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

WATER QUALITY MODELING OF WATER SUPPLY DISTRIBUTION SYSTEM

(The case of Tuffa-Bulbula 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 Birru Dula Habte Wold

Advised by Dr. Agizew Nigussie

Addis Ababa
Ethiopia
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List of Symbols & Acronyms

MOWE	Ministry of Water and Energy
TBTWSS	Tuffa- Bulbula 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, equipments 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, Tuffa-Bubula 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 deterioration 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*, *Tuffa-Bulbula, Ethiopia.***

Chapter 1

1. Introductions

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 “potable 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 vulnerable to water-borne diseases are those who use polluted drinking water sources. The report from UNICEF (2010), in the world 884 million people use unimproved drinking water sources in 2010, and in 2015 estimates about 672 million people will still using unimproved drinking water sources. The WHO (2000) revealed that seventy five percent of all diseases in developing countries arise from polluted drinking water.

Chlorine disinfection is of great importance 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 [II].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 is 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 [II] The decay constant values vary with water quality, water temperature, flow rate, pipe diameter, pipe materials, the type of inner coating, and pipe roughness.

Water Quality is a term that incorporates a large number of aspects of both potable and non-potable water. 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.

1.1 Existing Water Supply Sources

The first borehole was drilled in 1971E.C for Bulbula town water supply. This borehole is located at the center of the town at geographic coordinate of 461494mE and 823231mN. This borehole has not served the town for more than two years due to high concentration of dissolved solids, which can be easily sensible by ordinary test. The second borehole of depth 15m was drilled along Bulbula stream at geographic coordinate of 461145mE and 853314mN at which considerable recharge from previous Bulbula stream flow was possible, which dilutes ground water along the stream. The nearer the borehole or shallow/dug well to Bulbula stream the lesser the concentration of dissolved solids and fluoride ions. In 1995 E.C East Shoa Water Resource Office constructed three hand dug wells fitted with hand pumps near the dried-up Bulbula stream to lessen acute shortage of potable water The hand dug wells have depths of 16-25m.

The water utility taken as a case study is supplying the multivillage along to the Bulbula town. The distribution network is branched and grid types and it has one source. The length of the carrier and distribution networks is about 32 km with diameters from 25 mm to

400 mm of DCI (7.8% carrier lines with diameter from 150 mm to 250mm of DCI type and 82% of PVC pipes of distribution line with diameter from 50 mm to 200 mm). The network consists of more than one type of pipe such as (PVC – DCI – GSP), one 500m^3 water storage tank, about 600 customer service house connection and more than 40 public fountains both along the gravity line of 30km and in the distribution network of the town which was commissioned in the year 2009.

To alleviate the water shortage the Oromia Water Bureau has conducted study and design on Tuffa spring in March 2005[I] .This spring source is located in West Arsi Zone of Munessaa woreda and “Tuffa” Kebeles. The spring source is a cluster of at least six-spring eyes. The spring is emerging from acidic volcanic rocks, particularly ignimbrite and basaltic lava flow. It is a fault line spring and discharges adjacent to Langano lake.

The existence of hot and cold springs adjacent to one another is attributed to the existence of local fractures and/or faults and long stretched faults that allow recharge to groundwater at different locations. Currently, the supply system is from Tuffa spring which has a capacity of discharging 100l/s.Therefore, a great emphasis has been given for **“Tuffa” cold springs as** a source of water supply for Bulbula town and surrounding kebeles's and the rural communities existing along the main line. The spring has a maximum distance of 50km from the source to the Bulbula town which is fed by gravity. As discussed above, the springs have high yields with many eyes within an area of less than 0.1Km^2.

There are two types of springs with respect to water temperature and most of the springs are cold water springs with few of them being hot. The cold springs have temperature varying from 31 – 32 ^{O}C while the hot spring has 39.5 ^{O}C. The recharge to the springs is considered to be from Ketar River through eastern margin of rift fractures (escarpment) trending in NNE–SSW direction and Ketar River is losing flow to these fractures downstream which surges floods to the spring.

1.2 Description of the Study Area

Tuffa Bulbula Town multi-village water supply scheme is located in East Shoa Zone at 461145m Easting (UTM), and 853314m Northing (UTM) 184km far from Addis Ababa in southeast direction.

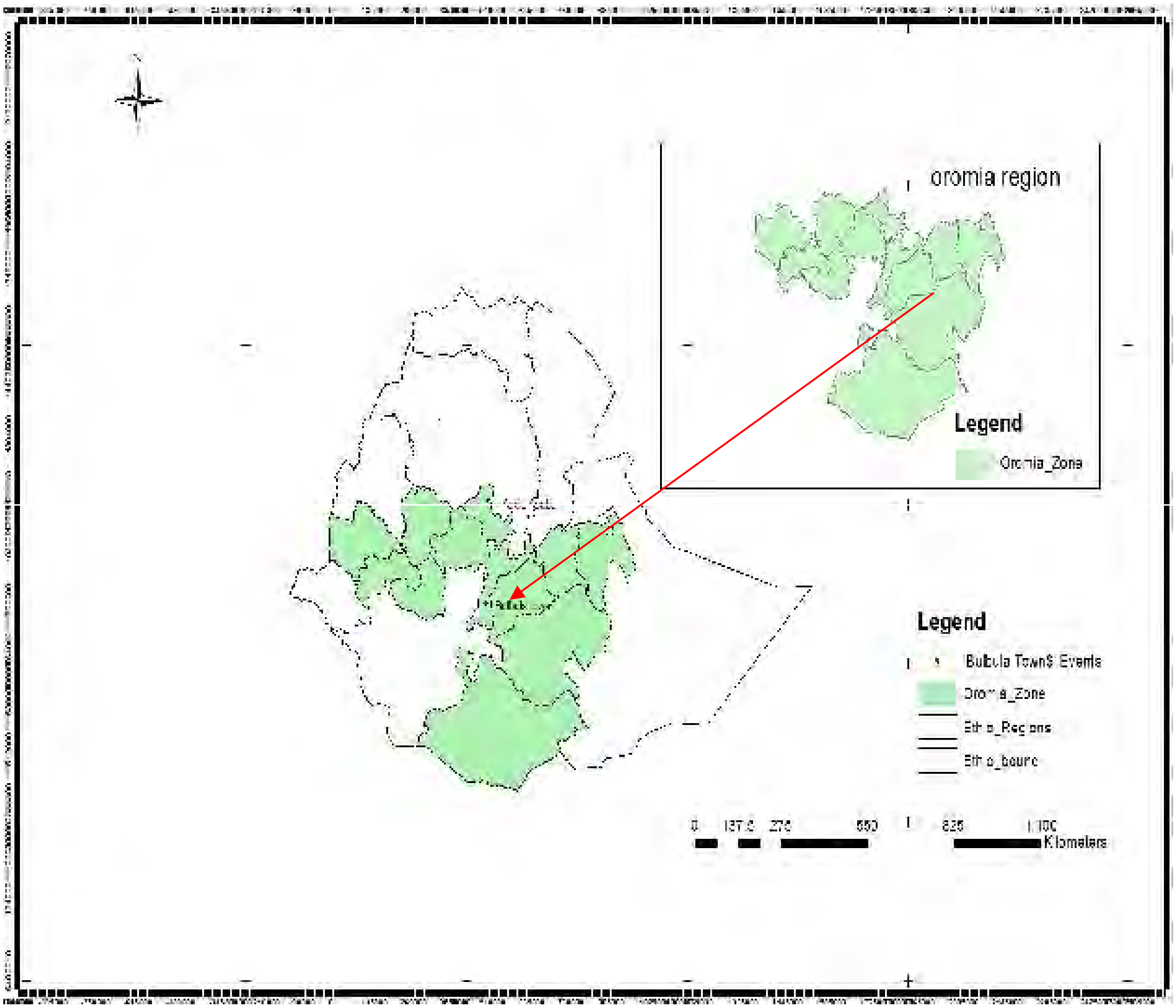


Figure 1.1Location map of Tuffa Bulbula multi-village water supply scheme

1.3 Population size of the water supply scheme

The population that is served by Bulbula water supply system resides in 15 villages with a projected total population of 40,811.The town's water supply system pipe is about 32 km long and provides water by gravity for dwellers in 10 villages and other additional five villages.

Table 1.1.Tuffa-Bulbula multi-village population Size

Year	2007	2012	2017	2022	2027
Bulbula Town	6,729	8,506	10,702	13,147	15,995
Rural village	16,758	18,776	20,812	22,799	24,816
Total	**23,487**	**27,282**	**31,514**	**35,946**	**40,811**

Source: (OWRB Design report, 2005)

1.4 Climatic Condition

Based on the meteorological data of Langano station, the mean annual rainfall of the area is 632mm while mean daily temperature is 22^0C. Bulbula town is situated in the Central Ethiopia Rift floor and hence characterized by arid to semi arid climate.

1.5 Surveillance Report of Health Center

According to the information obtained from Bulbula Health Centre, water related diseases are prevalent in the area due to poor water supply usage and personal hygiene.

Table 1.2 Health center reports of 10 top diseases of the year 2012/13

No	Description	Number of cases	Percentage of occurrences (%)
1	AFI	1952	23
2	URTI	1025	12.2
3	Malarias	787	9.3
4	Premiums	704	8.3
5	Helminthiness	592	7
6	Inflectional diseases	372	4.4
7	UTI	354	4.2
8	Trumta	334	4
9	STI	324	3.9
10	Dihiaria	321	3.8

Source; Bulbula Health Center, 2004 E.C.

1.6 Statement of problem

Water treatment has contributed to the prevention of waterborne disease and to safeguard human health in drinking water. The recharge to the springs is considered to be from Ketar River through eastern margin of rift fractures (escarpment) trending in

NNE–SSW direction and Ketar River is losing flow to these fractures downstream which floods to the spring source **(Appendix-A)**. As Tuffa spring is located adjacent to hot spring that is used by the community as traditional medication purposes, and since the wet-well is located at the downstream of the spring sources there is a possibility of being contaminated that induces coliforms and Ecoli micro-organisms in to the collection chamber and the disinfection work should be done and as the chlorine bulk decay rate test shows at constant temperatures there is potentially loss of chlorine and there are instants in which water shortage is accelerated by undesirable pressure within distribution system. leakage and pipe failure as well as provision of insufficient supply are situations which propagate water shortage within distribution system.

Plate 1 Source of Tuffa-Bulbula water supply

1.7 General objective:

The main aim of this research was to assess hydraulic and water quality performance of the water distribution system of Tuffa-Bulbula Multi-village water supply distribution system;.which includes proper application of chlorine, pipe net-work system leakage control, pump operation and maintenance.

1.7.1 Specific objective

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

- To assess the residual chlorine concentration at peak & low flow condition.
- To identify major water quality deterioration factors.
- To check whether the water from Tuffa spring is microbiologically safe to drink.
- To check whether the current water supply interruption is accelerated by occurrences of excessive pressures within the system.
- To recommend feasible improvement measures.

1.7.3 Research Questions

- How is the quality of the water supply scheme of Tuffa-Bulbula Multi-Village is assured?
- Is there appropriate disinfectant at the water supply Service?
- Is there significant chlorine loss in the multi-village water supply scheme?
- How the minimum chlorine is residual is maintained by initial concentration?
- Is there dependable residual chlorine in the system?

1.8 Scope of the Study

This study specifically focused on assessing factors affecting water Quality and hydraulic performance of water supply schemes. The study area is limited to Tuffa-Bulbula Multi-village Water Supply scheme. The study covered 15 rural kebeles and Bulbula town: the problem of enough budgets, 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, etc). Rather, attention has been given to identifying major factors water quality deterioration, 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 characteristics.

Chapter 2

2. Literature Review

This chapter presents a summary of authentic and relevant 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 Tuffa-Bulbula 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 are described by conservation of mass and energy and water quality is based upon conservation of mass at a node and advective 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 [4]. Here the pipe flow equation formulation is summarized. For a junction that connects two or more pipes, conservation of mass is written as:

$$\sum^{Pipes} Q_i - U = 0 \quad \text{.......} \quad 2.1$$

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

U = water used at node (L^3/T)

The term for accumulation of water at nodes is required to describe stored and withdrawn water from tanks, while xtended period simulation is regarded [26].

$$\sum^{Pipes} Q_i - U - ds/dt = 0 \quad \text{.......} \quad 2.2$$

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

2.1.2 Conservation of Energy

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

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

$$Z_1+P_1/\quad +v_1^2/2g+\ h_p= Z_2+ P_2/\quad + v_2^2/2g+\ \ h_l+\ \ h_m \text{.....................................} 2.3$$

Where, Z = elevation (L)

P = Pressure ($M/L/T_2$)

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

V = velocity (L/T)

g = gravitational acceleration constant (L/T_2)

h_p = head added at pump (L)

h_L = head loss in pipes (L)s

h_m = head loss due to minor losses (L)

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

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 [15]. While chlorine travels through the distribution system, it reacts with different materials inside the pipe. Reactions are impacted by the surrounding conditions due to the availability of reacting substances.

Reactants are present in the bulk water and may also occur at high concentrations on the surface of the pipe. Bulk reactions predominate in relatively less turbulent water. On the other hand, wall or surface reactions predominate in turbulent flows. Water quality transport mainly occurs by advective transport in which the constituent (chlorine) moves with water in the direction of flow with the magnitude of the main velocity component. In other words, advection transport is principles of conservation of mass coupled with reaction kinetics carrying constituent (chlorine) along with the flow of water [18].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 [27], advective transport within a pipe can be represented with the next equation.

$$\frac{\partial c_i}{\partial t} = \frac{\partial c_i}{\partial A_i} * \frac{\partial c_i}{\partial x} + \theta(C_i) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.4$$

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

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

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

θ(Ci) = reaction term (M/L3/T)

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

2.1.5 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.

$$C_{out} = \frac{\Sigma QC_i,n_i+U_i}{\Sigma Q_i} \text{……………………………...............................2.5}$$

Where, C_{OUTj} = 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 [20]. 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.

$$\frac{dc}{dt} = \frac{Q}{V}(Ci,np(t)-cK)+(cK)\text{........………………………………2.6}$$

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;

$$\frac{dc}{dt}K = \theta(c_k)\text{…………………………………….......2.7}$$

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;

$$\theta(C) = \pm K.C^{n} \text{………………………………………………………}2.8$$

Where, θ(C) = reaction term (M/L3/T)

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

C = concentration (M/L3)

n = reaction rate order constant

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

$$C_t = C_o * e^{-kt} \quad \ldots\ldots 2.9$$

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

$$T = -\left(\frac{0.693}{T}\right) \quad \ldots\ldots 2.10$$

2.1.8 Pipe Wall Reactions

While flowing through pipes, dissolved substances can be transported to the pipe wall and react with materials, such as corrosion products or biofilm that are on or close to the wall. The amount of wall area available for reaction and the rate of mass transfer between the bulk fluid and the wall also will influence the overall rate of this reaction. The surface area per unit volume, which for a pipe equals 2 divided by the radius, determines a mass transfer coefficient, the value of which depends on the molecular diffusivity of the reactive species and on the Reynolds number of the flow. For first-order kinetics, the rate of a pipe wall reaction can be expressed as:

$$r = \frac{2Kf*Kw*C}{R(k_w+K_f)} \quad \ldots\ldots 2.11$$

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;

$$K = K_b + \frac{2Kf*Kw*C}{R\,(k_w+K_f)} \quad \ldots\ldots 2.12$$

2.1.9 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.[13] 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;

$$\theta\ (C) = \pm KC \quad \text{.........} 2.13$$

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.

$$K = K_b + \frac{Kf*Kw}{R(k_w+K_f)} \quad \text{.........} 2.14$$

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.

$$K_f = \frac{SHd}{D} \quad \text{……………………………………………………………}2.15$$

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.

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

Where, Re = Reynolds number

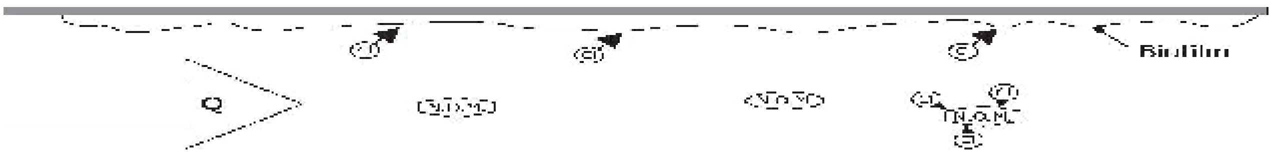
V= kinematic viscosity of fluid (L2/T)

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).

$$SH = 3.65 + \frac{0.0668(D^*Re\,(v/d)/L}{[1+0.04(D/L^*Re\,(v/d)^*0.66)]} \quad \text{….................................}2.17$$

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.



Fe^{+2} -----KW------→Fe^{+3} Boundary layer

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

2.1.9.1 Water Age Analysis

Many water distribution systems experience long retention times or increased water age, in part due to the need to satisfy fire fighting requirements. Although not a specific degradative process, water age is a characteristic that affects water quality; because, many deleterious 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. The cumulative residence time of water in the system, or water age, has come to be regarded as a reliable surrogate for 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).

2.1.9.2 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 [11].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.

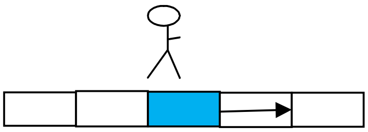


Figure 2.2: Eurelian solution methods

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 end of a pipe and mixes with water from other connecting pipes [11].

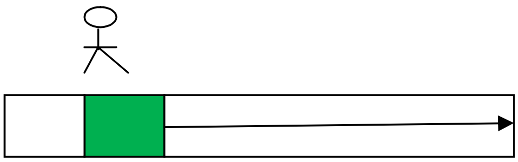


Figure 2.3 Lagrangian solution methods

Chapter 3

3.1 Methodology, Procedures and Materials used

Both primary and secondary data are 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 Bulbula 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 Tuffa Bulbula 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 flow meter reading for link flow data’s

10. Model Calibration and Validation.

Water sample 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 petridishes 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.

Water quality modeling was conducted by chlorine bulk decay bottle tests under controlled conditions like in dark room. 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 was measured in the laboratory and the residual chlorine at the selected location was measured accordingly.

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 [15]. 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'. 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.

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 (20) Twenty 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 2.3.1 Photo Field observation, 2014

Plate 3.2 sampling and pipe line leakage

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.

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

The Concentration is 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. Since at this time only single pump is operational the flow is about 36m3/hr.

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

Plate **3.3** Constant-head drip chlorinator A 500m3 Water tank

3.5 Source of Data's

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 on

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 July 2014 to September 2014 was 13,333m3, 14,028m3, 18,029m3 and the monthly Water consumption was 12,667m3, 13, 600m3, 15,098m3 and the monthly water loss shows 1,342m3.which is about 8.8%.

3.7 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, and is used world. Beside 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 merely 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 [5].

3.7.1 Hydraulic Modeling Capabilities

- models pressure-dependent flow issuing from emitters (sprinkler heads) EPANET contains a state-of-the-art hydraulic analysis engine that includes the following capabilities:
- places no limit on the size of the network that can be analyzed
- Computes friction head loss using the Hazen-Williams, Darcy- Weisbach, or Chezy-Manning formulas includes minor head losses for bends, fittings, etc.
- Models constant or variable speed pumps.
- Computes pumping energy and cost.
- models various types of valves including shutoff, check, pressure regulating, and flow control valves
- allows storage tanks to have any shape (i.e., diameter can vary with height)

3.7.2 Water Quality Modeling Capabilities

In addition to hydraulic modeling, EPANET provides the following water quality modeling Capabilities:

- Models the movement of a non-reactive tracer material through the network.
- models the movement and fate of a reactive material as it grows (e.g., a disinfection byproduct) or decays (e.g., chlorine residual) with time
- models the age of water throughout a network
- tracks the percent of flow from a given node reaching all other nodes over time
- models reactions both in the bulk flow and at the pipe wall
- uses n-th order kinetics to model reactions in the bulk flow
- uses zero or first order kinetics to model reactions at the pipe wall
- accounts for mass transfer limitations when modeling pipe wall reactions
- allows growth or decay reactions to proceed up to a limiting concentration
- employs global reaction rate coefficients that can be modified on a pipes.
- allows wall reaction rate coefficients to be correlated to pipe roughness
- allows for time-varying concentration or mass inputs at any location in the network
- models storage tanks as being either complete mix, plug flow, or two-compartment reactors

Chapter 4

4. Data analysis and Interpretation

4.1 Bacteriorological Analysis.

All the samples were found to be negative for fecal coliform and positive for total coliform, which should not occur in two consecutive samples. Depending on the risk level chlorination is recommended.The difference between the two is depicted below;

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 at this 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 [10]. 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 [23]. 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 [23]. An exception is *Escherichia coli* (*E.coli*), a thermotolerant 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 [29]. The presence of *E. coli* provides strong evidence of recent fecal contamination [23]. Temperature, producing acid and gas after 24–48 hours incubation.using membrane filtration provide at least a 'presumptive' result under normal laboratory conditions after 18 hours incubation.

4.1.2 Escherichia coli (Faecal coliform) count. Traditionally coliforms of faecal 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 faecal contamination. The presence of other 'thermotolerant' 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 [10]. 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 [23]. 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 [23]. An exception is Escherichia coli (E.coli), a thermotolerant 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 [29]. The presence of *E. coli* provides strong evidence of recent fecal contamination [23]. 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 [16] about risk classification for thermotolerant coliforms or *E. coli* of rural water supplies shown below.

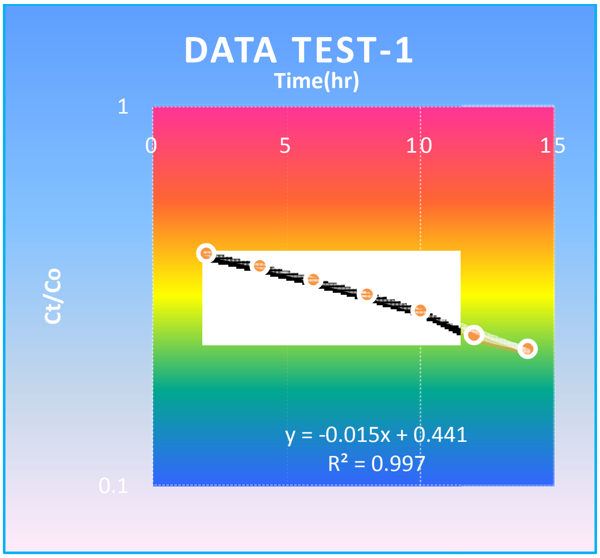
Table 4.1 water quality counts per 100mL and the associated risk

water quality counts per 100mL and the associated risk Count per 100ml	Risk Category
0	In conformity with WHO guidelines
1 – 10	Low risk
11 – 100	Intermediate risk
101 – 1000	High risk
> 1000	Very high risk

As the bacteriological test result (Appendix-Q) shows most of the system nodes have low risk number of coliforms except the hot spring near by the spring source, 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 [28].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 Bulbula 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.



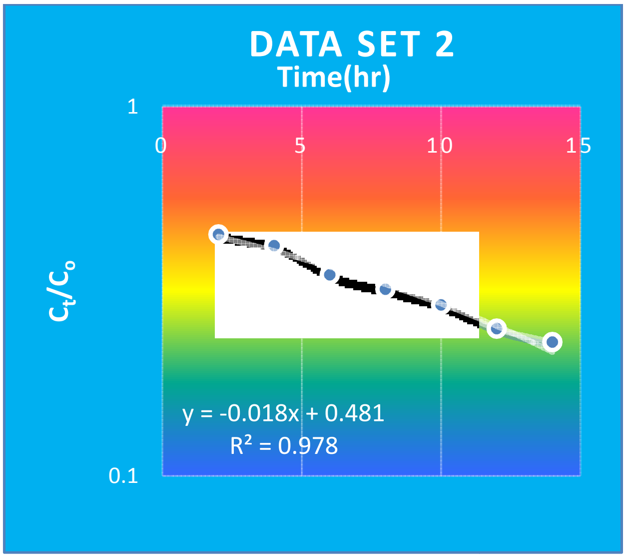


Figure 4.1 plot of residual chlorine tests 1 Figure 4.2 plot of residual chlorine test2

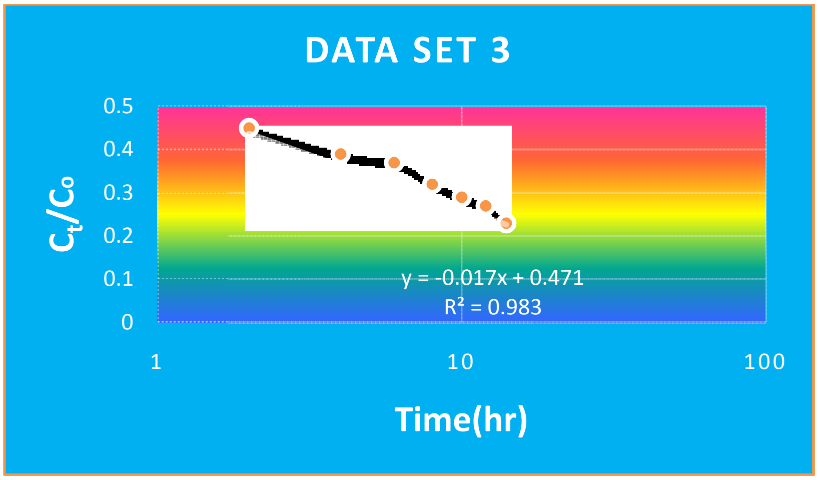


Figure 4.3 plot of residual chlorine test3

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 the best fitted line drawn through charted result where the slope of line is the bulk reaction coefficient. Accordingly, for the figure 4.4, figure 4.5 and figure 4.6 the slope of line is -0.015, -0.018, and -0.017 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, Pumps 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, Pumps and Valves are categorized as links.

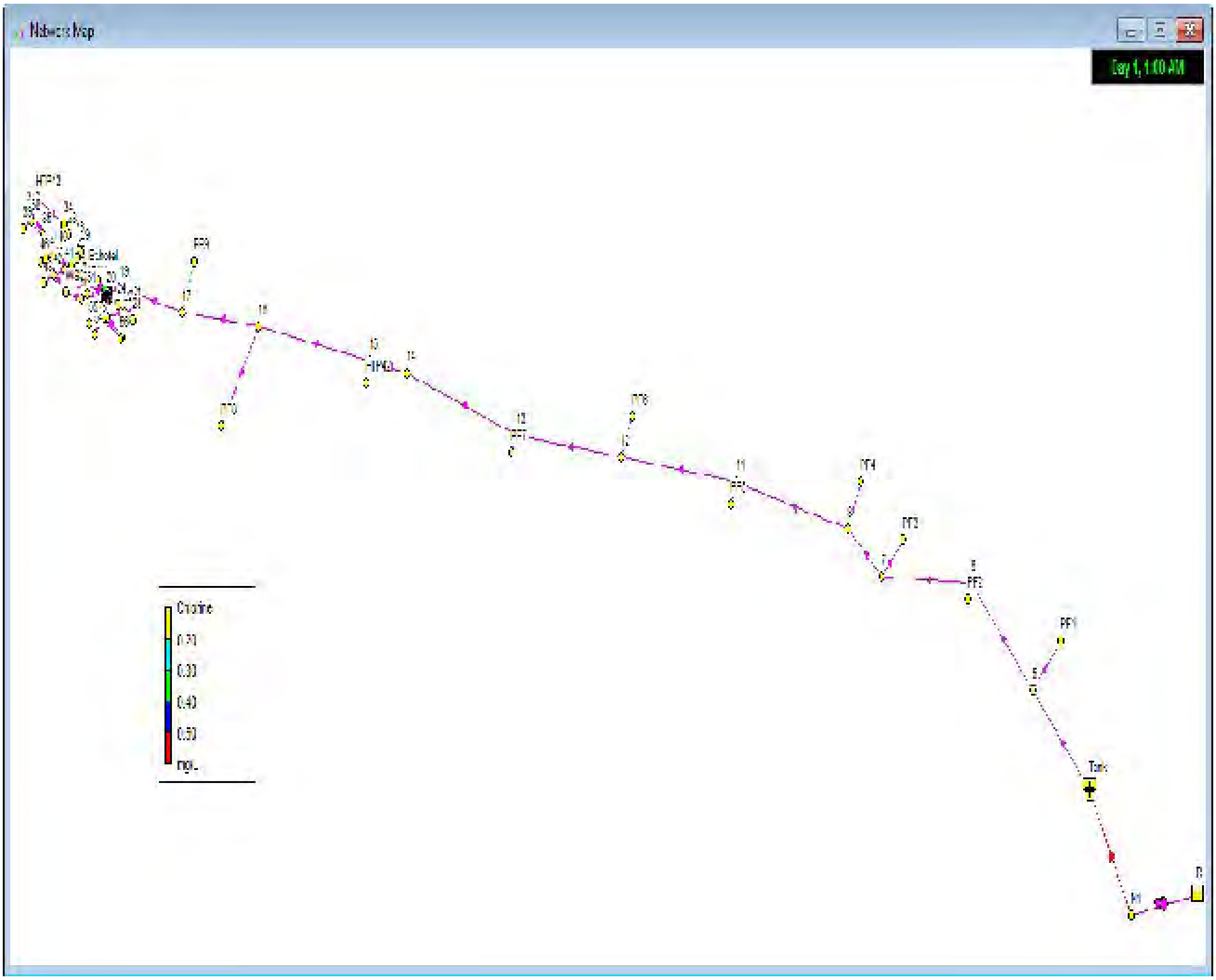


Figure 4.4. Illustrates layout of water distribution system

4.2.1 Model Calibration and Validation

For the majority of water distribution models, calibration is an iterative procedure of parameter evaluation and refinement, as a result of comparing simulated and observed values of interest. Model validation is in reality an extension of the calibration process. Its purpose is to assure that the calibrated model properly assesses all the variables and conditions which can affect model results, and demonstrate the ability to predict field observations for periods separate from the calibration.

4.2.1.1 Hydraulic Model Calibration

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

4.2.2 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:

- 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.

4.2.3 Calibration and Validation using Time-Series Datas

According to [27] a vital step in calibrating and validating an extended-period simulation is to compare time-series field data to model results. If the field data and model results are acceptably close, the model is calibrated. If significant variations exist, Adjustments can be made to various model parameters in order to improve the match. Ideally, one set of data should be available for calibration, and another set of data should be available to validate that the model is properly calibrated. Hydraulic measurements, water quality data, and tracer data are frequently used in combination in the extended period simulation (EPS) calibration and validation process.

4.2.4 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 [27].

4.2.4.1. 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.

4.2.4.2. 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.

4.2.4.3. 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.

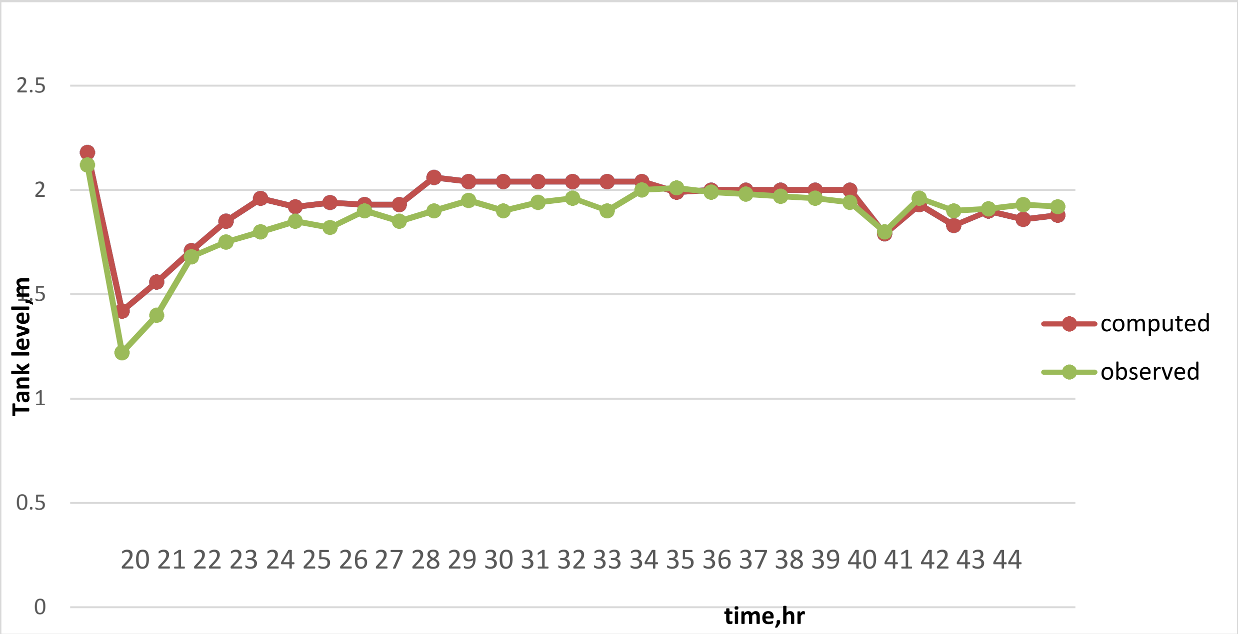
4.2.5 Model Calibration and Validation

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.

4.2.6 Hydraulic Calibration and Validation

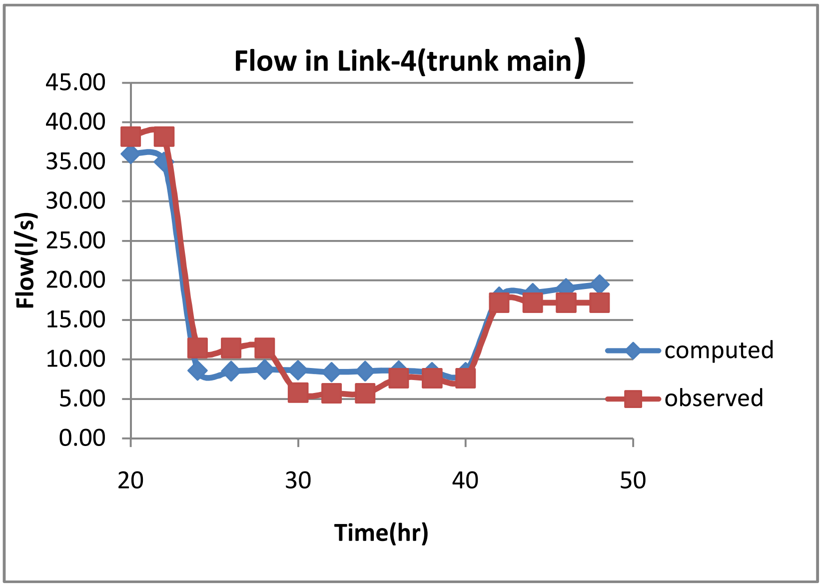
Two sets of data were collected for hydraulic model calibration and validation effort. The first sets of data were used for model calibration purpose and detailed in **Appendix R**. While the second sets of data were used for model validation purpose and detailed in Appendix S Figure 4.5 and Figure 4.6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

4.2.6.1 Tank level and Link flow calibration



R2=0.8385

Figure.4.5 Model calibration using time series data



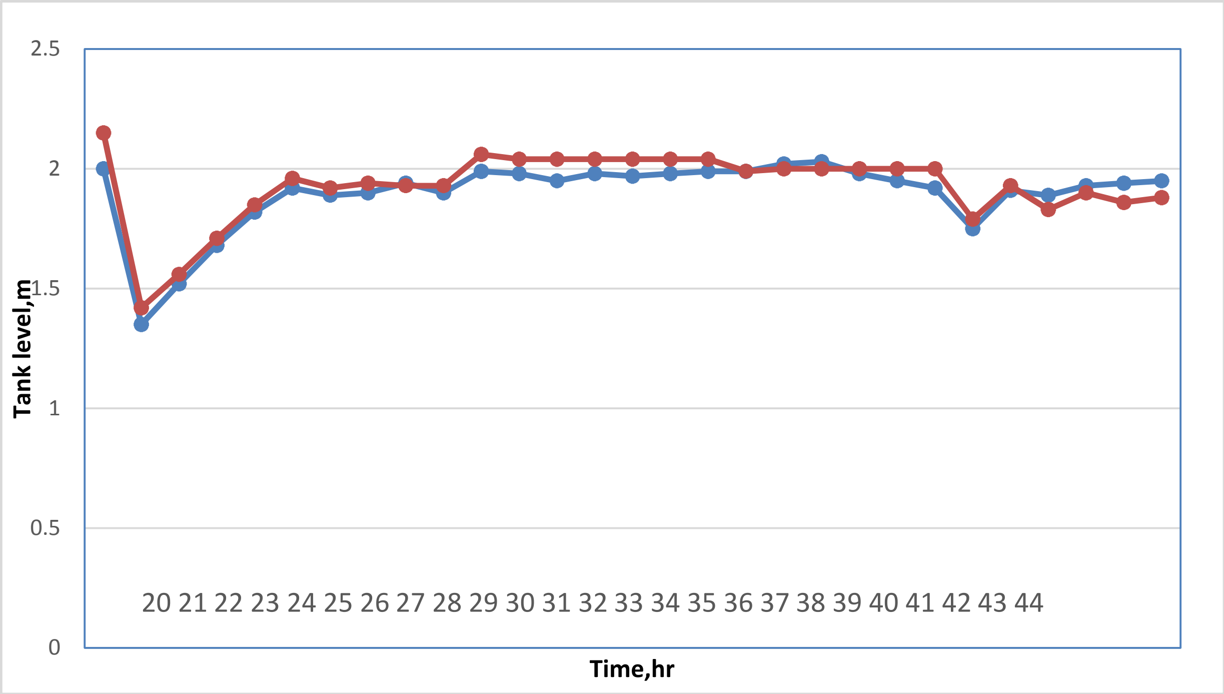
R2=0.9569

With regard to acceptable level of model calibration, there are different views. According to [27] 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 [27].

- The model should accurately reproduce tank water level fluctuations to within 3–6 ft (1–2 m) for EPS runs and match treatment plant/pump station/ well flows to 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.

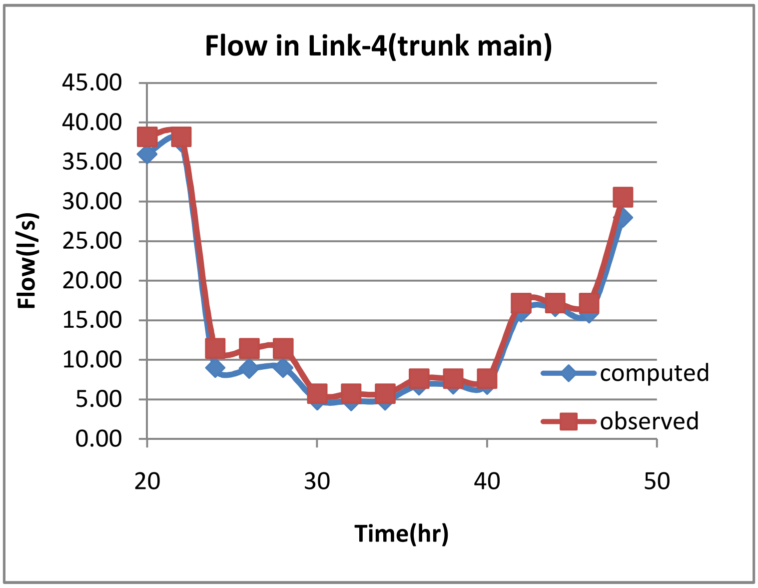
From data sets collected for calibration effort (24 data sets from tanks water level and 24 data sets from pumping flow) ,83.85 % of computed tank water level are found in the range of the guide line (1-2m). And 85% of computed link flow match pump station flow to within 10–20 percent.

4.2.6.2 Tank level and Link flow Validation



R2=0.8921

Figure.4.6 Model validation using time series data



R2=0.9952

From data sets collected for validation effort (24 data sets from tanks water level and 24 data sets from pumping flow) , 89.2 % of computed tank water level are in the range of the guide line (1-2m). And 95.0 % of computed link flow match pump station flow to within 10–20 percent.

4.7 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 [27]

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 4.1. Pipes roughness group

Group	Range	Cvalue 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 [17].

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.

$$R(C)= \frac{A}{V} *K_w*C^n \qquad 2.18$$

Where K_w=wall reaction coefficient
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.[17] postulated that the wall reaction coefficient is related to pipe roughness according to the following equation:

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

Where k_w=wall reaction coefficients
α=correlation coefficients which is constant value
Cc=adjusted roughness value

Table 4.2 Water quality model calibration for first data sets

Time (hr)	Node	Observed(mg/l)	Computed(mg/l)	Differences
20	pf2	0.45	0.35	-0.1
21	pf6	0.38	0.12	-0.26-
22	htp42	0.25	0.12	-0.13
23	Et.hotel	0.18	0.14	0.02
24	52	0.34	0.11	-0.23
25	33	0.26	0.09	-0.17
26	htp43	0.13	0.07	-0.06
			RMS	**0.159**

Table 4.3 Water quality model calibration for second data sets

Time (hr)	Node	Observed(mg/l)	Computed(mg/l)	Differences
44	pf2	0.60	0.35	-0.25
45	pf6	0.38	0.12	-0.26
46	htp42	0.22	0.12	-0.09
47	Et.hotel	0.28	0.14	-0.14
48	52	0.32	0.11	-0.21
49	33	0.26	0.09	-0.17
50	htp43	0.17	0.07	-0.10
			RMS	**0.184**

As shown in Table 4.2 and Table 4.3, 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.

4.8 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.

4.8.1 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 fire fighting (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 [25] 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, [25] 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.C

Table 4.4 Distribution of pressure at peak hour flow

Pressure(m)	Nodes(number)	Percentage
>70	11	18.97
60--70	0	0.00
50---60	1	1.72
40---50	3	5.17
30---40	3	5.17
20---30	11	18.97
15--20	15	25.86
<15	14	24.14

As indicated in the above table 4.4, 24.14% of nodes are failed to satisfy minimum pressure requirement during peak hour flow. And 18.97% of nodes exceed maximum allowable pressure of 70m. While 56.90% 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-C).

Table 4.5 Pressure distribution during low flow

Pressure(m)	Nodes(number)	Percentage
>70	53	91.38
60—70	0	0.00
50---60	2	3.45
40---50	0	0.00
30---40	0	0.00
20---30	2	3.45
15—20	0	0.00
<15	1	1.72

During low flow typically at mid-night distribution system of case study is marked by excessive pressure. As shown in table 4.5 and detailed in **Appendix D**, 91.38% of nodes are liable to extremely high pressure. This figure is relatively high. Minimum pressures are also observed during low consumption period. 1.72% of nodes are received water with low pressure and few of them obtain no water. Only 7.9% of nodes are obtaining water of optimum pressure.

4.8.2 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 elevation]ns 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 [27].

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

$$HGL_{max} < (\text{Elevation of lowest customer}) + C_f\ P_{max} \quad \text{............................} 2.21$$

Where ;HGLmin = minimum HGL (m)

HGL_{max} = maximum HGL (m

P_{min} = minimum acceptable pressure (kPa)

P_{max} = maximum acceptable pressure (kPa)

C_f = 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*.

4.8.3 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 by assessing in two cases,

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

i).Case 1. 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. These variations lead to use of large pipes and storage tanks. However, large pipes and storage tanks have negative impacts on water quality.

is about assessing water quality conditions in the distribution system of Tuffa-Bulbula 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 4.6 and table 4.7 depict water age distribution at peak and low hour flow.

Table 4.6 Water age distribution at low hour flow

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	7	11.86
12---14	6	10.17
14---16	7	11.86
>16	38	64.41

Table 4.7 Water age distribution at peak hour flow

Age(hr)	Node(number)	Percentage
<4	1	1.61
4—6	3	4.84
6---8	8	12.90
8----10	7	11.29
10----12	31	50.00
12---14	8	12.90
14---16	1	1.61
>16	0	0.00

As it is illustrated in table 4.6 and detailed in (Appendix F) at low hour flow majority of nodes 64.11% receive water with age exceeding 16 hours. Only 32.9% receive water with age less than 16 hours. While at peak hour flow nodes receiving water with age exceeding 16 hours is reduced to nill (Appendix E). 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 [30].Table 4.8 and Table 4.9 shows residual chlorine concentration distribution at low and peak hour flow.

Table 4.8 Residual chlorine distribution at low hour flow

Residual chlorine(mg/l)	Nodes(numbers)	percentages
0	5	8.47
0.01--0.1	20	33.90
0.1---0.2	5	8.47
0.2---0.5	22	37.29
>0.5	6	10.17

Table 4.9 Residual chlorine distribution at peak hour flow

Residual chlorine(mg/l)	Nodes(numbers)	Percentages
0	7	11.86
0.01--0.1	1	1.69
0.1---0.2	0	0.00
0.2---0.5	48	81.36
>0.5	3	5.08

As shown in table 4.8 and detailed in **(Appendix G)** at low hour flow only 37.29% of nodes receive water with residual chlorine between (0.2 mg/l - 0.5 mg/l). Around 8.47% of nodes obtain water of nil residual chlorine. While as it was shown in Table 4.9 and detailed **(Appendix H)** at peak hour flow 81.36% of nodes obtain water with residual chlorine between (0.2 mg/l – 0.5 mg/l). This indicates the quality of water in distribution system of Tuffa-Bulbula 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 4.10 Actual and modified pipe sizes.

Link Id	Existing size(mm)	Reduced size(mm)	Increased size(mm)
9	DCI 250	DCI 200	DCI 300
12	u-PVC 200	u-PVC 150	u-PVC 250
14	u-PVC 186	u-PVC150	u-PVC200
16	u-PVC 186	u-PVC150	u-PVC200
18	u-PVC 186	u-PVC150	u-PVC200
20	u-PVC 186	u-PVC150	u-PVC200
21	u-PVC 186	u-PVC150	u-PVC200
23	u-PVC 186	u-PVC150	u-PVC200
27	u-PVC 186	u-PVC150	u-PVC200

a. Water age for actual and modified pipe sizes

Table 4.11, Table 4.12 and Table 4.13 indicates average water age distribution at nodes for actual pipe sizes, decreased pipe size and increased pipe size.

Table 4.11 Average water age distribution for existing pipe sizes

Age(hr)	Node(number)	Percentage
<4	1	1.69
4—6	0	0.00
6---8	1	1.69
8----10	3	5.08
10----12	1	1.69
12---14	3	5.08
14---16	1	1.69
>16	49	83.05

Table 4.12 Average water age distribution for decreased pipe size

Age(hr)	Node(number)	Percentage
<4	1	1.75
4—6	1	1.75
6---8	0	0.00
8----10	2	3.51
10----12	3	5.26
12---14	2	3.51
14---16	1	1.75
>16	47	82.46

Table 4.13 Average water age distribution for increased pipe size

Age(hr)	Node(number)	Percentage
<4	1	1.89
4—6	0	0.00
6---8	0	0.00
8----10	2	3.77
10----12	4	7.55
12---14	4	7.55
14---16	3	5.66
>16	43	81.13

As illustrated in Table 4.11detailed in (**Appendix M**), for the case of existing pipe sizes majority of nodes 81.13 % receive average water age exceeding 16 hours. Only 24.0 % of nodes obtain water with average age less than 16 hour. For the case of down sized pipes as shown in Table 4.12 detailed in (**Appendix N**) 82.46% nodes obtain average water age exceeding 16 hours. 15.25% of nodes obtain water with average age less than 16 hour.

For the case of upper sized pipes as shown in Table 4.19 detailed in (**Appendix O**) 81.13% nodes obtain average water age exceeding 16 hours. 25.15 % 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 4.14, Table 4.15 and Table 4.16 depict distribution of minimum residual chlorine for actual pipe sizes, down sized pipes and upper sized pipes.

Table 4.14: Minimum residual chlorine distribution for existing pipe sizes

Residual chlorine(mg/l)	Nodes(numbers)	Percentages
0	15	25.42
0.01--0.1	4	6.78
0.1---0.2	7	11.86
0.2---0.5	22	37.29
> 0.5	11	18.64

Table 4.15 Minimum residual chlorine distribution for decreased pipe sizes

Residual chlorine(mg/l)	Nodes(numbers)	Percentages
0	9	15.25
0.01--0.1	14	23.73
0.1---0.2	12	20.34
0.2---0.5	13	22.03
>0.5	11	18.64

Table 4.16 Minimum residual chlorine distribution for increased pipe sizes

Residual chlorine(mg/l)	Nodes(numbers)	Percentages
0	10	16.95
0.01---0.1	0	0.00
0.1---0.2	1	1.69
0.2---0.5	37	62.71
> 0.5	11	18.64

As shown in table 4.14 (detailed in Appendix I), table 4.15 (detailed in Appendix K) and table 4.16 (detailed in Appendix J) only 37.29%, 22.03% and 62.71 %, 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 25.42%, 15.25% and 16.95% 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 Tuffa-Bulbula 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.

4.8.4 Proposed Solution to Improve Water quality deterioration.

Establishing booster disinfection stations is an intervention required to improve water quality deterioration in distribution system of Tuffa-Bulbula 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 [14].Maintaining disinfectant residuals at these 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.

Table 4.17 shows possible booster disinfection systems and concentration of chlorine dosage at each station.

Table 4.17 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 4.18 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	10	16.95	0	9	15.52
<0.2	1	1.69	<0.2	16	27.59
0.2	2	18.40	0.2	2	3.45
>0.2	46	62.71	> 0.2	31	53.45

As shown in table 4.18 establishments of booster disinfection system significantly improves residual chlorine concentration. At low hour flow in actual system only (53.45%) 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 are increased to (62.71%).This shows intervention work significantly improves the minimum residual chlorine concentration.

Table 4.19 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	9	15.52
<0.2	3	5.17	<0.2	16	27.59
>0.2	55	94.83	> 0.2	33	53.45

As shown in table 4.19 establishments of booster disinfection system significantly improves residual chlorine concentration, which is at low hour flow in existing system only (53.45%) 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 (94.83%) with the dosage of 0.5mg/l .This shows intervention work significantly improves the minimum residual chlorine concentration

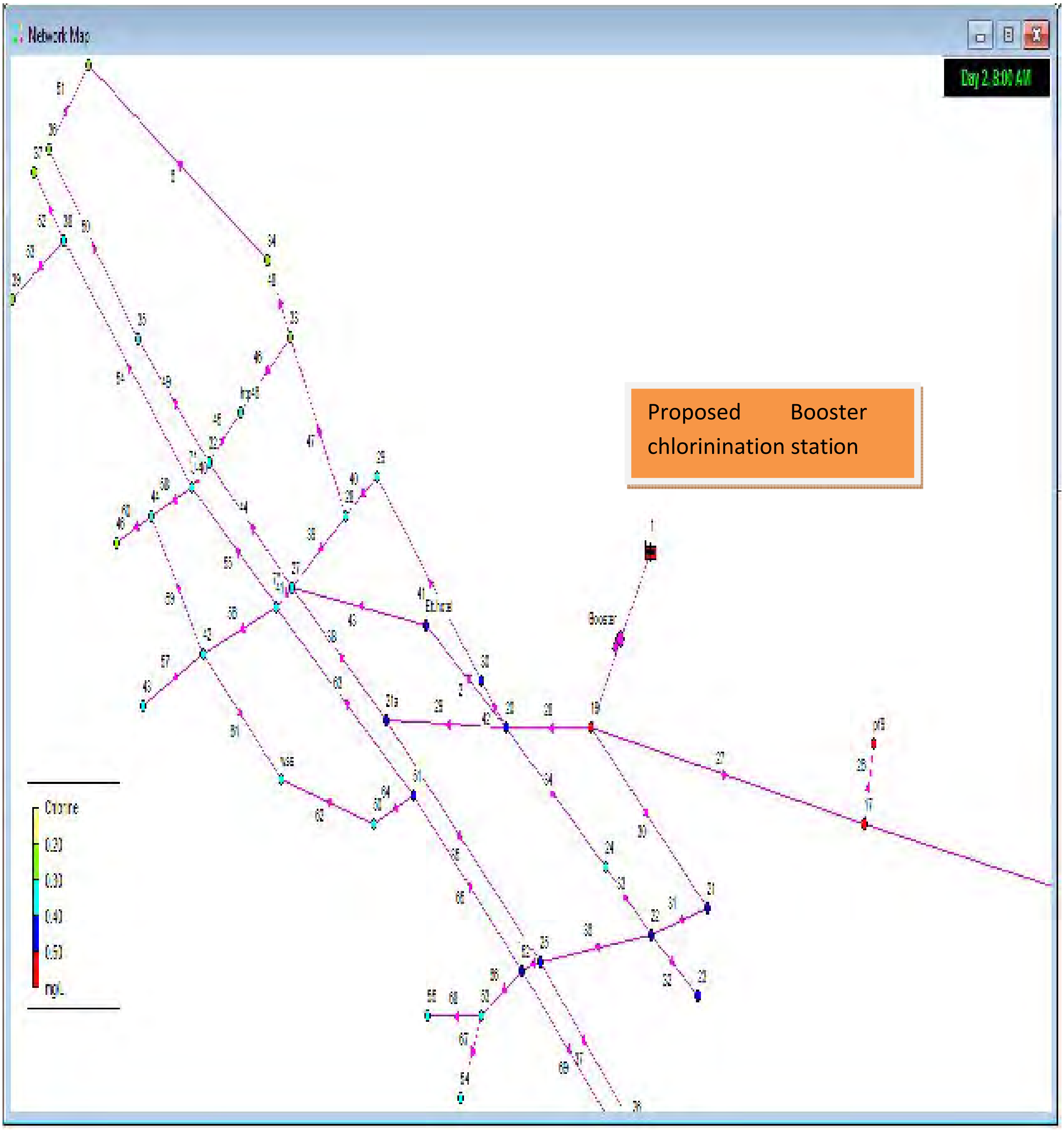


Figure 4.7 Location of Booster chlorination

Chapter 5

5. Conclusions and Recommendation

5. 1 Conclusions

To assess the current situation of Tuffa-Bulbula water supply network system modeling was found appropriate technique and accordingly modeling efforts were carried out for case study of the 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 of the pressure in the distribution system is below permissible minimum requirement. In line with this, about 90% of the nodes in 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 the water 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 current water shortage in the system. Variation in water use (water demand pattern) is the dominant factor currently contributing to water quality deterioration 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 within the system.In water quality calibrations the wall reaction coefficient (Kw) has an inverse relation with the C-factor.the residual chlorine concentration is maintained in the system even after C-values were adjusted. 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 Recommendations

To improve the current situation of Tuffa-Bulbula water 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 modify the hydraulic performance of the sub system, the design needs to be reviewed and pressure zones which serve customers situated in nearly equivalent elevation has to be established. If customers are less than approximately 37 m [27]

- 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 pre set 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.
- Modification in current flushing program which only targets storage tank cleaning per year is necessary. Periodic flushing of system elements associated with long water age may also minimize water quality degradation by removal of pipe scales and sediment associated with disinfectant consumption.
- 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 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.
- It is recommendable to boost the hot water (medication pond) away from the spring source so that the user community cannot access to the spring area.

This research has generated several significant 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. However, more comprehensive and detailed understanding was possible if plenty of time and resource were available. To have more complete understanding, future works are suggested to focus on:

- *Hydraulic modeling effort of the subsystem shall consider different scenario; 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*

Limitations:

- Water age analysis was based on assumption that the distribution system is loaded with and continuous flow
- Manual model calibration and validation was undertaken
- Gravity water supply system was assumed as an independent and isolated system

Chapter 6

6.1 References

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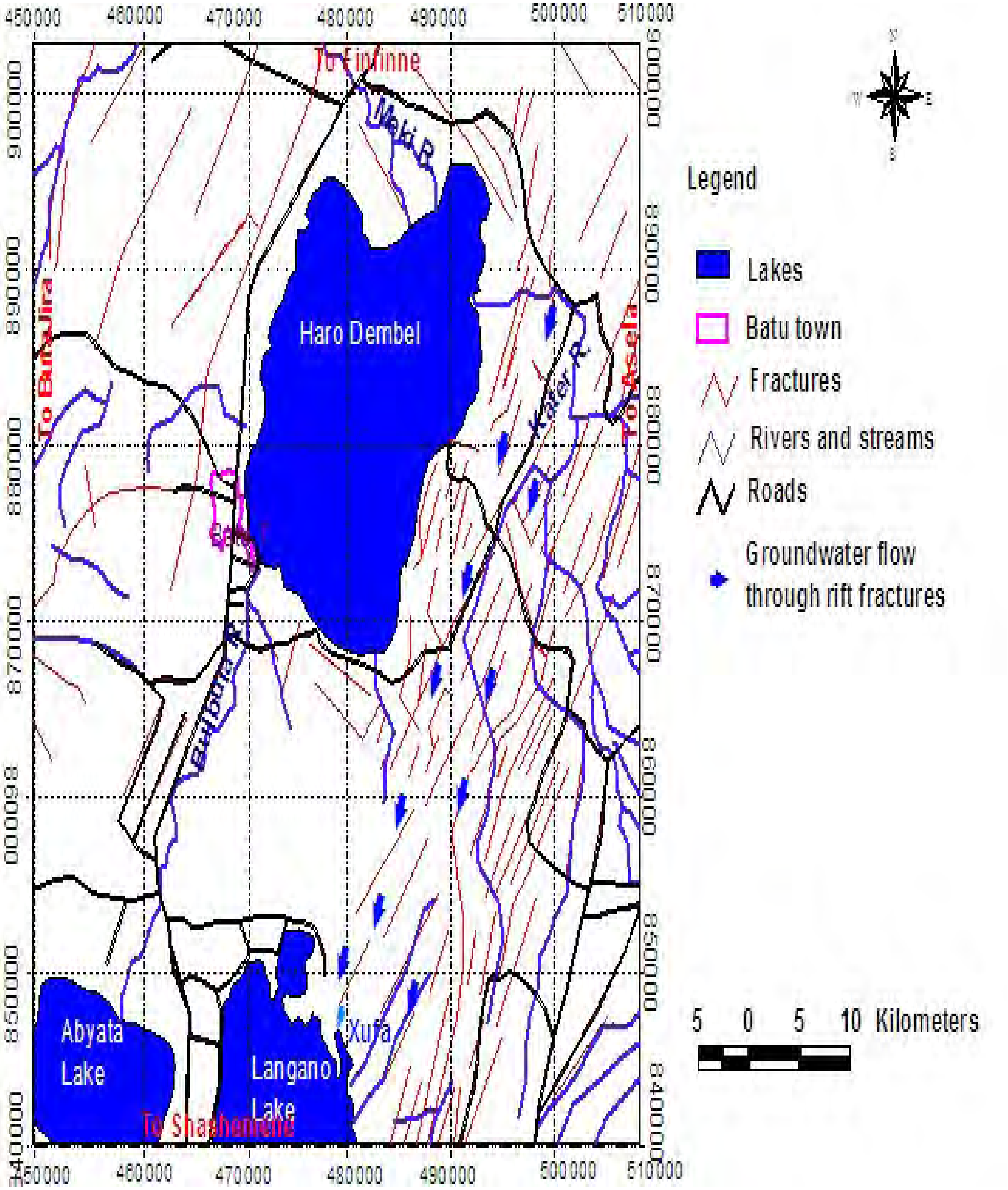
APPENDICS

Appendix A: Catchment Area of Tuffa Spring.



Appendix B: Demand pattern

Time(hr)	Demand pattern
1	0.4
2	0.9
3	1.6
4	2
5	0.6
6	0.3
7	0.4
8	0.9
9	1.6
10	2
11	0.6
12	0.3
13	0.4
14	0.9
15	1.6
16	2
17	0.6
18	0.3
19	0.4
20	0.9
21	1.6
22	2
23	0.6
24	0.3

Appendix C: Pressure at peak flow

Node	Easting(m)	Northing(m)	pressure(m)
N1	480129	846214	235.00
J5	478069	848751	18.72
Pwp	478090	849471	20.08
J6	477003	849797	49.63
wp2	476951	849656	49.01
J7	476951	849656	132.18
wp3	475823	850234	132.82
J8	474876	850346	126.71
wp4	475081	850802	126.26

J11	472930	850800	110.62
wp5	472845	850586	110.96
J12	470939	851049	87.01
wp6	471136	851448	74.25
J13	469139	851268	84.68
wp7	469050	851089	73.16
J14	467233	851886	54.51
J15	466577	852007	46.60
htp42	466514	851794	41.61
J16	464659	852349	33.02
wp8	464630	851182	33.82
J19	462257	852725	22.87
J20	461943	852747	25.41
J20a	461615	852756	22.78
J21	462497	852511	22.06
J22	462343	852476	22.87
J23	462472	852397	23.17
J24	462216	852564	24.10
J25	462036	852442	20.07
J26	462291	852227	17.70
J27	461354	852929	17.30
J28	461503	853024	18.76
J29	461589	853075	24.11
J30	461876	852807	25.21
J32	461125	853094	17.51
htp46	461213	853158	14.41
J33	461349	853257	13.42
J34	461287	853356	14.26
J35	460929	853255	13.03
J36	460683	853501	9.98
J37	460645	853471	17.14
Htp43	460795	853613	5.21
J38	460727	853383	16.22
J39	460584	853305	14.74
J40	461080	853059	17.52
J41	461310	852904	20.30
J42	461110	852842	18.15
J43	460944	852775	18.13
J44	460966	853022	16.97
J46	460870	852988	16.05
Wss	461324	852680	17.69
J50	461578	852621	19.25
J51	461688	852659	20.64
J52	461988	852430	19.97

J53	461875	852370	17.89
J54	461821	852263	17.65
J55	461728	852372	13.50
J56	462255	852213	15.90
Tank	479049	847765	4.20

Appendix D: Pressure at low flow

Node	Easting(m)	Northing(m)	pressure(m)
N1	480129	846214	235.04
J5	478069	848751	24.03
Pwp	478090	849471	26.02
J6	477003	849797	56.97
wp2	476951	849656	57.44
J7	476951	849656	143.84
wp3	475823	850234	144.83
J8	474876	850346	140.77
wp4	475081	850802	141.72
J11	472930	850800	135.44
wp5	472845	850586	136.42
J12	470939	851049	126.97
wp6	471136	851448	127.87
J13	469139	851268	127.56
wp7	469050	851089	128.50
J14	467233	851886	122.12
J15	466577	852007	118.97
htp42	466514	851794	119.80
J16	464659	852349	118.57
wp8	464630	851182	119.56
J19	462257	852725	119.32
J20	461943	852747	119.24
J20a	461615	852756	123.12
J21	462497	852511	121.15
J22	462343	852476	122.15
J23	462472	852397	122.64
J24	462216	852564	123.16
J25	462036	852442	122.07
J26	462291	852227	120.05
J27	461354	852929	120.04
J28	461503	853024	121.06
J29	461589	853075	124.13
J30	461876	852807	122.22
J32	461125	853094	122.96
htp46	461213	853158	120.93
J33	461349	853257	118.96

J34	461287	853356	119.95
J35	460929	853255	119.91
J36	460683	853501	118.85
J37	460645	853471	122.95
htp43	460795	853613	118.32
J38	460727	853383	121.95
J39	460584	853305	120.94
J40	461080	853059	122.96
J41	461310	852904	123.04
J42	461110	852842	122.01
J43	460944	852775	122.01
J44	460966	853022	122.94
J46	460870	852988	122.92
Wss	461324	852680	122.96
J50	461578	852621	122.04
J51	461688	852659	123.05
J52	461988	852430	122.06
J53	461875	852370	121.03
J54	461821	852263	123.93
J55	461728	852372	123.81
J56	462255	852213	120.00
Tank	479049	847765	4.20

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

Node	Easting(m)	Northing(m)	Water age(hr)
N1	480129	846214	0
J5	478069	848751	4
Pwp	478090	849471	5
J6	477003	849797	5
wp2	476951	849656	5
J7	476951	849656	6
wp3	475823	850234	6
J8	474876	850346	6
wp4	475081	850802	6
J11	472930	850800	7
wp5	472845	850586	7
J12	470939	851049	7
wp6	471136	851448	7
J13	469139	851268	8
wp7	469050	851089	8
J14	467233	851886	8
J15	466577	852007	9
htp42	466514	851794	9

J16	464659	852349	9
wp8	464630	851182	9
J19	462257	852725	10
J20	461943	852747	10
J20a	461615	852756	10
Ethiopia hotel	461723	852880	10
J21	462497	852511	10
J22	462343	852476	10
J23	462472	852397	10
J24	462216	852564	11
J25	462036	852442	11
J26	462291	852227	11
J27	461354	852929	11
J28	461503	853024	11
J29	461589	853075	11
J30	461876	852807	11
J32	461125	853094	11
htp46	461213	853158	11
J33	461349	853257	11
J34	461287	853356	11
J35	460929	853255	11
J36	460683	853501	11
J37	460645	853471	11
htp43	460795	853613	11
J38	460727	853383	11
J39	460584	853305	11
J40	461080	853059	11
J41	461310	852904	11
J42	461110	852842	11
J43	460944	852775	11
J44	460966	853022	11
J46	460870	852988	11
Wss	461324	852680	12
J50	461578	852621	12
J51	461688	852659	12
J52	461988	852430	12
J53	461875	852370	13
J54	461821	852263	13
J55	461728	852372	13
J56	462255	852213	13
Tank	479049	847765	16

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

Node	Easting(m)	Northing(m)	Water age(hr)
N1	480129	846214	0
J5	478069	848751	10
Pwp	478090	849471	11
J6	477003	849797	11
wp2	476951	849656	11
J7	476951	849656	12
wp3	475823	850234	12
J8	474876	850346	12
wp4	475081	850802	13
J11	472930	850800	13
wp5	472845	850586	13
J12	470939	851049	13
wp6	471136	851448	13
J13	469139	851268	14
wp7	469050	851089	14
J14	467233	851886	14
J15	466577	852007	14
htp42	466514	851794	15
J16	464659	852349	15
wp8	464630	851182	15
J19	462257	852725	16
J20	461943	852747	16
J20a	461615	852756	16
Ethiopia hotel	461723	852880	16
J21	462497	852511	16
J22	462343	852476	16
J23	462472	852397	16
J24	462216	852564	16
J25	462036	852442	16
J26	462291	852227	16
J27	461354	852929	16
J28	461503	853024	16
J29	461589	853075	16
J30	461876	852807	16
J32	461125	853094	16
htp46	461213	853158	16
J33	461349	853257	16
J34	461287	853356	16

J35	460929	853255	17
J36	460683	853501	17
J37	460645	853471	17
htp43	460795	853613	17
J38	460727	853383	17
J39	460584	853305	17
J40	461080	853059	17
J41	461310	852904	17
J42	461110	852842	17
J43	460944	852775	17
J44	460966	853022	17
J46	460870	852988	17
Wss	461324	852680	17
J50	461578	852621	17
J51	461688	852659	17
J52	461988	852430	17
J53	461875	852370	18
J54	461821	852263	18
J55	461728	852372	18
J56	462255	852213	18
Tank	479049	847765	20

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

Node	Easting(m)	Northing(m)	Residual chlorine(mg/l)
N1	480129	846214	0.00
J5	478069	848751	0.00
Pwp	478090	849471	0.00
J6	477003	849797	0.00
wp2	476951	849656	0.00
J7	476951	849656	0.00
wp3	475823	850234	0.02
J8	474876	850346	0.11
wp4	475081	850802	0.11
J11	472930	850800	0.16
wp5	472845	850586	0.17
J12	470939	851049	0.20
wp6	471136	851448	0.32
J13	469139	851268	0.56
wp7	469050	851089	0.56
J14	467233	851886	0.56
J15	466577	852007	0.57

htp42	466514	851794	0.57
J16	464659	852349	0.58
wp8	464630	851182	0.58
J19	462257	852725	0.58
J20	461943	852747	0.59
J20a	461615	852756	0.59
J21	462497	852511	0.59
J22	462343	852476	0.59
J23	462472	852397	0.59
J24	462216	852564	0.59
J25	462036	852442	0.59
J26	462291	852227	0.60
J27	461354	852929	0.60
J28	461503	853024	0.60
J29	461589	853075	0.60
J30	461876	852807	0.60
J32	461125	853094	0.60
htp46	461213	853158	0.61
J33	461349	853257	0.61
J34	461287	853356	0.61
J35	460929	853255	0.61
J36	460683	853501	0.61
J37	460645	853471	0.61
htp43	460795	853613	0.61
J38	460727	853383	0.61
J39	460584	853305	0.61
J40	461080	853059	0.62
J41	461310	852904	0.62
J42	461110	852842	0.62
J43	460944	852775	0.62
J44	460966	853022	0.62
J46	460870	852988	0.63
Wss	461324	852680	0.63
J50	461578	852621	0.63
J51	461688	852659	0.63
J52	461988	852430	0.63
J53	461875	852370	0.63
J54	461821	852263	0.92
J55	461728	852372	0.99
J56	462255	852213	1.26
Tank	479049	847765	1.32

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

Node	Easting(m)	Northing(M)	Residual chlorine(mg/l)
N1	480129	846214	0.00
J5	478069	848751	0.00
Pwp	478090	849471	0.00
J6	477003	849797	1.40
wp2	476951	849656	1.41
J7	476951	849656	1.07
wp3	475823	850234	0.76
J8	474876	850346	0.02
wp4	475081	850802	1.12
J11	472930	850800	0.69
wp5	472845	850586	0.64
J12	470939	851049	0.06
wp6	471136	851448	0.84
J13	469139	851268	0.71
wp7	469050	851089	0.58
J14	467233	851886	0.18
J15	466577	852007	0.41
htp42	466514	851794	0.57
J16	464659	852349	0.70
wp8	464630	851182	0.58
J19	462257	852725	0.23
J20	461943	852747	0.40
J20a	461615	852756	0.53
J21	462497	852511	0.30
J22	462343	852476	0.54
J23	462472	852397	0.57
J24	462216	852564	0.62
J25	462036	852442	0.56
J26	462291	852227	0.60
J27	461354	852929	0.62
J28	461503	853024	0.62
J29	461589	853075	0.61
J30	461876	852807	0.45
J32	461125	853094	0.60
htp46	461213	853158	0.62
J33	461349	853257	0.6
J34	461287	853356	0.6
J35	460929	853255	0.60

J36	460683	853501	0.60
J37	460645	853471	0.56
htp43	460795	853613	0.60
J38	460727	853383	0.58
J39	460584	853305	0.56
J40	461080	853059	0.6
J41	461310	852904	0.62
J42	461110	852842	0.60
J43	460944	852775	0.57
J44	460966	853022	0.59
J46	460870	852988	0.59
Wss	461324	852680	0.6
J50	461578	852621	0.60
J51	461688	852659	0.63
J52	461988	852430	0.62
J53	461875	852370	0.63
J54	461821	852263	0.62
J55	461728	852372	0.62
J56	462255	852213	0.61
Tank	479049	847765	0.00

Appendix I: Residual chlorine for existing pipe size

Node	Easting	Northing	Residual chlorine(mg/l)
N1	480129	846214	0.00
J5	478069	848751	0.50
PF1	478090	849471	0.00
J6	477003	849797	0.44
PF2	476951	849656	0.00
J7	476951	849656	0.92
PF3	475823	850234	0.00
J8	474876	850346	0.88
PF4	475081	850802	0.01
J11	472930	850800	1.20
PF5	472845	850586	0.00
J12	470939	851049	0.27
PF6	471136	851448	0.19
J13	469139	851268	0.73
PF7	469050	851089	0.64
J14	467233	851886	0.55
J15	466577	852007	0.62
htp42	466514	851794	0.63

J16	464659	852349	0.52
PF8	464630	851182	0.33
J19	462257	852725	0.48
J20	461943	852747	0.45
J20a	461615	852756	0.43
J21	462497	852511	0.45
J22	462343	852476	0.42
J23	462472	852397	0.40
J24	462216	852564	0.37
J25	462036	852442	0.4
J26	462291	852227	0.37
J27	461354	852929	0.36
J28	461503	853024	0.32
J29	461589	853075	0.34
J30	461876	852807	0.45
J32	461125	853094	0.29
htp46	461213	853158	0.29
J33	461349	853257	0.32
J34	461287	853356	0.22
J35	460929	853255	0.24
J36	460683	853501	0.21
J37	460645	853471	0.00
htp43	460795	853613	0.00
J38	460727	853383	0.00
J39	460584	853305	0.00
J40	461080	853059	0.26
J41	461310	852904	0.28
J42	461110	852842	0.24
J43	460944	852775	0.24
J44	460966	853022	0.22
J46	460870	852988	0.22
wss	461324	852680	0.25
J50	461578	852621	0.28
J51	461688	852659	0.35
J52	461988	852430	0.40
J53	461875	852370	0.38
J54	461821	852263	0.36
J55	461728	852372	0.36
J56	462255	852213	0.29
Tank	479049	847765	0.00

Appendix J: Residual chlorine for increased pipe size

Node	Easting	Northing	Residual chlorine(mg/l)
N1	480129	846214	0.00
J5	478069	848751	0.51
PF1	478090	849471	0.00
J6	477003	849797	0.00
PF2	476951	849656	1.55
J7	476951	849656	0.67
PF3	475823	850234	0.00
J8	474876	850346	0.19
PF4	475081	850802	0.56
J11	472930	850800	0.63
PF5	472845	850586	0.00
J12	470939	851049	0.61
PF6	471136	851448	0.49
J13	469139	851268	0.56
PF7	469050	851089	0.46
J14	467233	851886	0.55
J15	466577	852007	0.54
htp42	466514	851794	0.51
J16	464659	852349	0.50
PF8	464630	851182	0.00
J19	462257	852725	0.34
J20	461943	852747	0.31
J20a	461615	852756	0.29
J21	462497	852511	0.30
J22	462343	852476	0.28
J23	462472	852397	0.27
J24	462216	852564	0.25
J25	462036	852442	0.27
J26	462291	852227	0.18
J27	461354	852929	0.24
J28	461503	853024	0.20
J29	461589	853075	0.22
J30	461876	852807	0.31
J32	461125	853094	0.30
htp46	461213	853158	0.12
J33	461349	853257	0.14
J34	461287	853356	0.22
J35	460929	853255	0.00
J36	460683	853501	0.00

J37	460645	853471	0.00
htp43	460795	853613	0.00
J38	460727	853383	0.00
J39	460584	853305	0.00
J40	461080	853059	0.08
J41	461310	852904	0.10
J42	461110	852842	0.04
J43	460944	852775	0.00
J44	460966	853022	0.04
J46	460870	852988	0.00
Wss	461324	852680	0.05
J50	461578	852621	0.10
J51	461688	852659	0.20
J52	461988	852430	0.26
J53	461875	852370	0.25
J54	461821	852263	0.23
J55	461728	852372	0.23
J56	462255	852213	0.12
Tank	479049	847765	0.00

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

Node	Easting	Northing	Residual chlorine(mg/l)
N1	480129	846214	0.00
J5	478069	848751	0.00
PF1	478090	849471	0.00
J6	477003	849797	0.23
PF2	476951	849656	0.24
J7	476951	849656	0.25
PF3	475823	850234	0.27
J8	474876	850346	0.27
PF4	475081	850802	0.28
J11	472930	850800	0.28
PF5	472845	850586	0.28
J12	470939	851049	0.29
PF6	471136	851448	0.30
J13	469139	851268	0.31
PF7	469050	851089	0.31
J14	467233	851886	0.31
J15	466577	852007	0.32
htp42	466514	851794	0.33

J16	464659	852349	0.34
PF8	464630	851182	0.34
J19	462257	852725	0.34
J20	461943	852747	0.34
J20a	461615	852756	0.34
J21	462497	852511	0.36
J22	462343	852476	0.36
J23	462472	852397	0.36
J24	462216	852564	0.37
J25	462036	852442	0.37
J26	462291	852227	0.38
J27	461354	852929	0.38
J28	461503	853024	0.38
J29	461589	853075	0.39
J30	461876	852807	0.40
J32	461125	853094	0.40
htp46	461213	853158	0.40
J33	461349	853257	0.43
J34	461287	853356	0.43
J35	460929	853255	0.45
J36	460683	853501	0.45
J37	460645	853471	0.46
htp43	460795	853613	0.47
J38	460727	853383	0.51
J39	460584	853305	0.52
J40	461080	853059	0.52
J41	461310	852904	0.53
J42	461110	852842	0.54
J43	460944	852775	0.55
J44	460966	853022	0.56
J46	460870	852988	0.57
Wss	461324	852680	0.61
J50	461578	852621	0.74
J51	461688	852659	0.80
J52	461988	852430	0.82
J53	461875	852370	0.85
J54	461821	852263	0.89
J55	461728	852372	1.04
J56	462255	852213	1.54
Tank	479049	847765	1.61

Appendix L: Residual chlorine for decreased pipe size

Node	Easting	Northing	Residual chlorine(mg/l)
N1	480129	846214	0.00
J5	478069	848751	0.51
PF1	478090	849471	0.00
J6	477003	849797	0.55
PF2	476951	849656	0.43
J7	476951	849656	1.75
PF3	475823	850234	0.00
J8	474876	850346	1.66
PF4	475081	850802	0.15
J11	472930	850800	1.06
PF5	472845	850586	0.00
J12	470939	851049	0.07
PF6	471136	851448	0.04
J13	469139	851268	0.26
PF7	469050	851089	0.24
J14	467233	851886	0.26
J15	466577	852007	0.59
htp42	466514	851794	0.61
J16	464659	852349	0.51
PF8	464630	851182	0.53
J19	462257	852725	0.55
J20	461943	852747	0.52
J20a	461615	852756	0.49
J21	462497	852511	0.46
J22	462343	852476	0.43
J23	462472	852397	0.45
J24	462216	852564	0.40
J25	462036	852442	0.45
J26	462291	852227	0.44
J27	461354	852929	0.40
J28	461503	853024	0.33
J29	461589	853075	0.41
J30	461876	852807	0.43
J32	461125	853094	0.39
htp46	461213	853158	0.35
J33	461349	853257	0.41
J34	461287	853356	0.38
J35	460929	853255	0.39
J36	460683	853501	0.39

J37	460645	853471	0.00
htp43	460795	853613	0.32
J38	460727	853383	0.00
J39	460584	853305	0.00
J40	461080	853059	0.40
J41	461310	852904	0.43
J42	461110	852842	0.38
J43	460944	852775	0.40
J44	460966	853022	0.35
J46	460870	852988	0.37
wss	461324	852680	0.40
J50	461578	852621	0.43
J51	461688	852659	0.47
J52	461988	852430	0.44
J53	461875	852370	0.42
J54	461821	852263	0.42
J55	461728	852372	0.42
J56	462255	852213	0.45
Tank	479049	847765	0.00

Appendix M: Average water age for existing pipe size

Node	Easting(m)	Northing(m)	Water age(hr)
N1	480129	846214	0.00
J5	478069	848751	6.60
pwp	478090	849471	8.28
J6	477003	849797	9.30
wp2	476951	849656	9.45
J7	476951	849656	11.69
wp3	475823	850234	12.07
J8	474876	850346	13.37
wp4	475081	850802	13.93
J11	472930	850800	15.61
wp5	472845	850586	16.08
J12	470939	851049	17.14
wp6	471136	851448	17.31
J13	469139	851268	18.71
wp7	469050	851089	19.19
J14	467233	851886	19.40
J15	466577	852007	19.99
htp42	466514	851794	20.00
J16	464659	852349	20.00
wp8	464630	851182	20.00

J19	462257	852725	20.00
J20	461943	852747	20.00
J20a	461615	852756	20.00
Ethiopia hotel	461723	852880	20.00
J21	462497	852511	20.00
J22	462343	852476	20.00
J23	462472	852397	20.00
J24	462216	852564	20.00
J25	462036	852442	20.00
J26	462291	852227	20.00
J27	461354	852929	20.00
J28	461503	853024	20.00
J29	461589	853075	20.00
J30	461876	852807	20.00
J32	461125	853094	20.00
htp46	461213	853158	20.00
J33	461349	853257	20.00
J34	461287	853356	20.00
J35	460929	853255	20.00
J36	460683	853501	20.00
J37	460645	853471	20.00
htp43	460795	853613	20.00
J38	460727	853383	20.00
J39	460584	853305	20.00
J40	461080	853059	20.00
J41	461310	852904	20.00
J42	461110	852842	20.00
J43	460944	852775	20.00
J44	460966	853022	20.00
J46	460870	852988	20.00
wss	461324	852680	20.00
J50	461578	852621	20.00
J51	461688	852659	20.00
J52	461988	852430	20.00
J53	461875	852370	20.00
J54	461821	852263	20.00
J55	461728	852372	20.00
J56	462255	852213	20.00
Tank	479049	847765	5.5

Appendix N: Average water age for decreased pipe size

Node	Easting(m)	Northing(m)	Water age(hr)
N1	480129	846214	0.00
J5	478069	848751	6.60
pwp	478090	849471	8.28
J6	477003	849797	8.92
wp2	476951	849656	9.08
J7	476951	849656	10.44
wp3	475823	850234	10.66
J8	474876	850346	11.65
wp4	475081	850802	11.90
J11	472930	850800	12.68
wp5	472845	850586	13.14
J12	470939	851049	13.67
wp6	471136	851448	13.83
J13	469139	851268	14.84
wp7	469050	851089	15.22
J14	467233	851886	15.53
J15	466577	852007	16.41
htp42	466514	851794	17.99
J16	464659	852349	18.57
wp8	464630	851182	18.74
J19	462257	852725	18.77
J20	461943	852747	18.94
J20a	461615	852756	18.96
Ethiopia hotel	461723	852880	19.03
J21	462497	852511	19.03
J22	462343	852476	19.06
J23	462472	852397	19.09
J24	462216	852564	19.20
J25	462036	852442	19.24
J26	462291	852227	19.24
J27	461354	852929	19.25
J28	461503	853024	19.30
J29	461589	853075	19.37
J30	461876	852807	19.57
J32	461125	853094	19.62
htp46	461213	853158	19.73
J33	461349	853257	19.74
J34	461287	853356	19.75
J35	460929	853255	19.93
J36	460683	853501	19.93

J37	460645	853471	19.97
htp43	460795	853613	19.98
J38	460727	853383	20.00
J39	460584	853305	20.00
J40	461080	853059	20.00
J41	461310	852904	20.00
J42	461110	852842	20.00
J43	460944	852775	20.00
J44	460966	853022	20.00
J46	460870	852988	20.00
wss	461324	852680	20.00
J50	461578	852621	20.00
J51	461688	852659	20.00
J52	461988	852430	20.00
J53	461875	852370	20.00
J54	461821	852263	20.00
J55	461728	852372	20.00
J56	462255	852213	20.00
Tank	479049	847765	4.5

Appendix O: Average water age for increased pipe size

Node	Easting(m)	Northing(m)	Water age(hr)
N1	480129	846214	0.00
J5	478069	848751	8.28
pwp	478090	849471	20.00
J6	477003	849797	9.76
wp2	476951	849656	9.91
J7	476951	849656	13.26
wp3	475823	850234	20.00
J8	474876	850346	13.89
wp4	475081	850802	15.25
J11	472930	850800	16.89
wp5	472845	850586	20.00
J12	470939	851049	18.60
wp6	471136	851448	18.98
J13	469139	851268	19.81
wp7	469050	851089	19.88
J14	467233	851886	20.00
J15	466577	852007	20.00
htp42	466514	851794	20.00
J16	464659	852349	20.00

wp8	464630	851182	20.00
J19	462257	852725	20.00
J20	461943	852747	20.00
J20a	461615	852756	202.00
Ethiopia hotel	461723	852880	20.00
J21	462497	852511	20.00
J22	462343	852476	20.00
J23	462472	852397	20.00
J24	462216	852564	20.00
J25	462036	852442	20.00
J26	462291	852227	20.00
J27	461354	852929	20.00
J28	461503	853024	20.00
J29	461589	853075	20.00
J30	461876	852807	20.00
J32	461125	853094	20.00
htp46	461213	853158	20.00
J33	461349	853257	20.00
J34	461287	853356	20.00
J35	460929	853255	20.00
J36	460683	853501	20.00
J37	460645	853471	20.00
htp43	460795	853613	20.00
J38	460727	853383	20.00
J39	460584	853305	20.00
J40	461080	853059	20.00
J41	461310	852904	20.00
J42	461110	852842	20.00
J43	460944	852775	20.00
J44	460966	853022	20.00
J46	460870	852988	20.00
wss	461324	852680	20.00
J50	461578	852621	20.00
J51	461688	852659	20.00
J52	461988	852430	20.00
J53	461875	852370	20.00
J54	461821	852263	20.00
J55	461728	852372	20.00
J56	462255	852213	20.00
Tank	479049	847765	6.60

Appendix P: Bulk coefficients data tests

	data test-1	
Date	1-Nov-14	
Time	Sample	Residual chlorine(mg/l)
8:00 AM	Water Tank	0.80
8:00 AM	PF6	0.39
8:00 AM	Ethiopia hotel	0.21
8:00 AM	Junction 37	0.32
Date	2-Nov-14	
Time	Sample	Residual chlorine(mg/l)
10:00 AM	Water Tank	0.80
10:00 AM	PF6	0.23
10:00 AM	Ethiopia hotel	0.36
10:00 AM	Junction 37	0.31
Date	3-Nov-14	
Time	Sample	Residual chlorine(mg/l)
12:00 AM	Water Tank	0.80
12:00 AM	PF6	0.22
12:00 AM	Ethiopia hotel	0.30
12:00 AM	Junction 37	0.32
Date	4-Nov-14	
Time	Sample	Residual chlorine(mg/l)
2:00: PM	Water Tank	0.80
2:00: PM	PF6	0.08
2:00: PM	Ethiopia hotel	0.04
2:00: PM	Junction 37	0.50
Date	5-Nov-14	
Time	Sample	Residual chlorine(mg/l)
4:00 PM	Water Tank	0.80
4:00 PM	PF6	0.48
4:00 PM	Ethiopia hotel	0.03
4:00 PM	Junction 37	0.19
Date	6-Nov-14	

Time	Sample	Residual chlorine(mg/l)
6:00: PM	Water Tank	0.80
6:00: PM	PF6	0.09
6:00: PM	Ethiopia hotel	0.07
6:00: PM	Junction 37	0.45
Date	7-Nov-14	
Time	Sample	Residual chlorine(mg/l)
8:00: PM	Water Tank	0.80
8:00: PM	PF6	0.00
8:00: PM	Ethiopia hotel	0.29
8:00: PM	Junction 37	0.26
	data test-2	
Date	8-Nov-14	
Time	Sample	Residual chlorine(mg/l)
8:00 AM	Water Tank	0.80
8:00 AM	PF6	0.43
8:00 AM	Ethiopia hotel	0.33
8:00 AM	Junction 37	0.32
Date	9-Nov-14	
Time	Sample	Residual chlorine(mg/l)
10:00: AM	Water Tank	0.80
10:00: AM	PF6	0.44
10:00: AM	Ethiopia hotel	0.34
10:00: AM	Junction 37	0.31
Date	10-Nov-14	
Time	Sample	Residual chlorine(mg/l)
12:00 PM	Water Tank	0.80
12:00 PM	PF6	0.38
12:00 PM	Ethiopia hotel	0.42
12:00 PM	Junction 37	0.45
Date	11-Nov-14	
Time	Sample	Residual chlorine(mg/l)
2:00 PM	Water Tank	0.80
2:00 PM	PF6	0.44
2:00 PM	Ethiopia hotel	0.40

2:00 PM	Junction 37	0.13
Date	12-Nov-14	
Time	Sample	Residual chlorine(mg/l)
4:00 PM	Water Tank	0.80
4:00 PM	PF6	0.48
4:00 PM	Ethiopia hotel	0.03
4:00 PM	Junction 37	0.19
Date	13-Nov-14	
Time	Sample	Residual chlorine(mg/l)
6:00 PM	Water Tank	0.80
6:00 PM	PF6	0.09
6:00 PM	Ethiopia hotel	0.30
6:00 PM	Junction 37	0.20
Date	14-Nov-14	
Time	Sample	Residual chlorine(mg/l)
8:00 PM	Water Tank	0.80
8:00 PM	PF6	0.00
8:00 PM	Ethiopia hotel	0.29
8:00 PM	Junction 37	0.26
	data test-3	
Date	15-Nov-14	
Time	Sample	Residual chlorine(mg/l)
8:00 AM	Water Tank	0.80
8:00 AM	PF6	0.43
8:00 AM	Ethiopia hotel	0.33
8:00 AM	Junction 37	0.32
Date	16-Nov-14	
Time	Sample	
10:00: AM	Water Tank	0.80
10:00: AM	PF6	0.42
10:00: AM	Ethiopia hotel	0.40
10:00: AM	Junction 37	0.35
Date	17-Nov-14	
Time	Sample	Residual chlorine(mg/l)
12:00 AM	Water Tank	0.80
12:00 AM	PF6	0.20
12:00 AM	Ethiopia hotel	0.37

12:00 AM	Junction 37	0.32
Date	18-Nov-14	
Time	Sample	Residual chlorine(mg/l)
2:00: PM	Water Tank	0.80
2:00: PM	PF6	0.01
2:00: PM	Ethiopia hotel	0.40
2:00: PM	Junction 37	0.13
Date	19-Nov-14	
Time	Sample	Residual chlorine(mg/l)
4:00 PM	Water Tank	0.80
4:00 PM	PF6	0.48
4:00 PM	Ethiopia hotel	0.03
4:00 PM	Junction 37	0.19
Date	20-Nov-14	
Time	Sample	Residual chlorine(mg/l)
6:00 PM	Water Tank	0.80
6:00 PM	PF6	0.22
6:00 PM	Ethiopia hotel	0.25
6:00 PM	Junction 37	0.24
Date	21-Nov-14	
Time	Sample	Residual chlorine(mg/l)
8:00 PM	Water Tank	0.80
8:00 PM	PF7	0.15
8:00 PM	Ethiopia hotel	0.29
8:00 PM	Junction 37	0.26

Appendix-Q: Bacteriological Test Results

Ser .no	Sampling Nodes	Location			Field data			Remark
		Easting(m)	Northing (m)	Altitude(m)	date-time	E-coli/100ml	Total coliform /100ml	
1	Hot Spring	476940	851726	1612m	1/10/2014 ,8:30:00 AM	3	TNTC	High risk
2	Wet Well	479764	846357	1599	1/10/2014 ,9:30:00 AM	Nil	6	low risk
3	PF2	476951	849656	1598	1/10/2014 ,10:30:00 AM	Nil	4	,,
4	HTP-42	466518	851788	1600	1/10/2014 ,11:30:00 AM	Nil	3	,,
5	HTP-46	461478	852893	1602	1/10/2014 ,12:30:00 AM	Nil	2	,,
6	Ethiopian hotel	461723	852879	1604	1/10/2014 ,1:30:00 PM	Nil	3	,,
7	WSS	461324	852680	1605	1/10/2014 ,2:30:00 PM	Nil	3	,,
8	Junction-50	461586	852637	1609	1/10/2014 ,3:30:00 PM	Nil	7	,,
9	Juntion-46	460870	852988	1600	1/10/2014 ,4:30:00 PM	Nil	4	,,
10	Juntion-33	461349	853257	1604	1/10/2014 ,5:30:00 PM	Nil	3	,,
11	Juntion-27	461412	852949	1603	1/10/2014 ,6:30:00 PM	Nil	2	,,
12	Juntion-20	462038	852665	1601	1/10/2014 ,7:30:00 PM	Nil	5	,,
13	Juntion-25	462036	852442	1601	1/10/2014 ,8:30:00 PM	Nil	7	,,
14	Juntion-54	461821	852263	1599	1/10/2014 ,9:30:00 PM	Nil	2	,,
15	PF8	464022	851351	1604	1/10/2014 ,10:30:00 PM	Nil	3	,,
16	Junction-13	469139	851268	1597	1/10/2014 ,11:30:00 PM	Nil	8	,,
17	PF5	472845	850586	1589	1/10/2014 ,12:30:00 PM	Nil	3	,,
18	Junction-7	469050	851089	1596	1/10/2014 ,1:30:00 PM	Nil	4	,,
19	Juntion - 12	470939	851049	1598	1/10/2014 ,2:30:00 pM	Nil	2	,,
20	Juntion-40	461080	853059	1600	1/10/2014 ,3:30:00 PM	Nil	3	,,

Appendix R: Water Tank level Calibration data’s

Time	Tank level		
hr	computed(m)	observed(m)	difference(m)
20	2.18	2.12	0.06
21	1.42	1.22	0.2
22	1.56	1.4	0.16
23	1.71	1.68	0.03
24	1.85	1.75	0.1
25	1.96	1.8	0.16
26	1.92	1.85	0.07
27	1.94	1.82	0.12
28	1.93	1.9	0.03
29	1.93	1.85	0.08
30	2.06	1.9	0.16
31	2.04	1.95	0.09
32	2.04	1.9	0.14
33	2.04	1.94	0.1
34	2.04	1.96	0.08
35	2.04	1.9	0.14
36	2.04	2	0.04
37	1.99	2.01	-0.02
38	2	1.99	0.01
39	2	1.98	0.02
40	2	1.97	0.03
41	2	1.96	0.04
42	2	1.94	0.06
43	1.79	1.8	-0.01
44	1.93	1.96	-0.03
45	1.83	1.9	-0.07
46	1.9	1.91	-0.01
47	1.86	1.93	-0.07
48	1.88	1.92	-0.04

$$R^2 = \frac{(\quad \bar{})(\quad \bar{})}{\sqrt{\ (\quad \bar{})\ (\quad \bar{})}}$$

R^2=0.8385

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

Time(hr)	observed link flow(l/s)	computed link flow(l/s)	differences(l/s)
20	36.00	38.20	-2.20
22	35.00	38.20	-3.20
24	8.60	11.46	-2.86
26	8.50	11.46	-2.96
28	8.70	11.46	-2.76
30	8.65	5.80	2.85
32	8.40	5.73	2.67
34	8.50	5.73	2.77
36	8.60	7.64	0.96
38	8.40	7.64	0.76
40	8.40	7.64	0.76
42	17.90	17.19	0.71
44	18.40	17.19	1.21
46	19.00	17.19	1.81
48	19.50	17.19	2.31

$$R^2 = \frac{(\quad \bar{\ })(\quad \bar{\ })}{\sqrt{(\quad \bar{\ })\ (\quad \bar{\ })}}$$

R^2=0.9569

Appendix T: Water Tank level Validation data's

Time	Tank level		
hr	observed(m)	computed(m)	Difference
20	2	2.15	0.15
21	1.35	1.42	0.07
22	1.5	1.56	0.06
23	1.68	1.71	0.03
24	1.8	1.85	0.05
25	1.92	1.96	0.04
26	1.89	1.92	0.03
27	1.89	1.94	0.05
28	1.87	1.93	0.06
29	1.89	1.93	0.04
30	1.99	2.06	0.07
31	1.98	2.04	0.06
32	1.95	2.04	0.09
33	1.98	2.04	0.06

34	1.95	2.04	0.09
35	1.98	2.04	0.06
36	1.99	2.04	0.05
37	1.99	1.99	0
38	2.02	2	-0.02
39	2.03	2	-0.03
40	1.9	2	0.1
41	1.95	2	0.05
42	1.92	2	0.08
43	1.75	1.79	0.04
44	1.84	1.93	0.09
45	1.9	1.83	-0.07
46	1.93	1.9	-0.03
47	1.94	1.86	-0.08
48	1.95	1.88	-0.07

$$R^2=\frac{(\quad)(\quad)}{\sqrt{(\quad)\ (\quad)}}$$

R^2=0.8921

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

Time(hr)	observed link flow(l/s)	computed link flow(l/s)	differences(l/s)
20	36.00	38.20	-2.20
22	37.50	38.20	-0.70
24	9.00	11.46	-2.46
26	8.90	11.46	-2.56
28	9.00	11.46	-2.46
30	4.90	5.73	-0.83
32	4.80	5.73	-0.93
34	4.90	5.73	-0.83
36	6.80	7.64	-0.84
38	6.90	7.64	-0.74
40	6.85	7.64	-0.79
42	16.00	17.19	-1.19
44	16.70	17.19	-0.49
46	15.90	17.19	-1.29
48	28.00	30.56	-2.56

$$\mathbf{R^2}=\frac{(\quad)(\quad)}{\sqrt{(\quad)\ (\quad)}}=0.9952$$

Appendix V: Comparisons of Water Quality of standards of the Spring.

Parameter	Tuffa spring	WHO Guideline	Recommended for Ethiopia
PH(pH scale)	7.68	6.5 – 8.5	5.0-9.5
TDS (mg/l)	282	1000	2000
Total hardness $CaCO_3^{(mg/l)}$	69.3	300	600
Turbidity (NTU)	Trace	5	5
Fl (mg/l)	1.72	1.5	4
Na (mg/l)	78	200	
Fe (mg/l)	0.06	0.3	3
Mn(mg/l)	Trace	0.1	

So4(mg/l)	245.9	400	600
Chloride(mg/l)	14.4	250	800
Nitrate(mg/l)	0.39	10	40