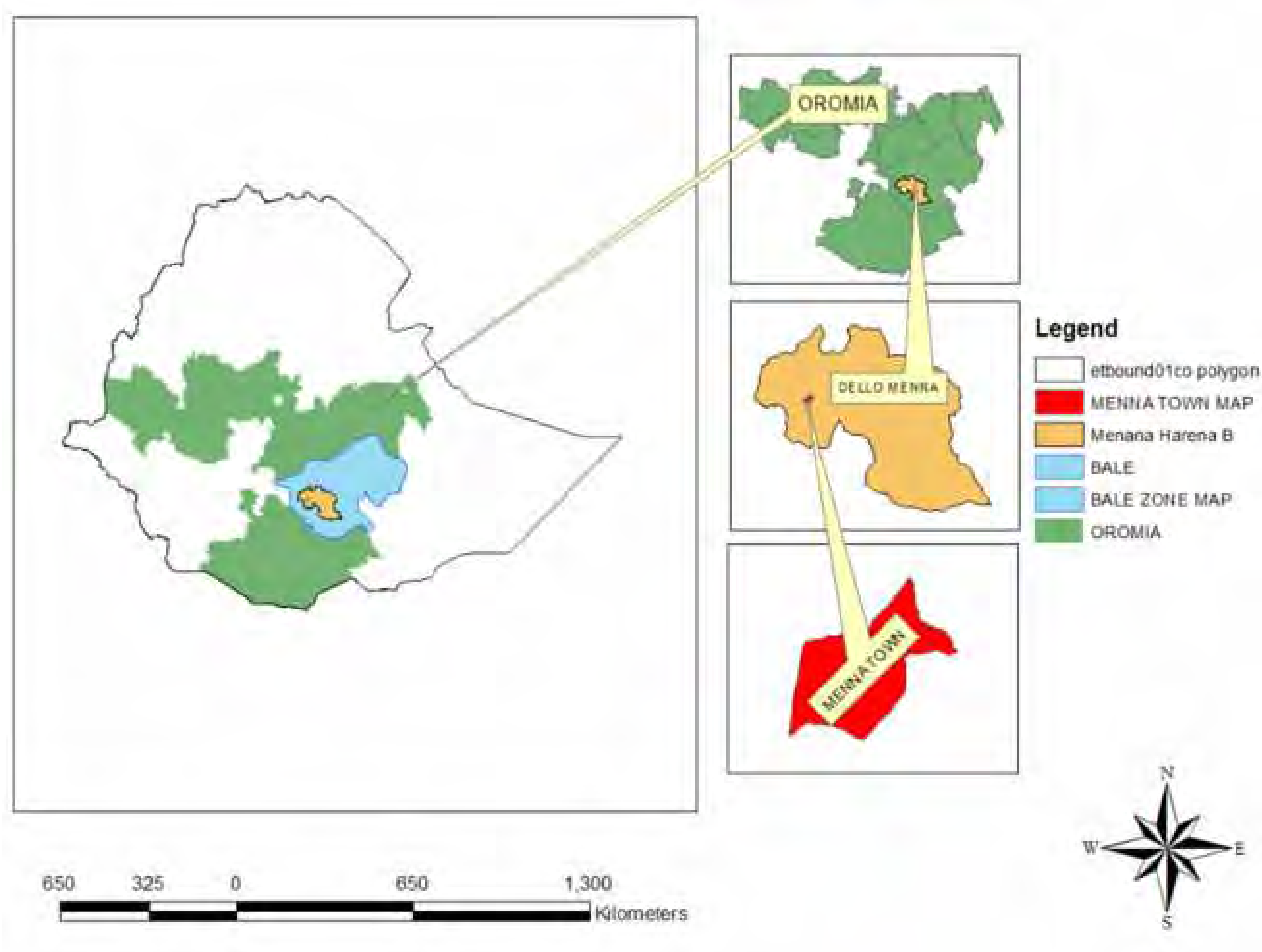


Figure1.1: Location map of Alalbada- Erba multi-village water supply scheme

1.3 Population size of the water supply scheme

The population that is served by Alalbada –Erba water supply system resides in 6 villages with a projected total population of 34,635.The town's water supply system pipe is about 31 km long and provides water by gravity for dwellers in 3 villages and other additional four villages

Table 1.1: Alalbada-Erba population size

Year (E.C)	2002	2007	2012	2017	2022
Menna Town	8,665	9,948	11,421	13,113	15,055
Rural village	14,224	16,331	18,746	21,522	24,709
Total	**22,889**	**26,279**	**30,167**	**34,635**	**39,764**

Source: (OWRB Design report, 2004)

1.4 Climatic Condition

The altitude of Manna town is about 1305 m. a.m.s.l. According to the climatic category of the country it is categorized under the tropical climatic zone with hot temperature and moderate rainfall. The mean annual rainfall and temperature are estimated to be 900-1300mm and 20^0-25^0 respectively.

1.5 Surveillance Report of Health Center

According to the information obtained from Menna Health Centre, The culture of using latrine and the awareness of the population towards sanitation is poor. In Menna town only about 10-15% of the total population has good latrine. Most of the dwellers use Yadot River for all purpose. The following are common diseases of the area:

1. Intestinal parasite-which is 23% of total diseases
2. Respiratory diseases (such as TB ,Neumonia, Bornchiates)
3. Ameabiasis, Jardiasis, Bacillary dysentery
4. Rheumatic pain

1.6 Statement of problem

Safe water is a precondition for health and development and a basic human right; yet it is still denied to hundreds of millions of people throughout the developing world. Water quality degradation is one of the major environmental problems of these days.

Contamination of surface and groundwater is the most serious problem affecting the health of the population.

Water treatment has contributed to the prevention of waterborne diseases and to safeguard human health in drinking water. The recharge to the springs is considered to be from Bale Mountain. Ogodo spring is located at high land area and is used by the community for traditional medication purpose; the surrounding community and animals use the spring as a source of water supply. Therefore, there is a possibility that the water in the collection chamber is contaminated by pathogenic microorganisms. The chlorine used at the source for disinfection purpose may not provide adequate residual concentration and ensure safe water supply.

1.7. General objective

The main objective of the present study is to study the hydraulic behavior of Mena town WDS through use of appropriate hydraulic model representing the main and selected secondary pipes. The study assessed the hydraulic and water quality performance of the water distribution system of Alalbada-Erba Multi-village water supply distribution system which included proper application of chlorine, pipe network system leakage control and maintenance.

1.7.1. Specific objective

To fulfill the above general objective the following specific objectives were used:

- To estimate the residual chlorine concentration at peak and low flow conditions
- To identify major water quality decline factors.
- To verify whether the water from Ogodo spring is microbiologically safe to drink.
- To check whether the current water supply interruption is accelerated by occurrences of excessive pressures within the system.

1.7.2 Research Questions

- How is the quality of the water supply scheme of Alalbada-Erba Multi-Village is guaranteed?
- Is there appropriate decontaminator at the water supply Service?
- Is there significant chlorine loss in the multi-village water supply scheme?
- How the least amount chlorine is residual is maintained by initial concentration?

1.8 Scope of the Study

This study specifically focuses on assessing factors affecting water Quality and hydraulic performance of water supply schemes. The study area is limited to Alalbada-Erba Multi-village Water Supply scheme. The study covers Menna town; the problem of inadequate budget and logistics were some of the main challenges/limitations for the research work. Water Quality modeling in water supply schemes is a multi-dimensional and dynamic concept which is the result of interactions of various factors (physical, chemical, biological, etc.). In this study attention has been given to identifying major factors responsible for water quality decline, pressure variation, and residual chlorine concentration of the water supply scheme. The result and findings of the modeling are the reflections of the study area. However, it might reflect problems in other areas of water Supply scheme with similar characters

CHAPTER 2

Literature Review

This chapter presents a summary of authentic and applicable literature on the topic. The selected readings represent a small sample from a broad range of literature. It is worthwhile mentioning that the literatures have been selected primarily for their relevance, accessibility, and clarity.

Based on the surveyed theoretical and empirical literature is expected to serve as a base for formulating the conceptual framework of the study, owing to the limitation of research works on the topic in the case of Alalbada-Erba Multi-village water supply scheme, only a summary of the overall current situation is presented.

This section presents review of water distribution system modeling conceptual frame work along with approaches of model calibration and validation. In the case of a water distribution system, network hydraulics is described by conservation of mass and energy and water quality is based upon conservation of mass at a node and adjective transport in a pipe.

2.1 Network Hydraulics

2.1.1 Conservation of Mass

Several formulations of conservation of mass and energy can be written for a water distribution system under steady conditions (Boulos *et al*, 2004). Here the pipe flow equation formulation is summarized. For a junction that connects two or more pipes, conservation of mass is written as:

$$\sum^{\text{Pipes}} Q - U = 0 \ldots\ldots\ldots\ldots\ldots\ldots 2.1$$

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 (Walski et al., 2003).

$$\sum^{\text{Pipes}} Q - U - \frac{d}{d} = 0 \quad \ldots\ldots 2.2$$

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:

$$Z1 + \frac{p}{r} + \frac{v2}{2g} + \Sigma h_p = Z2 + \frac{p}{r} + \frac{v2}{2g} + \Sigma h_l + \Sigma h_m \quad \ldots\ldots 2.3$$

Where, Z = elevation (L)

P = Pressure (M/L/T2)

γ = fluid specific weight ($M/L_2/T_2$)

V = velocity (L/T)

g = gravitational acceleration constant (L/T_2)

hp = head added at pump (L)

hL = head loss in pipes (L)s

hm = head loss due to minor losses (L)

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

2.1.3 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 (Mohan, 2003). 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, advective transport is the carrying of constituent (chlorine) along with the flow of water (Lansey and Boulos, 2005). 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.

2.1.4 Advective Transport in Pipe

According to (Walski et.al. 2003), advective transport within a pipe can be represented with the next equation.

$$\frac{\partial}{\partial} = \frac{\partial}{A} * \frac{\partial}{\partial} + (Ci) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.4$$

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 am a function of discharge divided by cross-sectional area.

2.1.5 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.

$$\text{Cout}=\frac{\Sigma \quad ,n \; +U}{\Sigma Q} \text{……………………….............................2.5}$$

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/

2.1.6 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 fill-and-draw conditions providing that sufficient momentum flux is imparted to the inflow (Rossman and Grayman, 1999). 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

$$\frac{d}{d} \text{X} \frac{Q}{V}(c_i , n \;\; (t) - c_i \;\;) + \boldsymbol{\theta}(\boldsymbol{c} \;\;) \text{…………………………………….2.6}$$

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

$$\frac{d}{d} k = (c_k) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.7$$

2.1.7 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.

2.1.8 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;

$\theta(C) = \pm K.C^n$………………………………………………………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 which is given by

$$C_t = C_o * e^{-k} \text{ ....2.9}$$

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

C_o= 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 2.8 for the time t when Ct/Co is equal to a value of 0.5.

$$T = -\left(\frac{0.693}{K}\right) \text{...2.10}$$

2.1.9 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:

$$r = \frac{2k \;\; *k \;\; *C}{R(k \;\; +K \;\;)} \text{..2.11}$$

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 k_b 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

$$K = K_b + \frac{2k \;\; *k \;\; *C}{R(k \;\; +K \;\;)} \text{...2.12}$$

2.1.10 Bulk and Wall Reactions

The most frequently modeled constituents in water distribution systems are disinfectants. Upon leaving the plant and entering the distribution system, disinfectants are subject to a poorly characterized set of potential chemical reactions. Typical chemical reactions occur with disinfectant is illustrated in figure 2.2.Chlorine (the most common disinfectant) is shown reacting in the bulk fluid with natural organic matter (NOM), and at the pipe wall, where oxidation reactions with biofilms and the pipe material (a cause of corrosion) can occur. The first-order decay model has been shown to be sufficiently accurate for most distribution system modeling applications and is well established. (Rossman, Clark, and Grayman (1994) proposed a mathematical framework for combining the complex reactions occurring within distribution system pipes. 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;

$$Q(C) = +KC \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.13$$

Where, K = overall reaction rate constant (1/T)

Equation 2.13 is a simple first-order reaction (n = 1). The reaction rate coefficient K, however, is now a function of the bulk reaction coefficient and the wall reaction coefficient, as indicated in the following equation.

$$K = K_b + \frac{k \; * k}{R(k \;\; + K \;)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.14$$

Where, k_b = bulk reaction coefficient (1/T)

Kw = wall reaction coefficient (L/T)

Kf = mass transfer coefficient, bulk fluid to pipe wall (L/T)

RH = hydraulic radius pipeline (L)

The rate that disinfectant decays at the pipe wall depends on how quickly disinfectant is transported to the pipe wall and the speed of the reaction once it is there. The mass transfer coefficient is used to determine the rate at which disinfectant is transported using the dimensionless Sherwood number, along with the molecular diffusivity coefficient (of the constituent in water) and the pipeline diameter.

$$Kf = \frac{S}{D} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.15$$

Where, SH = Sherwood number

d = molecular diffusivity of constituent in bulk fluid (L2/T)

D = pipeline diameter (L) for stagnant flow conditions (Re < 1), the

Sherwood number, SH, is equal to 2.0. For turbulent flow (Re > 2,300), the Sherwood number is computed using Equation 2.16.

$$SH=0.023Re^{0.83}\,(v/d)^{0.33} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.16$$

Where, Re = Reynolds number

V= kinematic viscosity of fluid (L2/T)

For laminar flow conditions (1 < Re < 2,300), the average Sherwood number along the length of the pipe can be used. To have laminar flow in a 6-in. (150-mm) pipe, the flow would need to be less than 5 gpm (0.3 l/s) with a velocity of 0.056 ft/s (0.017 m/s).

$$SH=3.65+\left[\frac{0.0668\frac{D*\ R\ \left(\frac{v}{d}\right)}{L}}{1+0.04\left(\frac{D}{L}*R\ \left(\frac{v}{d}\right)0.66\right.}\right]\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.17$$

Where, L = pipe length (L)

The framework developed above accounts both bulk fluid and pipe wall disinfectant decay reactions. Bulk decay coefficients can be determined experimentally. Wall decay coefficients, however, are more difficult to measure and are frequently estimated using disinfectant concentration field measurements and water quality simulation results.

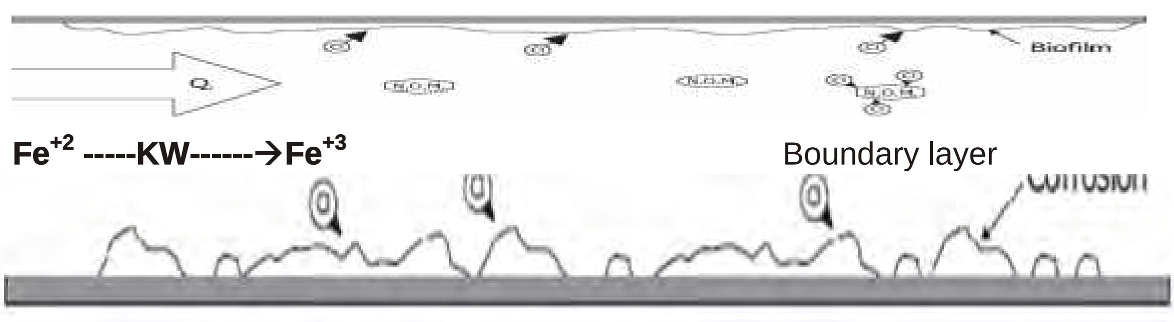


Figure 2.1: Disinfectant reactions occurring within a typical distribution system pipe

2.1.10.1 Water Age Analysis

Many water distribution systems experience long retention times or increased water age, in part due to the need to satisfy firefighting requirements. Although not a specific degradative process, water age is a characteristic that affects water quality; because, many toxic effects are time dependent. Typically, the loss of disinfectant residuals and the formation of Disinfection bi-products (DBPs) are due to increased water age.

The chemical processes that can affect distribution system water quality are a function of water chemistry and the physical characteristics of the distribution system itself (for example, pipe material and age). More generally, however, these processes occur over time, making residence time in the distribution system a critical factor influencing water quality. Water age is of particular concern when quantifying the effect of storage tank turnover on water quality.

It is also beneficial for evaluating the loss of disinfectant residual and the formation of disinfection by-products in distribution systems.

The chief advantage of a water age analysis when compared to a constituent analysis is that once the hydraulic model has been calibrated, no additional water quality calibration

Procedures are required. The water age analysis, however, will not be as precise as a constituent analysis in determining water quality; nevertheless, it is an easy way to leverage the information embedded in the calibrated hydraulic model. Consider a project in which a utility is analyzing mixing in a tank and its effect on water quality in an area of a network experiencing water quality problems. If a hydraulic model has been developed and adequately calibrated, it can immediately be used to evaluate water age. The water age analysis may indicate that excessively long residence times within the tank are contributing to water quality degradation. Using this information, a more precise analysis can be planned (such as an evaluation of tank hydraulic dynamics and mixing characteristics, or a constituent analysis to determine the impact on disinfectant residuals).

2.1.10.2 Dynamic models

Dynamic models of water quality in distribution systems take explicit account of how changes in flows through pipes and storage facilities occurring over an extended period of system operation affects water quality. Accordingly, Dynamic models provide a more practical picture of system behavior (W. Mays, 2000). Accordingly the dynamic water quality equations can be solved using two methods.

The first method is based on an Eulerian approach that divides each separate pipe into a series of equal length sub-links. The second method is a Lagrangian approach that tracks parcels of water of homogeneous water quality concentrations as they move through the pipe system. Solution methods for dynamic models can be classified spatially as either eulerian or lagrangian and temporally as either time driven or event-driven. Eulerian approaches divide the pipe network into a series of fixed, interconnected control volumes and record changes at the boundaries or within these volumes as water flows through them.

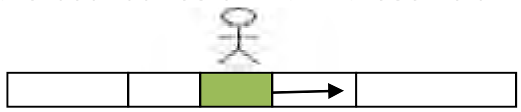


Figure 2.2: Eurelian solution method

Lagrangian models track changes in a series of discrete parcels of water as they travel through the pipe network. Time-driven simulations update the state of the network at fixed time intervals. Event-driven simulations update the state of the system only at times when a change actually occurs, such as when a new parcel of water reaches the e*nd of a* pipe and mixes with water from other connecting pipes'(W. Mays, 2000).

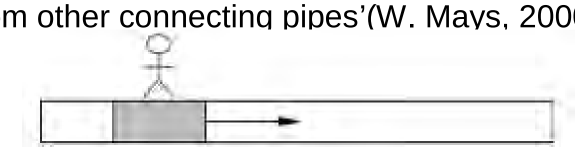


Figure 2.3: Lagrangian solution method

2.2 Modeling Tool

Water distribution network mathematical models have become increasingly accepted within the water industry as a mechanism for simulating the behavior of water distribution systems. They are intended to replicate the behavior of an actual or proposed system under various demand loading and operating conditions. Their purpose is to support the decision-making processes in various utility management applications including planning, design, operation, and water quality improvement of water distribution. For this study EPANET modeling software was preferred as analyzing tool due to its Hydraulic and Water Quality modeling capabilities. EPANET is an open-structured, public domain hydraulic and water quality model developed by the EPA. Besides this, EPANET is user friendly and accessible modeling software. While other hydraulic and water quality modeling software's such as: KY pipes, Info Water, H20Map Water, and Hydraulic CAD are not easily accessible.

EPANET has gained wide acceptance world-wide for the mere fact that it provides quantitative basis for undertaking new modeling activities and assessment of existing water distribution system. It has been used for: Long-range master planning, including new development and rehabilitation, Fire protection studies, Water quality investigations, Energy management, System design, Daily operational uses including operator training, emergency response, and Troubleshooting(For instance: EPA, 2005; Li, 2007; S. Abdel Meguid, 2011).

CHAPTER 3

3.1 Methodology, Procedures and Materials used

Both primary and secondary data were collected and analyzed using the following methodologies.

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

2. Conduct discussion with Mena water supply staff to get their views on the water supply services.

3. Discussions with relevant stakeholders such as municipality and Health center

4. Organizing and conducting of sample household survey on water quality situation.

5. Observing Records of surveillance report from health center

6. Conduct bacteriological analysis on the Alalbada-Erba spring source.

7. Determination of chlorine dosage and feeding rate.

8. Preparation of chlorine solution as disinfectant and applied to the system

9. Data Analysis and Interpretation of results.

9. Observing water tank level from staff gauge and meter reading for link flow data's

10. Model Calibration and Validation.

Water samples were collected in pre-sterilized plastic bags and were filtered on the spot using membrane filter with a pore size of 45micro-meter.The filters were incubated in a ELE pqualab 25 field incubator, in sterilized aluminum petri dishes with a bacterial medium of Lauryl-sulfate on absorbed pad, at 37^0C and 44^0C for total coli-forms and Fecal-coli-forms respectively. The filters were examined 24 hrs. Later for bacterial growth.

Plate 3.1: Photo Field observation Chlorine test

Water quality modeling required chlorine bulk decay bottle tests under controlled conditions. A chlorine decay test for the finished water was performed on monthly basis. A sample for the chlorine bulk decay tests was taken from the storage tank just before entering its distribution system. The 250 mL amber bottles screw capped with PTFE-lined septa were used for the tests. Chlorine concentrations were measured by DPD tablet, a pocket colorimeter (Pocket Colorimeter TM II, Hatch, Loveland, CO), spectrophotometer & incubator. Samples including Temperatures, initial chlorine, K_b is to be tested in the laboratory and the residual chlorine at the selected location will be measured.

3.2 Determination of chlorine dosage and feeding rate.

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 (Mohan Kumar 2003). 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. The principal factors influencing the disinfection efficiency of chlorine are free residual concentration, contact time, pH and water temperature. The term ‘free residual’ means the amount of free chlorine remaining after the disinfection process has taken place. Given adequate chlorine concentration and contact time, all bacterial organisms and most viruses can be inactivated. Thus a useful design criterion for the disinfection process is the product of contact time (*t* in minutes) and the chlorine-free residual concentration (*C* in mg/l) at the end of that contact time. This is known as the '*Ct* value 'or 'exposure value' (WHO, 1989). On this basis the WHO guide level of 0.5 mg/l free residual concentration after 30 minutes contact time would have a *Ct* value of 15 mg. min/l. This is shown to provide a 12.5-fold factor of safety so that a degree of inefficiency in the contact tank performance can be tolerated (Stevenson, 1998).

3.3. Random Sampling location of Residual chlorine measurement

One objective of surveillance is to assess the quality of the water supplied by the supply agency and of that at the point of use, so that samples of both should be taken. Any significant difference between the two has important implications for remedial strategies.
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.

A typical network representation of a water network may include hundreds or thousands of links and nodes. However, only a small percentage of representative sample measurements can be made available for the use of model simulation due to the limited financial requirements for data collection. Therefore, it is of utmost important to have a comprehensive methodology and efficient tool that can assist in achieving a highly accurate model under practical conditions.

Selection of sampling sites is typically compromise between selecting sites that provide the greatest amount of information and sites that are most amenable to sampling.

Sites should be spread throughout the study area and should reflect a variety of situations of Interest, such as transmission mains and local lines, areas served directly from a source, and areas under the influence of tanks.

Sampling taps should be placed close to seventeen representative sample nodes for residual chlorine measurement throughout the study area have been selected by random sampling technique for modeling. Samples were collected daily from different sampling point from spring water and at the exit of water tank before distribution, these samples collected from the origin in order to represent water with moderate and high residence time within the distribution system. The water samples were quenched with sodium thiosulfate ($Na_2S_2O_3$) and stored in 40-mL vials closed with Teflon-lined screw caps at 4°C until the analysis.

Plate 3.2: photo sample when taken

Typical chlorine dosages to final treated waters are frequently in the range **0.2–2.0 mg/l** of free chlorine to give a residual of about **0.02–0.3 mg/l** at the consumer's tap. The lower doses **(0.2–0.5 mg/l)** tend to be those used on clear well waters not subject to pollution; the higher doses relating to treated surface waters or to well or borehole supplies which are liable to experience sudden pollution, where super chlorination followed by partial dechlorination after adequate contact time may be advised. Some raw waters, particularly surface waters, can have a high chlorine demand of **6–8 mg/l**

The concentration of ammonia of the spring water is about 0.39mg/l that is much less than WHO standards. Chlorine dose is adopted as 0.8ppm and a constant-head drip chlorinator of 100 liter capacity is used. Feed rate=R, Q=flow rate, d=dosage of chlorine, 0.8mg/l as simulated by Epanet 2w software.

The Concentration of chlorine is in mg/l, chlorine concentration or strength=Q*d/C and V=volume of water and C=m/v with strength of 65%

Let C=10gm/l➔m=CVx100/65➔[10gm/l*100l*100/65] =1538gm=➔1.538kg of chlorine is required.

R=Q*d/C ➔[36m3/hr.*0.8mg/l(0.8gm/m3)/10gm/l]➔288/hr.*1/10=➔2.88l/hr.➔**3l/hr.**is determined. Therefore, using 100l of mixing tank, the chlorine application of 1.4 day time is required.

Plate 3.3: A 500m^3 Water tank Constant head drip chlorinator

3.4 Sources of Data

i. System Maps

System layout plan of the water supply systems 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 they are obtained from surveying data and from the design report.

System layout 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
- Elevations
- Miscellaneous notes or references for tank characteristics
- Background information, such as the locations of roadways, streams, planning zones, and so

ii. Utility water production reports

Model data were derived from water utility corporate data records. Specifically data such as water consumption and water billing records were obtained from utility Official records for water consumption and water billing data were used in this research to undertake water balance analysis and subsequently to quantify losses. The monthly water production from March 2016 to May 2016 was 13,444m3, 14,136m3, 16,137m3 and the monthly Water consumption was 12,840m3, 13, 705m3, 13,997m3 and the monthly water loss shows 3,176m3.which is about 7.26%.

3.5 Model Calibration and Validation

For the majority of water distribution models, calibration is an iterative procedure of parameter evaluation and modification, 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.

3.6 Hydraulic Model Calibration

Hydraulic behavior refers to flow conditions in pipes, valves and pressure/head levels at junctions and tanks. According to (EPA, 2005). for hydraulic model calibration Parameters that are typically set and adjusted 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.

3.7 Water Quality Model Calibration

Subsequent to the proper calibration of a hydraulic model, additional calibration of parameters in a water quality model may be required. '*The following parameters are used by water quality models that may require some degree of calibration*' (EPA, 2005):

- Initial Conditions: Defines the water quality parameter (concentration) at all locations 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 over the time period being simulated.

3.8 Calibration and Validation using Time-Series data

According to (Walski et al., 2003). 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.

3.9 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 (Walski et al., 2003).

3.9.1. 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.

3.9.2. 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.

***3.9.3. 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.

Chapter 4

4. Data analysis and Interpretation

4.1 Bacteriological Analysis.

All the samples were found to be negative for fecal coliform and positive for total coliform.

4.1.1 Coliform bacteria count. Coliform bacteria make it easy to detect and enumerate these bacteria when present in water. The term ‘coliforms’ traditionally referred to bacteria capable of growing at 37 °C in the presence of bile salts and of fermenting lactose. Testing the bacterial contaminants in water can be simplified by utilizing the presence of an indicator organism. An indicator organism may not necessarily pose a health risk but it can be easily isolated and enumerated, is present in large numbers, is more resistant to disinfection than pathogens, and does not multiply in water and distribution systems.

Traditionally, total coliform bacteria have been used to indicate the presence of fecal contamination; however, this parameter has been found to exist and grow in soil and water environments and is therefore considered a poor parameter for measuring the presence of pathogens (Stevens et al., 2003). Studies also show that due to their ability to grow in drinking water distribution systems and their unpredictable presence in water supplies during outbreaks of waterborne disease, the sanitary significance or quality of water is difficult to interpret in the presence of total coliforms (Stevens et al., 2003). An exception is Escherichia coli (E.coli), a thermo tolerant coliform, the most numerous of the total coliform group found in animal or human feces, rarely grows in the environment and is considered the most specific indicator of fecal contamination in drinking-water (WHO, 2004).The presence of E.Coli provides strong evidence of recent fecal contamination(WHO, 2004, Stevens et al., 2003). Temperature, producing acid and gas after 24¨ hour’s incubation. Using membrane filtrations provide at least a ‘presumptive’ result under normal laboratory conditions after 18 hours incubation.

4.1.2 Escherichia coli (Fecal coli form) count. Traditionally coliforms of fecal origin, characterized by E. coli, have been considered capable of growth and of expression of their fermentation properties at the higher temperature of 44 °C. This has resulted in them sometimes being referred to as ‘thermo tolerant’. As with coliform bacteria, a result that is at least ‘presumptive’ is available after 18 hours of incubation. These can provide a confirmed result after 18 hours. The detection of E.coli provides a reliable indicator of recent fecal contamination. The presence of other ‘thermo-tolerant’ coliform bacteria, particularly when detected in warmer tropical or subtropical waters, provides a less reliable indication of such contamination. However for many routine monitoring purposes an acceptable correlation between E.coli and faecal coliforms, as defined by thermo tolerance, is often assumed. Testing the bacterial contaminants in water can be simplified by utilizing the presence of an indicator organism. An indicator organism may not necessarily pose a health risk but it can be easily isolated and enumerated, is present in large numbers, is more resistant to disinfection than pathogens, and does not multiply in water and distribution systems (Gadgil, 1998). Traditionally, total coliform bacteria have been used to indicate the presence of fecal contamination; however, this parameter has been found to exist and grow in soil and water environments and is therefore considered a poor parameter for measuring the presence of pathogens (Stevens et al., 2003). Studies also show that due to their ability to grow in drinking water distribution systems and their unpredictable presence in water supplies during outbreaks of waterborne disease, the sanitary significance or quality of water is difficult to interpret in the presence of total coliforms (Stevens et al., 2003). An exception is Escherichia coli (E.coli), a thermo tolerant coliform, the most numerous of the total coliform group found in animal or human feces, rarely grows in the environment and is considered the most specific indicator of fecal contamination in drinking-water(WHO, 2004). The presence of E. coli provides strong evidence of recent fecal contamination (WHO, 2004, Stevens et al., 2003). The risk of coliform presence can depend on the health or sensitivity of the consumer. The risks of E.coli presence, slightly greater than WHO Guideline’s zero count per 100ml may be of only low or intermediate risk. According to IRC, 2002 as cited by Michael H., 2006 about risk classification for thermo tolerant coliforms or E. coli of rural water supplies shown below.

Table 4.1: water quality counts per 100mL and the associated risk

water quality counts per 100mL and the associated risk Count per 100ml	Risk Category
0	In conformity with WHO guide line
1-10	Low risk
11-100	Intermediate risk
101-1000	High risk

As the bacteriological test result (Appendix-P) shows most of the system nodes have low risk number of coliforms, therefore it is recommendable to apply chlorine to assure a quality drinking water.

4.1.2 Bottle test

Literatures recommend bulk coefficient has to be determined by performing bottle test in laboratory (Clark, 1996). 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. 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 starting 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 room being protected from direct sun light. The samples were pulled at designated times and measured which provides the experimental result for all tested samples. The first table contains three test results for samples collected from Menna water tank. Similarly the second table contains result for samples collected in the second period. The third table contains the result for samples collected from same locations.

For hydraulic analysis, simulation time step was 1, minute intervals over a 24 hour period. Along with hydraulic analysis, water age and thereby retention time was determined by setting initial estimates of water age in reservoirs and tanks. The calculated water age was used to fix maximum hour to run bottle test for the determination of bulk reaction coefficient in water quality laboratory. To determine bulk reaction coefficient; Water samples were stored in several amber bottles and kept at constant temperature. At several periods of time, a bottle was selected and analyzed for free chlorine. At the end of the test, the natural logarithms of the ratio of measured chlorine to initial chlorine (Ct/Co) values were plotted against time. The rate constant was the slope of the straight line

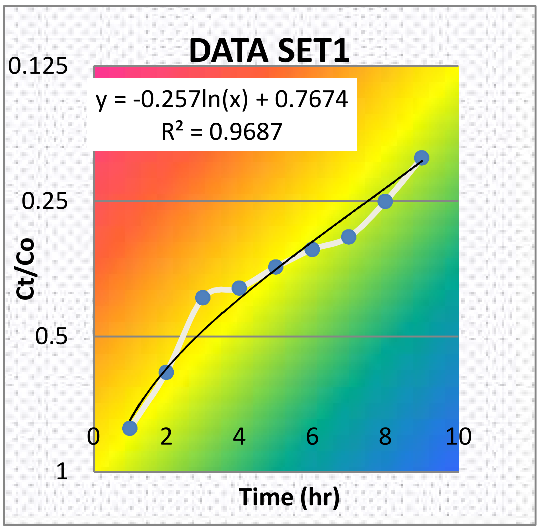


Figure 4.1: plot of residual chlorine tests 1

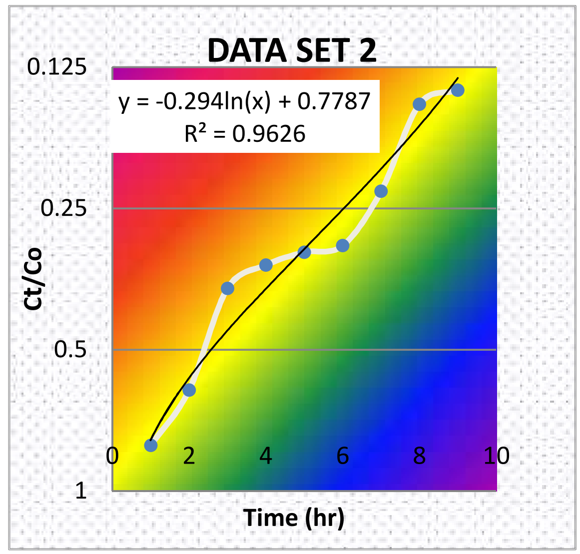


Figure 4.2: plot of residual chlorine test 2

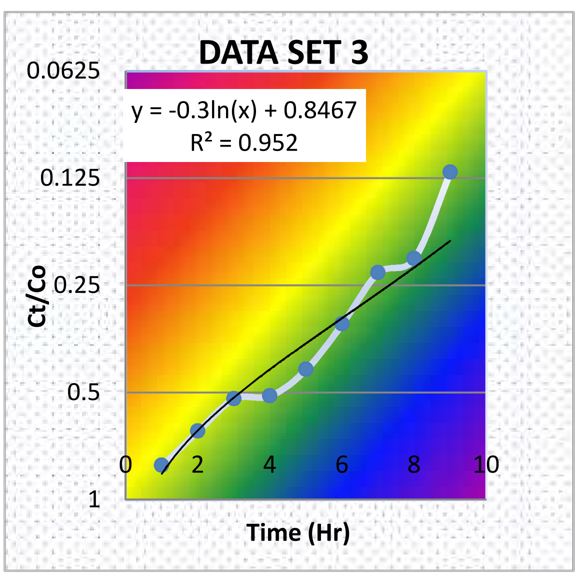


Figure 4.3: plot of residual chlorine test 3

Based on test results figure 4.1, figure 4.2 and figure 4.3 were plotted using the ratio of concentration at any time (Ct) to initial concentration (Co) as ordinate and time as abscissa. Best fit lines were drawn through the observed plots, where the slopes of lines represent the bulk reaction coefficients. Accordingly, for figure 4.4, figure 4.5 and figure 4.6 the slopes of the lines are -0.257, -0.294, and -0.30 respectively.

4.2 Model Representation

Frequently system maps are drawn as combination of various system components enclosed in water distribution system. It is common to include; Reservoirs, Tanks, Pipes, and Valves as much as possible and the resulting sketch fairly represent the actual water network. With little difference the real water distribution system represented as combination of nodes and links. Junctions, Reservoirs and Tanks are usually referred as nodes. Pipes and Valves are categorized as links.

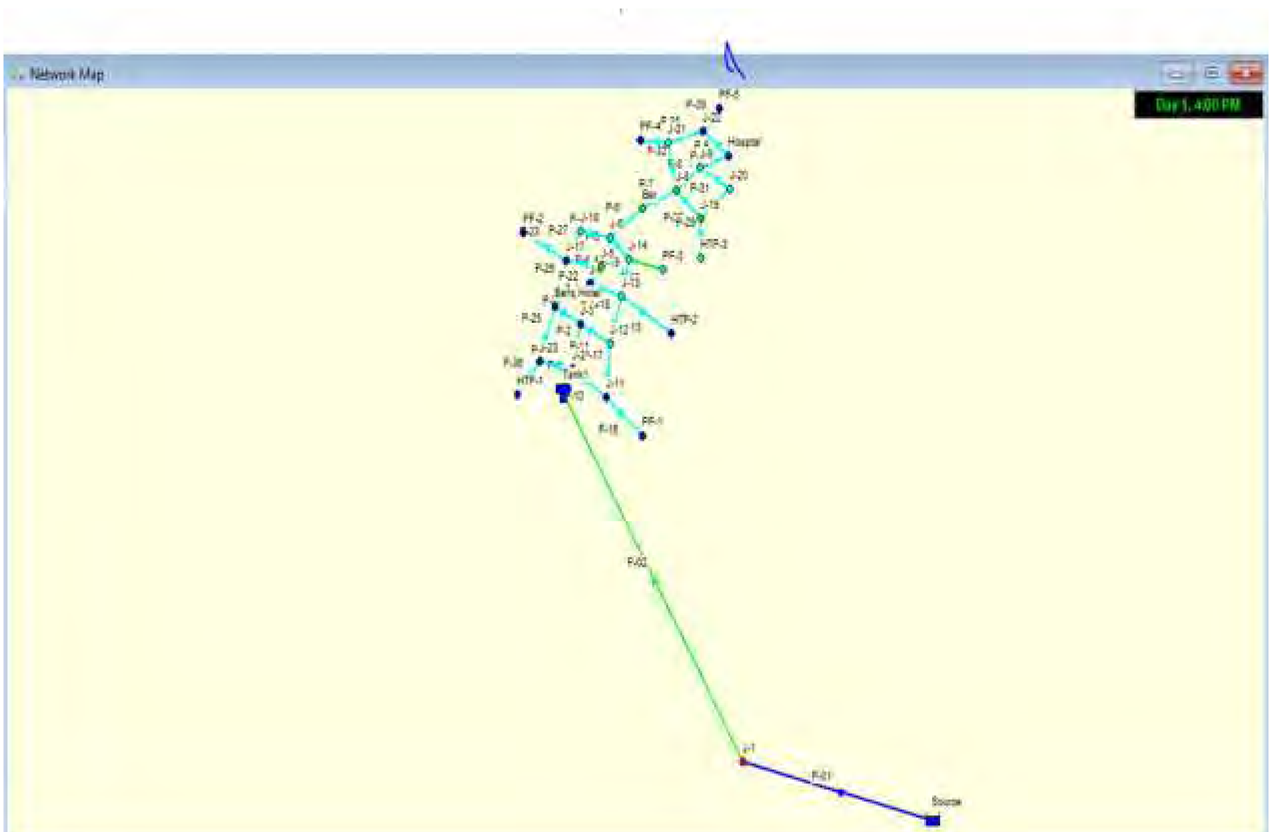


Figure 4.4: Illustrates layout of water distribution system

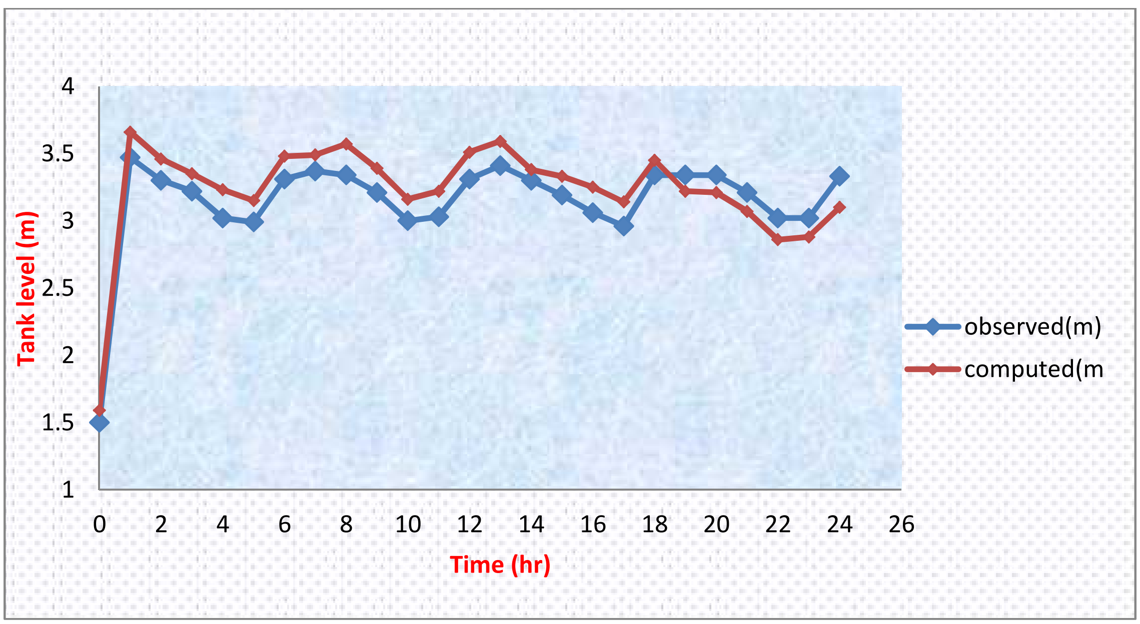
4.2.1. Model Calibration and Validation

The importance of a model is merely evident if a model result precisely reflects observed field values. Thus, to have a confidence on model result it needs to calibrate and validate a model. An effort to perform hydraulic and water quality model calibration and validation for this case study is presented as follows.

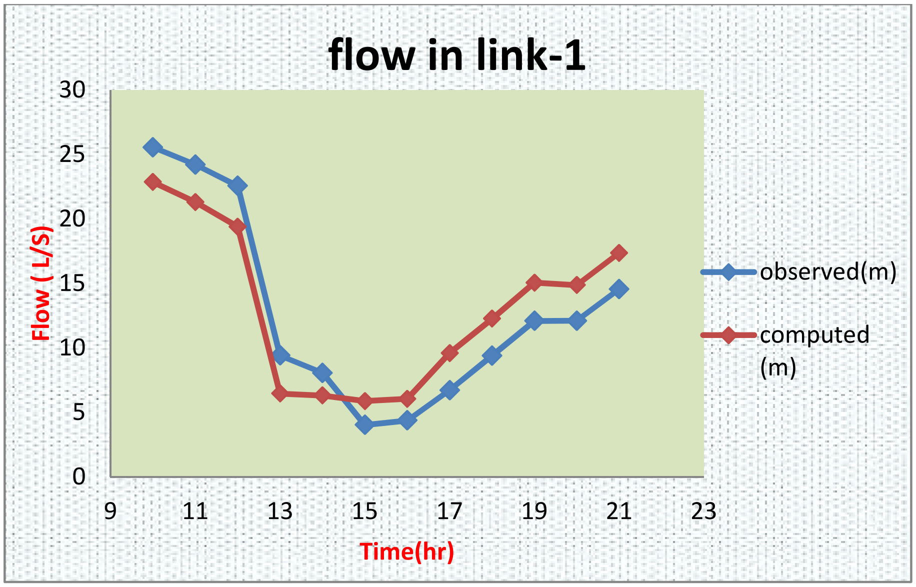
4.2.2 Hydraulic Calibration and Validation

Three sets of data were collected for hydraulic model calibration and validation effort. The first sets of data were used for model calibration purpose and detailed in Appendix N. While the second sets of data were used for model validation purpose and detailed in Appendix R. Figure 4.5 and Figure 4.6 illustrate plots of observed vs. computed values along with minimum and maximum difference.

4.2.2.1 Tank level and Link flow calibration



R^2=0.933794



R^2=0.936677

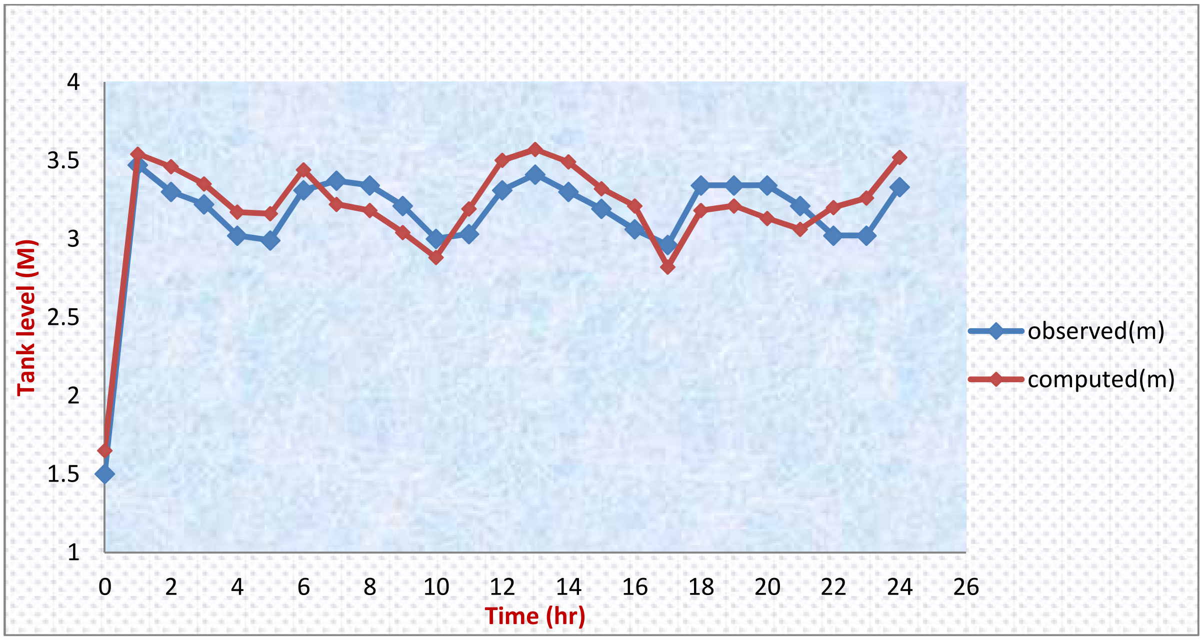
Figure 4.5: Model calibration using time series data

With regard to acceptable level of model calibration, there are different views. According to (Walski et al., 2003). 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 feels comfortable using the model to assist 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 (Walski et al., 2003). The model should accurately reproduce tank water level fluctuations to within 3–8 ft (1–2 m) for EPS runs and match treatment plant/pump station/ well flows to within 10–20 percent.

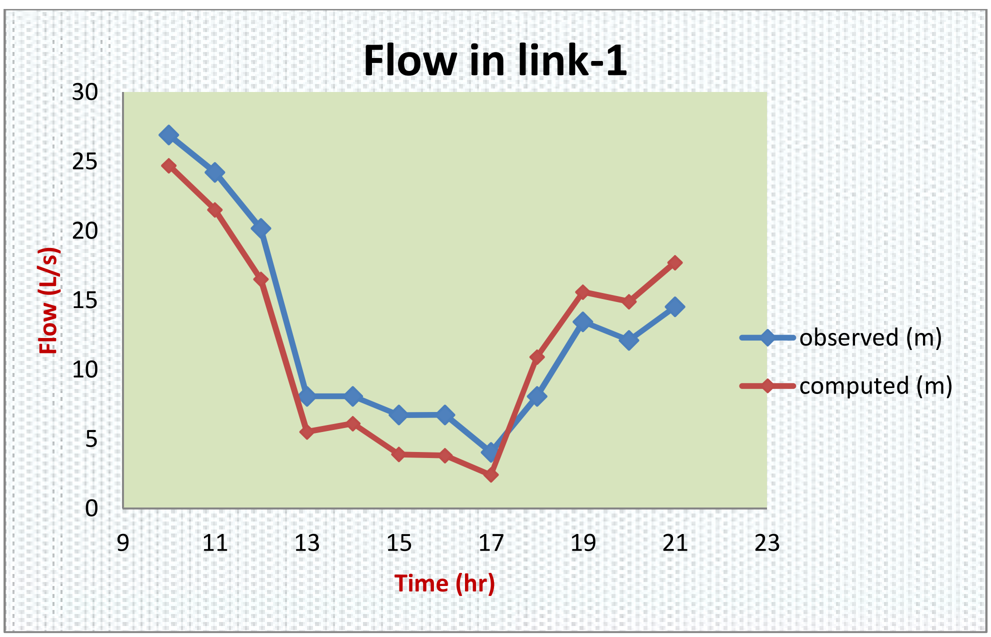
- The model should reproduce the pattern of observed disinfectant concentrations over the time. Samples were taken to an average error of roughly 0.1 to 0.2 mg/l, depending on the complexity of the system accordingly, model performance level was evaluated based on the above guide lines and presented as follows.

From data sets collected for calibration effort (24 data sets from tanks water level), 83.85 % of computed tank water level are found in the range of the guide line (1-3.5m).

4.2.2.2 Tank level and Link flow Validation



R^2=0.912477



R^2=0.936763

Figure 4.6: Model validation using time series data

From data sets collected for validation effort (24 data sets from tanks water level), 89.2 % of computed tank water level are in the range of the guide line (1-2).

4.2.3 Water Quality Model Calibration

Subsequent to hydraulic model calibration and validation, water quality model calibration has to be performed separately. To this effort data sets were collected from different part of water distribution system. Table 4.2 and Table 4.3 depict an attempt to water quality model calibration as guide line recommends: average error of roughly 0.1 to 0.2 mg/l (Walski et al., 2003).

The software can make each pipe's K_w be a function of the coefficient used to describe its roughness where *C* = Hazen-Williams C-factor, *e* = Darcy-Weisbach roughness=diameter, *n* = Manning roughness coefficient, and *F* = wall reaction - pipe roughness coefficient. The coefficient F must be developed from site-specific field measurements and will have a different meaning depending on which head loss equation is used. The advantage of using this approach is that it requires only a single parameter, *F*, to allow wall reaction coefficients to vary throughout the network in a physically meaningful way.

Table 4.2: Pipes roughness group

Group	Range	C value taken
Group 1(G.S.P)	(100-130)	120
Group 2(D.C.I)	(110-140)	130
Group 3(u-PVC)	(140-150)	140

Source; C-factors for various pipe materials, based on Lamont (1981).

Wall reactions depend on the bulk conditions, pipe dimensions (i.e., pipes with smaller diameters encourage greater solution/wall interaction and therefore greater reaction rates), and pipe wall condition The dependency of Kw and the reaction order on pipe material and condition (i.e., age, encrustation and corrosion) makes determining the coefficients difficult. Although conceptually Kw may be measured under ideal conditions (i.e., long isolated pipes, no connections, controlled flow, inline chlorine measurements), In real-world conditions such measurements are infeasible and therefore, models generally incorporate a calibrated Kw with initial estimates based on pipe roughness coefficients, flow velocity, and pipe diameter, This approach is practiced widely in the industry because wall decay coefficients vary greatly due to pipe condition (material, roughness, corrosion, and biofilms) and cannot be measured directly with in the distribution systems, but estimated by the following relation.

$R(C) = A/V*K_w*C^n$...2.18

Where $k_{W=}$wall reaction coefficients

A/V=surface area per unit volume with in a pipe.

The values for wall reaction coefficients are in the range of 0 to 5 ft /day (0 to 1.5 m/day). Because these values are difficult to measure, estimates can be based upon field concentration measurements and water quality simulation results as part of a calibration analysis. (Vasconcelos, Rossman, Grayman, Boulos, and Clark (1997)). Postulated that the wall reaction coefficient is related to pipe roughness according to the following equation:

$K_w = \alpha/Cc$...2.19

Where $k_{W=}$wall reaction coefficients

α=correlation coefficients

Cc=adjusted roughness value

Table 4.3: Water quality model calibration for first data sets

Time	Location	Mean Observed, mg/L	Computed Mean, mg/L	Deference
20	Tank	0.45	0.35	-0.1
21	PF-1	0.38	0.12	-0.26-
22	PF-2	0.25	0.12	-0.13
23	JN-8	0.18	0.14	0.02
24	JN-11	0.34	0.11	-0.23
25	JN-12	0.26	0.09	-0.17
26	HTP-1	0.13	0.07	-0.06
			RMS	**0.159**

Table 4.4: Water quality model calibration for second data sets

Time	Location	Mean Observed, mg/L	Computed Mean, mg/L	Deference
44	Tank	0.60	0.35	-0.25
45	PF-1	0.38	0.12	-0.26
46	PF-2	0.22	0.12	-0.09
47	JN-8	0.28	0.14	-0.14
48	JN-11	0.32	0.11	-0.21
49	JN-12	0.26	0.09	-0.17
50	HTP-1	0.17	0.07	-0.10
			RMS	**0.184**

As shown in Table 4.3 and Table 4.4, after adjustment of C-value of higher values of the truck mains the computed values are within an average error of 0.159 mg/l and 0.184mg/l to observed values. Hence, the model is well calibrated.

4.3. Simulation Results

Running single period simulation was helpful while performing preliminary model calibration. However, it should not be used for network assessment as water distribution system is likely to experience variations. Hence, only extended period simulation was exclusively used for entire model calibration and model assessment effort. Demand patterns used in simulating extended period simulation. An extended-period 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. For example, considering a storage tank with inadequate capacity operating within a system. The water level in the tank may be only slightly less at the end of each day than it was at the end of the previous day, which may go unnoticed when reviewing model results. If a duration of one or two weeks is used, the trend of the tank level dropping more and more each day will be more evident. Even in systems that have adequate storage capacity, simulation duration of 48 hours or longer can be helpful in better determining the tank draining and filling characteristics.

4.3.1 System Pressure

Pressure in water distribution system has to be maintained optimum; as to efficiently make water available to each demand category including at instances of firefighting (high withdrawal period) and as to reduce leakage as well as pipe breakage across the system. The former one is frequently achieved 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 (Swamee and et al., 2008).The minimum design nodal pressures are prescribed to discharge design flows onto the properties. It is based on population served, types of dwellings in the area, and 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 firefighting requirements. In case of high-rise buildings, booster pumps are installed in the water supply system to cater for the pressure head requirements. With these considerations, various codes recommend minimum ranging from 8 m to 20 m for residential areas.

Similarly, (M. Johnson and et al., 2009) recommend;

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

2. *Maximum static pressures during low demand periods:* typically at night, should be as low as practicable to minimize leakage. For flat areas a maximum static pressure in the range 30 to 45 m is desirable. However, there was no defined maximum and minimum pressure ranges set by the water supply office.

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 4.4 and detailed in Appendix –B

Table 4.5: Distribution of pressure at peak hour flow

Pressure(m)	Nodes (number)	Percentage
>70	2	6.25
60—70	1	3.13
50---60	4	12.50
40---50	7	21.88
30---40	5	15.63
20---30	7	21.88
15—20	3	9.38
<15	3	9.38

As indicated in the above table 4.5, 9.38% of nodes failed to satisfy minimum pressure requirement during peak hour flow. And 6.25% of nodes exceed maximum allowable pressure of 70m. While 84.37% of nodes are in the permissible pressure range of minimum 15m and maximum 70m. On the contrary minimum pressures are also observed mainly nodes situated near to tanks. In few instances nodes positioned in flat area of network are susceptible to low pressure. Whereas majority of nodes located in relative perfect loop region receive optimum pressure which doesn't violate minimum or maximum allowable pressure range (Appendix-B).

Table 4.6: Pressure distribution during low flow

Pressure(m)	Nodes(number)	Percentage
>70	7	21.88
60--70	6	18.75
50---60	2	6.25
40---50	4	12.50
30---40	4	12.50
20---30	5	15.63
15--20	2	6.25
<15	2	6.25

During low flow typically at mid-night distribution system of case study is marked by excessive pressure. As shown in table 4.6 and detailed in appendix C, 21.88% of nodes are liable to extremely high pressure. This figure is relatively high. Minimum pressures are also observed during low consumption period.6.25% of nodes received water with low pressure and few of them obtain no water. 71.87% of nodes are obtaining water of optimum pressure.

4.3.2 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 be adequately 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 to model the system. When laying out pressure zones, the elevations of the highest and lowest customers to be served are observed. If customers are less than Approximately 37 m apart vertically, then most likely a single pressure zone can serve them. If the elevation difference is significantly greater, more pressure zones are needed. In general, the elevation of the lowest and highest customers in the service area and the limits of the range of acceptable pressures are used to determine the HGL in a pressure zone. Equations 2.20 and 2.21 provide some useful guidelines for Selecting a HGL (Walski et al., 2003).

$$HGLmin > (Elevation\ of\ highest\ customer) + Cf\ Pmin \text{......................} 2.20$$

$$HGLmax < (Elevation\ of\ lowest\ customer) + Cf\ Pmax \text{........................} 2.21$$

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 2.20) ensures that the highest customer will have at least minimum pressure, while the second (Equation 2.21) 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 *Pmax*.

4.3.3 Scenarios for water quality degradations

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: water age and 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. Water quality variation is also observed as system pipes sizes are subjected to changes.

To obtain the dominant factors currently contributing to water quality deterioration specifically loss of residual chlorine in Alalbada-Erba Multi-village water supply distribution system of the following two possible scenarios were assessed.

i)Case1. Demand pattern

a. Water age (Residence time) at peak and low hour flow.

b. Residual chlorine at peak and low hour flow.

ii).Case 2. Pipe Sizes

a. Water age (Residence time) for existing and modified pipe sizes.

b. Residual chlorine for existing and modified pipe sizes.

i).Case 1. Demand pattern

Hourly water use in a given day is subjected to changes due to many factors. Frequently there is high water consumption at 8:00 PM and low water consumption at mid-night reading water matter for 24 hour. These variations lead to use of large pipes and storage tanks. However, large pipes and storage tanks have negative impacts on water quality.

It is about assessing water quality conditions in the distribution system of Alalbada Erba Multi-village water supply system under the influence of water demand variations. Water quality situation at peak hour flow and low hour flow was assessed taking water age or residence time and residual chlorine as water quality parameters.

a. Water age (Residence time) at peak and low hour flow

Analysis for water age was based on assumption that the distribution system was loaded with continuous flow. Thus, any findings for this parameter are limited to this assumption.

Table 4.7 and table 4.8 depict water age distribution at peak and low hour flow.

Table 4.7: Water age distribution at peak hour flow

Age(hr.)	Node(number)	Percentage
<4	1	3.13
4---6	0	0.00
6----8	0	0.00
8----10	2	6.25
10---12	21	65.63
12---14	5	15.63
14---16	5	15.63
>16	0	0.00

Table 4.8: Water age distribution at low hour flow

Age(hr.)	Node(number)	Percentage
<4	1	3.13
4---6	0	0.00
6----8	0	0.00
8----10	2	6.25
10---12	3	9.38
12---14	3	9.38
14---16	5	15.65
>16	18	56.25

As it is illustrated in table 4.8 and detailed in (Appendix E) at low hour flow majority of nodes 56.25% receive water with age exceeding 16 hours. Only 43.75% receive water with age less than 16 hours. While at peak hour flow nodes receiving water with age exceeding 16 hours is reduced to nil (Appendix D). There is significant difference between water age reaching nodes at high water consumption time and low water consumption time.

b. Residual chlorine at peak and low hour flow.

It had been long time since it was proved that microbiologically safe water would be supplied only while continuous disinfection prior to water distribution is possible along with letting distributed water with optimum residual chlorine to control recontamination in distribution system. Recontamination of water in a distribution system occurs due to various factors and their outcomes also differ. No matter what the reason is microbiologically unsafe water shall not be tolerated since its consequence is very appalling. The widespread strategy to tackle the probable recontamination is ensuring residual chlorine in water distribution system.

Recommended residual chlorine at the taps of the users usually lies between 0.2 mg/l to 0.5 mg/l (WHO, 2011; WHO, 1997). Table 4.8 and table 4.9 shows residual chlorine concentration distribution at low and peak hour flow.

Table 4.9: Residual chlorine distribution at low hour flow

Residual chlorine(mg/l)	Nodes(numbers)	percentages
0	2	6.25
0.01--0.1	0	0.00
0.1---0.2	2	6.25
0.2---0.5	24	75.00
>0.5	4	12.25

Table 4.10: Residual chlorine distribution at peak hour flow

Residual chlorine(mg/l)	Nodes(numbers)	Percentages
0	2	6.25
0.01--0.1	0	0.00
0.1---0.2	1	3.13
0.2---0.5	28	87.5
>0.5	1	3.13

As shown in table 4.9 and detailed in (Appendix F) at low hour flow only 75% of nodes receive water with residual chlorine between (0.2 mg/l - 0.5 mg/l). Around 6.25% of nodes obtain water of nil residual chlorine. While as it was shown in table 4.10 and detailed (Appendix F) at peak hour flow 87.5% of nodes obtains water with residual chlorine between (0.2 mg/l – 0.5 mg/l). This indicates the quality of water in distribution system of Alalbada-Erba Multi-village water supply scheme is much better at peak hour flow than low hour flow. At peak hours, when consumption is high, the water transits more rapidly in the network pipes and the chlorine has not enough time to react; thus chlorine residual concentration values remain quite high. At off-peak hours, when consumption is low (e.g. during the night), water velocity decreases and the chlorine concentration decaying process is more pronounced; thus chlorine residual concentration decreases down.

Case2: Pipe sizes

Case 2 is about assessing the impact of pipe geometry on water quality in distribution system of the water supply system. Two cases were assessed for modified pipe sizes taking water quality situation of existing pipe sizes as a reference. The first one is when actual pipe sizes decreased and the second one is when actual pipes are increased in size.

Table 4.11: Actual and modified pipe sizes

Link Id	Existing size(mm)	Reduced size(mm)	Increased size(mm)
01	DCI 300	DCI 250	DCI 350
02	u-PVC 200	u-PVC 150	u-PVC 250
03	u-PVC 150	u-PVC125	u-PVC200
04	u-PVC 150	u-PVC125	u-PVC200
05	u-PVC 150	u-PVC125	u-PVC200
06	u-PVC 150	u-PVC125	u-PVC200
07	u-PVC 125	u-PVC100	u-PVC150
08	u-PVC 100	u-PVC80	u-PVC 125
09	u-PVC 80	u-PVC65	u-PVC 100

a. Water age for actual and modified pipe sizes

Table 4.12, table 4.13 and table 4.14 indicates average water age distribution at nodes for actual pipe sizes, decreased pipe size and increased pipe size.

Table 4.12: Average water age distribution for existing pipe

Age(hr)	Node(number)	Percentage
<4	1	3.13
4—6	0	0.00
6---8	0	0.00
8----10	1	3.13
10----12	2	6.25
12---14	3	9.78
14---16	1	3.13
>16	24	75.00

Table 4.13: Average water age distribution for decreased pipe size

Age(hr.)	Node(number)	Percentage
<4	1	3.13
4—6	0	0.00
6---8	0	0.00
8----10	0	0.00
10----12	0	0.00
12---14	4	9.38
14---16	4	12.50
>16	24	76.80

Table 4.14: Average water age distribution for increased pipe size

Age(hr.)	Node(number)	Percentage
<4	1	1.69
4—6	0	0.00
6---8	0	0.00
8----10	0	0.00
10----12	2	6.25
12---14	5	12.63
14---16	3	9.38
>16	21	65.63

As illustrated in Table 4.12 (detailed in Appendix K), for the case of existing pipe sizes majority of nodes 75 % receive average water age exceeding 16 hours. Only 25 % of nodes obtain water with average age less than 16 hour. For the case of down sized pipes as shown in table 4.13 (detailed in Appendix L) 76.8% nodes obtain average water age exceeding 16 hours. 23.20% of nodes obtain water with average age less than 16 hour.

For the case of upper sized pipes as shown in table 4.14 (detailed in Appendix M) 65.63% nodes obtain average water age exceeding 16 hours. 34.37 % of nodes obtain water with average age less than 16 hour. There are no significant water age differences between actual and modified pipe sizes.

b. Residual chlorine for Existing and modified pipe sizes

Table 4.15, Table 4.16 and Table 4.17 depict distribution of minimum residual chlorine for actual pipe sizes, down sized pipes and upper sized pipes.

Table 4.15: Minimum residual chlorine distribution for existing pipe sizes

Residual chlorine(mg/l)	Nodes(numbers)	Percentages
0	2	6.25
0.01--0.1	0	0.0
0.1---0.2	3	9.38
0.2—0.5	25	78.13
> 0.5	2	6.25

Table 4.16: Minimum residual chlorine distribution for decreased pipe sizes

Residual chlorine(mg/l)	Nodes(numbers)	Percentages
0	2	6.25
0.01--0.1	0	0.0
0.1---0.2	5	15.63
0.2—0.5	24	75
> 0.5	1	3.13

Table 4.17: Minimum residual chlorine distribution for increased pipe sizes

Residual chlorine(mg/l)	Nodes(numbers)	Percentages
0	2	6.25
0.01--0.1	0	0.0
0.1—0.2	2	6.25
0.2—0.5	28	87.5
> 0.5	0	0.00

As shown in table 4.15 (detailed in Appendix H), table 4.16 (detailed in Appendix J) and table 4.17 (detailed in Appendix I) only 78.13%, 75% and 87.5 %, of nodes obtain water with residual chlorine in recommended range of (0.2 – 0.5 mg/l) the whole day for Existing , down sized and upper sized pipes respectively. While 6.25%, 6.25% and 6.25% of nodes of actual pipe sizes decreased size and increased size pipes receive water with nil residual chlorine. For the case of reduced pipes size percent obtaining optimum residual chlorine is decreased in comparison to existing pipe sizes. This is because for the case of increased pipe size percent receiving residual chlorine in the range of (0.2 – 0.5 mg/l) is increased in comparison to actual pipe sizes. Generally, both scenarios are identified as factors currently contributing to water quality deterioration in distribution system of Alalbada Erba Multi-village water supply system.

However, **Case 1** is the dominant factor due to observed significant variations of assessed parameters (water age and residual chlorine) whereas **Case 2** is identified as minor factor contributing to water quality deterioration since significant variation is observed only for the case of residual chlorine assessment. **Scenario 2** showed no significant variation for the case of water age.

4.3.4 Proposed Solution to Improve Water quality deterioration.

Establishing booster disinfection stations is an intervention required to improve water quality deterioration in distribution system of Alalbada Erba Multi-village water supply system. If an intrusion occurs in the storage tank, the water draining from it to reach consumers does not contain sufficient disinfectant to inactivate significant numbers of organisms (disinfectant concentration is depleted due to the long storage tank residence time) as the main cause of system water quality deterioration is the presence of reservoir tanks (Grayman and Clark, 1993). Maintaining disinfectant residuals at this location could improve protection to consumers. The booster station doesn’t add chlorine directly in the tank but to the outflow water when the tank is draining and the design increases the disinfectant concentration and the residual chlorine maintained in the storage tank resulting more efficient inactivation of pathogens present in the tank.

Below table shows possible booster disinfection systems and concentration of chlorine dosage at each station.

Table 4.18: Chlorine dosage concentration for proposed booster disinfection system

Selected Nodes	Chlorine Dosage (mg/L)
Water Tank upper to gravity line	1.25mg/l

Table 4.18 illustrates the distribution of residual chlorine during low hour flow for actual system and corresponding residual chlorine concentration for proposed system with intervention (with intervention of booster disinfection system).

Table 4.19: Distribution of residual chlorine at low hour flow for existing and proposed system

Increased pipe size			Existing pipe size		
Residual chlorine(mg/l)	Nodes(numbers)	percentage	Residual chlorine(mg/l)	Nodes(numbers)	Percentage
0	2	6.25	0	2	6.25
<0.2	2	6.25	<0.2	3	9.38
0.2	0.0	18.40	0.2	1	3.13
>0.2	28	87.5	>0.2	26	84.38

As shown in table 4.19 establishments of booster disinfection system significantly improves residual chlorine concentration. At low hour flow in actual system only (84.38%) nodes obtain water with residual chlorine of 0.2 mg/l and above. For new proposed system with booster disinfection system at low hour flow nodes obtain water with residual chlorine of (0.2 mg/l) and above increased to (87.50%).This shows the intervention work improves the minimum residual chlorine concentration.

Table 4.20: Residual chlorine concentration after Booster station injection of dosage 0.5mg/l.

Increased pipe size			Existing pipe size		
Min.Residual chlorine(mg/l)	Nodes(numbers)	Percentage	Min.Residual chlorine(mg/l)	Nodes(numbers)	Percentage
0	0	0	0	2	6.25
<0.2	2	6.25	<0.2	3	9.38
>0.2	30	93.75	> 0.2	27	84.38
Total	32				

As shown in table 4.20 establishments of booster disinfection system significantly improves residual chlorine concentration, which is at low hour flow in existing system only (84.38%) nodes obtain water with residual chlorine of 0.2 mg/l and above. But ,a new proposed system with booster disinfection system at low hour flow nodes obtain water with residual chlorine of 0.2 mg/l and above are increased to (93.75%) with the dosage of 0.5mg/l .This shows intervention work significantly improves the minimum residual chlorine concentration.

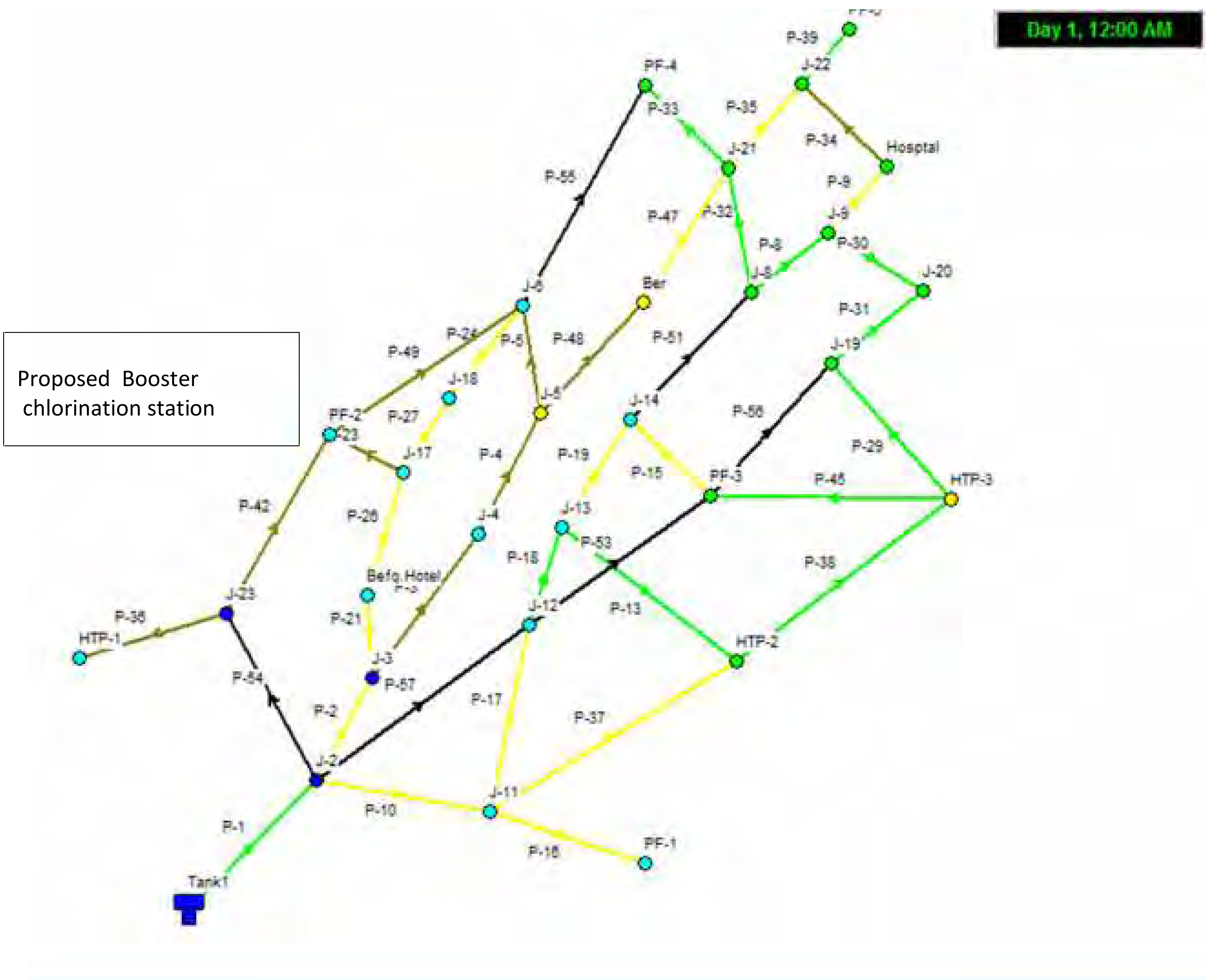


Figure 4.7: Location of Booster chlorination

Chapter 5

5. Conclusions and Recommendation

5.1 Conclusions

To assess the current situation of Alalbada-Erba water supply network system, modeling was found to be appropriate technique and accordingly modeling efforts were carried out for case study of supply system. The base of the assessment was evaluating hydraulic performance of the distribution system as well as analyzing issues of water quality transformations in the distribution system based on results of calibrated and validated model. With intended objectives the study was undertaken and it has come with significant outcomes. 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 receive water with low pressure and under some circumstances risk of obtaining no water is observed because the pressure in the distribution system is below acceptable minimum requirement. On the contrary, about one third of the distribution system is prone to undesirable pressures which exceed maximum allowable pressure. As a result, the distribution system is exposed to risks of high leakage and repeated pipe breakage during low flows.

Along with this, hydraulic modeling results revealed the existence of both design and 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 design problems. In general, the simulated hydraulic result indicated that the current hydraulic performance of Alalbada-Erba supply system is not satisfactory. But it doesn't mean that the subsystem is not functional. Rather the frequency of service interruption is relatively high. This interruption is partly contributing for the urgent water shortage in the system.

Variation in water use (water demand pattern) is the dominant factor currently contributing to water quality decline in distribution system. Disinfection modeling result showed that the distribution system lacks the ability to fully distribute microbiologically safe water due to absence of minimum allowable residual chlorine at least once per day. 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.

In general, from water quality point of view modeling results showed that the supply system is currently performing in a poor situation. Particularly, the subsystem is not maintaining the minimum permissible residual chlorine in which there is a recontamination of water during distribution.

5.2 Recommendation

To improve the current situation of Alalbada-Erba supply system, both design and operational modifications are necessary. From the study undertaken and modeling result the following sets of recommendations are drawn:

- To permanently adjust 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) apart vertically, then most likely a single pressure zone can serve them. If the elevation difference is significantly greater, more pressure zones are needed.
- Adjustment or implementations of pressure reducing valve devices which decrease pressure are recommended as solution to control occurrences of maximum pressures for isolated parts of network.
- 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 preset value as to guarantee allowable minimum pressure for isolated parts of network. Since most of the times these pressure reducing valves fails after some times therefore, it is recommendable to construct pressure breaking tank using local construction material.
- Adjustment 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.

- Installing new pipes as to eliminating dead-ends and letting water to route in the system is essential. Such an effort contributes reduction in water age. An attempt to eliminating dead- ends of case study by installing new pipes showed significant reduction of water age.
- Booster disinfection station has to be established at Reservoir 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.
- The model is good in predicting hydraulic and water quality parameters in distribution system. However, early and late simulation results were found less agree with observed measured values. Thus, a model result for early and late simulation is not recommended to be used for system assessment

This research has generated important results. Pressure, Water Age and Residual Chlorine were modeled to provide deeper understanding of the current situation of the water distribution network system. Subsequently valuable suggestions were drawn based on findings to improve the situation. To have more complete understanding, future works are suggested that focus on:

- Hydraulic modeling effort of the subsystem shall consider different situations; which entails impacts of excessive pressure in the system since most of the nodes obtain water by gravity system.
- Water quality modeling shall include modeling of disinfection by-product as to fully understand water quality transformations in water distribution system. Impacts of various development activities on performance of water distribution system

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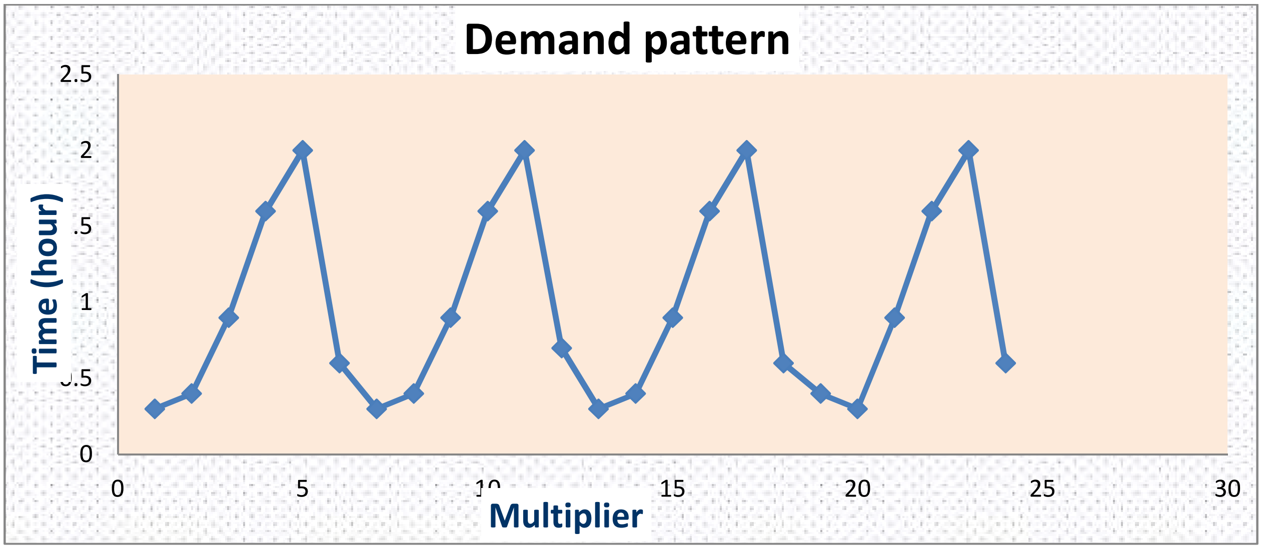
Appendix A: Demand pattern



Appendix B: Pressure at peak flow

Node	X-Coordinate	Y -Coordinate	Elevation	Pressure m
J-1	280191.07	4410349.30	1416	285.63
J-2	278496.00	4413498.20	1287	15.32
J-3	278576.60	4413850.00	1289	11.48
J-4	278671.70	4414179.90	1279	19.39
J-5	278776.90	4414309.70	1225	71.90
J-6	278871.60	4414549.50	1262	33.68
Bridge	279189.90	4414787.80	1228	61.08
J-8	279524.10	4414927.50	1233	54.21
J-9	279764.30	4415111.40	1244	41.81
Hospital	280052.30	4415203.90	1245	38.14
J-11	278836.00	4413265.60	1278	23.80
J-12	278872.81	4413703.27	1273	27.22
J-13	278981.80	4414081.80	1258	39.02
J-14	279056.50	4414373.50	1257	38.68
J-23	278171.86	4413552.34	1281	19.95
Be. Hotel	278322.10	4413992.90	1278	22.35
J-17	278432.80	4414368.40	1274	22.90

J-18	278578.92	4414598.48	1261	34.79
J-19	279774.90	4414706.00	1230	56.02
J-20	280059.81	4414933.76	1238	47.68
J-21	279451.30	4415310.90	1243	42.99
J-22	279796.32	4415401.92	1242	40.30
PF-1	279195.20	4412963.30	1277	23.80
HTP-2	279476.20	4413784.90	1248	45.91
PF-3	279395.96	4414298.05	1248	46.61
HTP-3	279775.85	4414383.75	1229	55.03
PF-4	279170.30	4415324.80	1231	52.04
PF-2	278007.40	4414592.20	1270	25.71
HTP-1	277947.04	4413293.80	1270	28.49
PF-5	279954.93	4415580.99	1239	42.08
Source	282083.05	4409879.80	1799	0.00
Tank	278399.80	4413303.20	1304	3.19

Appendix C: Pressure at low flow

Node	X-Coordinate	Y -Coordinate	Elevation	Pressure m
J-1	280191.07	4410349.30	1416	285.65
J-2	278496.00	4413498.20	1287	19.95
J-3	278576.60	4413850.00	1289	17.81
J-4	278671.70	4414179.90	1279	27.65
J-5	278776.90	4414309.70	1225	81.54
J-6	278871.60	4414549.50	1262	44.44
Bridge	279189.90	4414787.80	1228	77.94
J-8	279524.10	4414927.50	1233	72.79
J-9	279764.30	4415111.40	1244	61.69
Hospital	280052.30	4415203.90	1245	60.48
J-11	278836.00	4413265.60	1278	28.91
J-12	278872.81	4413703.27	1273	33.79
J-13	278981.80	4414081.80	1258	48.55
J-14	279056.50	4414373.50	1257	49.44
J-23	278171.86	4413552.34	1281	25.85

Be. Hotel	278322.10	4413992.90	1278	28.80
J-17	278432.80	4414368.40	1274	32.54
J-18	278578.92	4414598.48	1261	45.45
J-19	279774.90	4414706.00	1230	75.70
J-20	280059.81	4414933.76	1238	67.67
J-21	279451.30	4415310.90	1243	62.70
J-22	279796.32	4415401.92	1242	63.42
PF-1	279195.20	4412963.30	1277	29.84
HTP-2	279476.20	4413784.90	1248	58.31
PF-3	279395.96	4414298.05	1248	58.36
HTP-3	279775.85	4414383.75	1229	76.55
PF-4	279170.30	4415324.80	1231	74.47
PF-2	278007.40	4414592.20	1270	36.44
HTP-1	277947.04	4413293.80	1270	36.66
PF-5	279954.93	4415580.99	1239	66.32
Source	282083.05	4409879.80	1799	0.00
Tank	278399.80	4413303.20	1304	3.33

Appendix D: Peak hour flow water Age with in the distribution system

Node	X-Coordinate	Y -Coordinate	Elevation	Age hours
J-1	280191.07	4410349.30	1416	9
J-2	278496.00	4413498.20	1287	9
J-3	278576.60	4413850.00	1289	10
J-4	278671.70	4414179.90	1279	10
J-5	278776.90	4414309.70	1225	10
J-6	278871.60	4414549.50	1262	10
Bridge	279189.90	4414787.80	1228	10
J-8	279524.10	4414927.50	1233	10
J-9	279764.30	4415111.40	1244	11
Hospital	280052.30	4415203.90	1245	11
J-11	278836.00	4413265.60	1278	11
J-12	278872.81	4413703.27	1273	11
J-13	278981.80	4414081.80	1258	11

J-14	279056.50	4414373.50	1257	11
J-23	278171.86	4413552.34	1281	11
Be. Hotel	278322.10	4413992.90	1278	11
J-17	278432.80	4414368.40	1274	11
J-18	278578.92	4414598.48	1261	11
J-19	279774.90	4414706.00	1230	11
J-20	280059.81	4414933.76	1238	11
J-21	279451.30	4415310.90	1243	11
J-22	279796.32	4415401.92	1242	11
PF-1	279195.20	4412963.30	1277	11
HTP-2	279476.20	4413784.90	1248	12
PF-3	279395.96	4414298.05	1248	12
HTP-3	279775.85	4414383.75	1229	12
PF-4	279170.30	4415324.80	1231	13
PF-2	278007.40	4414592.20	1270	13
HTP-1	277947.04	4413293.80	1270	13
PF-5	279954.93	4415580.99	1239	14
Source	282083.05	4409879.80	1799	00
Tank	278399.80	4413303.20	1304	14

Appendix E: low hour flow water Age with in the distribution system

Node	X-Coordinate	Y -Coordinate	Elevation	Age hours
J-1	280191.07	4410349.30	1416	8
J-2	278496.00	4413498.20	1287	9
J-3	278576.60	4413850.00	1289	11
J-4	278671.70	4414179.90	1279	12
J-5	278776.90	4414309.70	1225	12
J-6	278871.60	4414549.50	1262	13
Bridge	279189.90	4414787.80	1228	13
J-8	279524.10	4414927.50	1233	13
J-9	279764.30	4415111.40	1244	16
Hospital	280052.30	4415203.90	1245	16
J-11	278836.00	4413265.60	1278	16

J-12	278872.81	4413703.27	1273	16
J-13	278981.80	4414081.80	1258	16
J-14	279056.50	4414373.50	1257	17
J-23	278171.86	4413552.34	1281	17
Be. Hotel	278322.10	4413992.90	1278	17
J-17	278432.80	4414368.40	1274	17
J-18	278578.92	4414598.48	1261	17
J-19	279774.90	4414706.00	1230	17
J-20	280059.81	4414933.76	1238	17
J-21	279451.30	4415310.90	1243	17
J-22	279796.32	4415401.92	1242	17
PF-1	279195.20	4412963.30	1277	17
HTP-2	279476.20	4413784.90	1248	17
PF-3	279395.96	4414298.05	1248	17
HTP-3	279775.85	4414383.75	1229	17
PF-4	279170.30	4415324.80	1231	17
PF-2	278007.40	4414592.20	1270	18
HTP-1	277947.04	4413293.80	1270	18
PF-5	279954.93	4415580.99	1239	18
Source	282083.05	4409879.80	1799	00
Tank	278399.80	4413303.20	1304	20

Appendix F: Residual chlorine at low hour flow with in the distribution system

Node	X-Coordinate	Y -Coordinate	Elevation	Chlorine mg/L
J-1	280191.07	4410349.30	1416	0.00
J-2	278496.00	4413498.20	1287	0.30
J-3	278576.60	4413850.00	1289	0.50
J-4	278671.70	4414179.90	1279	0.30
J-5	278776.90	4414309.70	1225	0.27
J-6	278871.60	4414549.50	1262	0.12
Bridge	279189.90	4414787.80	1228	0.57
J-8	279524.10	4414927.50	1233	0.15
J-9	279764.30	4415111.40	1244	0.27

Hospital	280052.30	4415203.90	1245	0.27
J-11	278836.00	4413265.60	1278	0.30
J-12	278872.81	4413703.27	1273	0.27
J-13	278981.80	4414081.80	1258	0.57
J-14	279056.50	4414373.50	1257	0.27
J-23	278171.86	4413552.34	1281	0.30
Be. Hotel	278322.10	4413992.90	1278	0.29
J-17	278432.80	4414368.40	1274	0.27
J-18	278578.92	4414598.48	1261	0.27
J-19	279774.90	4414706.00	1230	0.27
J-20	280059.81	4414933.76	1238	0.27
J-21	279451.30	4415310.90	1243	0.24
J-22	279796.32	4415401.92	1242	0.25
PF-1	279195.20	4412963.30	1277	0.57
HTP-2	279476.20	4413784.90	1248	0.27
PF-3	279395.96	4414298.05	1248	0.27
HTP-3	279775.85	4414383.75	1229	0.27
PF-4	279170.30	4415324.80	1231	0.24
PF-2	278007.40	4414592.20	1270	0.27
HTP-1	277947.04	4413293.80	1270	0.27
PF-5	279954.93	4415580.99	1239	0.25
Source	282083.05	4409879.80	1799	0.00
Tank	278399.80	4413303.20	1304	0.30

Appendix G: Residual chlorine at peak hour flow with in the distribution system

Node	X-Coordinate	Y -Coordinate	Elevation	Chlorine mg/L
J-1	280191.07	4410349.30	1416	0.00
J-2	278496.00	4413498.20	1287	0.28
J-3	278576.60	4413850.00	1289	0.28
J-4	278671.70	4414179.90	1279	0.28
J-5	278776.90	4414309.70	1225	0.28
J-6	278871.60	4414549.50	1262	0.28

Bridge	279189.90	4414787.80	1228	0.37
J-8	279524.10	4414927.50	1233	0.25
J-9	279764.30	4415111.40	1244	0.25
Hospital	280052.30	4415203.90	1245	0.25
J-11	278836.00	4413265.60	1278	0.18
J-12	278872.81	4413703.27	1273	0.28
J-13	278981.80	4414081.80	1258	0.25
J-14	279056.50	4414373.50	1257	0.25
J-23	278171.86	4413552.34	1281	0.18
Be. Hotel	278322.10	4413992.90	1278	0.18
J-17	278432.80	4414368.40	1274	0.28
J-18	278578.92	4414598.48	1261	0.27
J-19	279774.90	4414706.00	1230	0.25
J-20	280059.81	4414933.76	1238	0.25
J-21	279451.30	4415310.90	1243	0.25
J-22	279796.32	4415401.92	1242	0.25
PF-1	279195.20	4412963.30	1277	0.28
HTP-2	279476.20	4413784.90	1248	0.25
PF-3	279395.96	4414298.05	1248	0.59
HTP-3	279775.85	4414383.75	1229	0.35
PF-4	279170.30	4415324.80	1231	0.35
PF-2	278007.40	4414592.20	1270	0.28
HTP-1	277947.04	4413293.80	1270	0.28
PF-5	279954.93	4415580.99	1239	0.35
Source	282083.05	4409879.80	1799	0.00
Tank	278399.80	4413303.20	1304	0.28

Appendix H: Residual chlorine for existing pipe size

Node	X-Coordinate	Y -Coordinate	Elevation	Chlorine mg/L
J-1	280191.07	4410349.30	1416	0.00
J-2	278496.00	4413498.20	1287	0.19
J-3	278576.60	4413850.00	1289	0.18
J-4	278671.70	4414179.90	1279	0.28
J-5	278776.90	4414309.70	1225	0.28
J-6	278871.60	4414549.50	1262	0.28
Bridge	279189.90	4414787.80	1228	0.28
J-8	279524.10	4414927.50	1233	0.28
J-9	279764.30	4415111.40	1244	0.28
Hospital	280052.30	4415203.90	1245	0.27
J-11	278836.00	4413265.60	1278	0.51
J-12	278872.81	4413703.27	1273	0.28
J-13	278981.80	4414081.80	1258	0.28
J-14	279056.50	4414373.50	1257	0.26
J-23	278171.86	4413552.34	1281	0.28
Be. Hotel	278322.10	4413992.90	1278	0.28
J-17	278432.80	4414368.40	1274	0.28
J-18	278578.92	4414598.48	1261	0.28
J-19	279774.90	4414706.00	1230	0.28
J-20	280059.81	4414933.76	1238	0.20
J-21	279451.30	4415310.90	1243	0.26
J-22	279796.32	4415401.92	1242	0.27
PF-1	279195.20	4412963.30	1277	0.28
HTP-2	279476.20	4413784.90	1248	0.28
PF-3	279395.96	4414298.05	1248	0.57
HTP-3	279775.85	4414383.75	1229	0.27
PF-4	279170.30	4415324.80	1231	0.26
PF-2	278007.40	4414592.20	1270	0.28
HTP-1	277947.04	4413293.80	1270	0.28
PF-5	279954.93	4415580.99	1239	0.27
Source	282083.05	4409879.80	1799	0.00
Tank	278399.80	4413303.20	1304	0.29

Appendix I: Residual chlorine for increased pipe size

Node	X-Coordinate	Y -Coordinate	Elevation	Chlorine mg/L
J-1	280191.07	4410349.30	1416	0.00
J-2	278496.00	4413498.20	1287	0.18
J-3	278576.60	4413850.00	1289	0.28
J-4	278671.70	4414179.90	1279	0.28
J-5	278776.90	4414309.70	1225	0.28
J-6	278871.60	4414549.50	1262	0.28
Bridge	279189.90	4414787.80	1228	0.27
J-8	279524.10	4414927.50	1233	0.27
J-9	279764.30	4415111.40	1244	0.27
Hospital	280052.30	4415203.90	1245	0.27
J-11	278836.00	4413265.60	1278	0.28
J-12	278872.81	4413703.27	1273	0.28
J-13	278981.80	4414081.80	1258	0.27
J-14	279056.50	4414373.50	1257	0.27
J-23	278171.86	4413552.34	1281	0.28
Be. Hotel	278322.10	4413992.90	1278	0.28
J-17	278432.80	4414368.40	1274	0.28
J-18	278578.92	4414598.48	1261	0.24
J-19	279774.90	4414706.00	1230	0.27
J-20	280059.81	4414933.76	1238	0.26
J-21	279451.30	4415310.90	1243	0.26
J-22	279796.32	4415401.92	1242	0.26
PF-1	279195.20	4412963.30	1277	0.28
HTP-2	279476.20	4413784.90	1248	0.27
PF-3	279395.96	4414298.05	1248	0.18
HTP-3	279775.85	4414383.75	1229	0.26
PF-4	279170.30	4415324.80	1231	0.26
PF-2	278007.40	4414592.20	1270	0.27
HTP-1	277947.04	4413293.80	1270	0.28
PF-5	279954.93	4415580.99	1239	0.26
Source	282083.05	4409879.80	1799	0.00
Tank	278399.80	4413303.20	1304	0.29

Appendix J: Minimum Residual chlorine for increased pipe size with booster injection

Node	X-Coordinate	Y -Coordinate	Elevation	Chlorine mg/L
J-1	280191.07	4410349.30	1416	0.00
J-2	278496.00	4413498.20	1287	0.11
J-3	278576.60	4413850.00	1289	0.11
J-4	278671.70	4414179.90	1279	0.13
J-5	278776.90	4414309.70	1225	0.27
J-6	278871.60	4414549.50	1262	0.20
Bridge	279189.90	4414787.80	1228	0.32
J-8	279524.10	4414927.50	1233	0.32
J-9	279764.30	4415111.40	1244	0.56
Hospital	280052.30	4415203.90	1245	0.57
J-11	278836.00	4413265.60	1278	0.57
J-12	278872.81	4413703.27	1273	0.58
J-13	278981.80	4414081.80	1258	0.58
J-14	279056.50	4414373.50	1257	0.58
J-23	278171.86	4413552.34	1281	0.59
Be. Hotel	278322.10	4413992.90	1278	0.59
J-17	278432.80	4414368.40	1274	0.59
J-18	278578.92	4414598.48	1261	0.59
J-19	279774.90	4414706.00	1230	0.59
J-20	280059.81	4414933.76	1238	0.59
J-21	279451.30	4415310.90	1243	0.59
J-22	279796.32	4415401.92	1242	0.60
PF-1	279195.20	4412963.30	1277	0.60
HTP-2	279476.20	4413784.90	1248	0.60
PF-3	279395.96	4414298.05	1248	0.60
HTP-3	279775.85	4414383.75	1229	0.60
PF-4	279170.30	4415324.80	1231	0.60
PF-2	278007.40	4414592.20	1270	0.61
HTP-1	277947.04	4413293.80	1270	0.61
PF-5	279954.93	4415580.99	1239	0.61
Source	282083.05	4409879.80	1799	0.00
Tank	278399.80	4413303.20	1304	0.61

Appendix K: Residual chlorine for decreased pipe size

Node	X-Coordinate	Y -Coordinate	Elevation	Chlorine mg/L
J-1	280191.07	4410349.30	1416	0.00
J-2	278496.00	4413498.20	1287	0.19
J-3	278576.60	4413850.00	1289	0.29
J-4	278671.70	4414179.90	1279	0.29
J-5	278776.90	4414309.70	1225	0.28
J-6	278871.60	4414549.50	1262	0.28
Bridge	279189.90	4414787.80	1228	0.28
J-8	279524.10	4414927.50	1233	0.28
J-9	279764.30	4415111.40	1244	0.28
Hospital	280052.30	4415203.90	1245	0.28
J-11	278836.00	4413265.60	1278	0.29
J-12	278872.81	4413703.27	1273	0.29
J-13	278981.80	4414081.80	1258	0.28
J-14	279056.50	4414373.50	1257	0.28
J-23	278171.86	4413552.34	1281	0.19
Be. Hotel	278322.10	4413992.90	1278	0.29
J-17	278432.80	4414368.40	1274	0.29
J-18	278578.92	4414598.48	1261	0.28
J-19	279774.90	4414706.00	1230	0.28
J-20	280059.81	4414933.76	1238	0.18
J-21	279451.30	4415310.90	1243	0.28
J-22	279796.32	4415401.92	1242	0.27
PF-1	279195.20	4412963.30	1277	0.29
HTP-2	279476.20	4413784.90	1248	0.28
PF-3	279395.96	4414298.05	1248	0.18
HTP-3	279775.85	4414383.75	1229	0.28
PF-4	279170.30	4415324.80	1231	0.27
PF-2	278007.40	4414592.20	1270	0.28
HTP-1	277947.04	4413293.80	1270	0.51
PF-5	279954.93	4415580.99	1239	0.17
Source	282083.05	4409879.80	1799	0.00
Tank	278399.80	4413303.20	1304	0.29

Appendix L: Average water age for existing pipe size

Node	X-Coordinate	Y -Coordinate	Elevation	Age hours
J-1	280191.07	4410349.30	1416	9.37
J-2	278496.00	4413498.20	1287	11.90
J-3	278576.60	4413850.00	1289	12.37
J-4	278671.70	4414179.90	1279	12.93
J-5	278776.90	4414309.70	1225	13.61
J-6	278871.60	4414549.50	1262	13.08
Bridge	279189.90	4414787.80	1228	13.19
J-8	279524.10	4414927.50	1233	13.40
J-9	279764.30	4415111.40	1244	15.71
Hospital	280052.30	4415203.90	1245	17.99
J-11	278836.00	4413265.60	1278	17.00
J-12	278872.81	4413703.27	1273	19.00
J-13	278981.80	4414081.80	1258	19.00
J-14	279056.50	4414373.50	1257	20.00
J-23	278171.86	4413552.34	1281	20.00
Be. Hotel	278322.10	4413992.90	1278	20.00
J-17	278432.80	4414368.40	1274	20.00
J-18	278578.92	4414598.48	1261	20.00
J-19	279774.90	4414706.00	1230	20.00
J-20	280059.81	4414933.76	1238	20.00
J-21	279451.30	4415310.90	1243	20.00
J-22	279796.32	4415401.92	1242	20.00
PF-1	279195.20	4412963.30	1277	20.00
HTP-2	279476.20	4413784.90	1248	20.00
PF-3	279395.96	4414298.05	1248	20.00
HTP-3	279775.85	4414383.75	1229	20.00
PF-4	279170.30	4415324.80	1231	20.00
PF-2	278007.40	4414592.20	1270	20.00
HTP-1	277947.04	4413293.80	1270	20.00
PF-5	279954.93	4415580.99	1239	20.00
Source	282083.05	4409879.80	1799	0.00
Tank	278399.80	4413303.20	1304	20.00

Appendix M: Average water age for decreased pipe size

Node	X-Coordinate	Y -Coordinate	Elevation	Age hours
J-1	280191.07	4410349.30	1416	14.89
J-2	278496.00	4413498.20	1287	15.25
J-3	278576.60	4413850.00	1289	16.89
J-4	278671.70	4414179.90	1279	12.00
J-5	278776.90	4414309.70	1225	18.60
J-6	278871.60	4414549.50	1262	18.98
Bridge	279189.90	4414787.80	1228	19.81
J-8	279524.10	4414927.50	1233	19.88
J-9	279764.30	4415111.40	1244	20.00
Hospital	280052.30	4415203.90	1245	20.00
J-11	278836.00	4413265.60	1278	20.00
J-12	278872.81	4413703.27	1273	20.00
J-13	278981.80	4414081.80	1258	20.00
J-14	279056.50	4414373.50	1257	20.00
J-23	278171.86	4413552.34	1281	20.00
Be. Hotel	278322.10	4413992.90	1278	20.00
J-17	278432.80	4414368.40	1274	20.00
J-18	278578.92	4414598.48	1261	20.00
J-19	279774.90	4414706.00	1230	20.00
J-20	280059.81	4414933.76	1238	20.00
J-21	279451.30	4415310.90	1243	20.00
J-22	279796.32	4415401.92	1242	20.00
PF-1	279195.20	4412963.30	1277	20.00
HTP-2	279476.20	4413784.90	1248	20.00
PF-3	279395.96	4414298.05	1248	20.00
HTP-3	279775.85	4414383.75	1229	20.00
PF-4	279170.30	4415324.80	1231	20.00
PF-2	278007.40	4414592.20	1270	20.00
HTP-1	277947.04	4413293.80	1270	20.00
PF-5	279954.93	4415580.99	1239	20.00
Source	282083.05	4409879.80	1799	0.00
Tank	278399.80	4413303.20	1304	20.00

Appendix N: Average water age for increased pipe size

Node	X-Coordinate	Y -Coordinate	Elevation	Age hours
J-1	280191.07	4410349.30	1416	11.65
J-2	278496.00	4413498.20	1287	11.90
J-3	278576.60	4413850.00	1289	12.68
J-4	278671.70	4414179.90	1279	13.14
J-5	278776.90	4414309.70	1225	13.67
J-6	278871.60	4414549.50	1262	13.83
Bridge	279189.90	4414787.80	1228	13.84
J-8	279524.10	4414927.50	1233	14.22
J-9	279764.30	4415111.40	1244	15.53
Hospital	280052.30	4415203.90	1245	15.41
J-11	278836.00	4413265.60	1278	17.99
J-12	278872.81	4413703.27	1273	18.57
J-13	278981.80	4414081.80	1258	18.74
J-14	279056.50	4414373.50	1257	18.77
J-23	278171.86	4413552.34	1281	18.94
Be. Hotel	278322.10	4413992.90	1278	18.96
J-17	278432.80	4414368.40	1274	19.03
J-18	278578.92	4414598.48	1261	19.03
J-19	279774.90	4414706.00	1230	19.06
J-20	280059.81	4414933.76	1238	19.09
J-21	279451.30	4415310.90	1243	19.20
J-22	279796.32	4415401.92	1242	19.24
PF-1	279195.20	4412963.30	1277	19.24
HTP-2	279476.20	4413784.90	1248	19.25
PF-3	279395.96	4414298.05	1248	19.30
HTP-3	279775.85	4414383.75	1229	19.37
PF-4	279170.30	4415324.80	1231	19.57
PF-2	278007.40	4414592.20	1270	19.62
HTP-1	277947.04	4413293.80	1270	19.73
PF-5	279954.93	4415580.99	1239	19.74
Source	282083.05	4409879.80	1799	0.00
Tank	278399.80	4413303.20	1304	19.93

Appendix O: Bulk coefficients data tests

1st bottle test data set		
Date	**April 7,2016**	
Time	S ample	**Observed residual chlorine(mg/l)**
1:30 pm	Clear water Tank	0.8
2:00 pm	JN-2	0.30
2:30 PM	JN-4	0.60
3:00 pm	Hospital	0.20
3:30 PM	JN-8	0.25
4:00 PM	JN-11	0.41
4:30 pm	Be .Hotel	0.39
5:00 PM	PF-3	0.35
5:30 PM	HTP3	0.32
2nd bottle test data set		
Date	**April 8,2016**	
Time	S ample	**Observed residual chlorine(mg/l)**
8:00 AM	Clear water Tank	0.8
8:30 AM	JN-2	0.3
9:00 AM	JN-4	0.61
9:30 AM	Hospital	0.31
10 :00AM	JN-8	0.14
10:30 AM	JN-11	0.37
11:00 AM	Be .Hotel	0.3
11:30 AM	PF-3	0.31
12:00 AM	HTP3	0.15
3rd bottle test data set		
Date	**may 10,2015**	
Time	S ample	**Observed residual chlorine(mg/l)**
1:00 PM	Clear water Tank	0.8
1:30 PM	JN-2	0.23
2:00 PM	JN-4	0.65
2:30 PM	Hospital	0.23
3:00 PM	JN-8	0.43
3:30 PM	JN-11	0.52
4: 00 PM	Be .Hotel	0.21
4: 30 PM	PF-3	0.51
5:30 PM	HTP3	0.02

Appendix-P: Bacteriological Test Results

S.N	Sampling Nodes	Location			Field data			Remark
		Easting (m)	Northing (m)	Altitude (m)	date-time	E-coli/100ml	Total coliform/ 100ml	
1	Source	282083	4409880	1799	07/04/16, 2:30 pm	Nil	3	Low risk
2	Jnc. 1	280191	4410349	1416	07/04/16, 3:45pm	Nil	2	‘’
3	Pf-1	279195	4412963	1277	07/04/16, 4:30pm	Nil	4	‘’
4	HTp. 1	277947	4413294	1270	07/04/16, 5:45pm	Nil	3	‘’
5	Jnc. 3	278577	4413850	1289	07/04/16, 6:30pm	Nil	8	‘’
6	Jnc.13	278982	4414082	1258	08/04/16, 8:00pm	Nil	3	‘’
7	Jnc. 12	278873	4413703	1273	08/04/16, 8:30am	Nil	2	‘’
8	BefHotel	278322	4413993	1278	08/04/16, 9:15am	Nil	7	‘’
9	Pf-2	278007	4414592	1270	08/04/16, 9:50am	Nil	5	‘’
10	Jnc. 8	279524	4414928	1233	08/04/16, 10:30am	Nil	2	‘’
11	Jnc. 5	278777	4414310	1225	08/04/16, 11:25am	Nil	3	‘’
12	Jnc. 21	279451	4415311	1243	08/04/16, 12:00am	Nil	4	‘’
13	Pf-4	279170	4415325	1231	08/04/16, 1:30pm	Nil	7	‘’
14	Hospital	280052	4415204	1245	08/04/16, 1:55pm	Nil	3	‘’
15	Jnc. 8	279524	4414928	1233	08/04/16, 2:30pm	Nil	3	‘’
16	Jnc. 14	279057	4414374	1257	08/04/16, 3:00pm	Nil	2	‘’
17	Tank	278400	4413303	1304	08/04/16, 3:35pm	Nil	3	‘’

Appendix Q: Water Tank level Calibration data's

Time (hour)	observed(m)	computed(m	difference(m)
0	1.5	1.59	0.09
1	3.47	3.66	0.19
2	3.3	3.46	0.16
3	3.22	3.35	0.13
4	3.02	3.23	0.21
5	2.99	3.15	0.16
6	3.31	3.48	0.17
7	3.37	3.49	0.12
8	3.34	3.57	0.23
9	3.21	3.39	0.18
10	3	3.16	0.16
11	3.03	3.22	0.19
12	3.31	3.51	0.2
13	3.41	3.59	0.18
14	3.3	3.38	0.08
15	3.19	3.33	0.14
16	3.06	3.25	0.19
17	2.96	3.14	0.18
18	3.34	3.45	0.11
19	3.34	3.22	-0.12
20	3.34	3.21	-0.13
21	3.21	3.07	-0.14
22	3.02	2.86	-0.16
23	3.02	2.88	-0.14
24	3.33	3.1	-0.23

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

R^2=0.933794

Appendix R: Pipe Link no-4 Flow Calibration data's

Time (hour)	observed(m)	computed (m)	differences(l/s)
10	25.56	22.86	-2.7
11	24.21	21.31	-2.9
12	22.56	19.4	-3.16
13	9.42	6.46	-2.96
14	8.07	6.31	-1.76
15	4.03	5.88	1.85
16	4.38	6.05	1.67
17	6.73	9.6	2.87
18	9.41	12.27	2.86
19	12.1	15.06	2.96
20	12.11	14.87	2.76
21	14.56	17.37	2.81

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

R^2=0.936677

Appendix S: Pipe Link no-4 Flow Validation data's

Time (hour)	observed(m)	computed(m)	Difference
0	1.5	1.65	0.15
1	3.47	3.54	0.07
2	3.3	3.46	0.16
3	3.22	3.35	0.13
4	3.02	3.17	0.15
5	2.99	3.16	0.17
6	3.31	3.44	0.13
7	3.37	3.22	-0.15
8	3.34	3.18	-0.16
9	3.21	3.04	-0.17
10	3	2.88	-0.12
11	3.03	3.19	0.16

12	3.31	3.5	0.19
13	3.41	3.57	0.16
14	3.3	3.49	0.19
15	3.19	3.32	0.13
16	3.06	3.21	0.15
17	2.96	2.82	-0.14
18	3.34	3.18	-0.16
19	3.34	3.21	-0.13
20	3.34	3.13	-0.21
21	3.21	3.06	-0.15
22	3.02	3.2	0.18
23	3.02	3.26	0.24
24	3.33	3.52	0.19

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

R^2=0.912477

Appendix T: Water Tank level Validation data’

Time(hour)	observed (m)	computed (m)	differences(l/s)
10	26.9	24.7	-2.2
11	24.21	21.51	-2.7
12	20.17	17.51	-2.66
13	8.07	10.63	2.56
14	8.07	10.03	1.96
15	6.72	3.89	-2.83
16	6.73	3.8	-2.93
17	4.04	2.41	-1.63
18	8.07	10.91	2.84
19	13.45	15.59	2.14
20	12.11	14.9	2.79
21	14.52	17.71	3.19

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

R^2=0.932503

Appendix U: Comparisons of Water Quality of standards of the spring.

Parameter	**Alalbada-Erba spring**	**WHO Guideline**	**Recommended for Ethiopia**
PH(pH scale)	6.69	6.5 – 8.5	5.0-9.5
TDS (mg/l)	93	1000	2000
Total hardness $CaCO_3^{(mg/l)}$	65	300	600
Turbidity (NTU)	Nil	5	5
Fl (mg/l)	0.12	1.5	4
Na (mg/l)	15	200	
Fe (mg/l)	0.08	0.3	3
Mn(mg/l)	0.05	0.1	
So4(mg/l)	3	400	600
Chloride(mg/l)	Nil	250	800
Nitrate(mg/l)	8.80	10	40