

**KWAME NKRUMAH UNIVERSITY OF SCIENCE AND TECHNOLOGY**

**KUMASI, GHANA**



**MODELLING AND SIMULATING FREE CHLORINE DECAY  
USING A PILOT DISTRIBUTION SYSTEM**

**KOFI AKODWAA-BOADI  
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**COLLEGE OF ENGINEERING**

**DEPARTMENT OF CIVIL ENGINEERING**

**MODELLING AND SIMULATING FREE CHLORINE DECAY  
USING A PILOT DISTRIBUTION SYSTEM**

**By**

**KOFI AKODWAA-BOADI**

**BSc. Chemistry (Hons.)**

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In Partial Fulfillment of the Requirements for the degree of

**MASTER OF SCIENCE**

**In**

**Water Supply and Environmental Sanitation**

**August, 2012**

**CERTIFICATION**

I hereby declare that this submission is my own work towards the M.Sc. and, that to the best of my knowledge, it contains no material previously published by another person nor material which has been accepted for the award of any other degree of the University, except where due acknowledgement has been made in the text.

Kofi Akodwaa-Boadi .....  
(PG 4777810) Signature Date

**Certified by:**

Dr. Richard Akwasi Buamah .....  
(Principal Supervisor) Signature Date

Mr. E. K Baah-Ennumh .....  
(Supervisor) Signature Date

Prof. M. Salifu .....  
(Head of Department) Signature Date

**DEDICATION**

This work is dedicated to the Lord God Almighty

And to my Father

Akodwaa-Boadi, K.A.N

And my Mother,

Maame Serwaa

**ABSTRACT**

Residual chlorine decays “naturally” in water due to its reaction with constituents in the bulk water and/or materials (like Fe and Mn Oxides, biofilm etc.) deposited on pipe walls. As such, residual chlorine concentrations at the taps have often been found to be low and water quality is compromised at the expense of the consumer’s health. This study aimed at modelling chlorine decay in the Kumasi Water Distribution Network. This was achieved by determining the chlorine decay coefficients (bulk ( $K_b$ ), overall wall ( $K_w$ ) and overall decay ( $K$ )) using 6 inch diameter pipes of PVC (15 - 20 years old), AC (40 - 50 years old) and Cast Iron pipes (84 years old) which are in use in the Kumasi Water Distribution Network. The decay was further simulated for 3 and 9 inches pipe diameters of the pipes under study. A double - jacketed batch reactor and a Pilot Distribution System were used in the experimental determination of the bulk and wall decay coefficients respectively. The first-order decay kinetics as developed by Rossman was applied to the empirical data, and under the conditions tested,  $K_b$  was found to be  $0.053 \text{ hr}^{-1}$  at  $26 \text{ }^\circ\text{C}$  whereas  $K_w$  was found to be 0.25, 0.19 and  $0.38 \text{ hr}^{-1}$  for PVC, AC and Cast Iron pipes respectively. Furthermore, the overall decay coefficient ( $K$ ) was found to be 0.30, 0.24 and  $0.43 \text{ hr}^{-1}$  respectively for PVC, AC and Cast Iron pipes. The decay in the Cast Iron pipe was greatest and may be due to large deposits of probably oxides of Iron and manganese observed in the pipe lumen. It was also observed, that the overall decay in the aged PVC pipe was 2.14 times that of the newer pipe. In addition, the simulation revealed that chlorine consumption in 3 inch diameter pipes is greater. Using the SynerGEE<sup>®</sup> hydraulic model of the Kumasi Water Distribution Network and the decay coefficients obtained from the experiment, the model predicted, that in order to maintain residual chlorine concentrations  $\geq 0.20 \text{ mg/L}$  in the entire distribution network, the Achiase booster, Suame reservoirs, Pankrono, Buokrom and KNUST booster points should maintain a free residual chlorine concentration of 1.0, 0.74, 0.50, 0.50, 0.50 mg/L respectively.

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**LIST OF ABBREVIATION**

|        |   |  |
|--------|---|--|
| AC     | - | Asbestos Cement                                      |
| AWWA   | - | American Water Works Association                     |
| AWWARF | - | American Water Works Association Research Foundation |
| CI     | - | Cast Iron  |
| DBP    | - | Disinfection By-product                              |
| DO     | - | Dissolved Oxygen                                     |
| DVM    | - | Discrete Volume Method                               |
| EDM    | - | Event Driven Method                                  |
| FDM    | - | Final Difference Method                              |
| Fr     | - | Froude number  |
| GSA    | - | Ghana Standards Authority                            |
| GUWL   | - | Ghana Urban Water Limited                            |
| GV     | - | Guideline Value                                      |
| MG     | - | Million Gallons                                      |
| NOM    | - | Natural Organic Matter                               |
| PDS    | - | Pilot Distribution System                            |
| PVC    | - | Polyvinyl Chloride                                   |
| TDM    | - | Time-Driven Method                                   |
| TDS    | - | Total Dissolved Solids                               |
| THM    | - | Trihalomethane                                       |
| TOC    | - | Total Organic Carbon                                 |
| TOC    | - | Total Organic Carbon                                 |
| U.V    | - | Ultra Violet   |
| WTP    | - | Water Treatment Plant                                |

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## CHAPTER 1

### 1.0 INTRODUCTION

#### 1.1 Background

Water scarcity already affects every continent and around 1.2 billion people, or almost one-fifth of the world's population, live in areas of physical scarcity. This means that one in every six people lack access to safe drinking water ([www.un.org/waterforlifedecade](http://www.un.org/waterforlifedecade)). In Africa, the struggle for access to clean drinking water is one of today's most obvious examples of how water scarcity leads to the stalling and reversal of human progress and sub-Saharan Africa has the largest number of water-stressed countries on the globe ([www.uneca.org/awich/Water in Africa](http://www.uneca.org/awich/Water%20in%20Africa)). While each individual living in the United States uses on average 100 to 175 gallons of water per day in the home, the average African family uses only 5 gallons of water per day ([www.un.org/waterforlifedecade](http://www.un.org/waterforlifedecade)). In Ghana today, about 63 % of the urban populace are supplied with piped water and according to a recent survey conducted by Water and Sanitation for the Urban Poor, more than 900,000 people lack access to safe drinking water in Kumasi, Ashanti region.

Although water is a scarce commodity, its quality should be assured no matter how small the quantity may be. Thus, the goal of a water treatment and distribution system is to deliver water in sufficient quantities and of good quality where and when it is needed at the required pressure.

Firstly, this quality can be expressed in terms of the water being physically and chemically free from carcinogens and contaminants that impart colour, taste and odour. Secondly, water quality can be expressed as water devoid of pathogenic microorganisms. This suggest that ideally, treatment processes at water treatment plants as well as pipes and storage facilities of a good drinking water distribution system should constitute a network of uncontrolled chemical and biological reactors capable of withstanding significant variations to maintain

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water quality. But unfortunately, even before these pipes are networked to form a distribution system, their sterility may not be guaranteed during storage and transportation to construction sites and they may have been left outdoors for months or years leading to contamination potentially caused by a variety of animal, plants and microbiological life entering it.

Consequently, after installation, if such mains are not properly “purified” and flushed, then although treated water may meet the required quality criteria when it leaves the treatment headworks and travels through such a distribution system, the quality will deteriorate. In cases where these distribution mains are properly disinfected and flushed after installation, as water continues to run through for some number of years, there may be sediment build up (which may lead to encrustation), corrosion and subsequent leaching of pipe materials, formation of biofilms etc. These occurrences normally compromise the quality of the distributed water. Generally, some indicators of water quality deterioration in distribution systems include;

- Loss of disinfectant residual,
- Corrosion of iron pipes,
- Dissolution of Pb and Cu from pipe walls,
- Biofilm formation,
- Occurrence of compounds that confer poor taste and odour,
- Formation of disinfection by-products (some of which are carcinogens). These compounds are normally products of reactions between organic and/or inorganic soluble compounds and disinfectants.
- Increased turbidity caused by particulate re-suspension.

To attain microbiological safety, potable water should be free of pathogenic microorganisms, and this is achieved through purification by a specific treatment step called disinfection.

Generally, disinfection can be defined as the inactivation of pathogenic microorganism purposed to eliminate any microbiological risks of water-related diseases. The use of U.V light, ozonation and chlorination are all forms of disinfection, but chlorination is usually preferred and employed in most drinking water systems because of its efficiency and durability (leaves adequate residuals) and it is relatively cheap (Mays, 2000).

Thus, to reduce the microbiological risk of potable water, it should have adequate residual chlorine to ensure bacteriological safety of the water. In view of this, the World Health Organisation (W.H.O) recommends a chlorine residual concentration of 0.5 mg/L in final water whereas the Ghana Standard Authority (GSA) recommend a concentration not lesser than 0.2 mg/L at the point of consumption. To ensure that potable water is bacteriologically safe at the taps, a chlorine residual concentration between 0.20 – 0.5 mg/L should be maintained in the treated water (WHO, 1997). However, the physical, chemical and microbiological transformations (existing pipe material/age, water age, biofilm formation, encrustation etc.) occurring within and along a distribution system may defeat this purpose by “using up” the chlorine thereby enhancing its decay. This decay may occur within the bulk of the water (bulk decay) or as a result of the interaction between the water and pipe walls (wall decay). Therefore, to ensure that adequate chlorine residuals persist in distribution lines till consumption, studies must be done to find out how these transformations occurring along a particular distribution system contribute to the Bulk and Wall reactions.

## **1.2 Urban Water Supply in Ghana**

The Ghana Water Company Limited (GWCL) was established on 1<sup>st</sup> July 1999, following the conversion of Ghana Water and Sewerage Corporation into a state-owned limited liability company under the Statutory Corporations (Conversion to Companies) Act 461 of 1993 as amended by LI 1648. Presently, the company operates 83 urban water supply systems

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throughout the country and the design capacity of all the systems is about 949,000 m<sup>3</sup>d<sup>-1</sup>. Presently, potable water demand in the urban areas is estimated at about 1,101,032 m<sup>3</sup>d<sup>-1</sup> while average daily production is about 691,690.90 m<sup>3</sup>d<sup>-1</sup>. Effective urban water supply is about 63 % and customer strength is currently 421,363 of which 42.8 % are metered and 57.2 % unmetered (<http://www.gwcl.com.gh>).

On 22<sup>nd</sup> November 2005, the GWCL signed a management contract with Vitens Rand water services to operate the urban water systems and this contract came to an end on 6<sup>th</sup> June 2006. Presently, a new company, Ghana Urban Water Limited (GUWL) has been formed by government to take over the management of urban water systems in the country for 12 months.

### **1.2.1 Water supply in Kumasi**

The Kumasi Water Supply System currently consists of two treatment plants; the Barekese and the Owabi headworks. The Owabi headworks was constructed in 1924 and currently, it produces an average of 2.7 million gallons (MG) of treated water daily whereas the Barekese headworks started operations in 1974, and it currently produces an average of 22 million gallons (MG) of treated water daily. Water supply to the main city centre is via the Barekese headworks. The Kumasi Water Distribution Network has a total length of approximately 1,016,958 m and it constitutes different pipe materials with the three most aged ones being Asbestos Cement (AC, 40 – 50 years), Polyvinyl chloride (PVC, 15 – 20 years) and Cast Iron (CI, 84 years). Some challenges faced by the distribution network are illegal connections, pipe burst (due to pipe age or high pressures) and low residual chlorine concentrations.

According to existing protocols for production at the water treatment plants, disinfection residuals in the final water should range between 0.2 – 0.5 mg/l, and 0.1 – 0.2 mg/l at the farthest point of the distribution system. However, these production requirements are not met

sometimes because the G.U.W.L currently has no empirical data on the physical, chemical or microbiological transformations occurring within their distribution system and how that affects the chlorine residual decay (bulk and wall decay).

The rate of chlorine decay can be affected by several factors including microbiological activity, Water quality/chemistry (including physico-chemical parameters), Hydraulic effects, Pipe material and age. Thus, in this research the following parameters will be considered to find out their contributions to chlorine residual decay;

- **Physico-chemical parameters:** Temperature, pH, Colour, Turbidity, Conductivity, Total Dissolved Solids (TDS), Calcium, Magnesium, Nitrate, Nitrite, Total iron, Manganese, Free Chlorine residual.
- **Bacteriological tests:** Total coliform, *E. Coli*, *Salmonella*.

### **1.3 Problem statement**

Residual chlorine decays “naturally” in water as a result of the reaction of chlorine with the bulk water and/or materials (like Fe and Mn, biofilms, encrustation etc.) deposited on pipe walls. Therefore, the possibility of pipe wall demand or encrustations contributing to chlorine consumption should not be underestimated. Furthermore, brown deposits and particles are usually observed in tap water after supply disruptions. As such, free residual chlorine concentrations at the taps may be low and the water quality may be compromised at the expense of the consumer’s health. However, there is no empirical data to predict this chlorine loss and to substantiate top-up quantities in the distribution network. Thus, it is necessary to predict this chlorine loss and the top-up quantities in order to maintain required residual chlorine concentrations throughout the distribution system.

## **1.4 Justification**

This research intends to determine the chlorine decay coefficients (bulk, wall and overall decay coefficients) per the pipe materials under investigation. Secondly, using these decay coefficients to model chlorine decay in the Kumasi Water Distribution Network would assist in predicting nodes where free chlorine residuals are below required standards so that probable chlorine boosting points could be identified and the appropriate chlorine concentrations predicted. Finally, this research will provide data for future research and references, and research findings could serve as a basis for policy.

## **1.5 Research question**

Is there a way of maintaining free chlorine residual concentrations within acceptable concentrations in the Kumasi Water Distribution Network?

## **1.6 Objectives**

### **1.6.1 Main objective**

To model and simulate chlorine residual decay in the Kumasi Water Distribution Network under prevailing distribution conditions.

### **1.6.2 Specific objectives**

- To determine the bulk and overall wall decay coefficients ( $K_b$  and  $K_w$  respectively), and the overall decay coefficient ( $K$ ) for Cast Iron (84 years), Asbestos Cement (40 – 50 years) and Polyvinyl Chloride (15 – 20 years) pipes under distribution conditions (e.g. pressure, water age).
- To assess the effect of water quality, pipe material, pipe age and hydraulic retention time on chlorine residual decay.

- To use data obtained to simulate free residual chlorine decay in different diameter of AC, PVC and CI pipes found in the Kumasi Water Distribution network.
- To model chlorine decay in the Kumasi Water Distribution Network and predict probable points within the distribution network for chlorine boosting.

## CHAPTER 2

### 2.0 LITERATURE REVIEW

#### 2.1 Disinfection

In water supplies, disinfection may be defined as the process of using chemical or physical means to inactivate harmful microorganisms that might be present inside the lumen of distribution mains or in filtered water in order to protect distributed water from pathogen re-growth or recontamination (Mays, 2000).

##### 2.1.1 Types of disinfection

The use of U.V light, Ozonation and Chlorination are all forms of disinfection, but Chlorination is usually preferred and employed in most drinking water systems because of its efficiency and capacity to leave adequate residuals and relatively cheaper to apply. However, it must be noted, that certain enteroviruses, cysts of *Giardia intestinalis* and Oocysts of *Cryptosporidium* are resistant to the bactericidal effects of chlorine and its residual compounds.

#### 2.2 Chlorination

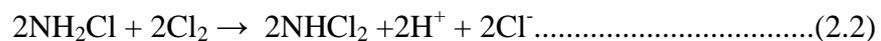
It is a chemical disinfection method that uses various types of chlorine or chlorine-containing substances. Apart from its ability to disinfect, chlorine controls taste, odour and colour in water. Chlorine was first discovered in Sweden by a Chemist, Karl W. Scheele in 1774 and it was used to remove odour from water in 1835, but it was not until 1890 that chlorine was found to be an effective agent for disinfecting; a way to reduce the amount of disease transmitted through water. With this new find, chlorination began in Great Britain and then expanded to the United States in 1908 and Canada by 1917. Today, chlorination is the most popular method of disinfection and it is used for water treatment all over the world. Chlorine inactivates a microorganism by damaging its cell membrane. Once the cell membrane is

weakened, the chlorine can enter the cell and disrupt cell respiration and DNA activity; two processes that are necessary for cell survival (www.safewater.org).

In order to guarantee the water supply systems' disinfection, adequate residual concentrations are needed to prevent re-contamination by pathogenic or indicator micro-organisms, which can originate in the biofilm formed or in negative pressure areas (created by pipe cracks, fissures, etc.) of the water supply system.

### **2.2.1 Chlorine demand and breakpoint chlorination**

Chlorine demand may be defined as the difference between the amount of chlorine added to a water system and the amount of free available chlorine or combined available chlorine remaining at the end of a specified time period. In other words, the demand is the amount of chlorine consumed by oxidation or substitution reactions with microbiological, organic and inorganic materials, such as H<sub>2</sub>S, Fe<sup>2+</sup>, Mn<sup>2+</sup>, NH<sub>3</sub>, phenols, amino acids, proteins, and carbohydrates. When all these demands are met, a point is reached when any further addition of chlorine to a water supply system leaves adequate residuals for the purposes of disinfection. The reactions leading to the breakpoint reactions are as shown;



Equation 2.4 is the complete reaction step, which is called the chlorine breakpoint reaction.

The principal purpose of breakpoint chlorination is to ensure that adequate chlorine residuals are left for disinfection after satisfying the chlorine demand of the water.

## **2.2.2 Stages in Chlorine disinfection**

To ensure efficiency, chlorine disinfection is typically carried out in three stages. These are primary, secondary and booster disinfection.

### **2.2.2.1 Primary disinfection**

It is the treatment step used to inactivate pathogens at the treatment headworks. It is accomplished using a variety of chemicals, the three most common being;

- Liquid chlorine ( $\text{Cl}_2$ ) which is usually available in 45.5 kg or 68 kg pressurised containers,
- Sodium hypochlorite ( $\text{NaOCl}$ ) which is a liquid stored in glass, rubber-lined or plastic containers of varying sizes. It is usually 5 – 15 % chlorine but it has finite shelf-life,
- Calcium hypochlorite ( $\text{Ca(OCl)}_2$ ) which is approximately 65 % chlorine by weight.

### **2.2.2.2 Secondary or post disinfection**

It refers to the practise of adding additional disinfectant to the treated water before it is released to the distribution system to maintain residual (leftover) disinfection capability.

### **2.2.2.3 Booster disinfection**

It is the practise of adding disinfectants at the points in the distribution system beyond the point of raw water treatment. It is typically used at the entrance point to a service area being supplied by a wholesale provider or at distant points in a system subject to large disinfectant demands.

### **2.2.3 Disinfection of distribution mains**

According to Mays (2000), three methods are employed in the disinfection of distribution mains; the tablet, continuous and slug method.

#### **2.2.3.1 The Tablet method**

This method can be used if the distribution mains have been kept clean and dry during construction and it involves placing hypochlorite granules or tablets in the pipes during installation at intervals no greater than 150 m. The number of 5.0 g tablet per length of pipe can be estimated from:

$$N = 0.0012 LD^2 \dots\dots\dots(2.5)$$

Where N = number of 5.0 g of tablet

L = length of pipe (ft)

D = diameter of pipe (in)

The mains must be filled with potable water at a velocity slower than 1ft/s (0.3 ms<sup>-1</sup>) making sure to eliminate all air pockets. If the temperature is higher than 5 °C, the water must be kept in the pipe for at least 24 hours, if the temperature is lower than 5 °C, it must be kept in the pipe 48 hours. The tablet method is acceptable for smaller mains, mains without solvent welded or threaded steel joints.

#### **2.2.3.2 The Continuous method**

Consists of first flushing the mains at a velocity of at least 0.76 ms<sup>-1</sup> (2.5 ft/s) to remove any sediment and air pockets. For larger mains, where flushing may not be effective, brooming or swabbing can be used. Chlorine must then be fed at a rate that maintains a concentration of 25.0 mg/L for 24 hours. At the end of that time, the free chlorine residual must be greater than 10.0 mg/L at all points in the pipe.

### **2.2.3.3 The Slug method**

This method consists of placing hypochlorite granules as in the tablet method, and flushing the mains as in the continuous feed method. Then a slug of highly chlorinated water with a concentration of at least 100 mg/L is passed through the mains so that the slug does not drop below 50 mg/L. The slug method is used most commonly for larger mains, where the volume of water required for the continuous feed method is impractical.

Highly chlorinated water must be disposed off in an environmentally safe manner in compliance with all applicable water quality regulations. This may necessitate the use of a reducing chemical (e.g. sodium sulphite) at the downstream end of the pipe being disinfected to react with excess chlorine in the water.

### **2.2.4 Disinfection of storage tanks**

The AWWA Standard C652 (1992 b) recognises three methods for disinfecting storage tanks when they have been taken out of service and drained.

#### **2.2.4.1 Method 1**

Liquid chlorine or sodium hypochlorite is added to the influent pipe while filling or calcium hypochlorite is placed on the bottom of the tank before filling so that the chlorine concentration shall be at least 10 mg/L after the retention period. A 6 hour retention period is used when chlorine has been fed uniformly with the influent water, whereas a 24-hour period is used when the chemicals are mixed into the tank.

#### **2.2.4.2 Method 2**

The tank is sprayed or brushed entirely with a solution of 200 mg/L of available chlorine. Drain pipes are filled with a solution of 10 mg/L chlorine, as in method 1. The highly chlorinated solution is left on the surface for at least 30 minutes. Only surfaces that will be in contact with potable water need to be sprayed or brushed. Upon filling and bacteriological

testing, this method may produce water that can be delivered to the distribution system rather than be discharged.

### **2.2.4.3 Method 3**

Chlorine is added using the procedures in method 1, except that the target concentration when the tank is approximately 5 % full is 50 mg/L. This water is held for 6 hours, then the tank is slowly brought to full level with potable water and the water is kept in the tank for an additional 24 hours. Following bacteriological testing, this tank may be placed in service, provided any drain lines have been purged of highly chlorinated water and the chlorine residual is at least 2 mg/L.

### **2.2.5 Reaction of chlorine with water**

According to Gang (2003), when chlorine gas is added to water, rapid hydrolysis forms Hydrochloric (HCl) and Hypochlorous acid (HOCl) as shown:



At pH values above 4.4, essentially no molecular chlorine remains in aqueous solution. HOCl is a weak acid with a  $\text{pK}_a$  of 7.5 (at 25 °C), which further hydrolyzes to yield a hydronium ion ( $\text{H}_3\text{O}^+$ ) and a hypochlorite ion ( $\text{OCl}^-$ ) according to the following equation:

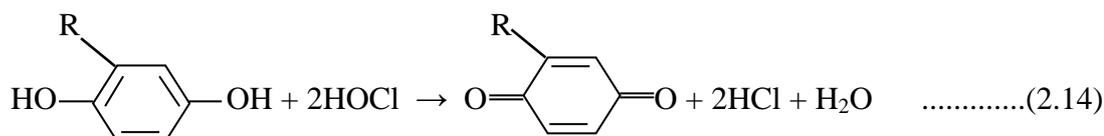
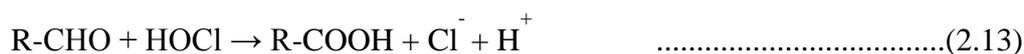
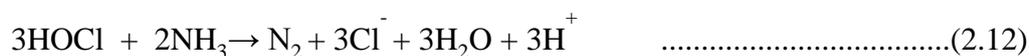
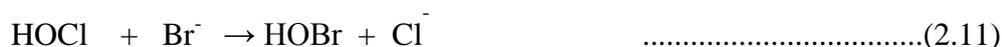


With increasing pH, at a given temperature, the fraction of the anionic species increases. Chlorine added to water “disappears” by numerous pathways. In general, four kinds of reactions lead to the “disappearance” of chlorine (Gang *et al.*, 2003):

- Oxidation,
- Addition,
- Substitution and

- Catalyzed or light decomposition.

When oxidation of a substance by HOCl occurs, the  $\text{Cl}^+$  radical accepts two electrons from the substance being oxidized to become a chloride ion ( $\text{Cl}^-$ ). The following equations are common chlorine oxidation reactions occurring in waters:

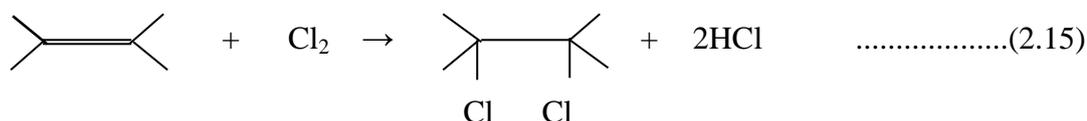


Where R is an alkyl substituent.

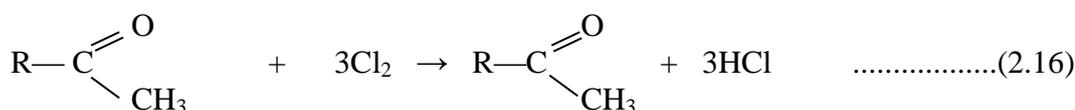
Equations (2.8) to (2.12) are the reactions of HOCl with inorganic water constituents, which cause the immediate chlorine residual consumption. Equations (2.13) and (2.14) are the reactions between HOCl and the functional groups that exist in the Natural Organic Matter (NOM) molecules in water. All of the above reactions do not produce chlorinated Disinfection By-Products (DBPs), as they only consume the chlorine residuals.

In addition and substitution reactions, chlorine is added or substituted into the NOM molecular structure to form chlorinated organic intermediates, which might further decompose to form many of the DBPs (Van Hoof, 1992). Addition reactions with free chlorine could be significant in waters containing highly unsaturated plant pigments.

However, they are slow reactions unless the double bonds are activated by substituent groups (Brezonik, 1994). In equation 10, the alkyl groups are substituent groups that activate the double bounds;



Substitution reactions involving chlorine are usually electrophilic (Brezonik, 1994) substitution (Equation 11).



The decomposition of hypochlorous acid (HOCl) at pH 5 to 9 proceeds according to the following reaction stoichiometry (Adam *et al.* 1992):



The most common forms of chlorine residual referenced in technical literature and in environmental regulations are listed below:

*Free Available Chlorine Residual:* concentration of hypochlorous acid and hypochlorite ions existing in chlorinated water.

*Free Chlorine Residual:* refers to chlorinated water in which at least 85 % of the total measured chlorine residual is hypochlorous acid.

*Combined Chlorine Residual:* refers to the chlorine residual that consists of chloramines.

*Total Chlorine Residual (or Total Available Chlorine):* refers to the sum of Free Available Chlorine Residual and Combined Chlorine Residual.

Excessive levels of chlorine produce taste and odour problems, and accelerate pipe corrosion and subsequently enhance formation of harmful by-products.

### **2.2.5.1 Formation of disinfectant by-products**

Disinfection by-products (DBPs) are formed when the disinfectant used reacts chemically with the organic material in the treated water, and the types of DBPs formed vary with the type of primary disinfectant used. For instance, chlorine can react with naturally occurring organic matter in treated water usually of humic origin to produce literally hundreds of halogenated by-products, most of which are present in only trace quantities. Some of these by-products such as Chloroform and Dichloroacetic acid have been found to be potential human carcinogens. These by-product produced in the greatest quantities as a result of disinfection are Trihalomethanes (THMs) - methane ( $\text{CH}_4$ ) molecules in which three of the hydrogen atoms have been replaced by some combination of chlorine and bromine atoms. According to Regli (1993), the main factors influencing formation of DBPs include temperature, pH, contact time, chlorine dose and residual, concentration of precursors and bromide ions. pH levels can increase in some storage facilities caused by the leaching of hydroxides and carbonates from concrete surfaces, thereby increasing the potential for formation of THMs.

Generally, chlorine half lives might be reduced to only few hours in systems with large demands. This reduction of half life is termed chlorine decay.

## **2.3 Chlorine decay**

By their very nature, disinfectants are extremely reactive and do not persist for long, and in that regard, most waters exhibit a rapid and immediate consumption of chlorine when the chemical is first added during primary disinfection. Again, as the water travels through the distribution system, physical, chemical or microbiological transformations may occur within the bulk of the water as well as through interaction with the pipe wall which also leads to

chlorine consumption or decay. Theoretically, these decay can be broadly classified as bulk or wall decay, but put together, they constitute overall decay.

### 2.3.1 Overall Decay

In a first order reaction involving bulk and pipe wall reactions, the overall rate constant  $K$  ( $T^{-1}$ ) that incorporates both the bulk and the wall reactions can be written as,

$$\frac{dC}{dt} = -K_b C - K_w C \quad \dots\dots\dots(2.18)$$

$$\frac{dC}{dt} = - (K_b + K_w) C$$

$$\frac{dC}{dt} = - KC$$

Integration of this equation gives;

$$C_t = C_o e^{-Kt} \quad \dots\dots\dots(2.19)$$

$$\text{Thus, } K = \frac{1}{t} \ln (C_o/C_t) \quad \dots\dots\dots(2.20)$$

Where  $C_o$  = Initial residual chlorine concentration (mg/L)

$C_t$  = Final residual chlorine concentration (mg/L) at time  $t$ .

$K$  = Overall decay constant ( $T^{-1}$ ) =  $K_b + K_w$

$K_b$  = Bulk reaction rate coefficient ( $T^{-1}$ )

$K_w$  = Overall wall reaction rate coefficient ( $T^{-1}$ )

Note that even if  $K_b$  and  $K_w$  were the same throughout a system, the apparent rate,  $K$  could still vary from one pipe to the other because of the variety in pipe material. It is therefore essential to measure chlorine residuals because it suggests the activity or inactivity of microorganisms, growth of biofilms and the variance in water quality (due to contamination resulting from leakages or cross-connections) along a distribution system.

### 2.3.2 Bulk Decay

While a substance travels down a pipe or resides in storage, it can undergo reactions with constituents in the water column. The rate of the reaction, R can generally be described as a power function of concentration;

$$R = -K'C^n \dots\dots\dots(2.21)$$

Where  $K'$  = a reaction constant

n = reaction order

In 1995, Vasconcelos *et al.*, applied this rate expression to distribution systems as;

$$\frac{dC}{dt} = -K'C \dots\dots\dots(2.22)$$

This is the first order kinetic expression for the decay of residual chlorine due to reactions with materials in the aqueous phase at different residence times. In this equation, C is the chlorine concentration (mg/L),  $K'$  is the first-order decay constant ( $T^{-1}$ ) (and in the bulk water phase, it is denoted by  $K_b$ , the bulk decay coefficient) and t is the residence time (T).

Integration of Equation 2.22 gives;

$$C_{(t)} = C_o e^{-K_b t} \dots\dots\dots(2.23)$$

Where  $C_{(t)}$  = chlorine concentration (mg/L) at time t

$C_o$  = initial chlorine concentration (mg/L)

$K_b$  = bulk decay coefficient ( $hr^{-1}$ )

t = is the hydraulic retention time (s)

The bulk decay rate constants can be adjusted for temperature variations using the Van't Hoff – Arrhenius equation;

$$K_T = A \times e^{(-E_a/RT)} \dots\dots\dots(2.24)$$

By means of this equation, the rate constant ( $K_T$ ) at any operating water temperature (T), can be calculated from the rate constant defined at a base temperature;

$$K_{b1} = K_{b2} \times \theta^{(T1-T2)} \dots\dots\dots(2.25)$$

$$K_T = K_{b2} \times \theta^{(T-T2)} \dots\dots\dots(2.26)$$

Where Ea = activation energy

R = ideal gas law constant

T = temperature

$K_T$  = dissipation constant at temperature T

$K_{b1}$  = bulk dissipation constant at temperature T1

$K_{b2}$  = bulk dissipation constant at temperature T2

A = constant

$\theta$  = temperature correction factor

### 2.3.3 Wall Decay

While flowing through pipes, dissolved substances can be transported to the pipe wall to react with materials such as iron released (because of the corrosion) or the organic slime associated with a biofilm which is also found on or close to the pipe wall. The amount of wall area available and the rate of mass transfer between the bulk fluid and the wall will also influence the overall rate of the reaction. By comparing wall and bulk decay coefficients, Cook *et al.*, (1997) suggested that the greatest level of total chlorine residual decay occurs as a result of the combination of wall decay and mass transfer. Castro *et al.*, (2003) concluded that the contribution of pipe walls is more significant than that of bulk fluid reactions. For first order kinetics, the expression for the wall reaction rate by Rossman, Clark and Grayman (1994) is as given by;

$$\left( \frac{dC}{dt} \right)_{wall} = \left[ - \frac{A}{V} \right] \times K_{w'} \times C_w \dots\dots\dots(2.27)$$

Where  $K_w$  = Wall reaction rate constant ( $LT^{-1}$ )

$A$  = Surface area ( $L^2$ )

$V$  = Pipe volume ( $L^3$ )

For a pipe fully filled with water, the hydraulic radius reduces to 4 divided by the pipe diameter;

$$\left(\frac{dC}{dt}\right)_{\text{wall}} = \left[-\frac{4}{D}\right] \times K_w \times C_w \dots\dots\dots(2.28)$$

Where  $D$  = pipe diameter (m)

$C_w$  = chlorine concentration at the pipe wall (mg/L)

Chlorine concentration at the pipe wall,  $C_w$ , is difficult to measure and an expression based on the residual concentration in the bulk liquid,  $C$ , should be used instead.  $K_w$  must be adjusted for mass transfer limitations that are dependent on the flow regime within the pipe segment. A film resistance model was developed by Rossman, Clark and Grayman (1994), which shows the mass transfer of chlorine towards the pipe wall is proportional to the difference of chlorine concentration between the bulk liquid and the pipe wall;

$$\left(\frac{dC}{dt}\right)_{\text{mass transfer}} = \left[-\frac{4}{D}\right] \times K_f \times (C - C_w) \dots\dots\dots(2.29)$$

Where  $K_f$  = Mass transfer coefficient

$D$  = Pipe diameter

$C$  = Chlorine concentration in the bulk

$C_w$  = Chlorine concentration at the pipe wall

The mass transfer coefficient has to be determined for the flow conditions. Thus, if it is assumed that chlorine reacts as it reaches the pipe wall and there is no accumulation (Rossman, Clark and Grayman (1994)), the rate of mass transfer of the disinfectant would be equal to the rate of chlorine decay at the pipe wall as shown in Equation 2.30;

$$\left(\frac{dC}{dt}\right)_{\text{wall}} = \left(\frac{dC}{dt}\right)_{\text{mass transfer}} \dots\dots\dots(2.30)$$

This implies that,  $-\frac{4}{D} \times K_{w'} \times C_w = -\frac{4}{D} \times K_f \times (C - C_w)$

$$K_{w'} \times C_w = K_f \times (C - C_w)$$

A first order differential equation for chlorine dissipation at the pipe wall can be developed by solving this equation for  $C_w$  and substituting in Equation 2.28;

$$C_w = \frac{K_f C}{(K_{w'} + K_f)}$$

This implies that,  $\left(\frac{dC}{dt}\right)_{\text{wall}} = \left(-\frac{4}{D}\right) \times K_{w'} \times \frac{K_f C}{(K_{w'} + K_f)}$

But Rossman (2000) expressed the wall decay  $K_w$  as;

$$K_w = \frac{4K_{w'} K_f}{D(K_{w'} + K_f)} \dots\dots\dots(2.31)$$

Thus the resulting model is;

$$\left(\frac{dC}{dt}\right)_{\text{wall}} = -K_w C \dots\dots\dots(2.32)$$

Where  $K_w$  = overall wall reaction rate ( $T^{-1}$ )

$K_{w'}$  = wall decay constant ( $LT^{-1}$ )

$K_f$  = mass transfer coefficient ( $LT^{-1}$ )

$D$  = Pipe diameter (m)

Mass transfer coefficients ( $K_f$ ), are usually expressed in terms of a dimensionless Sherwood number (Sh) ;

$$K_f = \frac{Sh \times d}{D} \dots\dots\dots(2.33)$$

Where d = Molecular diffusivity of the species (in this case chlorine) being transported ( $L^2/T$ )

D = Pipe diameter (L).

In fully developed Laminar flow ( $Re < 2000$ ), the average Sherwood number along the length of a pipe can be expressed as;

$$Sh = 3.65 + \frac{0.0668 (D/L) Re \times Sc}{1 + 0.04 [(D/L) Re \times Sc]^{2/3}} \dots\dots\dots(2.34)$$

Where Reynolds number ( $Re$ ) =  $\frac{V \times D}{\nu}$  .....(2.35)

Schmidt number ( $Sc$ ) =  $\frac{\nu}{d}$  .....(2.36)

Kinematic viscosity ( $\nu$ ) =  $\frac{497 \times 10^{-6}}{(T + 42.5)^{1.5}}$  ( $L^2T^{-1}$ ) .....(2.37)

V = Velocity of flow ( $LT^{-1}$ )

For Turbulent flow ( $Re > 4000$ ), the empirical correlation of Notter and Sleicher (1971) can be used;

$$Sh = 0.0149 Re^{0.88} Sc^{0.33} \dots\dots\dots(2.38)$$

Rossmann (2000) provided the Sherwood number as a function of the Reynolds number (Re), for stagnant conditions;  $Re < 1$ , and  $Sh = 2.0$

Thus, if the overall wall decay constant ( $K_w$ ) and the mass transfer coefficient ( $K_f$ ) is known, then the wall decay constant ( $K_w$ ) can be computed.

## **2.4 Parameters affecting Chlorine residual decay**

Four main categories of parameters are associated with chlorine decay – Water quality/chemistry, Pipe material, Microbiological activity (Biofilm formation) and Hydraulic effects.

### **2.4.1 Water quality/chemistry**

A water quality problem may be related to a physical occurrence, which in-turn can lead to other chemical or microbiological problems. Some of these factors are discussed.

#### **2.4.1.1 pH and Temperature**

In chemically pure water, molecular chlorine reacts with water and rapidly hydrolyzes to hypochlorous acid (HOCl) and hydrochloric acid (HCl). At pH levels above 4.0 and in dilute solutions, the hydrolysis reaction is completed within a fraction of a second. For all practical purposes, the reaction is irreversible.

Hypochlorous acid is a weak acid and dissociates to form a hydrogen ion (H<sup>+</sup>) and a hypochlorite ion (OCl<sup>-</sup>). The concentration of each species at equilibrium depends on pH and temperature. Between pH 6.5 and 8.5, the dissociation reaction is incomplete, and both hypochlorous acid and hypochlorite ions are present.

At constant pH and increasing temperature, chemical equilibrium favours the OCl<sup>-</sup> ion over HOCl. Shi and Taylor, (2007) reported from a PDS, that residual chlorine dissipated too rapidly during the summer because of high temperature.

Also, the speed at which chemical substances are oxidized (by OCl<sup>-</sup> or HOCl) is determined by pH and temperature. As pH increases, the chemical reactivity of chlorine decreases, and as temperature increases, reactions proceed more rapidly. Mutoti *et al.*, (2007) observed that the bulk decay coefficient increases by threefold for every 10 °C rise in temperature.

#### **2.4.1.2 Iron and Manganese Build-up**

Dissolved Iron and Manganese in the main source water can precipitate out of the solution and settle in storage facilities, particularly if there's a long detention time or an oxidant such as chlorine or oxygen is present. Iron and Manganese and other precipitates settled in a finished water storage facility may be re-introduced to the finished water during rapid discharge from the facility. Since chlorine is a strong oxidising agent, iron and manganese in their +2 states use up chlorine residuals to oxidise to their +3 and +4 states respectively. Furthermore, Manganese dioxide is nearly insoluble and it is carried as a colloidal suspension in water.

#### **2.4.1.3 Leachate from internal coatings**

Chemicals contained in protective coatings/linings may leach out into the finished water over time depending on the chemical composition, the rate of migration, and the water temperature. An American Water Works Association Research Foundation (AWWARF) study of five organic coatings used in potable water distribution systems determined that drinking water contaminants at levels above 1 µg/L are common during the first thirty days in service (Alben *et al.*, 1989). Organic paint and coatings often contain nutrients that can support bacterial growth, and this enhances residual chlorine decay, as it is used up to inactivate the bacteria cells.

#### **2.4.1.4 Hydrogen Sulphide occurrence due to Sulphates**

Larson and Skold (1958) studied the corrosion of iron materials and found that sulphate promoted corrosion which in turn enhances residual chlorine decay. Hydrogen Sulphide gas is an aesthetic concern because of its rotten egg odour. It has a greater tendency to form in distribution systems when the following conditions are present; high levels of sulphate ions, sulphate-reducing bacteria, excess electrons and little or no DO. The hydraulic retention time

in the distribution and storage system is a contributing factor, affecting the reaction time and the resultant hydrogen sulphide concentration (Pettie, 1990). An aeration or ventilation system at a storage facility also may improve conditions.

#### **2.4.1.5 Nitrification**

It is a 2-step process caused by bacteria that enhances the conversion of ammonia to nitrite, then to nitrate. Certain water quality and system conditions can result in water distribution system nitrification. Important water quality factors include chloramines residual, pH, temperature, ammonia concentration, chlorine-to-ammonia-nitrogen ratio and concentrations of organic compounds. Important system factors include system detention time, reservoir detention and operation, sediment and tuberculation in piping, biofilm and absence of sunlight.

Nitrification can have several effects on water quality. For example, it may increase heterotrophic bacterial populations and degrade chloramines residuals in the distribution system. Nitrification can be reduced by decreasing the overall detention time and practise breakpoint chlorination. Operationally, changing the chlorine-to-ammonia ratios may help to reduce the occurrence of nitrification by reducing the available ammonia.

#### **2.4.1.6 Sediment build-up**

Sediments are potential causes of water-quality degradation by contributing to the waters' chlorine demand. Sediments frequently accumulate in storage tanks where the velocities are minimal and they can be re-suspended (because of surges in flow) to exert demand on the residual chlorine. One utility traced elevated levels of coliform bacterial to accumulated sediment (Beuhler *et al.*, 1994). In another study, the authors speculated that a taste and odour problem was linked to the bottom sediment, where substrate is abundant for diverse biological growth (Burlingame and Brock, 1995). Once settling has occurred, one method of

preventing the accumulated sediments from re-entering the distribution system and exerting demand on the residual chlorine is to avoid flow situations that would scour the bottom surface, cleaning the storage facilities to minimize sediments build-up and regular flushing of the distribution system.

#### **2.4.1.7 Water age**

Deterioration in water quality is frequently associated with the age of the water. Water age is the amount of time that water remains in the distribution system after it leaves the treatment headworks. The water quality changes that can occur within finished water-storage tanks and reservoirs are influenced significantly by the degree of mixing and subsequent residence time that water experiences within these facilities. Loss of disinfectant residual can result from aging of water because, as water stagnates in distribution systems, there is more interaction between the water and the pipe walls - which usually, has a higher demand for chlorine. As a result, an implicit objective in both the design and operation of distribution storage facilities is the minimization of detention time and the avoidance of parcels of water that remain in the storage facility and distribution lines for long periods.

For a storage facility operating in fill-and-draw mode, the detention time can be estimated by dividing the duration of an average fill-and-draw cycle by the fraction of the water that is exchanged during the cycle. Mathematically, this can be stated by;

$$\text{Average detention time} = [0.5 + (V/\Delta V)] (\tau_d + \tau_f)$$

Where  $\tau_f$  = fill time

$\tau_d$  = draw time

V = Volume of water at start of the fill period

$\Delta V$  = Change in water volume during the fill period

Sharp *et al.*, (1991) showed how chlorine residuals can vary throughout the day at different locations in a water distribution system depending on the residence time of the water, and in 2007, Shi and Taylor also observed that iron released decreased with the reduction in hydraulic retention time. Furthermore, simulations conducted by Mutoti *et al.*, (2007) revealed a considerable loss of chlorine residuals at a reduced Reynolds number that can be attributed to the dominant effect of the increased hydraulic retention time.

#### **2.4.2 Pipe material**

Chlorine decay is a function of a number of site-specific properties such as pipe material (Sharp *et al.*, 1993). Different pipe materials demonstrate different levels of chlorine demand; LeChevallier *et al.*, (1990) found that free chlorine residuals achieved greater biofilm inactivation compared to chloramine for PVC and copper pipes. For the galvanized pipes, monochloramine provided greater biofilm inactivation than free chlorine, and the iron pipes seemed to exert the greatest disinfectant demand. In the same study, the disinfectant demand of biofilm on iron pipes was as much as ten times greater than for biofilms grown on other pipe materials.

Wable *et al.*, (1991) observed that chlorine decay rates in single lengths of pipe is several times greater than the decay rate of the same water in a flask, suggesting that the pipe wall can contribute to overall chlorine demand. Kiéné, Lu and Lévi (1998) found that chlorine was consumed more rapidly in unlined old cast iron and steel than in synthetic material pipes. In a related study, Mutoti *et al.*, (2007) observed differences between the overall first-order dissipation coefficient for Galvanised, Unlined Cast Iron, PVC and Lined Cast Iron, implying that different pipe materials exert different rates of decay on chlorine residuals. The research also showed that for the same blend of water, chlorine residual decay varied differently with different pipe materials.

#### **2.4.2.1 Hydraulic effect**

Chlorine decay is a function of a number of site-specific properties such as flow rate (Sharp *et al.*, 1993). Kiene *et al.*, (1993) found that the decay rate can be correlated directly with the flow regime. There are two theoretical ways in which water can flow through a storage tank or reservoir; in a completely mixed state or in a completely unmixed state. These two ideal flow regimes are termed mixed and plug flow.

In mixed flow, water entering the reservoir mixes instantaneously and completely with water already in the reservoir, resulting in a uniform mixture at all times. The composition of water discharging from the reservoir is the same as the uniform composition in the reservoir at the time of discharge.

In plug flow, “fresh” water travels through the reservoir without mixing with the water already in the reservoir. This produces a first-in-first-out ordering. Many factors affect the actual mixing processes in a reservoir, thus resulting in flow that is neither fully mixed nor plug flow. Factors that can lead to non-ideal flow conditions include thermal effects interaction between inlets and outlets, resulting in short-circuiting, stagnant zones - in which flow is incomplete and small scale eddies. Unit processes such as chlorine contact chambers and clear wells at water treatment plants are generally designed for plug-flow operations to ensure that all parcels of water experience approximately the same residence time. A major factor influencing the selection of a flow regime is the minimization of disinfectant loss. Since the rate of disinfectant loss with respect to time is concentration dependent (Fair *et al.*, 1948), the higher the concentration of chlorine, the faster it decays because of reactions with other constituents in the water. When the reaction kinetics exhibit this type of behaviour, a tank operating under plug flow will have more disinfectant loss than it would have under

mixed flow (Grayman *et al.*, 1999). Under turbulent flow conditions, there is greater chlorine mobility from the bulk phase to the wall.

Plug flow is best achieved in long, narrow basins or through the use of baffling, diffusers or stilling chambers to reduce flow momentum.

### **2.4.3 Microbiological Activity**

Microorganisms can be introduced into open reservoirs from windblown dust, debris and algae. Covered storage facilities are generally better protected than open reservoirs; however, they can be susceptible to airborne microorganisms entering through access hatches, roofs, vents and other penetrations. Several hundred cases of illness and five deaths were reported in a Missouri community that had a system where bird droppings and avian remains were found inside a closed storage tank (Atkinson, 1995).

Additionally, microorganisms can enter from outside sources such as poorly constructed or inadequately maintained storage facility covers, roofs, or side wall joints and faulty vents, hatches, and other penetrations. For example, several documented cases of *Salmonellosis* were caused by drinking water reservoirs contaminated with bird droppings (Ongerth, 1971; Smith and Burlingame, 1994). However, it must be stated, that whether a person contracts these diseases or not depends on the type of pathogen, the number of organism (virulence), the volume of water ingested, and the susceptibility of the individual. Other sources of microbial contamination include inadequately treated water, surface or ground water infiltration into ground storage facilities and water mains breakage (Antoun *et al.*, 1995).

#### **2.4.3.1 Bacterial growth**

Smith *et al.*, (1990) identified the following factors that provide a favourable environment for microbial growth in the distribution system; the seasonal variation in water temperature, the availability of growth-promoting nutrients and minerals, the occurrence of corrosion products

in the distribution system, distribution system disinfection practises, and distribution hydraulics (flow and velocity). Factors that provide optimum conditions for microorganisms to multiply include long water-detention times, adequate nutrient levels, and warm temperatures. The presence of microorganisms can be a regulatory concern and can contribute to increased chlorine demand. Organic paints and coatings often contain nutrients that can support bacterial growth (Sonntag, 1986). This process can lead to biocorrosion of the reservoirs structure or to increased porosity of the walls, thus creating spaces for bacterial colonies. All materials used in the distribution system are not necessarily to contact water directly, however in some instances, poor linings and fittings have enabled these materials to come into direct contact with water and release chemical agents that may be nutritive to bacteria.

#### **2.4.3.2 Biofilm**

Biofilms are often a consortium of different microorganisms bound to each other and to pipe surfaces by a polysaccharide matrix. Biofilms can also provide ecological niches that are suited to the potential survival of pathogens (Walker and Morales, 1997). They play a major role in the microbial characterisation of drinking water quality in distribution systems. These films or slimes become established in sediments, corrosion tubercles, static areas of slow water flow, dead ends, and storage tanks. It is important to note, that not all pipe sections will show evidence of deterioration even after years of service, the reason being the nature of the chemistry of water in the system and the continuous movement of water in high demand areas (Geldreich and LeChavelier, in press). Water supply reservoirs and stand pipes may also accumulate deposits of biofilm on the side walls and in sediments on the bottom of the tank. These deposits are a particular nuisance in stratified static water because of anaerobic growth of various organisms that can cause taste and odours. For organisms that can endure

this harsh aquatic environment, growth proceeds slowly at first as the organisms adjust to the constraints of the pipe environment. In time, these microcolonies attract other organisms (such as *Legionella*), the more exacting nutritional needs of which may be found only in the by-products released in the metabolism of various organisms in the pioneering microbial community. Thus a progressive diversity is brought into the biofilm as the site becomes populated with a variety of bacteria, protozoans, nematodes and worms as Geldreich, (1996) observed. Biofilms provide a variety of microenvironments for growth including aerobic and anaerobic zones caused by limited oxygen diffusion (Characklis and Marshall, 1990). Within this complex structure is a community of diverse organisms embedded in a matrix of extracellular organic polymers adhering to moist surfaces. This matrix is interlaced with water channels that have been reported to constitute as much as 40 – 60 % of the total volume of biofilm in a conceptual model proposed at the Centre for Biofilm Engineering in 1994.

The most drastic measures of biofilm control have involved the replacement of old pipe sections that did not respond to elevated levels of disinfectants and flushing. In Halifax, Nova Scotia, a 5.0 km section of new pipe was lined with cement to seal off *Klebsiella* colonisation of wood forms left in the construction of a new pipe section (Martin *et al.*, 1982). Avoiding the incipient releases of biofilm can be achieved through manipulation of water chemistry (pH, alkalinity, Langlier Index) or the careful application of corrosion inhibitors not only to protect pipe materials but also to firm up the coating of sediments that harbour microbial communities.

LeChavelier *et al.*, (1990) showed that disinfection of organisms on iron pipes was ineffective even when the organisms were exposed to 5.0 mg/L of free chlorine for several weeks. However, using 1.0 mg/L of free chlorine or monochloramine was effective for disinfection of biofilm on Galvanised, Copper and P.V.C pipes. Although the occurrence of

biofilm coliform may not be an immediate public health concern, they should not be ignored because their contamination suggests;

- Existence of a habitat that could be used by pathogens
- Possible leaks in the distribution system and
- The accumulation of chlorine-demanding material that interferes with maintaining a disinfectant residual throughout the pipe network.

There is always a concern that biofilm occurrences could be shielding faecal contamination either from inadequate treatment or contamination of the distribution system. Any occurrences of faecal coliform bacteria or *E. coli* during a biofilm episode should not be brushed aside as an aberration in the laboratory results. These occurrences suggest very recent contamination because these organisms are not normally able to colonise biofilms permanently.

## **2.5 Water Quality Modelling**

According to Mays (2000), chlorine residuals have been found to decay as water travels through distribution systems and the loss of chlorine residual is a chemical process resulting in the decrease of the secondary disinfectant, generally either free chlorine or total chlorine. In that regard, water quality managers over the years have used monitoring as a means to understand these residual transformations. But, it is difficult to use monitoring alone to understand the fate and transformations of chlorine as the water travels through a distribution system; even medium-sized cities can have thousands of miles of pipes, making it impossible to achieve widespread monitoring. Furthermore, the flow pathways and travel times of water through these systems are highly variable because of the looped layout of the pipe network and the differing peak factors in the water usage over space and time coupled with the use of storage facilities in the system makes things even more variable. Thus, it is usually

impractical to experiment on the entire distribution system by seeing how changes in pumping schedules, storage facilities operations or treatment methods affect the quality of water received by the consumer.

For these reasons, Mays (2000) states that mathematical modelling of water quality behaviour in distribution systems has become an attractive supplement to monitoring. These models offer a cost effective way to study the spatial and temporal variations of a number of water quality constituents including;

- The fraction of water originating from a particular source,
- The age of water in the system,
- The concentration of non-reactive tracer compound either added or removed from the system (example; Fluoride or Sodium),
- The concentration and growth rate of disinfection by-products such as THMs and
- The numbers and mass of attached and free-flowing bacteria in the system.

The water quality model can be used to assist managers to perform a variety of water-quality related studies. Examples include the following;

- Locating points of low residuals to control chlorine injecting rate at booster stations in order to maintain desired disinfection levels throughout the system,
- Calibrating and testing hydraulic models of the system through the use of chemical tracers,
- Locating and sizing storage facilities and modifying system operations to reduce the age of the water,
- Modifying the design and the operations of the system to provide a desired blend of waters from different sources,

- Assessing and minimising the risk of consumer exposure to disinfection by-products and the systems vulnerability to incidents of external contamination.

### **2.5.1 History of Water - Quality Models**

Although the use of mathematical models for hydraulic analysis of distribution systems dates back to the 1930's (Cross, 1936), water - quality models of distribution systems are a relative recent development. Early work was limited to Steady state representations of networks. In a study of slurry flow in a pipe network, Wood (1980) presented an extension to a steady state hydraulic model in which a series of simultaneous equations are solved for each node. A similar formulation was used by Chan and Selznick (1985) in a 166 – link representation of the Alameda County, California, Water District with three sources of water of differing hardness and by Metzger (1985) in studying blending, settling and flushing in distribution systems.

In a generalisation of this formulation, Males *et al.*, (1985) used simultaneous equations to calculate the spatial distribution of concentration, travel times, costs and any other variables that could be associated with links or nodes. This model called SOLVER was a component of the Water Supply Simulation Model, an integrated database management, modelling and display system (Clark and Males, 1986).

Steady state water quality models proved to be useful tools for investigating the movement of a contaminant under constant conditions. However the need for models that would represent the dynamics of contaminant movement led to the development of models that simulate the movement and transformation of contaminants in a distribution system under temporally varying conditions. Three of such models were initially introduced at the American Water Works Association Distribution System Symposium in 1986 (Clark *et al.*, 1986; Hart *et al.*, 1986; Liou and Kroon, 1986). Early applications of these models to trace contaminants and water age were reported by Grayman *et al.*, (1988) and Kroon and Hunt (1989). More recent

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developments include the effects of both bulk and wall reactions in simulations of chlorine residuals (Rossman *et al.*, 1994; Vasconcelos *et al.*, 1997), biofilm growth models (Servais *et al.*, 1995), and THM formation models (Vasconcelos *et al.*, 1996) and an important development in the 1990's was the development of hydraulic and water - quality models into graphically based user-friendly Packages. Examples of commercial network modelling packages that contain water quality Modelling capabilities include CYBERNET<sup>®</sup>, EPANET<sup>®</sup>, H2ONET<sup>®</sup> and Stonersoftware<sup>™</sup> SynerGEE<sup>®</sup>. Each of these approaches assumes that a hydraulic model has determined the flow direction and velocity of flow in each pipe at specific intervals over an extended period. Those intervals are referred to as hydraulic time step and the velocity within each pipe remains constant. Constituent transport and reaction proceed at smaller intervals of time known as the water - quality time step. Adjustments are made at the start of a new hydraulic time step to account for possible changes in flow velocity and direction.

## **2.5.2 Solution Methods**

### **2.5.2.1 Dynamic models**

They are models of water - quality distribution systems and they take explicit account of how changes in flows through pipes and storage facilities occurring over an extended period of system operation affects water quality. These models thus provide a more realistic picture of system behaviour. 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 while Lagrangian models track changes in a series of discrete parcels of water

as they travel through the pipe network. Two solution methods for dynamic models are discussed;

- Time-Driven Method (TDM) simulations

They update the state of the network at fixed time intervals. This Lagrangian method tracks the concentration and size of a series of non-overlapping segments of water that fill each link of the network (Liou and Kroom, 1987). As time progresses, the size of the most upstream segment in a link increases as water enters the link while an equal loss in size of the most downstream segments remain unchanged. This sequence of steps is repeated until the time when a new hydraulic condition occurs. The network is then re-segmented to reflect changes in pipe travel times, mass is re-apportioned from the old segmentation to the new one, and the computations are continued.

- Event-driven simulations (EDM)

They 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. This is a Lagrangian method that is similar to TDM, except that rather than updating the entire network at fixed time steps, individual link/ node conditions are updated only at times when the leading segment in a link completely disappears through its downstream node (Boulos *et al.*, 1995).

Comparison of the alternative solution techniques suggests that the Lagrangian TDM is the most efficient and versatile of the methods available for solving dynamic water - quality network models.

### **2.5.2.2 Steady State Models**

They compute the spatial distribution of water quality throughout a pipe network under assumptions that hydraulic conditions do not change and that storage does not affect water

quality. Although steady state models are simple to set up and solve, their restrictive assumptions limit their applicability. Two solution methods for steady state models are discussed;

- Final Difference Method (FDM)

It is an Eulerian approach that approximates the advective transport equations derivatives with their finite differences equivalents along a fixed grid of points in time and space (Islam *et al.*, 1997).

- Discrete Volume Method (DVM)

It is an Eulerian approach that divides each pipe into a series of equally-sized completely-mixed volume segments (Grayman *et al.*, 1988; Rossman *et al.*, 1993). At each successive water-quality step, the concentration within each volume segment is then reacted and then transferred to the adjacent downstream segment. When the adjacent segment is a junction node, the mass and flow entering the node is added to any mass and flow already received from other pipes. After these reactions, transport steps are completed for all pipes and the resulting mixture concentration at each junction node is computed and released into the first segments of pipes with flows leaving the node.

### **2.5.3 Data requirements**

#### **2.5.3.1 Hydraulic data**

A water quality model uses flow solutions of a hydraulic model as part of its input data and having a good hydraulic understanding of a network is essential for Modelling water quality challenges. However, a poorly calibrated model will invariably lead to a poorly performing water quality model.

### **2.5.3.2 Water quality data**

Dynamic models require a set of initial water quality conditions; one is to use the results from a field monitoring survey. This approach is often used when calibrating the model to field observations. Sites in the model corresponding to sampled sites can have initial quality set to the measured value.

### **2.5.3.3 Reaction rate data**

First-order rate constants for chlorine decay in the bulk flow can be estimated by performing a bottle test or by using a batch reactor in the laboratory. With the bottle tests, water samples are stored in several amber bottles and kept at a constant temperature. At several periods of time, a bottle is selected and analysed for the free chlorine. Similarly, with the batch reactor, water is stored inside at a constant temperature and sampled at various time intervals. At the end of the test, the natural-logarithm of the measured chlorine values is plotted against time. The rate constant is the slope of the straight line through these points. There is currently no similar direct test to estimate wall-reaction rate constants. Instead, one must rely on calibration against measured field data.

### **2.5.4 Model Calibration**

It is the process of adjusting characteristics and parameters of the model so that it matches actual observed field data to some acceptable level. If a mathematical model exactly represented the actual physical processes and if one had perfect knowledge of all the parameters required by the model, calibration would not be necessary. Unfortunately, neither criterion listed above is generally met, thus calibration is an important consideration in all studies.

Frequently, hydraulic models are calibrated to match pressures measured in the field. Since only pressure calibration does not guarantee that flows and velocities are accurately predicted

(Grayman, 1998), additional hydraulic calibration may be required when performing water quality monitoring.

For water quality models, adjustable model inputs include initial water quality conditions and reaction – rate coefficients and observable model outputs are pressures, flows, tank levels, and water – quality predictions.

One can perform two levels of calibration; one level serves as a reality check that the model is producing reasonable, but not necessarily highly accurate results. The modeller should check for the following problematic behaviour;

- Unreasonably low (example, negative) or high pressures
- Pumps operating outside of their allowable range or being shut down for this reason
- Pumps cycling on/off in an unreasonable fashion
- Tanks that continuously keep filling or emptying
- Nodes disconnected from any source because of closed pipes, pumps or valves.

The second level of calibration involves adjustments to model input parameters that match best with field observations.

#### **2.5.4.1 Calibration of conservative substances**

In water quality modelling of conservative substances and water age, the constituents travel with the flow and no transformations occur. At junctions, the constituent concentration or age is calculated based on a linear combination of the incoming inflows. As a result, no intrinsic calibration is required or possible. Field results can be matched only by varying inflow concentration or by the hydraulic parameters that control the flows in the system.

#### **2.5.4.2 Calibration of non-conservative substances**

The concentration of non-conservative substances changes over time as the substances travel through the distribution system because of reactions with other constituents in the water or through interaction with pipe walls and appurtenances. For transformations that involve interactions with the distribution system itself, limited general information is available, therefore laboratory and field data are generally required to establish both the form of the reaction and the reaction coefficients (Mays, 2000).

### **2.6 Water Quality Models**

#### **2.6.1 EPANET®**

After ground work for computer Modelling of distribution systems was laid down by Hardy Cross in the 1930's for analysing looped pipe networks (Cross, 1936), newer and more powerful algorithms were discovered (Epp and Fowler, 1970; Haman and Brameller, 1971) and techniques for Modelling non-pipe elements such as pumps and valves were developed (Chandrashekar, 1980; Jeppson and Davis, 1976).

EPANET is a public – domain, water-distribution system Modelling package developed by the U.S EPA's Water Supply & Water Resources Division. It performs extended – period simulation of hydraulic and water quality behaviour within pressurized pipe networks and it is designed to be a research tool that improves our understanding of the movement and fate of drinking – water constituents within distribution systems. EPANET first appeared in 1993 (Rossman, 1993).

EPANET models a distribution system using objects including Junctions (where pipe connects and water consumption occurs), Reservoirs (which represents fixed head boundaries), Tanks (which are variable – volume storage facilities), Pipes (which can contain either shut-off valves or check valves), Pumps (which can include fixed speed, variable

speed, constant horse-power pumps), Control valves (which can include pressure-reducing valves, pressure – sustaining valves, flow control valves, and throttle – control valves).

In addition to these physical objects, the following informational objects can also be used to represent a distribution system; Time patterns (sets of multipliers used to model diurnal water demands), Curves (x – y data used to represent head discharge curves for pumps and water level-volume curves of tanks), Operational controls (rules that change link status depending on such conditions as tank levels, nodal pressures, and time), Hydraulic analysis options (choice of headloss formula, flow units, viscosity and specific gravity), Water quality options (choice of type of water quality analysis, type of reaction mechanism and global reaction rate coefficient), Time parameters (simulation duration, time steps for hydraulic and water quality analyses, and time interval at which output results are reported (Rossman, 2000).

Furthermore, apart from its steady – state or extended – period hydraulic analysis, EPANET<sup>®</sup> can be used to run the following kinds of water quality analysis;

- Modelling the fate of chlorine and chloramines, which decay with time and can react both in the water phase and at the pipe wall,
- Modelling the growth of certain disinfection by-products such as THM's that grow with time, up to a limiting value,
- Tracking the propagation of a non-reactive constituent, such as one that would be used in tracer study or for reconstruction of a contamination event,
- Determining what percentage of water from a particular source is received by each location in a network,
- Estimating the age of water received at various locations in the network.

### **2.6.2 STONERSOFTWARE™ SynerGEE®**

SynerGEE® provides an integrated water quality Modelling capability to simulate the steady-state equilibrium, distribution, and time-varying (quasi-dynamic) propagation of waterborne substances, such as contaminants, disinfectants, and other water quality parameters. Water quality Modelling provides valuable information about the spatial and temporal concentrations of these substances. Inert or conservative substances (such as nitrates or fluoride) and reactive substances (such as chlorine or THMs) can be modeled. For reactive substances, reactions in the bulk and reactions at the pipe wall can be accounted for separately. Global bulk and wall reaction rate coefficients can be assigned, or each pipe can be assigned unique reaction rate coefficients. The effect of source water on bulk reaction rate coefficients can also be modeled. Furthermore, temperature correction can be applied to bulk and wall reaction rate coefficients. SynerGEE applies temperature correction, if necessary, before every first analysis after a time page. SynerGEE also computes the age of the water and the contribution each source makes at any point in the system. Source contribution information can be used to determine blended water quality parameters such as hardness, turbidity, and nitrate concentrations. These powerful water quality Modelling capabilities let you:

- Identify and correct stagnant water problems, low disinfectant residual problems, and a host of other water quality related problems,
- Investigate patterns of customer complaints about water quality,
- Focus on hydrant flushing efforts,
- Locate sampling locations and re-chlorination facilities,
- Identify the probable location of a cross-connection accident,
- Evaluate the effect of various operating schemes on storage turnover.

- Direct system maintenance efforts to improve circulation and quality.
- One can also simulate the spread of a contaminant through a network, evaluate the probable extent and concentration of contamination, and predict when the water is safe again (SynerGEE<sup>®</sup> Water 4.5.0 User Guide). In this study, the SynerGEE<sup>®</sup> hydraulic model of the Kumasi Water Distribution Network will be used together with bulk and wall decay coefficients obtained from the experiment to model the free residual chlorine decay in the Network. Further adjustments will be made to predict optimum dosages at various points in the distribution network .

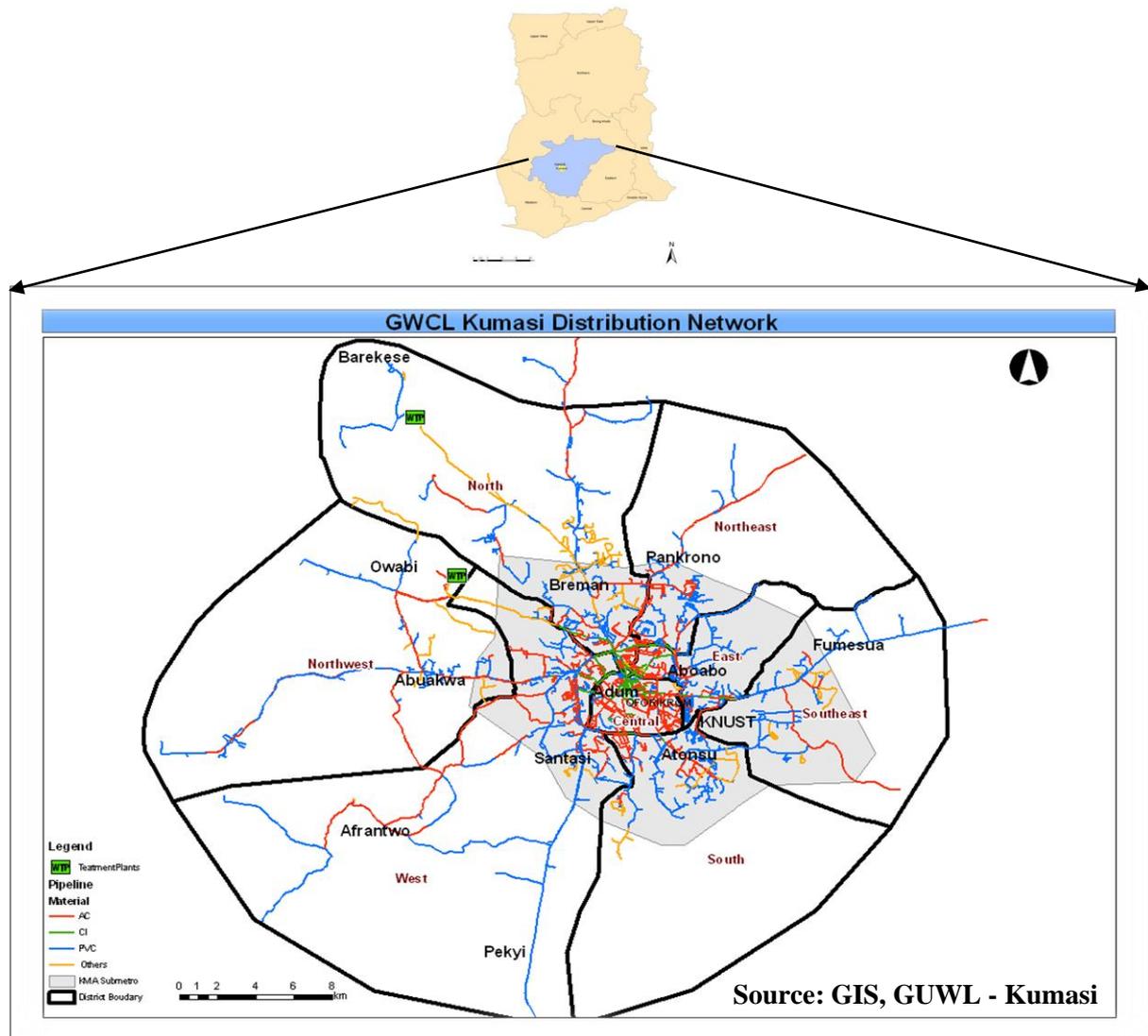
## CHAPTER 3

### 3.0 MATERIALS AND METHODS

#### 3.1 Study Area

The Kumasi Water Distribution Network has a total length of approximately 1,016,958 m and it constitutes different pipe materials such as Cast Iron (5 %), Ductile iron (0.001 %), Ductile Cast Iron (0.037 %), Asbestos Cement (35 %), Polyvynl chloride (48 %), High Density Polyethylene (8 %), Steel (3.4 %), Galvanised steel (0.033 %) and Copper (0.005 %). However, till date about 0.5 % of pipe materials have not yet been classified.

Plate 3 - 1: Map of Kumasi Water Distribution Network



The distribution system is currently fed by treated water from two treatment headworks; the Barekese and the Owabi headworks. The Owabi headworks was constructed in the year 1924 and currently, it produces an average of 2.7 million gallons (MG) of treated water daily. The system currently operates on three pumps, where two are in constant operation and one on standby. The pump has a Head of up to 9.5 bars and a maximum speed of 1485 rpm. The treatment process includes Aeration (using a cascade), Flocculation (the use of Alum), Sedimentation, Liming, Filtration, and Disinfection (use of Calcium Hypochlorite or Chlorine gas). The treated water is then pumped for distribution through a 10, 14 and 18 inches diameter pipes.

The Barekese headworks started operations in 1974, and it currently produces an average of 22 million gallons (MG) of treated water daily. Field observations made at the headworks revealed that it operates on three pumps with a head of up to 8.5 bars. The raw water is abstracted from the Offin River through a 360 mm diameter pipe. Treatment processes include Aeration (6 weir aerator), Flocculation (2 Clariflocculators with volume of 1 MG each), Filtration (24 number filters), Liming, disinfection (using Calcium Hypochloride or Chlorine gas). The filters have a 48 hour filter-run and backwashing is done every 2 days. Currently, 9 % of treated water is used in backwashing.

However, it must be noted, that areas where the pipelines were excavated for this study are areas supplied with water from the Barekese headworks, thus the experimental will assume prevailing water quality conditions at the Barekese headworks.

### **3.2 Preparation of “Model” water**

“Model” water was prepared to mimic the characteristics of the final water produced at the Barekese Headworks and in that regard, a five year (2007 - 2011) water quality data of the

finished water of the aforementioned headworks was obtained from the GUWL and the average value per parameter has been computed as shown;

Table 3 - 1: 5 year water quality data of the Barekese final water

| <b>Parameter</b>         | <b>Average value</b> | <b>Parameter</b>                    | <b>Average value</b> |
|--------------------------|----------------------|-------------------------------------|----------------------|
| <b>pH</b>                | 7.15                 | <b>Total Dissolved Solid (mg/L)</b> | 67.27                |
| <b>Temperature</b>       | 25.63                | <b>Sulphate (mg/L)</b>              | 35.79                |
| <b>Colour</b>            | 5.35                 | <b>Aluminium (mg/L)</b>             | 0.11                 |
| <b>Turbidity</b>         | 1.93                 | <b>Iron (mg/L)</b>                  | 0.20                 |
| <b>R-chlorine (mg/L)</b> | 0.74                 | <b>Manganese (mg/L)</b>             | 0.01                 |
| <b>E-coli</b>            | 0                    | <b>Fluoride (mg/L)</b>              | 0.52                 |
| <b>Hardness (mg/L)</b>   | 63.40                | <b>Nitrate (mg/L)</b>               | 0.20                 |
| <b>Chloride (mg/L)</b>   | 18.10                | <b>Arsenic (mg/L)</b>               | 0.00                 |

**SOURCE: GUWL, Regional Water Quality Assurance Laboratory, Kumasi**

The model water was thus prepared based on the following selected parameters;

Table 3 - 2: "Model" water characteristics

| <b>Parameter</b>              | <b>Value</b> |
|-------------------------------|--------------|
| Temperature (°C)              | 28 - 30      |
| pH                            | 6.5 - 8.5    |
| Free Chlorine Residual (mg/L) | 0.74         |
| Total iron (mg/L)             | 0.20         |
| Manganese (mg/L)              | 0.01         |
| Fluoride (mg/L)               | 0.52         |
| Sulphate (mg/L)               | 36.0         |

After chlorination, a hydraulic retention time of 30 minutes is allowed for bactericidal action on the model water.

### **3.3 Experimental procedure**

#### **3.3.1 Laboratory experiments to determine Bulk decay coefficient**

A 600 L plastic tank and an inert double-jacketed batch reactor were filled with chlorinated water of free residual chlorine concentration of 10 mg/L and made to stand overnight. A volume of 600 L of the model water was prepared in the reservoir, and then a chlorine demand test was carried out after which the water was dosed with 1 % hypochlorite solution to obtain a target residual chlorine concentration of 0.74 mg/L after a residence time of 30 minutes. A volume of 1.4 L of the chlorinated water (0.74 mg/L) was then transferred into the inert double-jacketed batch reactor which had the exterior covered with aluminium foil to exclude light rays from entering the reactor (See Plate 3 - 2). The entire system was then allowed to equilibrate for an hour after which hourly samples for residual chlorine measurement were taken for 8 hours. The experiment was repeated with a higher target dosage of 1.44 mg/L free residual chlorine.

Plate 3 - 2: Double-Jacketed batch reactor



### **3.3.2 Pilot-scale experiments**

A Pilot Distribution System (PDS) was constructed such that two pipe materials under test could be fitted parallel to each other (See Figure 3 - 1 and Plate 3 - 3 ). The pipe materials under test were PVC (15 - 20 years old), AC (40 - 50 years old) and Cast Iron pipes (84 years old). These pipe materials (all 6 inches in diameter) were existing service lines which were excavated from the Kumasi Water Distribution Network and each was cut into three pieces of 1.0 m lengths. For each set of the pipe material under test, the PDS was run with two batches of chlorinated model water. Since the 5 year water quality data obtained for the final water at Barekese headworks showed that the average free residual chlorine concentration is 0.74 mg/L, the first batch of chlorinated water was prepared to have the same influent concentration of 0.74 mg/L. This concentration was then doubled for the second batch of chlorinated water to yield an influent of 1.44 mg/L, so that probable effects of doubling the current concentration at the Barekese headworks could be studied. To determine the wall demand, a duplicate (out of a triplicate) of unit lengths of the same pipe material (starting with PVC pipes) which had already been excavated from existing service lines within the Kumasi distribution network were connected in parallel in the laboratory (as shown in the experimental set-up). The lines were then flushed continuously under pressure of 0.5 bars with tap water for several hours to equilibrate the system and also to obtain constant readings in terms of Colour, Turbidity, Conductivity and TDS.

In the first experimental protocol, model water was prepared and chlorinated to yield a target free residual concentration of 0.74 mg/L. In protocol 2, the water was then fed into the PDS at 0.5 bars, and samples were taken (via sampling taps) just at the entrance of the focus pipes to ascertain the residual chlorine concentration entering the pipe. Afterwards, the valves were closed so that the water stagnates in the focus pipes. After a hydraulic retention time of 2 and

4 hours, monitoring was done by sampling (my means of taps) and measuring residual chlorine concentrations at exit points of each of the 1.0 meter lengths. After monitoring, the water was pumped out and pipes were refilled with chlorinated water of same free residual chlorine concentration. Protocol 1 and 2 were then repeated for each pair of the hydraulic retention times 6 and 12, 18 and 24 hours before sampling and monitoring were done. These pipes were then replaced with the last of the triplicate PVC pipe and Protocols 1 and 2 were repeated. For protocol 3, a new batch of chlorinated water (second batch) with a target free residual chlorine concentration of 1.44 mg/L was prepared and protocol 1 and 2 were repeated for the various pair of hydraulic retention times. Thus, after all triplicate of the PVC pipes have been test-run, Cast Iron pipes were fixed into the PDS and protocols 1, 2 and 3 were repeated. After the Cast Iron pipes, AC pipes were connected and protocols 1 to 3 were again repeated. To serve as a control, newer pipes for the PVC was obtained and subjected to protocols 1, 2 and 3. However, for the AC and Cast Iron pipes, due to the unavailability of newer pipes, the encrustations in a single length of each pipe material was removed by mechanical means, washed thoroughly and used as a control. This is however a limitation to the work. It must also be emphasised, that it was ensured that the internal chemical and physical structures of the pipe lines were maintained during excavation, transport, storage and assembling of the PDS. Again, before the focus pipes were fixed in the PDS each time, it was ensured that newer PVC pipes were first fixed and the entire system was super-chlorinated and left to stand overnight to ensure that any microbe in the line is destroyed.

Rigorous quality assurance and quality control were performed throughout the project on all pilot and laboratory test data, including the use of control, duplicate and in some cases triplicate samples.

Figure 3 - 1: Pilot Distribution System for wall decay determination (Design)

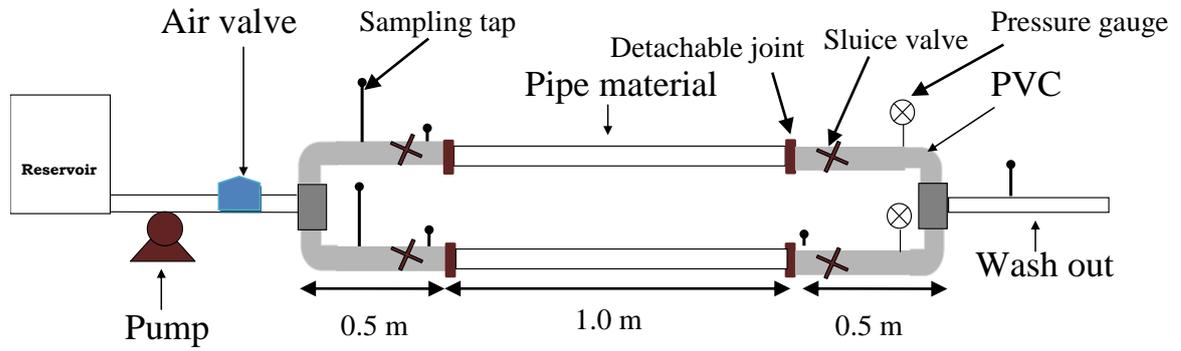


Plate 3 - 3: Pilot Distribution System (PDS)



1

<sup>1</sup> A – Influent Sampling Taps, B – Effluent Sampling Taps

### 3.4 Sampling

Table 3 - 3: Sampling protocol for PVC pipes when influent free residual chlorine concentration is 0.74 mg/L (1<sup>st</sup> Batch).

| Item                    | Number of samples per Hydraulic retention Time |         |         |          |          |          | Total     |
|-------------------------|--|---------|---------|----------|----------|----------|-----------|
|                         | 2 hours  | 4 hours | 6 hours | 12 hours | 18 hours | 24 hours |           |
| <b>Feed water</b>       | 1  |         | 1       |          | 1        |          | 3         |
| Control Pipe (Influent) | 1  |         | 1       |          | 1        |          | 3         |
| PVC 1 (Influent)        | 1  |         | 1       |          | 1        |          | 3         |
| Control Pipe (Effluent) | 1  | 1       | 1       | 1        | 1        | 1        | 6         |
| PVC 1 (Effluent)        | 1  | 1       | 1       | 1        | 1        | 1        | 6         |
| <b>Feed water</b>       | 1  |         | 1       |          | 1        |          | 3         |
| PVC 2 (Influent)        | 1  |         | 1       |          | 1        |          | 3         |
| PVC 3 (Influent)        | 1  |         | 1       |          | 1        |          | 3         |
| PVC 2 (Effluent)        | 1  | 1       | 1       | 1        | 1        | 1        | 6         |
| PVC 3 (Effluent)        | 1  | 1       | 1       | 1        | 1        | 1        | 6         |
| <b>Total</b>            | 14   |         | 14      |          | 14       |          | <b>42</b> |

2

<sup>2</sup> All feed water samples and effluent samples were also used for the determination of Fe, Mn, Ca<sup>2+</sup>, Mg<sup>2+</sup>, and NO<sub>3</sub><sup>-</sup> determination. Only one batch of experiment was carried out.

Table 3 - 4: Sampling protocol for AC pipes.

| Item                    | Number of samples when Influent free residual chlorine concentration is 0.74 mg/L (1 <sup>st</sup> Batch) |   |    |    |    |    | Number of samples when Influent free residual chlorine concentration is 1.44 mg/L (2 <sup>nd</sup> Batch) |   |    |    |    |    | Total     |
|-------------------------|---|---|----|----|----|----|---|---|----|----|----|----|-----------|
|                         | Hydraulic Retention Time (hours)  |   |    |    |    |    | Hydraulic Retention Time (hours)  |   |    |    |    |    |           |
|                         | 2   | 4 | 6  | 12 | 18 | 24 | 2   | 4 | 6  | 12 | 18 | 24 |           |
| <b>Feed water</b>       | 1   |   | 1  |    | 1  |    | 1   |   | 1  |    | 1  |    | 6         |
| Control Pipe (Influent) | 1   |   | 1  |    | 1  |    | 1   |   | 1  |    | 1  |    | 6         |
| AC 1 (Influent)         | 1   |   | 1  |    | 1  |    | 1   |   | 1  |    | 1  |    | 6         |
| Control Pipe (Effluent) | 1   | 1 | 1  | 1  | 1  | 1  | 1   | 1 | 1  | 1  | 1  | 1  | 12        |
| AC 1 (Effluent)         | 1   | 1 | 1  | 1  | 1  | 1  | 1   | 1 | 1  | 1  | 1  | 1  | 12        |
| <b>Feed water</b>       | 1   |   | 1  |    | 1  |    | 1   |   | 1  |    | 1  |    | 6         |
| AC 2 (Influent)         | 1   |   | 1  |    | 1  |    | 1   |   | 1  |    | 1  |    | 6         |
| AC 3 (Influent)         | 1   |   | 1  |    | 1  |    | 1   |   | 1  |    | 1  |    | 6         |
| AC 2 (Effluent)         | 1   | 1 | 1  | 1  | 1  | 1  | 1   | 1 | 1  | 1  | 1  | 1  | 12        |
| AC 3 (Effluent)         | 1   | 1 | 1  | 1  | 1  | 1  | 1   | 1 | 1  | 1  | 1  | 1  | 12        |
| <b>Total</b>            | 14  |   | 14 |    | 14 |    | 14  |   | 14 |    | 14 |    | <b>84</b> |

Table 3 - 5: Sampling protocol for CI pipes

| Item                    | Number of samples when Influent free residual chlorine concentration is 0.74 mg/L (1 <sup>st</sup> Batch) |   |    |    |    |    | Number of samples when Influent free residual chlorine concentration is 1.44 mg/L (2 <sup>nd</sup> Batch) |   |    |    |    |    | Total     |
|-------------------------|---|---|----|----|----|----|---|---|----|----|----|----|-----------|
|                         | Hydraulic Retention Time (hours)  |   |    |    |    |    | Hydraulic Retention Time (hours)  |   |    |    |    |    |           |
|                         | 2   | 4 | 6  | 12 | 18 | 24 | 2   | 4 | 6  | 12 | 18 | 24 |           |
| <b>Feed water</b>       | 1   |   | 1  |    | 1  |    | 1   |   | 1  |    | 1  |    | 6         |
| Control Pipe (Influent) | 1   |   | 1  |    | 1  |    | 1   |   | 1  |    | 1  |    | 6         |
| CI 1 (Influent)         | 1   |   | 1  |    | 1  |    | 1   |   | 1  |    | 1  |    | 6         |
| Control Pipe (Effluent) | 1   | 1 | 1  | 1  | 1  | 1  | 1   | 1 | 1  | 1  | 1  | 1  | 12        |
| CI 1 (Effluent)         | 1   | 1 | 1  | 1  | 1  | 1  | 1   | 1 | 1  | 1  | 1  | 1  | 12        |
| <b>Feed water</b>       | 1   |   | 1  |    | 1  |    | 1   |   | 1  |    | 1  |    | 6         |
| CI 2 (Influent)         | 1   |   | 1  |    | 1  |    | 1   |   | 1  |    | 1  |    | 6         |
| CI 3 (Influent)         | 1   |   | 1  |    | 1  |    | 1   |   | 1  |    | 1  |    | 6         |
| CI 2 (Effluent)         | 1   | 1 | 1  | 1  | 1  | 1  | 1   | 1 | 1  | 1  | 1  | 1  | 12        |
| CI 3 (Effluent)         | 1   | 1 | 1  | 1  | 1  | 1  | 1   | 1 | 1  | 1  | 1  | 1  | 12        |
| <b>Total</b>            | 14  |   | 14 |    | 14 |    | 14  |   | 14 |    | 14 |    | <b>84</b> |

3

<sup>3</sup> Feed water samples and effluent samples were also used for the determination of Fe, Mn, Ca<sup>2+</sup>, Mg<sup>2+</sup>, and NO<sub>3</sub><sup>-</sup> determination.

### 3.5 Analytical Procedure

Table 3 - 6: Analytical Procedures

| <b>PARAMETER</b>   | <b>METHOD</b>       |
|--------------------|---------------------|
| pH                 | Membrane probe      |
| Temperature        | Direct reading      |
| Colour             | Spectrometer        |
| Turbidity          | Spectrometer        |
| Conductivity       | Membrane probe      |
| TDS                | Membrane probe      |
| Chlorine, Residual | DPD colorimetric    |
| Chlorine, Total    | DPD colorimetric    |
| Total iron         | AAS                 |
| Manganese          | AAS                 |
| Nitrate            | AAS                 |
| <i>Coliforms</i>   | Membrane filtration |
| <i>E. coli</i>     | Membrane filtration |
| <i>Salmonella</i>  | Membrane filtration |

## CHAPTER 4

### 4.0 RESULTS & DISCUSSION

#### 4.1 Chlorine Decay Patterns

Generally, free chlorine was observed to decay in three patterns; the first could be described as a more rapid decay which occurs within the first two hours of distribution where nearly over 50 % of the initial chlorine dose is consumed. This initial “heavy” demand could be attributed to the oxidation of some readily oxidizable ions such as  $\text{Fe}^{2+}$ ,  $\text{Mn}^{2+}$ ,  $\text{S}^{2-}$  and the reaction of free residual chlorine with biofilms and NOMs. The second pattern of decay, an “intermediate” one was observed to be relatively less rapid and occurred between 2 to 6 hours. The third pattern of decay was observed to be the slowest, and it occurred in situations where the free residual chlorine concentration was below 0.1 mg/L.

##### 4.1.1 Bulk Decay Coefficient ( $K_b$ ) Determination

The batch-scale experiments to determine the bulk decay coefficient using the double - jacketed batch reactor yielded the following results;

Table 4 - 1: Results for Bulk decay plots

| <b>Time (hr)</b> | <b>Free Chlorine Residual (mg/L) (first batch)</b> | <b>Free Chlorine Residual (mg/L) (Second batch)</b> |
|------------------|--|---|
| 0                | 0.74   | 1.50  |
| 1                | 0.70   | 1.40  |
| 2                | 0.67   | 1.27  |
| 3                | 0.66   | 1.23  |
| 4                | 0.63   | 1.14  |
| 5                | 0.58   | 1.06  |
| 6                | 0.58   | 1.05  |
| 7                | 0.50   | 1.05  |
| 8                | 0.50   | 0.99  |

The First-order decay kinetics was assumed in determining the bulk decay coefficient ( $K_b$ ) for the two batches of chlorinated water. A plot using non-linear regression by the least squares method showed that the  $K_b$  for the first and second batch of chlorinated water was  $0.054 \text{ hr}^{-1}$  and  $0.052 \text{ hr}^{-1}$  respectively (See Figure 4 - 1). Again, the graphs showed that within 8 hours, the initial chlorine dose reduced by 32 – 34 % in the bulk fluid.

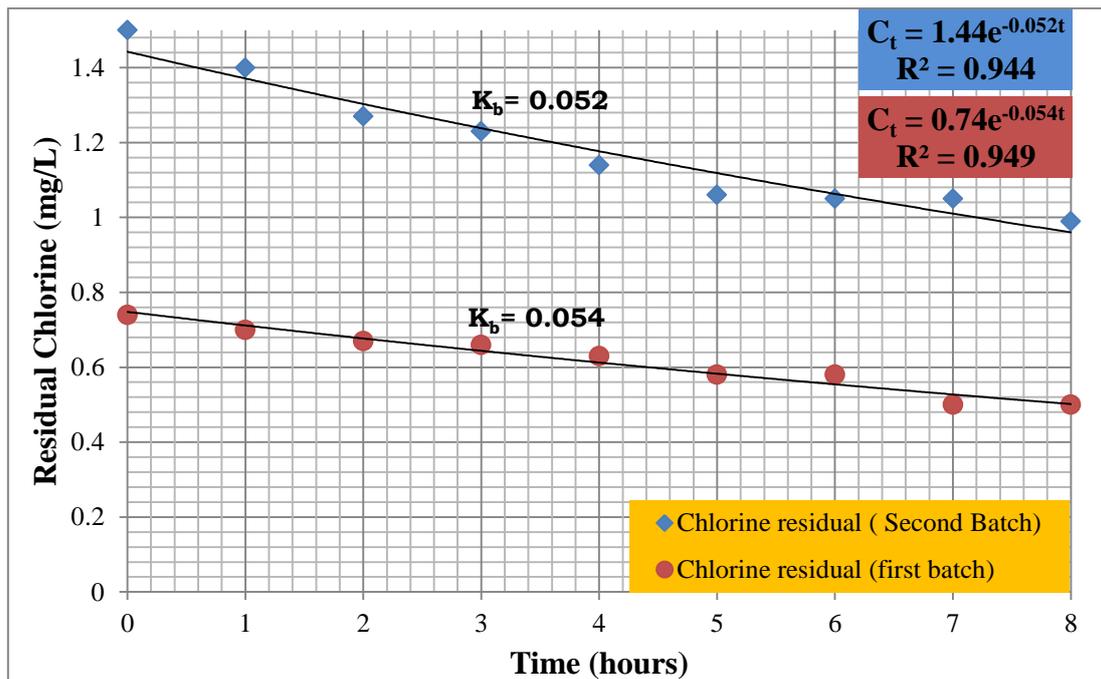


Figure 4 - 1: Trend in Chlorine decay in the bulk fluid

Since the bulk decay coefficients were determined at a temperature different from the operating temperature (approximately  $26.0 \text{ }^\circ\text{C}$ ) of treated water at the Barekese Water Treatment Plant (WTP), the  $K_b$  was adjusted for the variations in temperature using the Vant Hoff - Arrhenius equation;

$$K_{b1} = K_{b2} \times \theta^{(T1-T2)}$$

Where  $K_{b1}$  = bulk dissipation constant at temperature T1

$K_{b2}$  = bulk dissipation constant at temperature T2.

$\theta$  = Temperature correction factor

From the graph,  $K_{b1} = 0.054 \text{ hr}^{-1}$ ,  $T1 = 29.00 \text{ }^\circ\text{C}$

$$K_{b2} = 0.052 \text{ hr}^{-1}, T2 = 28.00 \text{ }^\circ\text{C}$$

$$0.054 = 0.052 \times \theta^{(29-28)}$$

$$\theta = 1.04$$

But  $K_T = K_{b1} \times \theta^{(T-T1)}$ ,

Where  $K_T$  = is the bulk decay coefficient at an operation temperature T

T = the operation temperature (approximately 26.0 °C) of the Barekese final water.

Thus,  $K_T = 0.054 \times 1.04^{(26-29)}$

$$K_T = 0.053 \text{ hr}^{-1}$$

Therefore the bulk decay constant,  $K_b = 0.053 \text{ hr}^{-1}$ .

Generally, Conductivity and TDS values in the bulk fluid increased per time for both batches of chlorinated water. For the first batch of chlorinated water (with influent free residual concentration,  $C_o = 0.74 \text{ mg/L}$ ), conductivity increased from 168.2  $\mu\text{S/cm}$  to 179  $\mu\text{S/cm}$  whereas TDS values increased from 84.2 mg/L to 86.6 mg/L. For the second batch of chlorinated water (with initial residual concentration,  $C_o = 1.44 \text{ mg/L}$ ), Conductivity values within the bulk fluid ranged from a minimum of 178.2  $\mu\text{S/cm}$  to a maximum of 182.2  $\mu\text{S/cm}$  while the TDS values also ranged from a minimum of 89.1 mg/L to a maximum of 91.3 mg/L. However, it must be stated, that fluctuations in conductivity values corresponded positively with that of the TDS. The marginal increment in conductivity values may be due to the fact that slight changes in pH values favoured the conversion of HOCl to OCl<sup>-</sup>. Increase in TDS values may be attributed to the conversion of oxidizable ions like Fe<sup>2+</sup> to Fe<sup>3+</sup> and

Mn<sup>2+</sup> to Mn<sup>4+</sup>; this was evident by corresponding increases in turbidity values as water became more turbid with increasing TDS. However within the bulk fluid, the average Turbidity values were found to 0.50 and 0.52 NTU for the first and second batch of chlorinated water respectively.

Table 4 - 2: Experimental results for first batch of chlorinated water

(Influent free residual chlorine concentration = 0.74 mg/L, T = 29 °C)

| <b>Hydraulic Retention Time (hr)</b> | <b>pH</b>   | <b>Turbidity (NTU)</b> | <b>Conductivity (µS/cm)</b> | <b>TDS (mg/L)</b> | <b>Free Chlorine Residuals (mg/L)</b> |
|--------------------------------------|-------------|------------------------|-----------------------------|-------------------|---------------------------------------|
| 0                                    | 6.70        | 0.45                   | 168.2                       | 84.2              | 0.74                                  |
| 1                                    | 6.88        | 0.44                   | 168.3                       | 84.4              | 0.70                                  |
| 2                                    | 6.87        | 0.46                   | 168.9                       | 84.5              | 0.67                                  |
| 3                                    | 6.87        | 0.44                   | 169.2                       | 84.8              | 0.66                                  |
| 4                                    | 6.88        | 0.48                   | 171.1                       | 85.3              | 0.63                                  |
| 5                                    | 6.90        | 0.44                   | 171.8                       | 85.8              | 0.58                                  |
| 6                                    | 6.92        | 0.49                   | 172.2                       | 86.2              | 0.58                                  |
| 7                                    | 6.94        | 0.62                   | 172.7                       | 86.4              | 0.50                                  |
| 8                                    | 6.95        | 0.65                   | 172.9                       | 86.6              | 0.48                                  |
| <b>AVERAGE</b>                       | <b>6.88</b> | <b>0.50</b>            | <b>170.59</b>               | <b>85.36</b>      |                                       |

Table 4 - 3: Experimental results for second batch of chlorinated water

(Initial free residual chlorine concentration = 1.44 mg/L, T = 28 °C)

| <b>Hydraulic Retention Time (hr)</b> | <b>pH</b>   | <b>Turbidity (NTU)</b> | <b>Conductivity (µS/cm)</b> | <b>TDS (mg/L)</b> | <b>Free Chlorine Residual (mg/L)</b> |
|--------------------------------------|-------------|------------------------|-----------------------------|-------------------|--------------------------------------|
| 0                                    | 7.08        | 0.46                   | 179.0                       | 89.3              | 1.50                                 |
| 1                                    | 7.10        | 0.44                   | 178.2                       | 89.1              | 1.40                                 |
| 2                                    | 7.11        | 0.52                   | 179.6                       | 89.9              | 1.27                                 |
| 3                                    | 7.14        | 0.53                   | 180.8                       | 90.5              | 1.23                                 |
| 4                                    | 7.15        | 0.59                   | 181.4                       | 90.7              | 1.14                                 |
| 5                                    | 7.14        | 0.50                   | 181.5                       | 90.8              | 1.06                                 |
| 6                                    | 7.15        | 0.50                   | 182.0                       | 91.0              | 1.05                                 |
| 7                                    | 7.15        | 0.55                   | 182.6                       | 91.3              | 1.05                                 |
| 8                                    | 7.17        | 0.55                   | 182.2                       | 91.0              | 0.99                                 |
| <b>AVERAGE</b>                       | <b>7.13</b> | <b>0.52</b>            | <b>180.81</b>               | <b>90.4</b>       |                                      |

#### 4.1.2 Overall Decay Coefficient (K) Determination

The overall decay is a combination of both bulk and pipe wall reactions. In the experimental determination of the overall decay coefficient, two batches of chlorinated water with influent concentrations of 0.74 and 1.44 mg/L were used. The temperature for the feed water ranged from 27.83 – 30.97 °C. Results obtained from the Pilot Distribution System (PDS) were subjected to non-linear regression by the least squares method and Equation 2.20 was used to calculate the overall decay coefficients for the various pipes. It must however be stated, that in the plots and the subsequent determination of the overall decay constants, concentrations considered were those  $\geq 0.1$  mg/L; a situation encountered when the ratio of Residual chlorine dose to the Total Organic Carbon (TOC) is high (Rodriguez *et al.*, 2002).

##### 4.1.2.1 Overall decay coefficient for Control Pipes

With the exception of the PVC pipe where a new pipe was used as a control pipe, the AC and CI control pipes were actually aged pipes whose encrustation had been removed. This is however a limitation to the work. For both batches of chlorinated water, experimental results for the overall decay coefficient are as shown;

Table 4 - 4: Experimental results for overall decay coefficient determination for control pipes.

| Hydraulic Retention Time (hours) | Free Residual Chlorine concentrations (mg/L) when influent concentration = 0.74 mg/L |      |           | Free Residual Chlorine concentrations (mg/L) when influent concentration = 1.44 mg/L |           |
|----------------------------------|--|------|-----------|--|-----------|
|                                  | PVC  | AC   | Cast Iron | AC   | Cast Iron |
| 0                                | 0.74   | 0.74 | 0.74      | 1.44   | 1.44      |
| 2                                | 0.52   | 0.43 | 0.33      | 1.23   | 0.82      |
| 4                                | 0.34   | 0.33 | 0.25      | 0.77   | 0.52      |
| 6                                | 0.26   | 0.25 | 0.19      | 0.51   | 0.39      |
| 12                               | 0.13   | 0.15 | 0.10      | 0.18   | 0.12      |
| 18                               | 0.02   | 0.10 | 0.02      | 0.18   | 0.11      |
| 24                               | 0.02   | 0.02 | 0.01      | 0.17   | 0.08      |

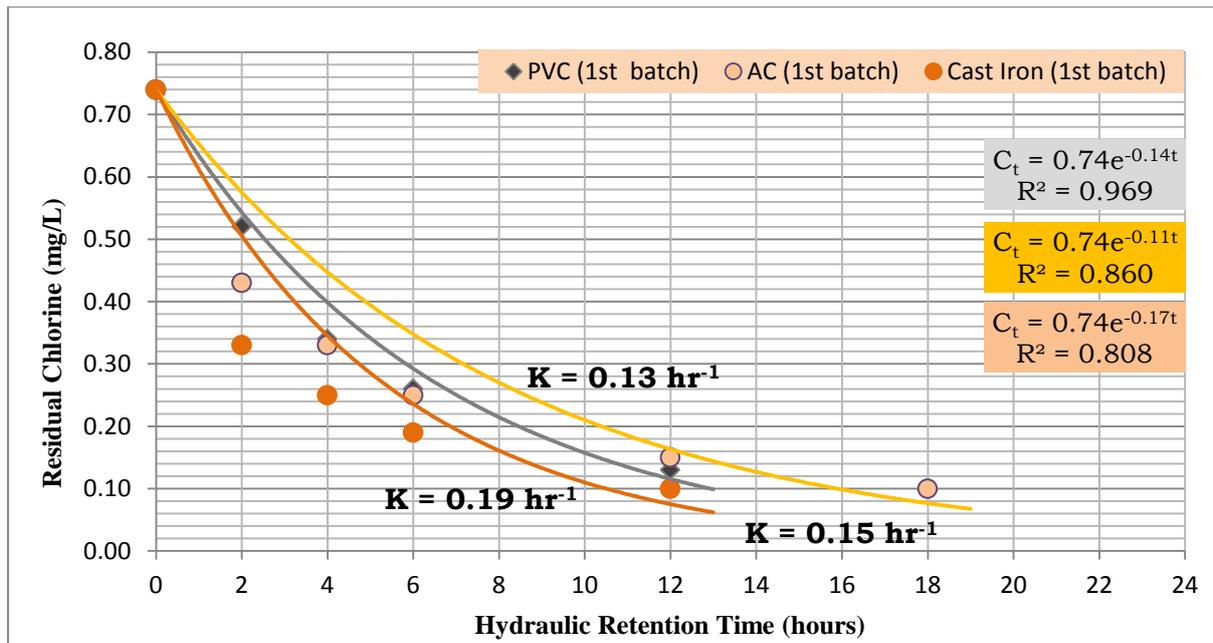


Figure 4 - 2: Trend in Free Residual chlorine decay for control pipes when influent concentration = 0.74 mg/L

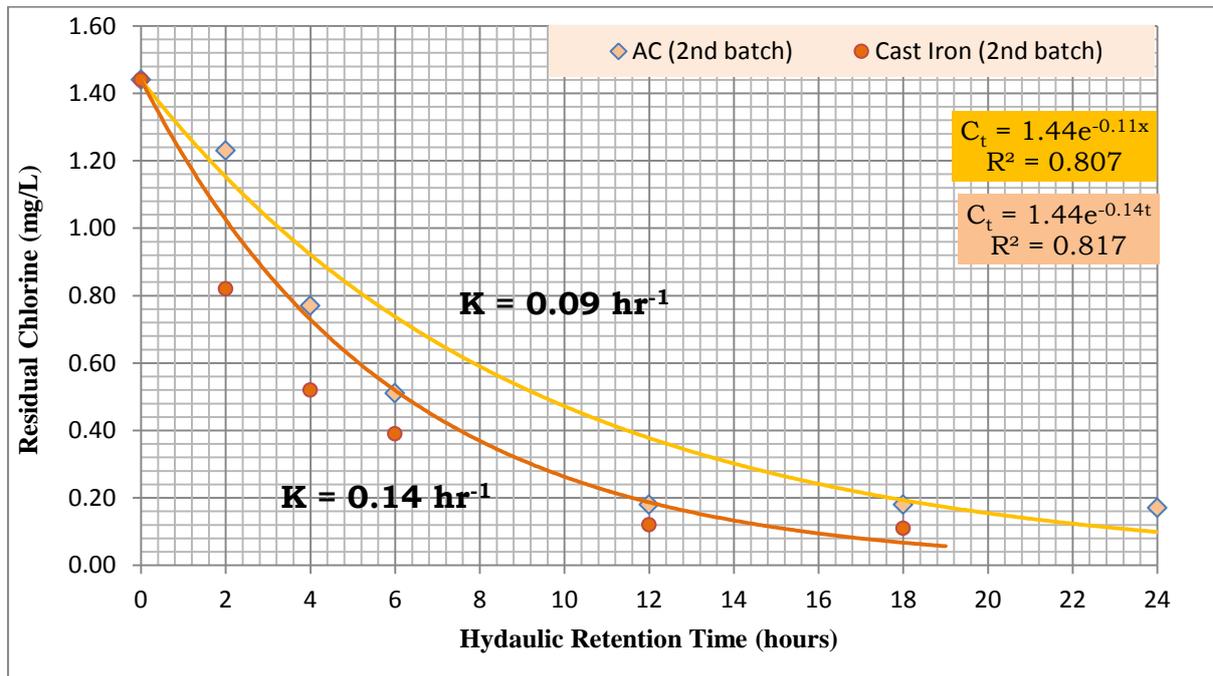


Figure 4 - 3: Trend in Free Residual chlorine decay for control pipes when influent concentration = 1.44 mg/L

From the coefficients obtained from both plots, the overall decay coefficients for the control pipes were obtained by finding the average values.

Table 4 - 5: Experimental results for overall decay coefficient determination for control pipes

| Pipe Material | Overall decay, K (hr <sup>-1</sup> )        |  |                                  |
|---------------|---|--|----------------------------------|
|               | First Batch<br>(C <sub>0</sub> = 0.74 mg/L) | Second Batch<br>(C <sub>0</sub> = 1.44 mg/L) | AVERAGE<br>K (hr <sup>-1</sup> ) |
| PVC           | 0.14  | -  | 0.14                             |
| AC            | 0.11  | 0.09   | 0.10                             |
| CI            | 0.17  | 0.14   | 0.16                             |

The highest decay, which was observed in the Cast iron pipes, could be attributed to the interaction of free chlorine with Iron deposits or particulate Iron released from the walls of the pipe material.

#### 4.1.2.2 Overall decay coefficient for Aged Pipes

Table 4 - 6 : Experimental results for Average free residual chlorine concentrations per triplicate of each pipe material (C<sub>0</sub> = 0.74 (mg/L))

| Hydraulic Retention Time (hours) | Free Chlorine Concentration           |                                   |                                    |
|----------------------------------|---------------------------------------|-----------------------------------|------------------------------------|
|                                  | PVC<br>(C <sub>0</sub> = 0.74 (mg/L)) | AC<br>(C <sub>0</sub> = 0.74mg/L) | CI<br>(C <sub>0</sub> = 0.74 mg/L) |
| 0                                | 0.74                                  | 0.74                              | 0.74                               |
| 2                                | 0.33                                  | 0.33                              | 0.20                               |
| 4                                | 0.20                                  | 0.25                              | 0.12                               |
| 6                                | 0.14                                  | 0.17                              | 0.07                               |
| 12                               | 0.03                                  | 0.08                              | 0.03                               |
| 18                               | 0.01                                  | 0.05                              | 0.02                               |
| 24                               | 0.01                                  | 0.00                              | 0.00                               |

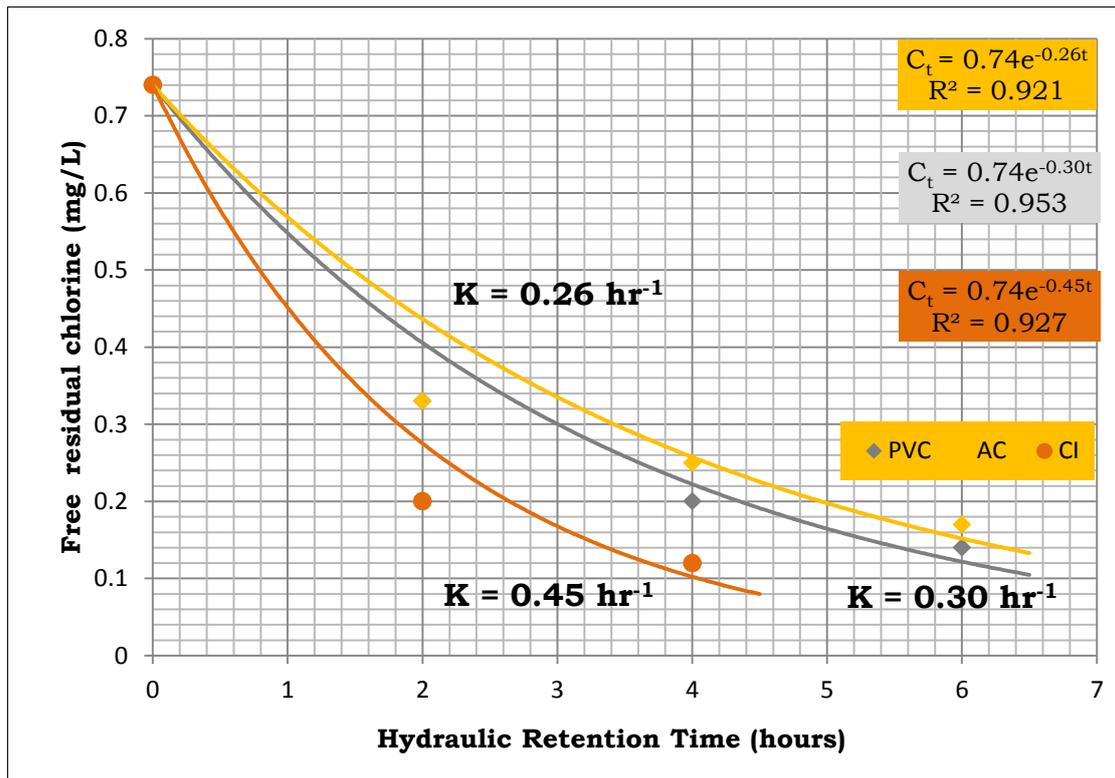


Figure 4 - 4: Trend in Chlorine residual decay observed for PVC, AC and Cast Iron pipe when  $C_0 = 0.74$  mg/L

Table 4 - 7: Experimental results for Average free residual chlorine concentrations per triplicate of AC and Cast Iron pipe ( $C_0 = 1.44$  mg/L)

| Hydraulic Retention Time (hours) | Free Chlorine Concentration  |                              |
|----------------------------------|------------------------------|------------------------------|
|                                  | AC Pipe ( $C_0 = 1.44$ mg/L) | CI Pipe ( $C_0 = 1.44$ mg/L) |
| 0                                | 1.44                         | 1.44                         |
| 2                                | 0.96                         | 0.50                         |
| 4                                | 0.76                         | 0.24                         |
| 6                                | 0.45                         | 0.13                         |
| 12                               | 0.10                         | 0.04                         |
| 18                               | 0.08                         | 0.02                         |
| 24                               | 0.03                         | 0.00                         |

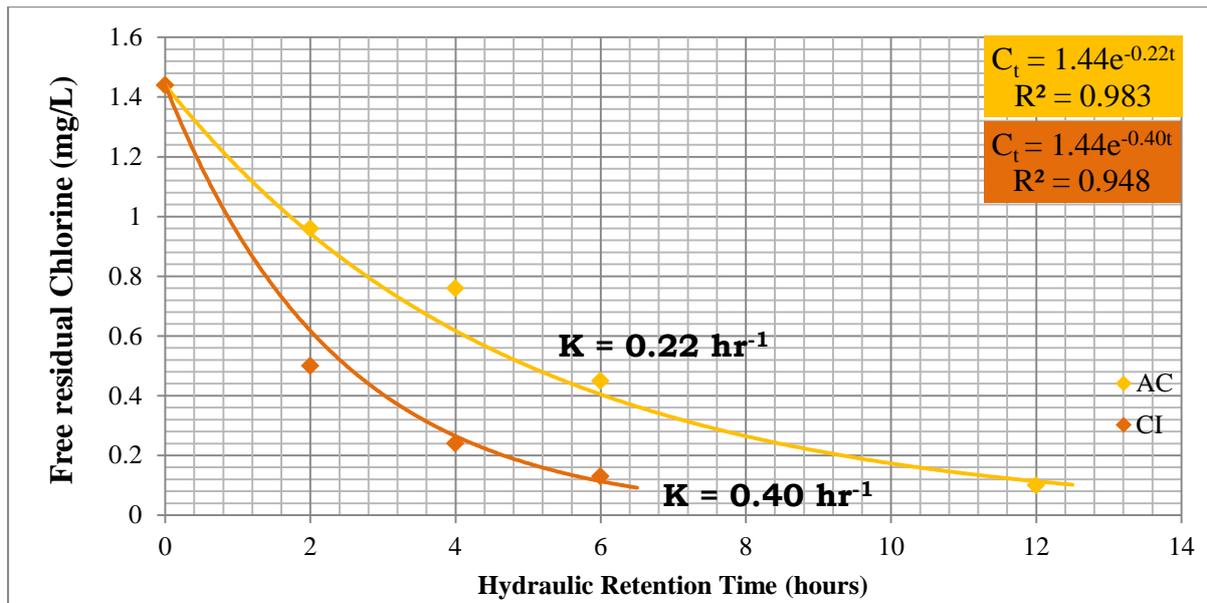


Figure 4 - 5: Trend in Chlorine residual decay observed for AC and Cast Iron pipe when  $C_o = 1.44$  mg/L

The final Overall decay coefficient was obtained by finding an average of the two initial overall decay coefficients obtained per each batch of chlorinated water. The K computed was found to be  $0.24 \text{ hr}^{-1}$ ,  $0.30 \text{ hr}^{-1}$ ,  $0.43 \text{ hr}^{-1}$  for AC, PVC and Cast Iron pipes respectively.

#### 4.1.3 Wall Decay Coefficients ( $K_w$ and $K_w'$ )

##### 4.1.3.1 Overall Wall decay coefficient ( $K_w$ )

Determination of this coefficient was achieved by subtracting the bulk decay coefficients from the overall decay coefficients of the various pipe materials.

Table 4 - 8: Calculated values for overall wall decay coefficient

| Pipe Material            | $K$ ( $\text{hr}^{-1}$ ) | $K_b$ ( $\text{hr}^{-1}$ ) | $K_w$ ( $\text{hr}^{-1}$ ) |
|--------------------------|--------------------------|----------------------------|----------------------------|
| PVC (control pipe)       | 0.14                     | 0.053                      | 0.09                       |
| PVC (15 – 20 yrs)        | 0.30                     | 0.053                      | 0.25                       |
| AC (control pipe)        | 0.10                     | 0.053                      | 0.05                       |
| AC (40 - 50 yrs)         | 0.24                     | 0.053                      | 0.19                       |
| Cast Iron (control pipe) | 0.16                     | 0.053                      | 0.11                       |
| Cast Iron (84 yrs)       | 0.43                     | 0.053                      | 0.38                       |

From the  $K_b$  and  $K_w$  coefficients obtained for all pipes under study, it can be inferred that Chlorine decay was predominantly due to pipe wall reactions. Again, comparing the wall decay constants of the control pipes to that of the aged, it can be deduced that the  $K_w$  of the old PVC, AC and CI pipes were greater than their control pipes by a factor of 2.77, 3.8 and 3.45 respectively. Furthermore, the wall decay coefficients of the pipe materials vis-à-vis their ages indicate that for every year the  $K_w$  increases at a rate of 0.0125, 0.0038, 0.0045  $\text{hr}^{-1}$  for PVC, AC and CI pipes respectively.

#### 4.1.3.2 Wall decay coefficient ( $K_w$ )

From Equation 2.31, the wall decay coefficient was calculated for the various aged pipe materials by assuming that for stagnant flow, Reynolds number,  $Re < 1$ , thus Sherwood number,  $Sh = 2$ .

Table 4 - 9: Calculation of Wall decay constant, ( $K_w'$ )

| Pipe Material | Sh | Molecular Diffusivity of Chlorine ( $\text{m}^2\text{s}^{-1}$ ) | Pipe Diameter (m) | $K_f$ ( $\text{hr}^{-1}$ ) | $K_b$ ( $\text{hr}^{-1}$ ) | K ( $\text{hr}^{-1}$ ) | $K_w'$ ( $\text{mhr}^{-1}$ ) |
|---------------|----|---|-------------------|----------------------------|----------------------------|------------------------|------------------------------|
| PVC           | 2  | 1.21E-09  | 0.1524            | 5.72E-05                   | 0.053                      | 0.30                   | -5.75E-05                    |
| AC            | 2  | 1.21E-09  | 0.1524            | 5.72E-05                   | 0.053                      | 0.24                   | -5.74E-05                    |
| Cast Iron     | 2  | 1.21E-09  | 0.1524            | 5.72E-05                   | 0.053                      | 0.43                   | -5.76E-05                    |

#### 4.1.4 Model Development

In view of the fact that the Pilot Distribution System was limited to only pipe materials with diameter of 6 inches (150 mm), a mathematical model was developed by modifying Equation 2.19 in order to simulate how free residual chlorine will decay in different diameters of the pipes under test. The initial target concentration for the simulation was set to 0.74 mg/L and 1.44 mg/L to synchronise with that of the experimental.

From Equation 2.19;  $C_t = C_0 e^{-Kt}$

Where t = hydraulic retention time (Length/Velocity)

But  $K = K_b + K_w$ , and  $K_w = \frac{4K_{w'}K_f}{D(K_{w'} + K_f)}$

$$- \left[ \left( K_b + \frac{4K_{w'}K_f}{D(K_{w'} + K_f)} \right) \times \text{HRT} \right] \dots\dots\dots(4.1)$$

This implies that;  $C_t = 0.74 \times e$

$$C_t = 1.44 \times e^{- \left[ \left( K_b + \frac{4K_{w'}K_f}{D(K_{w'} + K_f)} \right) \times \text{HRT} \right]} \dots\dots\dots(4.2)$$

$$C_t = 0.74 \times e^{- \left[ \left( K_b + \frac{4K_{w'}K_f}{D(K_{w'} + K_f)} \right) \times \frac{\text{Length}}{\text{Velocity}} \right]} \dots\dots\dots(4.3)$$

$$C_t = 1.44 \times e^{- \left[ \left( K_b + \frac{4K_{w'}K_f}{D(K_{w'} + K_f)} \right) \times \frac{\text{Length}}{\text{Velocity}} \right]} \dots\dots\dots(4.4)$$

Thus, these two mathematical models (Equation 4.1 and 4.2) were further developed (Equation 4.3 and 4.4) and used to simulate free chlorine decay for various pipe lengths.

#### **4.1.5 Simulation of Chlorine decay**

##### **4.1.5.1 Simulated effect of Pipe length, diameter and HRT on free chlorine decay**

Simulation of chlorine decay was done to study the effects of pipe diameter, pipe length and hydraulic retention time on chlorine decay. For the reason that all three pipe materials under study were 6 inches (0.1524 m) diameter pipes and 1.0 m lengths, the simulation was carried

out using pipe diameters 9 inches (0.229 m), 6 inches (0.1524 m) and 3 inches (0.0762 m). The simulation assumed a flow velocity of  $1.0 \text{ ms}^{-1}$  (typical of a distribution system).

To calculate the mass transfer coefficient ( $K_f$ ) and the wall decay constants for each of the pipe materials, the Temperature (T) was assumed to be equal to the average temperature of the Barekese final water ( $26 \text{ }^\circ\text{C}$ ) and the molecular diffusivity of chlorine was taken to be  $1.21 \times 10^{-9} \text{ m}^2\text{s}^{-1}$ . Thus, substituting these constants into Equations 2.33 - 2.38 yielded the Kinematic viscosity ( $\nu$ ) to be  $8.766 \times 10^{-07} \text{ m}^2\text{s}^{-1}$ ,  $\text{Re} = 173,846$ ,  $\text{Sh} = 5,350$ ,  $\text{Sc} = 724$ ,  $K_f = 0.153 \text{ mhr}^{-1}$ . The  $K_f$  together with the bulk and the overall decay coefficients for the various pipes was then used to calculate their wall decay coefficients ( $K_w'$ ).

#### 4.1.5.2 PVC pipes

Substituting  $K_b = 0.053 \text{ hr}^{-1}$ ,  $K_w = 0.25 \text{ hr}^{-1}$  into Equation 2.13 yielded  $K_w' = 0.010 \text{ mhr}^{-1}$ . Final substitution into the mathematical models in Equation 4.1 and 4.2 using diameters 0.072 m and 0.229 m resulted in the simulated values in Table 4 – 10.

Table 4 - 10: Simulated decay for different pipe diameters (PVC)

| Length<br>(m) | HRT<br>(hrs) | Simulated decay<br>( $C_0 = 0.74 \text{ mg/L}$ ) |          |          | Simulated decay<br>( $C_0 = 1.44 \text{ mg/L}$ ) |          |          |
|---------------|--------------|--|----------|----------|--|----------|----------|
|               |              | (3 inch)   | (6 inch) | (9 inch) | (3 inch)   | (6 inch) | (9 inch) |
| 0             | 0.0          | 0.74   | 0.74     | 0.74     | 1.44   | 1.44     | 1.44     |
| 1800          | 0.5          | 0.56   | 0.64     | 0.66     | 1.10   | 1.24     | 1.29     |
| 3600          | 1.0          | 0.43   | 0.55     | 0.60     | 0.83   | 1.07     | 1.16     |
| 5400          | 1.5          | 0.33   | 0.47     | 0.53     | 0.63   | 0.92     | 1.04     |
| 7200          | 2.0          | 0.25   | 0.41     | 0.48     | 0.48   | 0.79     | 0.93     |
| 9000          | 2.5          | 0.19   | 0.35     | 0.43     | 0.37   | 0.68     | 0.84     |
| 10800         | 3.0          | 0.14   | 0.30     | 0.39     | 0.28   | 0.59     | 0.75     |
| 12600         | 3.5          | 0.11   | 0.26     | 0.35     | 0.21   | 0.50     | 0.67     |
| 14400         | 4.0          | 0.08   | 0.22     | 0.31     | 0.16   | 0.43     | 0.60     |
| 16200         | 4.5          | 0.06   | 0.19     | 0.28     | 0.12   | 0.37     | 0.54     |
| 18000         | 5.0          | 0.05   | 0.17     | 0.25     | 0.09   | 0.32     | 0.49     |
| 19800         | 5.5          | 0.04   | 0.14     | 0.22     | 0.07   | 0.28     | 0.44     |
| 21600         | 6.0          | 0.03   | 0.12     | 0.20     | 0.05   | 0.24     | 0.39     |

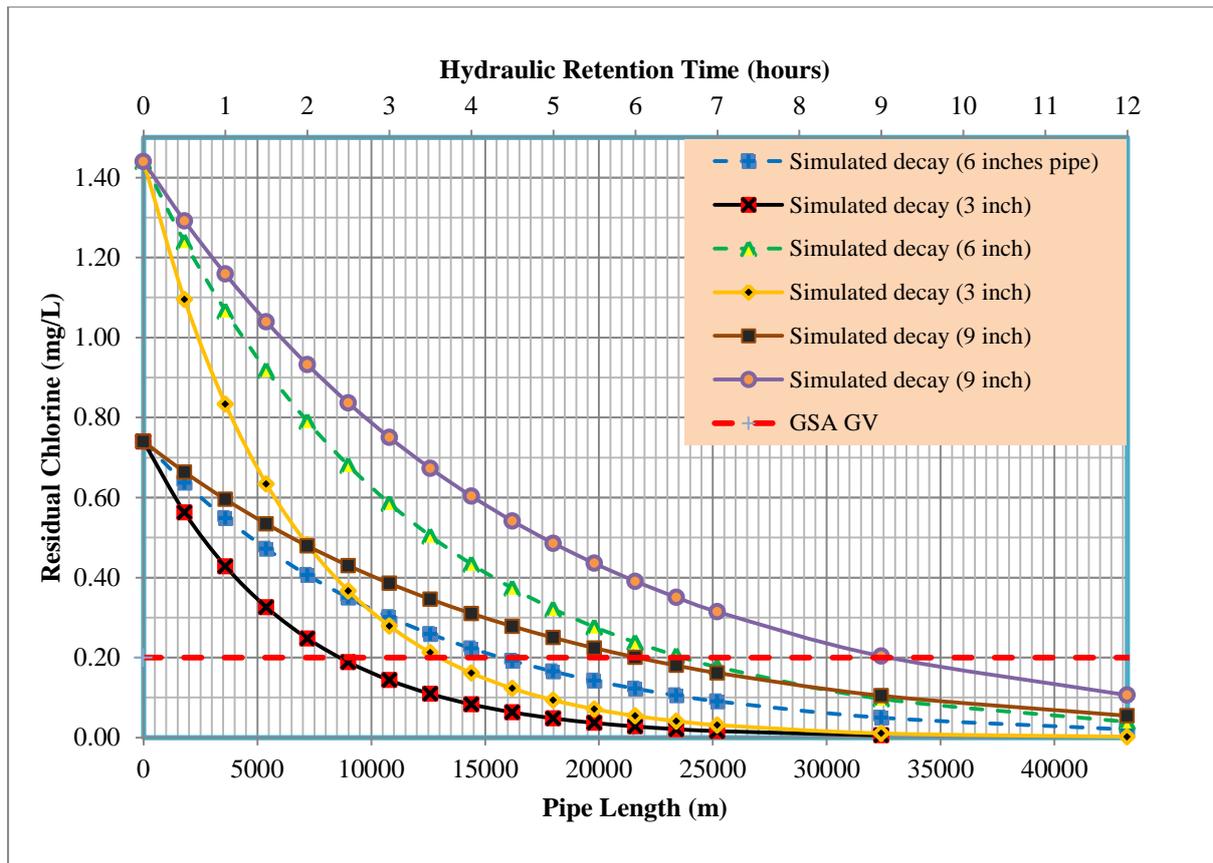


Figure 4 - 6: Simulated effect of Pipe diameter, length and HRT (PVC)

#### 4.1.5.3 AC pipe

Substituting  $K_b = 0.053 \text{ hr}^{-1}$ ,  $K_w = 0.19 \text{ hr}^{-1}$  into Equation 2.13 yielded  $K_w' = 0.008 \text{ mhr}^{-1}$ .

Final substitution into the mathematical models in Equation 4.1 and 4.2 using diameters 0.072 m and 0.229 m resulted in the simulated values in Table 4 - 11.

Table 4 - 11: Simulated decay for different pipe diameters (AC)

| Length<br>(m) | HRT<br>(hrs) | Simulated decay<br>(C <sub>0</sub> = 0.74 mg/L) |          |          | Simulated decay<br>(C <sub>0</sub> = 1.44 mg/L) |          |          |
|---------------|--------------|---|----------|----------|---|----------|----------|
|               |              | (3 inch)  | (6 inch) | (9 inch) | (3 inch)  | (6 inch) | (9 inch) |
| 0             | 0.0          | 0.74  | 0.74     | 0.74     | 1.44  | 1.44     | 1.44     |
| 1800          | 0.5          | 0.60  | 0.66     | 0.68     | 1.16  | 1.28     | 1.32     |
| 3600          | 1.0          | 0.48  | 0.58     | 0.62     | 0.94  | 1.13     | 1.21     |
| 5400          | 1.5          | 0.39  | 0.52     | 0.57     | 0.76  | 1.00     | 1.10     |
| 7200          | 2.0          | 0.32  | 0.46     | 0.52     | 0.61  | 0.89     | 1.01     |
| 9000          | 2.5          | 0.25  | 0.41     | 0.47     | 0.50  | 0.79     | 0.92     |
| 10800         | 3.0          | 0.21  | 0.36     | 0.43     | 0.40  | 0.70     | 0.85     |
| 12600         | 3.5          | 0.17  | 0.32     | 0.40     | 0.32  | 0.62     | 0.77     |
| 14400         | 4.0          | 0.13  | 0.28     | 0.36     | 0.26  | 0.55     | 0.71     |
| 16200         | 4.5          | 0.11  | 0.25     | 0.33     | 0.21  | 0.49     | 0.65     |
| 18000         | 5.0          | 0.09  | 0.22     | 0.30     | 0.17  | 0.43     | 0.59     |
| 19800         | 5.5          | 0.07  | 0.20     | 0.28     | 0.14  | 0.38     | 0.54     |
| 21600         | 6.0          | 0.06  | 0.18     | 0.26     | 0.11  | 0.34     | 0.50     |

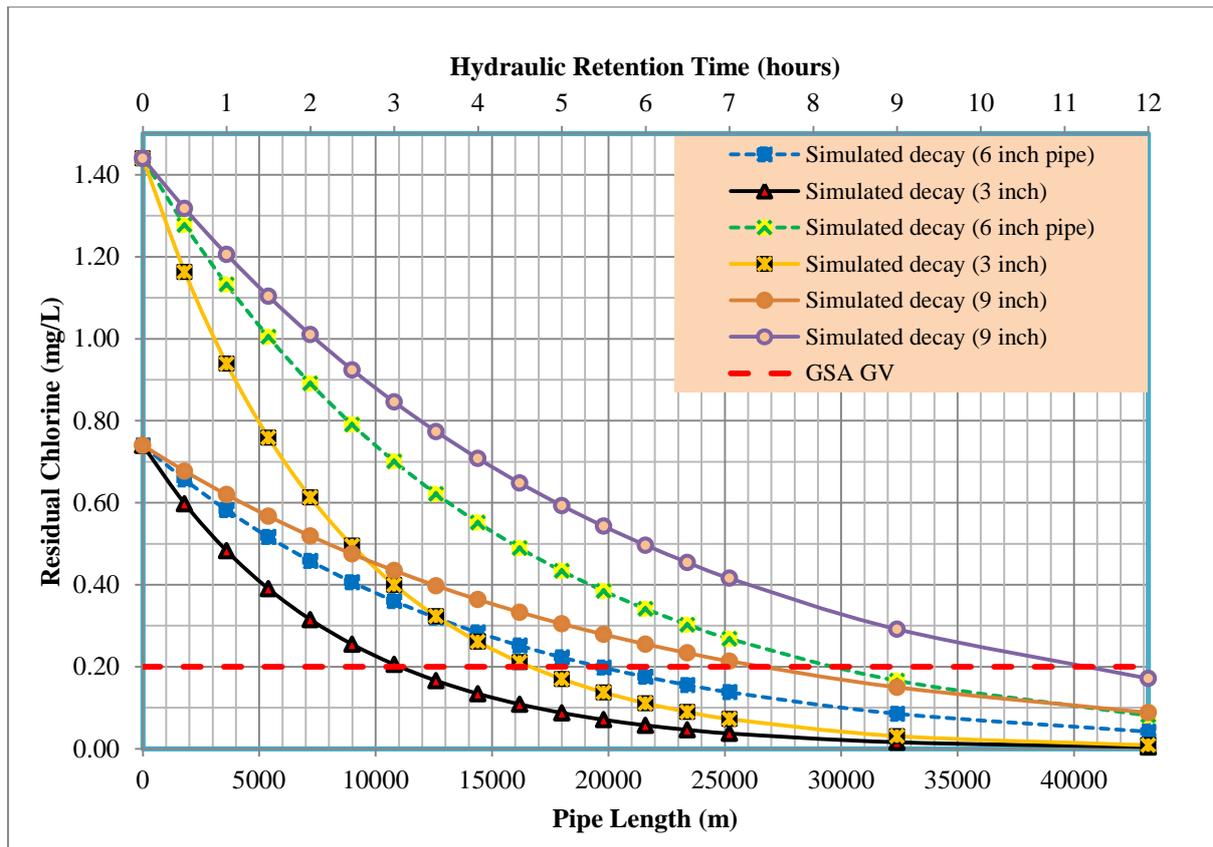


Figure 4 - 7: Simulated effect of Pipe diameter, length and HRT (AC)

#### 4.1.5.4 Cast Iron pipes

Substituting  $K_b = 0.053 \text{ hr}^{-1}$ ,  $K_w = 0.38 \text{ hr}^{-1}$  into Equation 2.13 yielded  $K_w' = 0.016 \text{ mhr}^{-1}$ .

Final substitution into the mathematical models in Equation 4.1 and 4.2 using diameters 0.072 m and 0.229 m resulted in the simulated values in Table 4 – 12.

Table 4 - 12: Simulated decay for different pipe diameters (CI)

| Length (m) | HRT (hrs) | Simulated decay (C <sub>0</sub> = 0.74 mg/L) |          |          | Simulated decay (C <sub>0</sub> = 1.44 mg/L) |          |          |
|------------|-----------|--|----------|----------|--|----------|----------|
|            |           | (3 inch)                                     | (6 inch) | (9 inch) | (3 inch)                                     | (6 inch) | (9 inch) |
| 0          | 0.0       | 0.74   | 0.74     | 0.74     | 1.44   | 1.44     | 1.44     |
| 1800       | 0.5       | 0.49   | 0.60     | 0.64     | 0.96   | 1.16     | 1.24     |
| 3600       | 1.0       | 0.33   | 0.48     | 0.55     | 0.64   | 0.94     | 1.06     |
| 5400       | 1.5       | 0.22   | 0.39     | 0.47     | 0.43   | 0.76     | 0.91     |
| 7200       | 2.0       | 0.15   | 0.31     | 0.40     | 0.29   | 0.61     | 0.78     |
| 9000       | 2.5       | 0.10   | 0.25     | 0.35     | 0.19   | 0.49     | 0.67     |
| 10800      | 3.0       | 0.07   | 0.20     | 0.30     | 0.13   | 0.40     | 0.58     |
| 12600      | 3.5       | 0.04   | 0.16     | 0.26     | 0.09   | 0.32     | 0.50     |
| 14400      | 4.0       | 0.03   | 0.13     | 0.22     | 0.06   | 0.26     | 0.43     |
| 16200      | 4.5       | 0.02   | 0.11     | 0.19     | 0.04   | 0.21     | 0.37     |
| 18000      | 5.0       | 0.01   | 0.09     | 0.16     | 0.03   | 0.17     | 0.32     |
| 19800      | 5.5       | 0.01   | 0.07     | 0.14     | 0.02   | 0.14     | 0.27     |
| 21600      | 6.0       | 0.01   | 0.06     | 0.12     | 0.01   | 0.11     | 0.23     |

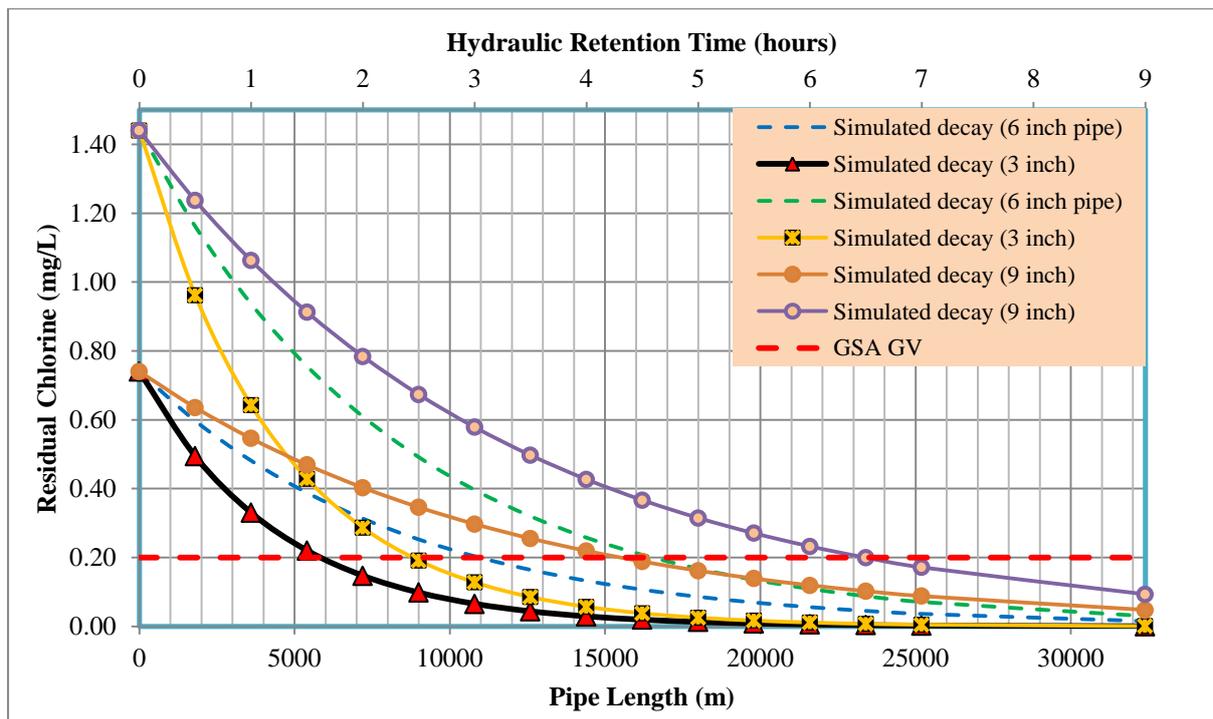


Figure 4 - 8: Simulated effect of Pipe diameter, Length and HRT (CI)

It was observed that for the various pipe materials, and with the simulations involving either initial concentration of 0.74 or 1.44 mg/L free residual chlorine, the chlorine decay in the

bigger diameter (9 inch) pipe was relatively slower than that of the smaller diameter pipes (3 inch). This may be due to the fact that under distribution conditions, as pipe diameter increases, there is very little interaction between the bulk of the distributed water and the deposits on pipe walls such as biofilms and oxidisable ions, thus the residual chlorine in the bulk water persist for longer hours. In the case of the smaller diameter pipes, the decay was relatively faster due to the fact that there is more interaction between the bulk water and the demand on the pipe walls, thus the residual chlorine is consumed more rapidly. The plots showed that for the 9 inch diameter pipes with initial free residual chlorine concentration of 1.44 mg/L, the concentration falls below 0.2 (the GSA GV) at a hydraulic retention time greater than 9.0, 11 and 6.5 hours for PVC, AC and Cast Iron pipes respectively. Similarly, for an influent free residual chlorine concentration of 0.74 mg/L, the chlorine concentration falls below the GSA GV at hydraulic retention time greater than 6.0, 7.5 and 4.3 hours for PVC, AC and Cast Iron pipes respectively.

For the smaller diameter pipe (3 inch), it was observed that for an initial free residual chlorine concentration of 0.74 mg/L, the concentration falls below the GSA GV at hydraulic retention times greater than 2.5, 3.1 and 1.5 hours for PVC, AC and CI pipes respectively, whereas for an initial concentration of 1.44 mg/L it fell below the GSA GV at hydraulic retention times greater than 3.5, 4.5 and 2.5 hours for PVC, AC and Cast iron pipe respectively. In all the cases, it was observed that the decay in the cast iron pipes were more rapid. This could be due to the relatively high pipe wall demand in the lumen (encrustation) of the said pipe. Again, this site could also be a habitat for various microorganisms and opportunistic bacteria whose destruction places an appreciable demand on the free residual chlorine in the water.

## **4.2 Pipe Material**

Differences in overall decay coefficient was observed for the PVC, AC and CI pipes, implying that different pipe materials will consume chlorine differently. Generally, it was observed that AC pipe material had a relatively lower overall wall decay constant ( $0.24 \text{ hr}^{-1}$ ) as compared to PVC and CI pipes ( $0.30$  and  $0.43 \text{ hr}^{-1}$  respectively). Thus, for the same blend of water being distributed through these pipes, distributed water from the AC pipe will have relatively higher free residual chlorine than the others. Again, the experimental result shows that water distributed by the AC pipes relatively had better colour and turbidity values than the PVC and CI pipe. Thus, water from such a pipe would be more aesthetically pleasing than water distributed by the PVC and CI pipes. Furthermore, it must be stated that water distributed by the cast iron pipes had the worst quality in terms of all physico-chemical parameters analysed. This water was often associated with high levels of colour, turbidity and iron concentrations. It is probable that such waters will have taste and odour problems and it will be unsuitable for domestic and industrial purposes. Thus, considering the parameters discussed above, the AC pipes may be preferable in the construction of a distribution system. However, because of the carcinogenicity associated with Asbestos (in its dissolved state), the PVC pipe material may be a better substitute.

## **4.3 Pipe Age**

Pipe age is usually associated with the build-up of sediments in the lumen of the pipe material. From Table 4.8, it was also observed, that the overall decay in the aged PVC pipe was 2.14 times that of the newer pipe. The lower wall decay coefficients observed for all the control pipes as compared to that of the aged pipes confirms that chlorine consumption may increase with pipe age if the age of the pipe material is associated with an increase in wall demand. In the case of the CI pipes, the excessive deposits of Iron and Mn together with

sediments forming an encrusted layer could produce hydraulic effects like reduction of the available quantity and pressure of the water supply.

#### **4.4 Water Quality**

##### **4.4.1 pH**

The pH represents the balance between hydrogen ions and hydroxide ions in water, and a unit change in pH represents a ten-fold change in the  $H^+$  or  $OH^-$  concentration. For both the first and second batches of chlorinated water in the aged PVC and AC pipes, pH over the HRT of 24 hours was fairly constant ranging from a minimum of 6.7 to a maximum of 7.3. A similar trend was observed for the control pipes where pH values were fairly constant for the PVC pipe ranging from a minimum of 7.02 to a maximum of 7.30. These values were found to be within the GSA GV of 6.5 – 8.5. However, with the Cast Iron pipes, the pH ranges from a minimum 5.7 to a maximum of 6.9 for the aged pipe, and 5.20 to 6.9 for the control pipe. It was realised that pH drops to a value below 6.5 at a  $HRT \geq 2$  hours. This means that at a  $HRT \geq 2$  hours, water in the cast iron pipe becomes unwholesome with respect to pH. This decrease in pH with HRT could be attributed to an increase in  $H^+$  ions in the water; as the water stagnates in the Cast Iron pipeline, there is more interaction between the chlorine and the encrustation (mainly made up of sand and organics like NOMs) deposited on the pipe walls. This interaction of chlorine with NOMs which usually proceed with a release of a proton or HCl tends to make the water acidic over time. Furthermore, adsorption of iron normally results in the release of protons and thus a drop in pH is expected. The high pH associated with water in the cast iron pipes may also impart taste problems.

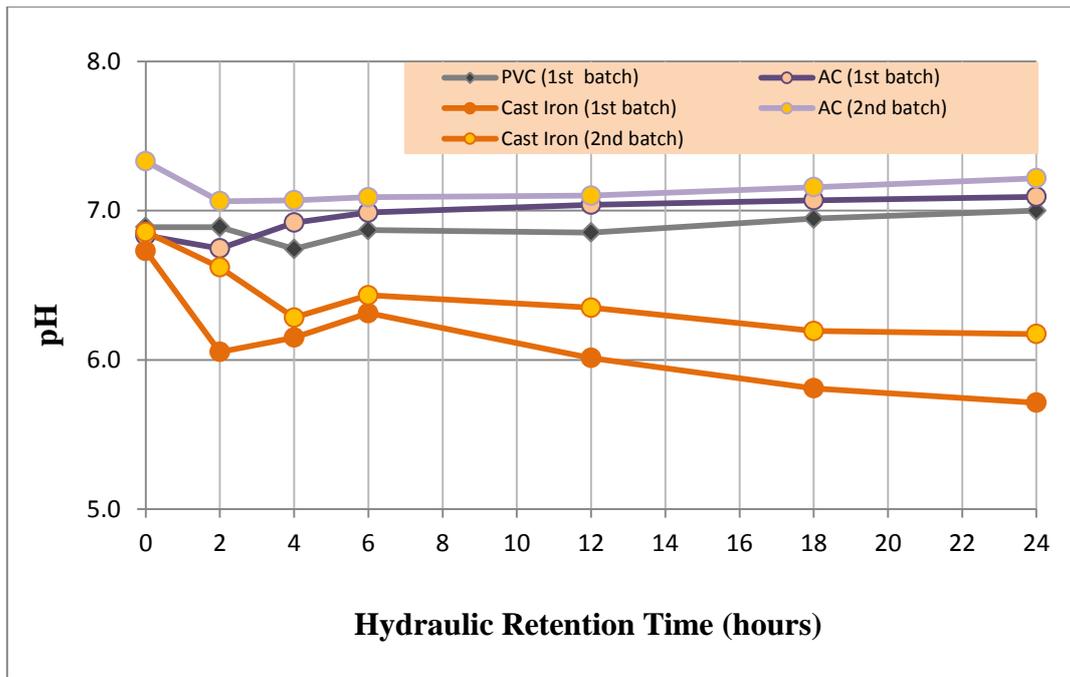


Figure 4 - 9: Trend in pH (Aged Pipes)

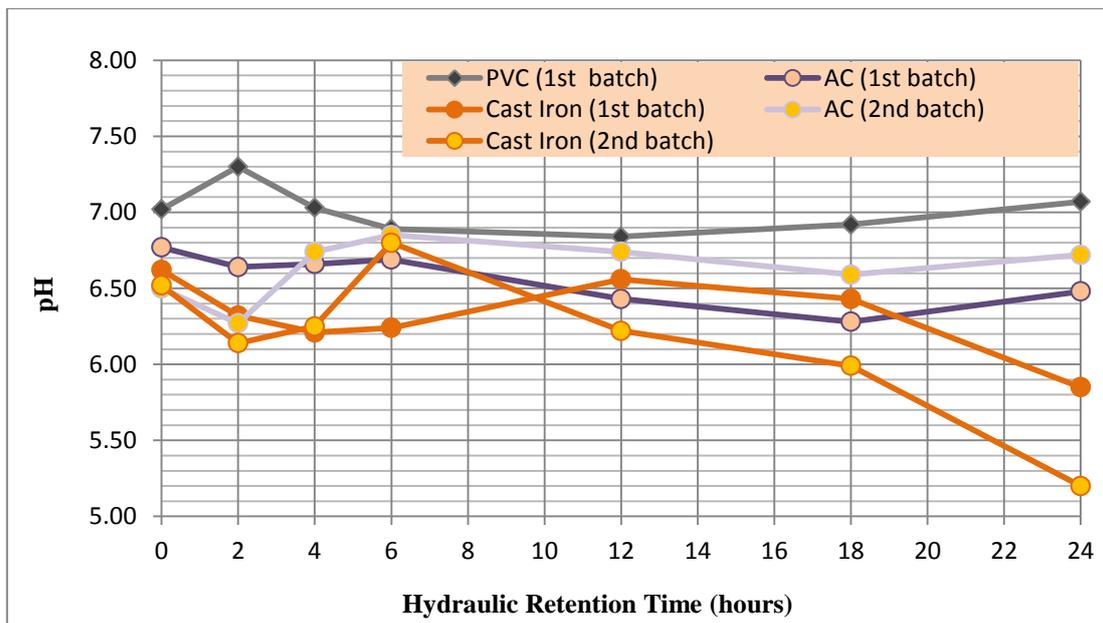


Figure 4 - 10: Trend in pH (control pipes)

#### **4.4.2 Colour**

Colour in water may be of natural mineral or animal origin. It may be caused by metallic substances, humus material, peat, algae, weeds or protozoa. For all the pipe materials under study, colour was observed to increase with HRT. As oxidisable ions like  $\text{Fe}^{2+}$  and  $\text{Mn}^{2+}$  are oxidized to their +3 and +4 states respectively, the reddish brown colour of  $\text{Fe}^{3+}$  and the dark brown (or black colour) of  $\text{Mn}^{4+}$  is imparted to the water. Again the destruction of microorganisms by free residual chlorine also imparts colour to the water. It was observed that with higher initial free residual concentrations of 1.44, the reactions proceeded faster within the AC and Cast Iron pipes with a corresponding release of higher colour. It was observed that, for the PVC and AC pipes, at a  $\text{HRT} \geq 6$  hours, the colour falls below the GSA guideline limit of 15 Hazen Units. However for the Cast Iron pipes, it was observed that after a HRT of just 2 hours, the colour falls below the GSA GV. Increase in HRT of water in the Cast Iron pipes rose to 120 Hazen units which is about 8 times the GSA GV. This high colour in the distributed water could also be as a result of the sand or silt which forms part of the encrustation in the pipes especially the Cast Iron and AC pipes. High colour makes the water aesthetically objectionable, may stain clothing when used in washing and may be associated with taste and odour problems.

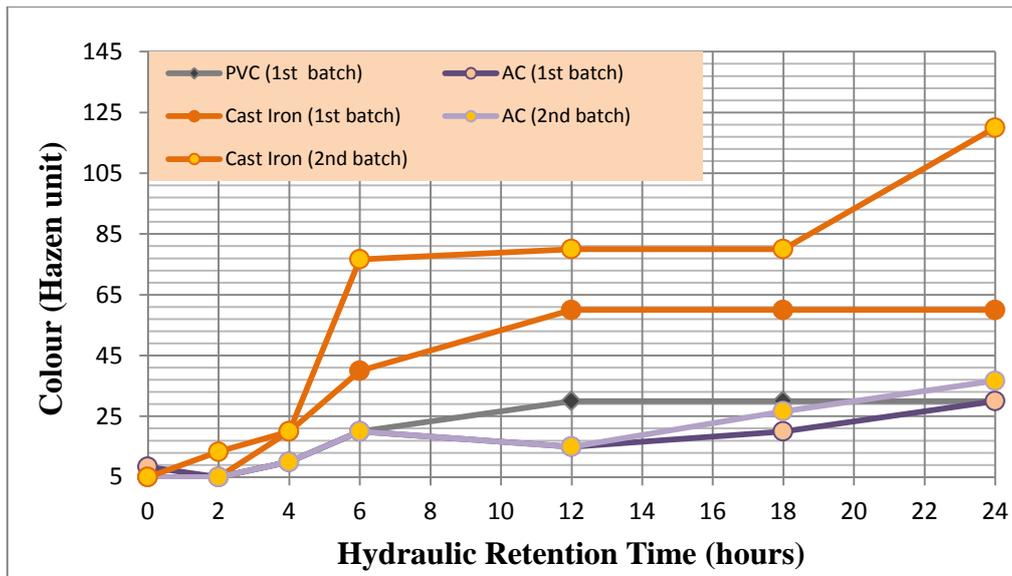


Figure 4 - 11: Trend in Colour (Aged pipes)

#### 4.4.3 Turbidity

Turbidity is a measure of water clarity (how far light can travel through water). The more particles suspended in a sample of water, the more difficult it is for light to travel through it and the higher the water's turbidity. The turbidity values correlated positively with colour for all pipes. As colour increased, there was a corresponding increase in turbidity; this increase could be attributed to the dissolved ions like Fe and Mn whose presence make water coloured and turbid. For the PVC and AC pipes, increase in turbidity values could be attributed to the oxidation of such oxidisable ions present in the feed water. It was observed that, the CI pipes recorded higher turbidity values as high as 24 and 28 NTU for the aged and control pipes respectively and these values do not fall within the GSA GV of 5 NTU. Higher Turbidity values may be as a result of the interaction of chlorine (which acts as an oxidising agent) with the pipe material, thus releasing Fe into the water. This higher turbidity may also be associated with lower oxygen levels thus making the water stale. In the case of the Cast iron

pipes, lower pH favoured the solubility of the Ferric ion, thus imparting high turbidity to the water;

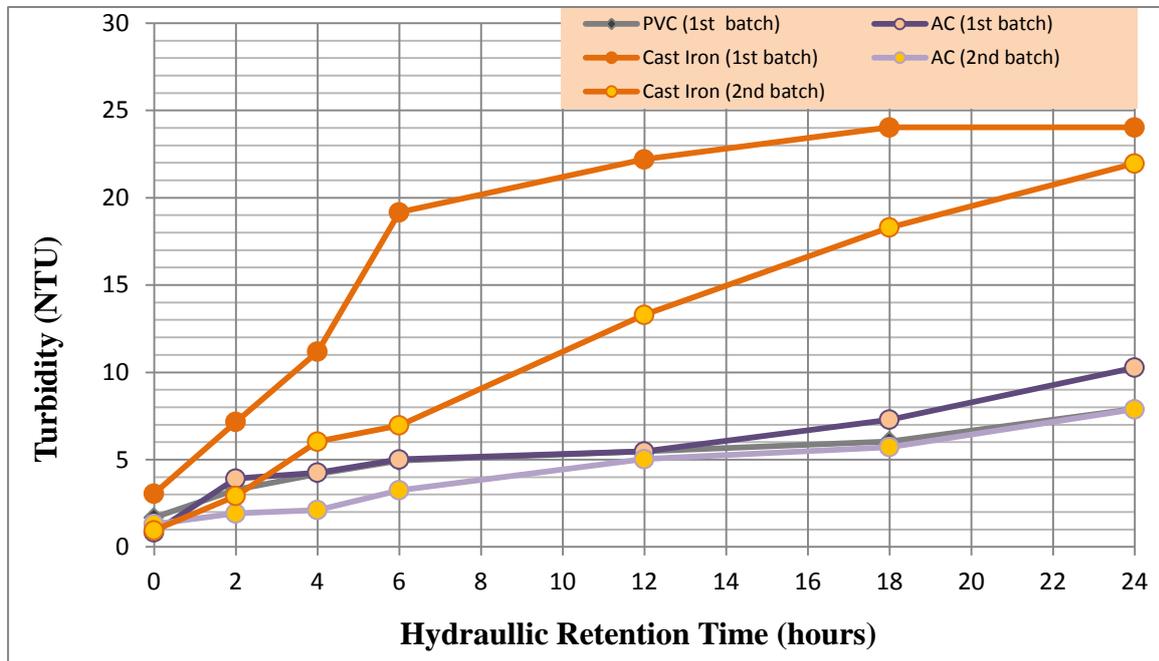


Figure 4 - 12: Trend in turbidity

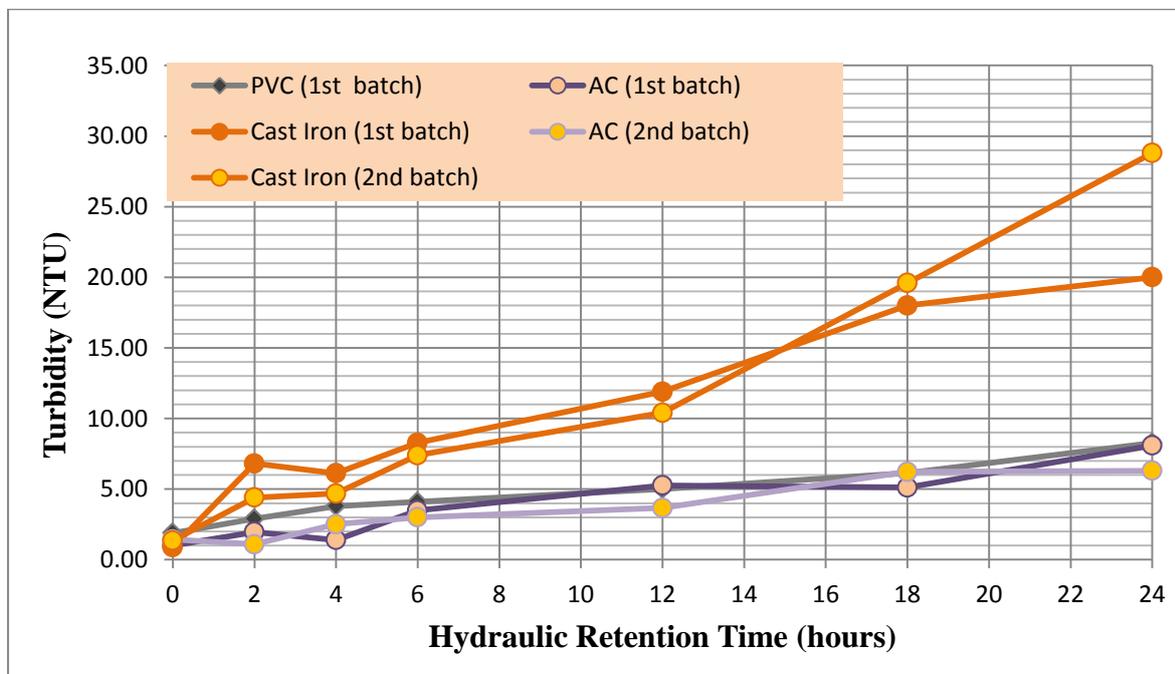


Figure 4 - 13: Trend in turbidity (control pipes)

#### **4.4.4 Conductivity and TDS**

Dissolved ions in water influences the ability of that water to conduct an electrical current and the electrical conductivity of water is directly related to the concentration of dissolved solids in the water. TDS is a measure of the total amount of all the materials that are dissolved in water. These materials, both natural and anthropogenic (made by humans) are mainly inorganic solids and a minor amount of organic material. The conductivity observed in the PVC pipe was fairly constant, with an average value of 142 and 143.98  $\mu\text{S}/\text{cm}$  for the aged and control pipes respectively. Marginal increments in conductivity values with respect to HRT were also observed for the AC pipes. This may be attributed to slight leaching of ions from the walls of the AC pipes. However, high conductivity increments were observed for the CI pipes; with a minimum of 148  $\mu\text{S}/\text{cm}$  and a maximum of 271  $\mu\text{S}/\text{cm}$  for the aged pipes while the control pipe recorded a minimum of 157.53  $\mu\text{S}/\text{cm}$  and a maximum of 176.5  $\mu\text{S}/\text{cm}$ . This increment in conductivity for CI pipes may be indicative of leaching of iron from the walls of the CI pipe into the distributed water. On the whole, Conductivity values correlated positively with TDS values and were all within the GSA GV.

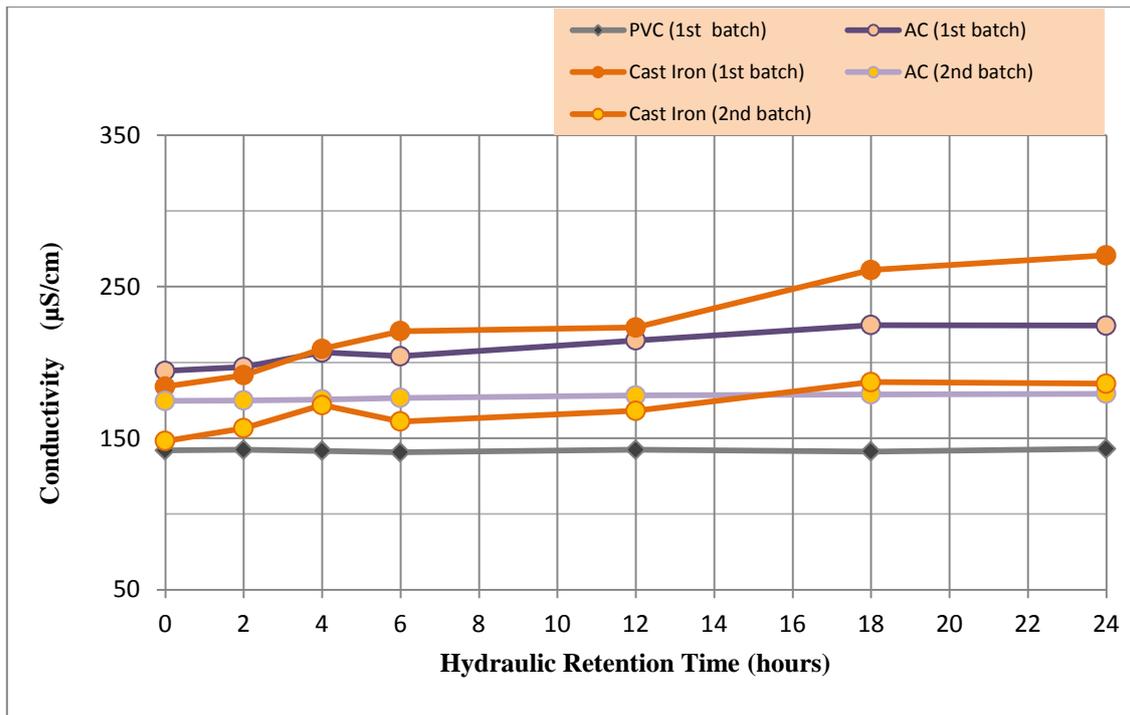


Figure 4 - 14: Trend in conductivity (Aged Pipes)

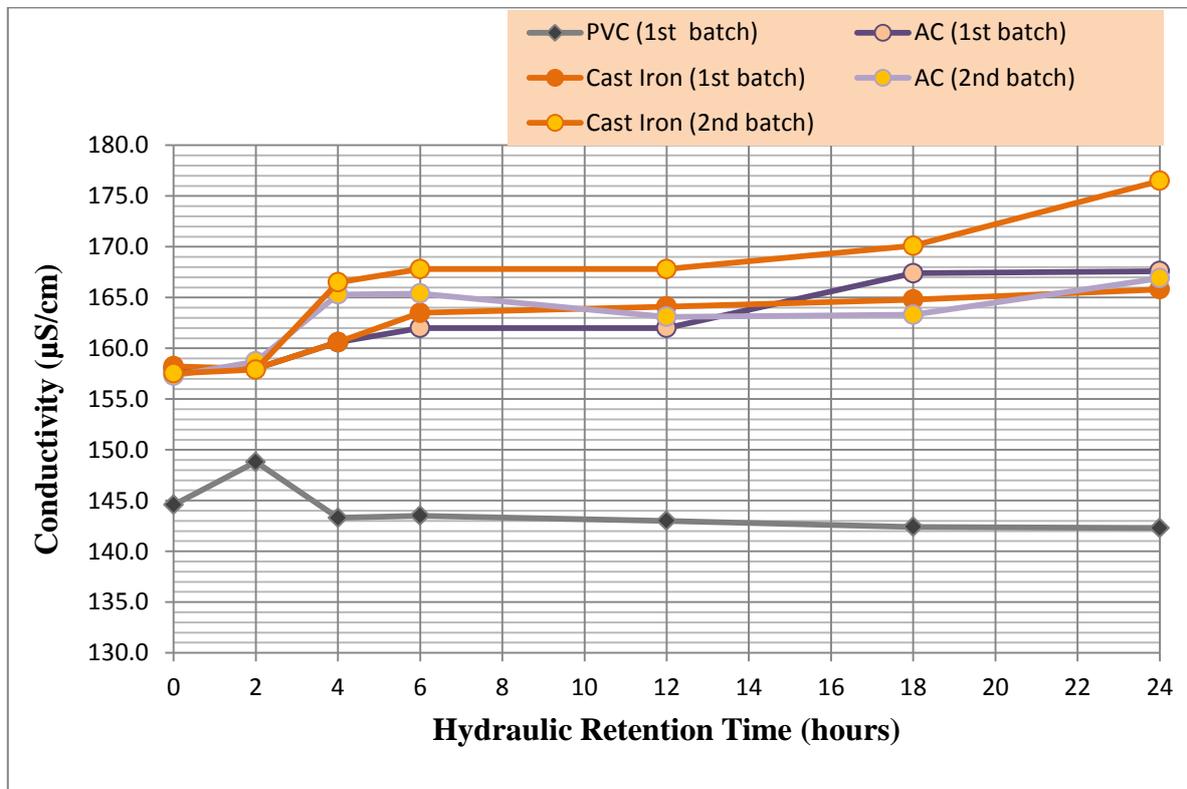


Figure 4 - 15: Trend in Conductivity (Control pipes)

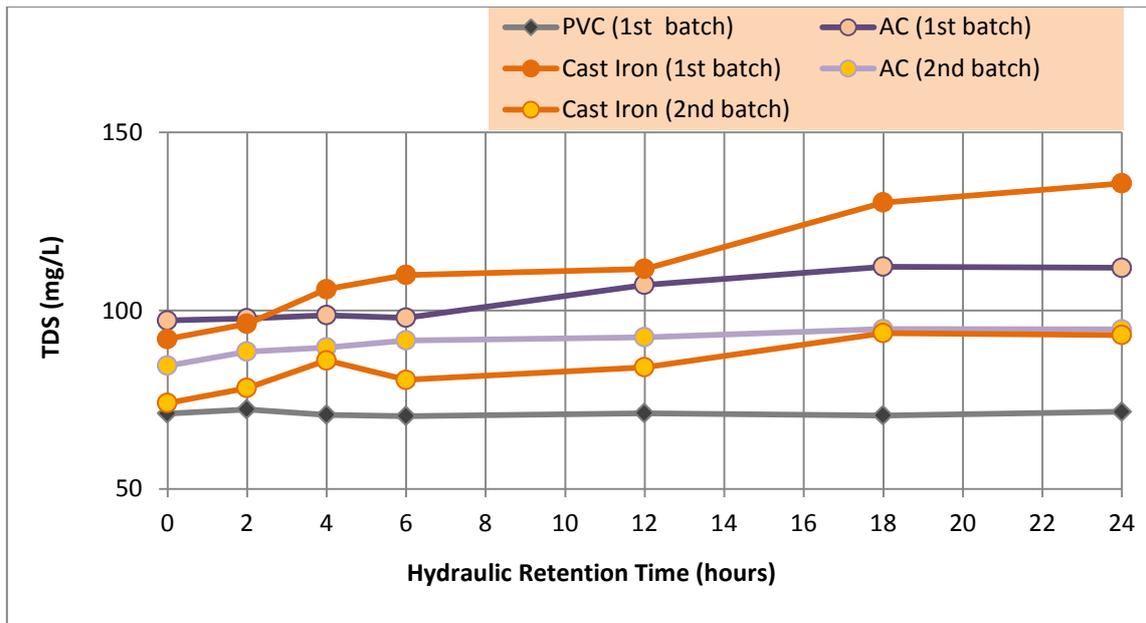


Figure 4 - 16: Trend in TDS (Aged Pipes)

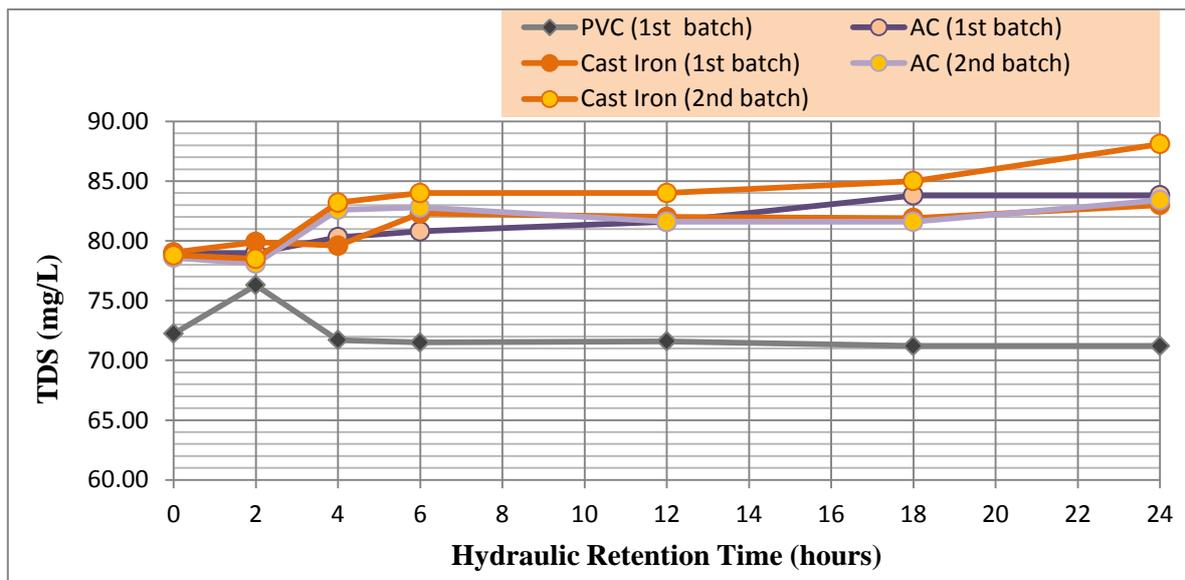


Figure 4 - 17: Trend in TDS (Control pipes)

#### 4.4.5 Iron and Manganese

Lower pH favours the solubility of the  $Fe^{3+}$  and  $Mn^{4+}$ , thus imparting colour and turbidity. High concentrations of iron and manganese in drinking waters make the water aesthetically unpleasant for domestic and other use.

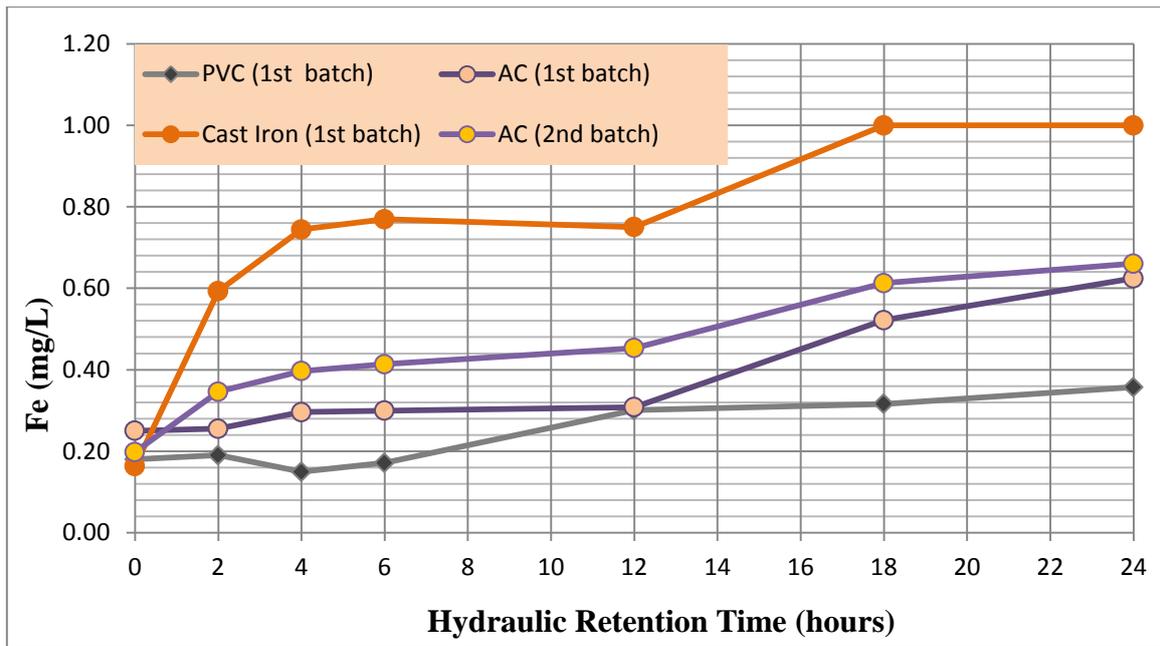


Figure 4 - 18: Trend in Fe concentration for aged pipes

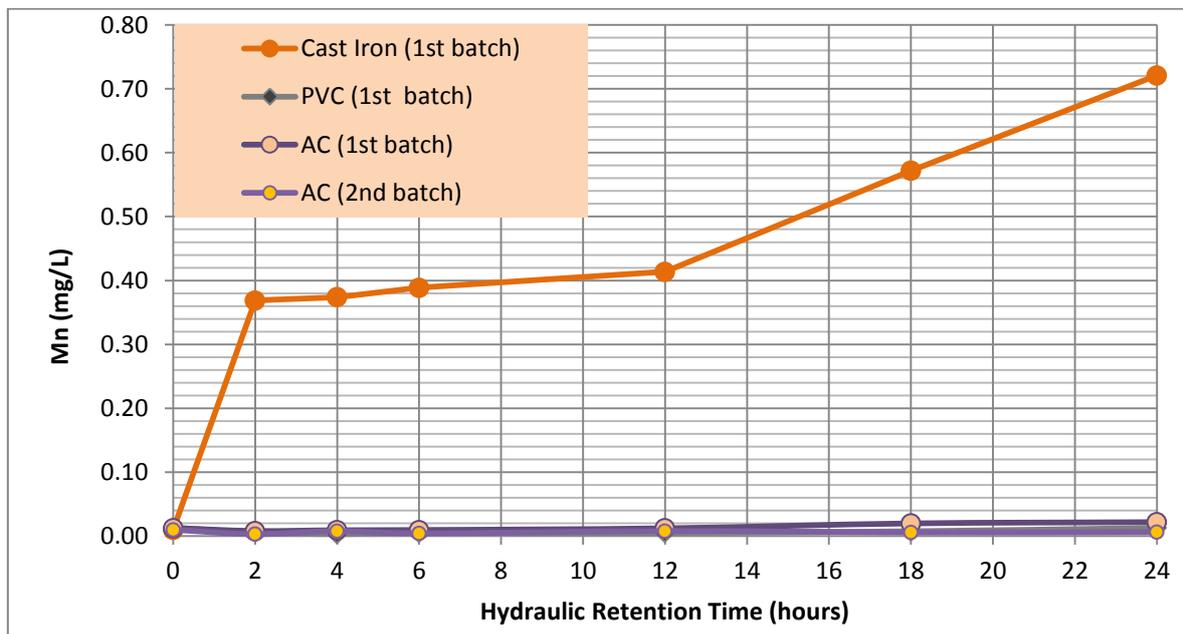


Figure 4 - 19: Trend in Mn concentration for aged pipes

Table 4 - 13: Average concentrations of Fe and Mn for Aged and control pipes

| <b>Pipe Material</b> | <b>Average Fe concentration (mg/L)</b> | <b>Average Mn concentration (mg/L)</b> |
|----------------------|--|--|
| PVC (control pipe)   | 0.21                                   | 0.011                                  |
| PVC                  | 0.24                                   | 0.010                                  |
| AC (control pipe)    | 0.33                                   | 0.010                                  |
| AC                   | 0.40                                   | 0.010                                  |
| CI (control pipe)    | 0.74                                   | 0.060                                  |
| CI                   | 0.72                                   | 0.410                                  |

For the PVC and AC pipes, Mn concentration was fairly constant (and same as the feed water) with both having an average concentration of 0.01 mg/L over a HRT of 24 hours. A similar observation was recorded for the control pipes of AC and PVC pipes which also had average Mn concentrations of 0.011 and 0.01 mg/L respectively. This implies that concentrations observed were entirely from the feed water and it clearly indicate that deposits (encrustation) observed in the aged PVC and AC pipes may have very negligible or no Mn deposits. However, these Mn concentrations were within the GSA GV of 0.1 mg/L. Averagely, Fe concentrations were also 0.24 and 0.4 mg/L for PVC and AC pipes respectively. This compares favourably with the observed average concentrations of 0.21 and 0.33 mg/L for PVC and AC control pipes respectively. This suggests that there may be negligible or no iron oxides deposited on the pipe walls of the aged PVC and AC pipes, thus concentrations observed were purely as a result of the presence of iron in the feed water. However, for the CI pipes, very high concentration of Fe and Mn were recorded. Average concentrations recorded for Fe and Mn were 0.72 and 0.41 mg/L respectively. The average Fe and Mn concentrations for the control pipes were also found to be 0.74 and 0.06 mg/L respectively. The equivalent values observed in terms of Fe concentration for the aged CI and control pipes suggest that the “large” encrustation on the aged CI walls may have formed a protective layer that prevented direct reaction of free residual chlorine with the walls of the

pipe material to leach Fe from the walls of the pipe into the water. Rather, it is probable that the increase in Fe concentration observed over the HRT could be attributed to leaching of Fe from the Iron oxides already encrusted onto the pipe walls. These high concentrations corresponded with high colour (as high as 120 Hazen unit) and turbidity (as high as 24 NTU) values. This confirms that Fe and Mn could impart colour to water. Again, the rise in Fe and Mn concentration correlated positively with chlorine decay. Within a HRT of 2 hours, Fe and Mn concentrations exceeded the GSA GV of 0.3 and 0.1 mg/L respectively and it was observed, that during the first 2 hours where the free residual chlorine reacts rapidly with the bulk water and pipe walls, there was about 73 % increase in the concentration of Fe and about 97 % increase in the concentration of Mn. This confirms that chlorine consumption in CI pipes could be attributed to the oxidation of the Fe and Mn deposits encrusted on the pipe walls. The increase in Mn and Fe concentrations correlated positively with acidity. As distributed water became more and more acidic (low pH in the CI pipes), the acidic medium created favoured the leaching of Fe from the walls of the CI pipelines into the water and the dissolution of Fe and Mn. High levels of Mn recorded can be attributed to large deposits of Mn which was observed as black layers on the sediments (encrustation) in the lumen of the CI pipes, thus in the presence of strong oxidizing agents like chlorine, it reacts with the Mn deposits which in turn increases the concentration of Mn in the distributed water. This could be further explained with the variance in Mn concentration between the aged and control CI pipes which suggest that increase in Mn concentration was entirely due to Mn deposits in the encrusted material in the lumen of the CI pipe.

Again, it can be inferred that, Fe and Mn deposits form an appreciable quantity of ions in the encrustation observed in the lumen of the CI pipes. It is therefore not advisable to retain distributed water in the CI pipes for long hours as concentrations of these ions will increase

over time. It is advised that supplies to areas where CI pipes dominate should be done during peak hours so that water doesn't stagnate in the pipe lines for longer hours. Also it will be advisable to regularly flush the CI pipes with high pressurized water to remove Fe and Mn deposits.

Plate 4 - 1: Deposits of probably oxides of Fe and Mn encrusted in the lumen of CI Pipes



#### **4.4.6 Calcium and Magnesium**

Waters that contain a significant concentration of dissolved minerals like calcium and magnesium are called “hard” and when hard waters are heated, they leave a mineral deposit called “scale.” Generally, calcium and magnesium concentrations were very low and within the GSA GV of 200 and 150 mg/L respectively. Average calcium concentrations recorded were 14, 16 and 16 mg/L for PVC, AC and CI pipes respectively. Similarly, average magnesium concentrations recorded were 8.97, 7.25 and 8.53 mg/L for PVC, AC and CI pipes respectively. It was observed that the trend in Mg concentration was fairly constant for both aged and control pipes, but that of the calcium increased marginally over time. This slight increase especially for distributed water in the AC and CI pipes may be attributed to the

cement lining in the case of the AC pipe and the sediments build-up (the sand component) in the CI pipes which may all be sources of calcium. However, there was no direct correlation observed between the trend in free residual chlorine decay and that of calcium and magnesium.

Table 4 - 14: Average Calcium and Magnesium concentrations

| Pipe material      | Average calcium concentration (mg/L) | Average magnesium concentration (mg/L) |
|--------------------|--------------------------------------|--|
| PVC (control pipe) | 14.16                                | 6.03                                   |
| PVC                | 14.00                                | 8.97                                   |
| AC (control pipe)  | 22.10                                | 8.28                                   |
| AC                 | 16.00                                | 7.46                                   |
| CI (control pipe)  | 18.09                                | 8.62                                   |
| CI                 | 16.00                                | 8.53                                   |

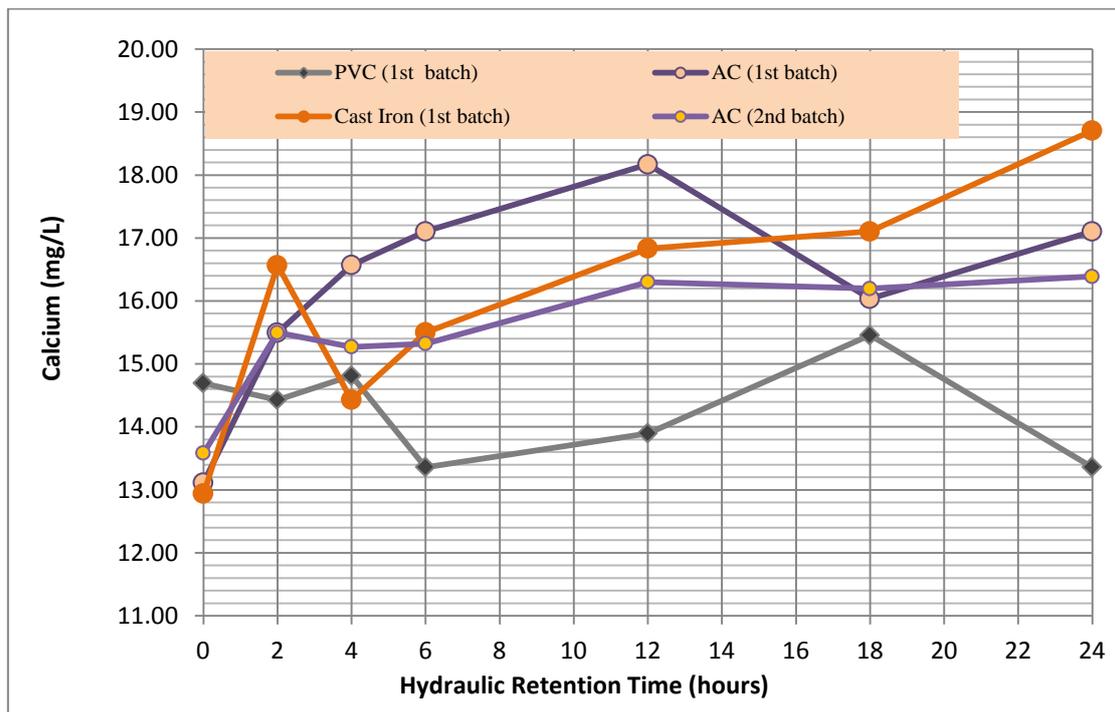


Figure 4 - 20: Trend in calcium concentration for aged pipes

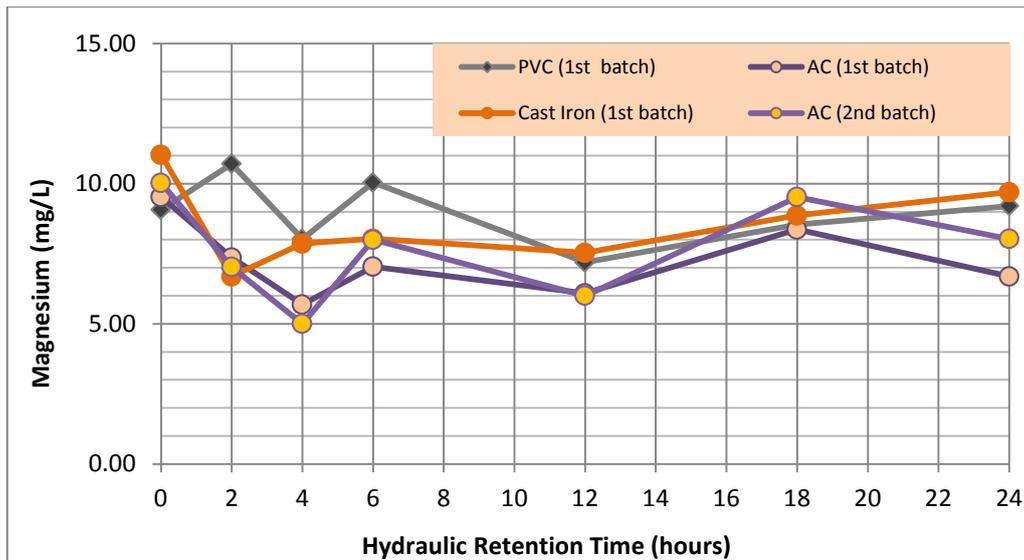


Figure 4 - 21: Tend in magnesium concentration for aged pipes

#### 4.4.7 Nitrates

Once in water, nitrates can stimulate excessive plant and algae growth. For the PVC pipes, the relative increases in nitrate concentrations could be due to the organic coatings of the pipe material which could be a source of this nutrient. However, for the AC and CI pipes, a nearly constant trend was observed with average nitrate concentration of 2.29 and 2.51 mg/L respectively. Again, It was observed that within the first 6 hours where free chlorine consumption was high, there were increases in nitrate concentrations; this could be due to the fact that chlorine as an oxidant favoured the conversion of compounds like  $\text{NH}_3$  to  $\text{NO}_2^-$  and then to  $\text{NO}_3^-$ . Again, for all pipes, in the event of trace residuals usually at  $\text{HRT} > 6$  hours, there were significant drop in nitrate concentrations, this could be due to the fact that during anaerobic or anoxic conditions,  $\text{NO}_3^-$  reverts to  $\text{NO}_2^-$ . However, between HRT of 12 to 18 hours, an appreciable increase in  $\text{NO}_3^-$  concentration was observed. This could be associated with microbial or algae die-off which usually results in the release of  $\text{NH}_3$  which

subsequently is converted to  $\text{NO}_3^-$ . Furthermore there were no significant differences in nitrate concentrations between the aged and control pipes.

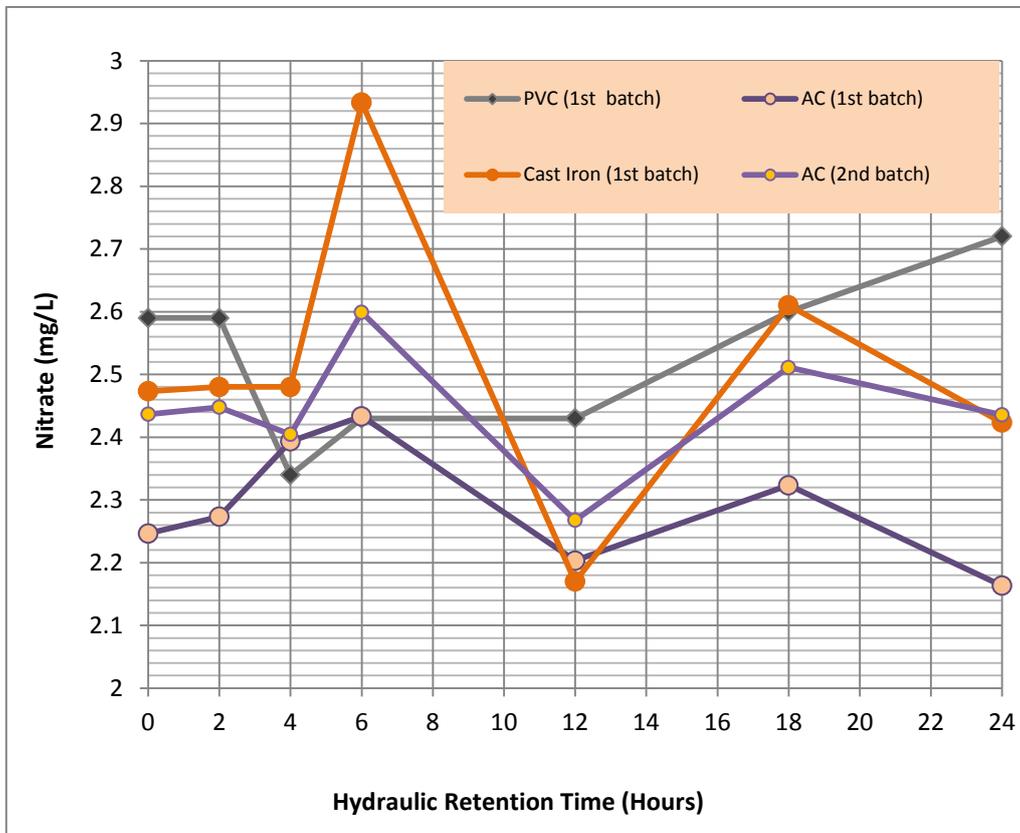


Figure 4 - 22: Trend in Nitrate Concentration for the aged pipes

Table 4 -15: Average nitrate concentrations for various pipes

| Pipe Material      | Average $\text{NO}_3^-$ concentration (mg/L) |
|--------------------|--|
| PVC (control pipe) | 2.40   |
| PVC                | 2.53   |
| AC (control pipe)  | 2.44   |
| AC                 | 2.37   |
| CI (control pipe)  | 2.31   |
| CI                 | 2.51   |

#### 4.5 Microbiological Quality

The presence of microorganisms may suggest the existence of a habitat that could be used by pathogens or the accumulation of chlorine-demanding material that interferes with maintaining a disinfectant residual throughout the pipe network.

For the first batch of chlorinated water with influent free residual concentration of 0.74 mg/L, microbial growths were observed for the feed water after a HRT of 30 minutes. However, usually within 2 to 4 hours all microbes were observed to die-off. This may suggest that at optimum pH, a HRT of 30 minutes may not be sufficient for 100 % bactericidal effect for chlorinated water with influent free residual chlorine concentration  $\leq 0.74$  mg/L, but rather this could be achieved between HRT greater than 30 minutes to a maximum of 4 hours.

However, for the feed water with initial free residual chlorine concentration of 1.44 mg/L, no growths were observed. This means that after a HRT of 30 minutes, chlorinated water with free residual chlorine concentrations  $\geq 1.44$  mg/L may not support the survival of these microbes.

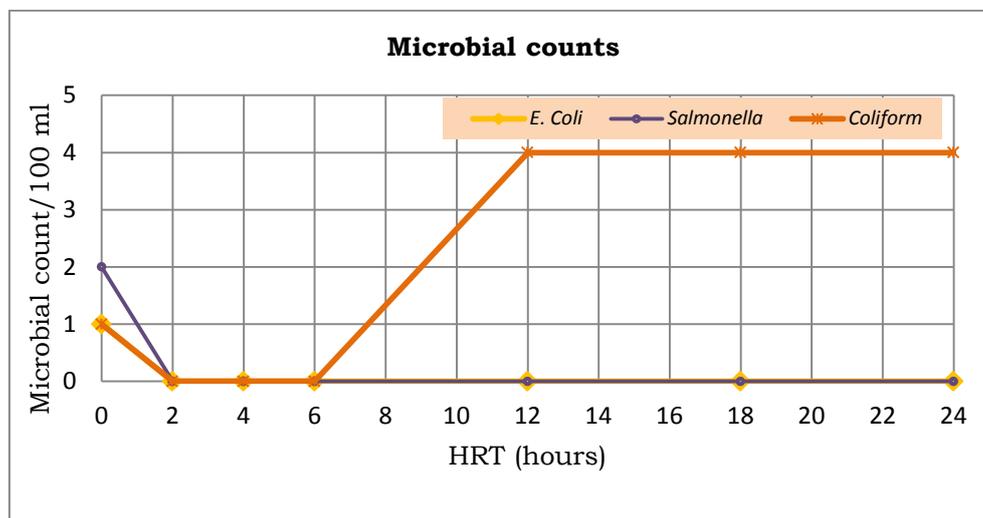


Figure 4 - 23: Trend in microbial growth observed in PVC pipes

For the PVC and CI pipes, after a HRT of 6 hours where trace residuals were recorded, *Coliform* growths were observed and this could be due to the presence of biofilm on the pipe

walls. The presence of these biofilm results in the re-growth of the microbes in limited amounts of chlorine.

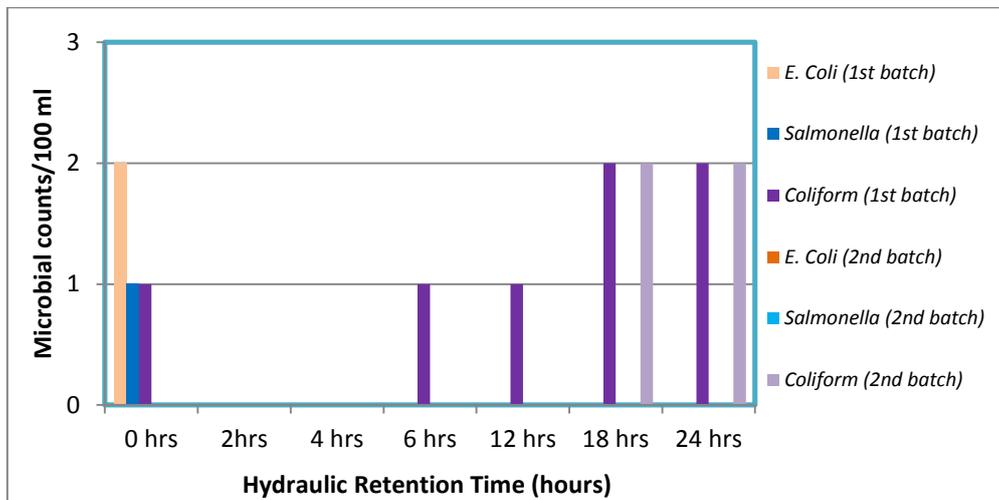


Figure 4 - 24: Microbial growth observed for the aged AC pipe

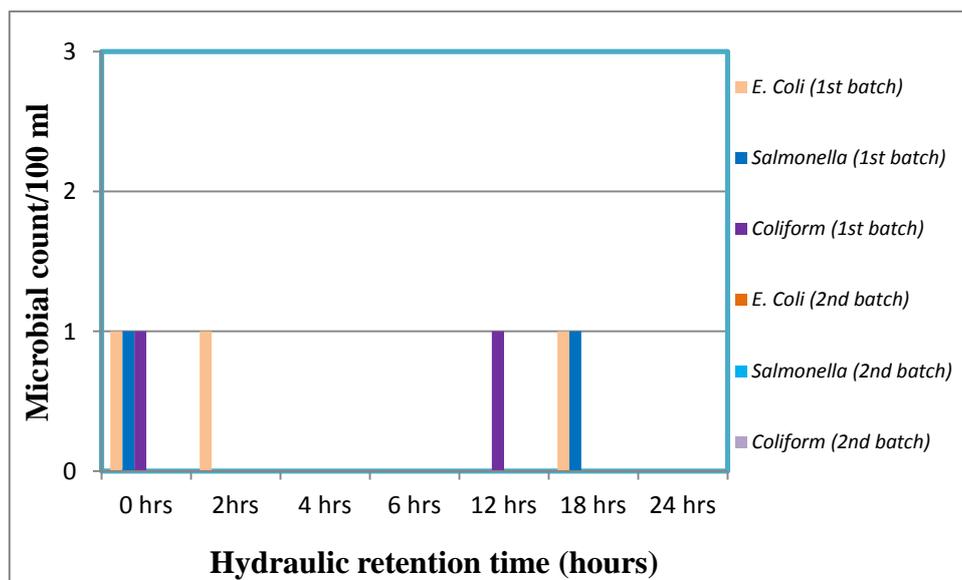


Figure 4 - 25: Microbial growth observed in AC control pipes

With the CI pipes, relatively few microbial growths were observed after 6 hours for the first batch of chlorinated water (influent concentration 0.74 mg/L); this hour marks the start of a pH drop ( $\text{pH} \leq 6$ ) in the CI pipe, thus it may suggest that although there were trace free residual chlorine concentrations in the water, the acidic nature of the water did not support

the metabolism of the microbes, thus lower counts were observed in the CI pipes. However, for the second batch of chlorinated water (influent concentration of 1.44 mg/L), few microbial counts were associated with trace free residual chlorine, this could be due to the fact that the pH of the water ( $\geq 6.2$ ) favoured the metabolism of the microbes. Furthermore, microbial growths observed in the control pipes were relatively fewer than that of the aged pipes. This could suggest that the presence of biofilms in the aged pipe contributed to relatively higher microbial growths observed. In addition, no microbial growth were observed when influent free residual concentration of the control pipes was 1.44 mg/L and this could mean that ideally for newer pipes, such a concentration may prevent re-growth of microbes.

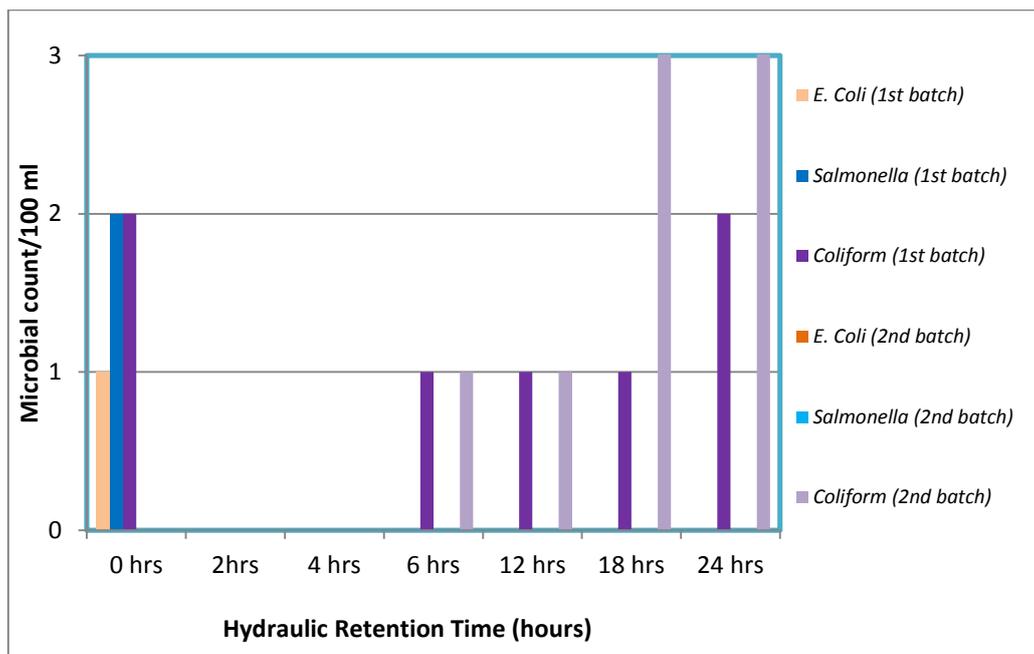


Figure 4 - 26: Trend in Microbial growth observed in aged CI pipes

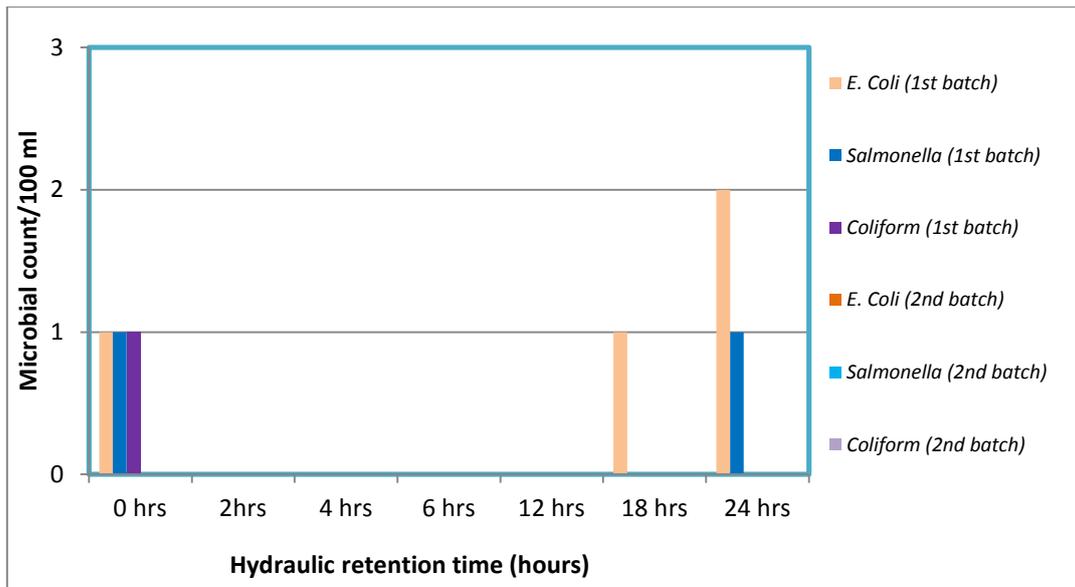


Figure 4 - 27: Microbial growth observed in CI control pipes

#### 4.6 Modelling Free residual chlorine decay in the Kumasi Distribution Network

The wall decay coefficients ( $K_w$ ) obtained for the aged pipes (see Table 4 – 8) together with the bulk decay coefficient ( $K_b$ ) were assigned for the aged pipe materials in the SynerGEE<sup>®</sup> water quality model of the Kumasi Water Distribution Network with an influent concentration of 0.74 mg/L (synonymous with the experimental) at the Achiase booster station which is about 9 km from the Suame reservoir; the main point of distribution to the Kumasi Water Distribution Network. Since the main transmission line that transports high pressurised water from Achiase to Suame is a 900 mm steel pipe, it was assumed that the wall demand in such a pipe will be relatively smaller, thus this decay was equated to that observed for the AC pipes ( $0.19 \text{ hr}^{-1}$ ) which is the least wall decay coefficient observed out of the different pipe materials under study. Also, apart from the pipes under test, all other pipes which occurred in smaller quantities spread within the distribution network were assumed to have a decay constant equal to that of the AC pipe. The output of the model suggested that the nodes in the distribution network experiences the worst residual chlorine concentrations at 7:00, 12:00 and 23:00 hours, but the scenarios observed for the 7:00 and 23:00 hours were

same. Thus, the model was corrected to boost chlorine concentration at 7:00 and 12:00 hours with the view that such an intervention will equally favour the other scenarios where relatively better nodal free residuals chlorine was observed in the network.

Figure 4 - 28: Nodal free residual chlorine concentrations at 7:00 hours

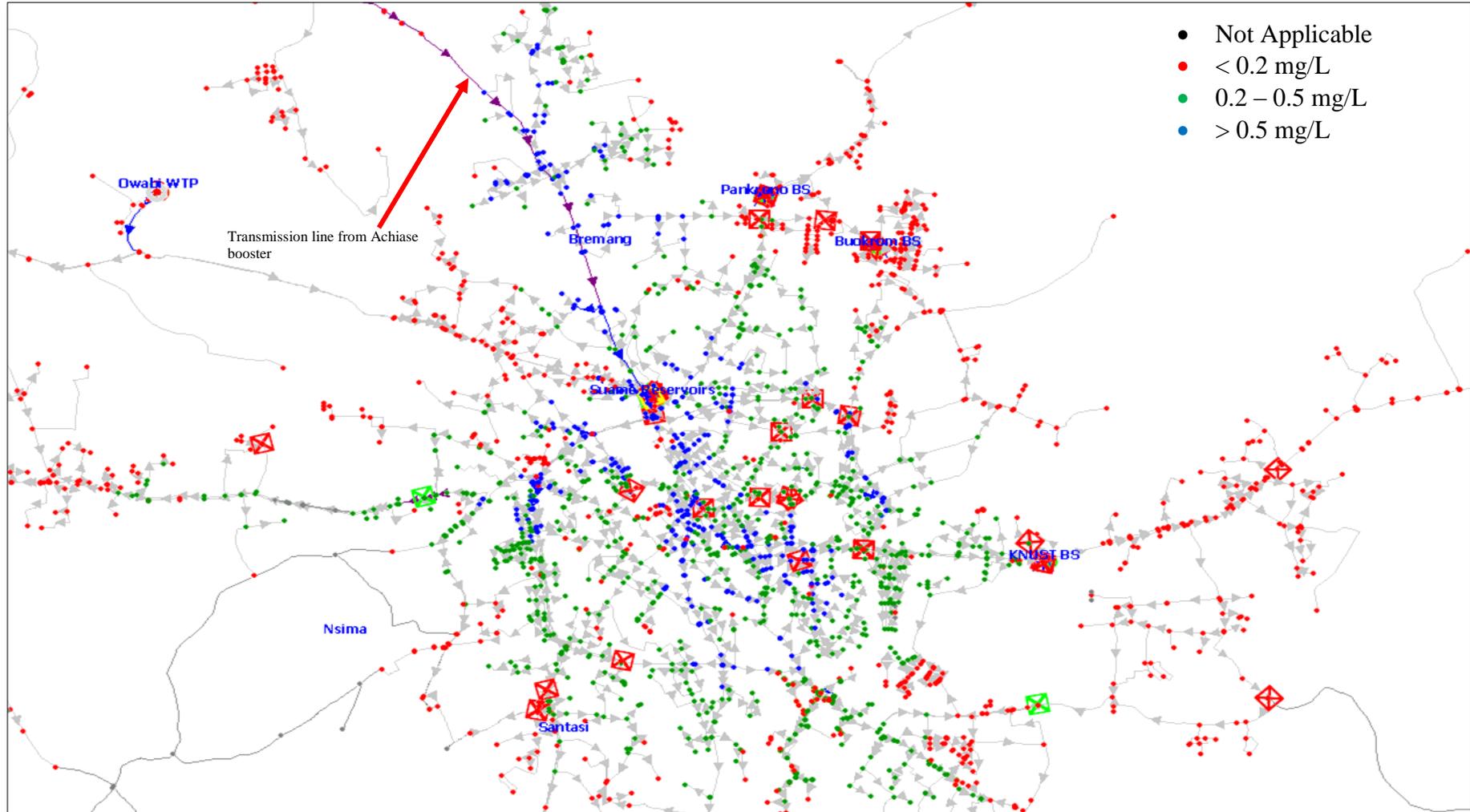


Figure 4 - 29: Nodal free residual chlorine concentrations at 12:00 hours

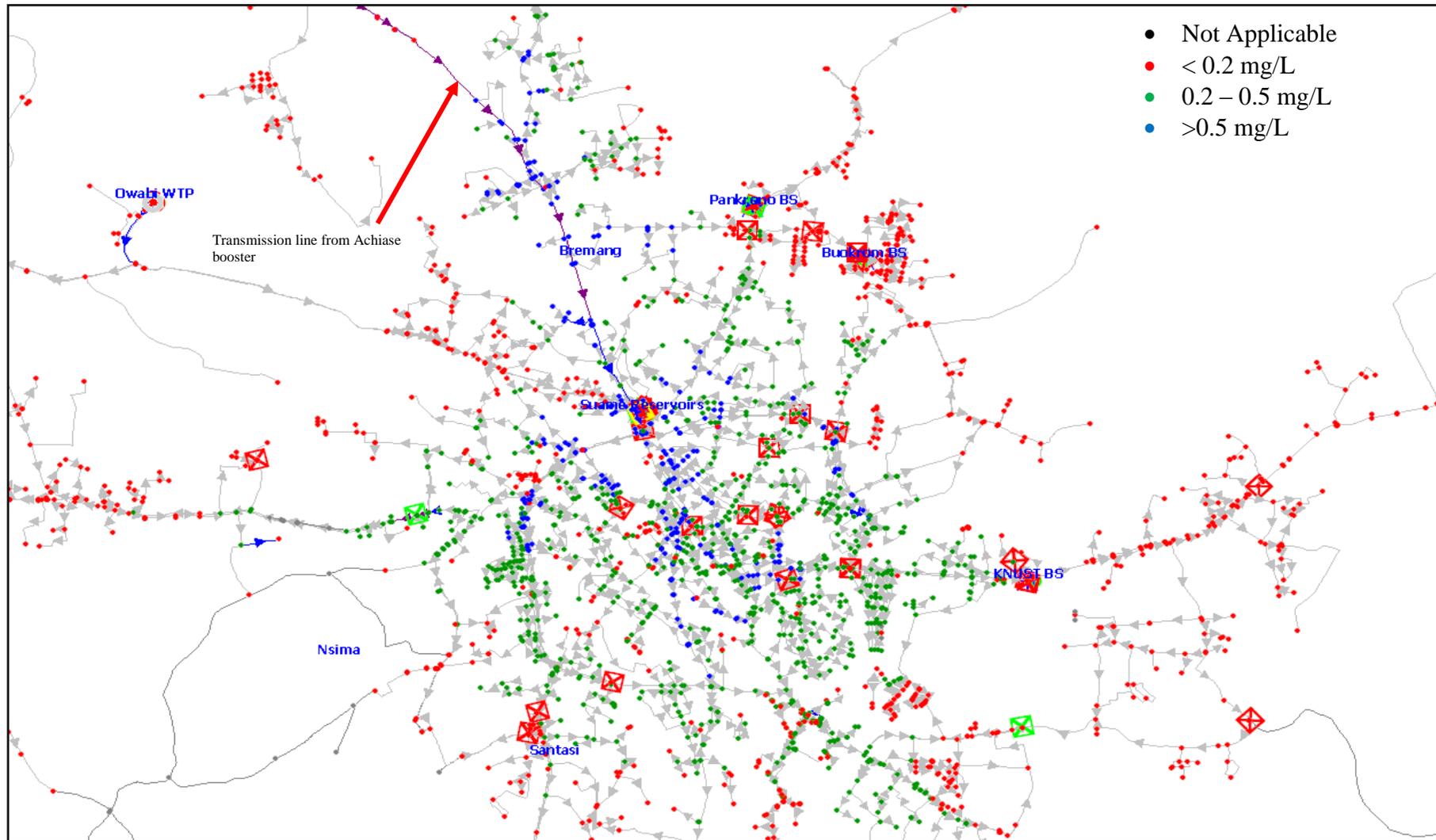
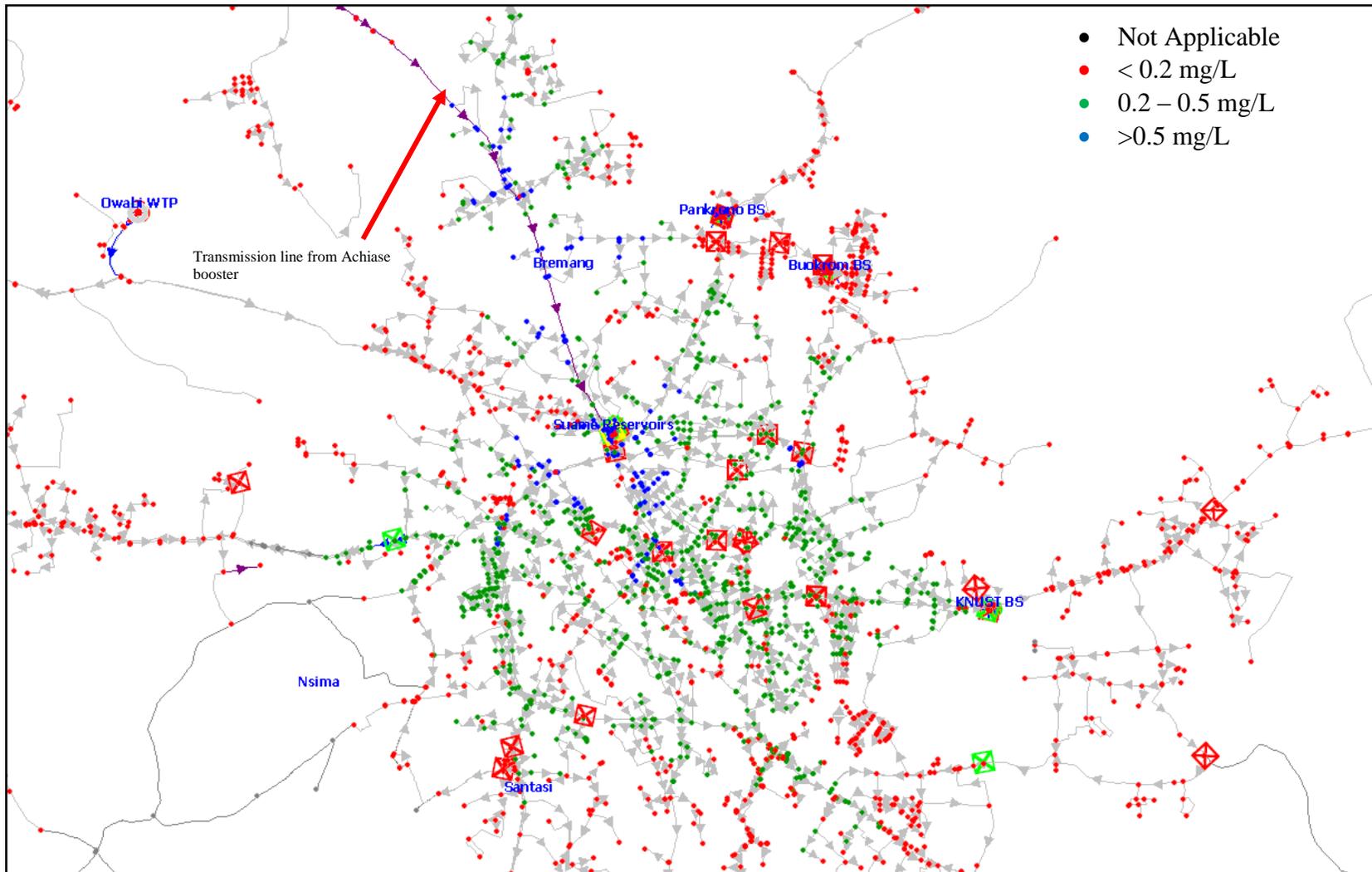


Figure 4 - 30: Nodal free residual chlorine concentrations at 23:00 hours



The result of the model shows that there are several nodes where the free residual concentration is below the GSA GV and the WHO GV of 0.2 mg/L. Thus, probable re-chlorination points were predicted and the network optimised to ensure a free residual concentration  $\geq 0.2$  mg/L for every node in the entire distribution system. This was done by employing the trial-and-error method and also using different free residual concentrations at the source and at some selected re-chlorination points. In effect, four re-chlorination points were selected and their respective probable free chlorine boosting concentrations are as shown in Table 4 -16.

Table 4 -16: Probable points for re-chlorination and their predicted concentrations

| <b>Selected probable re-chlorination points</b> | <b>Name of probable re-chlorination point</b> | <b>Location (District)</b> | <b>Predicted free residual chlorine concentration (mg/L)</b> |
|---|---|----------------------------|--|
| Main transmission point                         | Achiase booster station                       | Kumasi North               | 1.00   |
| 1 <sup>st</sup> Re-chlorination point           | Pankrono booster station                      | Kumasi East                | 0.50   |
| 2 <sup>nd</sup> Re-chlorination point           | Buokrom booster station                       | Kumasi East                | 0.50   |
| 3 <sup>rd</sup> Re-chlorination point           | Suame reservoirs                              | Kumasi Central             | 0.74   |
| 4 <sup>th</sup> Re-chlorination point           | KNUST booster station                         | Kumasi South               | 0.50   |

These free residual concentrations were assigned for the various points and the model was run. The output for 7:00 and 12:00 hours is shown in Figure 4 - 31 and Figure 4 - 32 respectively. It must however be stated that areas in the North West district which are served by Owabi WTP was not considered, thus all red nodes in surrounding areas of the said treatment plant depict “no flow.”

Consequently, areas which still observed lower residuals were as a result of very negligible or poor flow situations encountered.

Figure 4 - 31: Nodal free residual chlorine concentrations at 7:00 and 23:00 hours

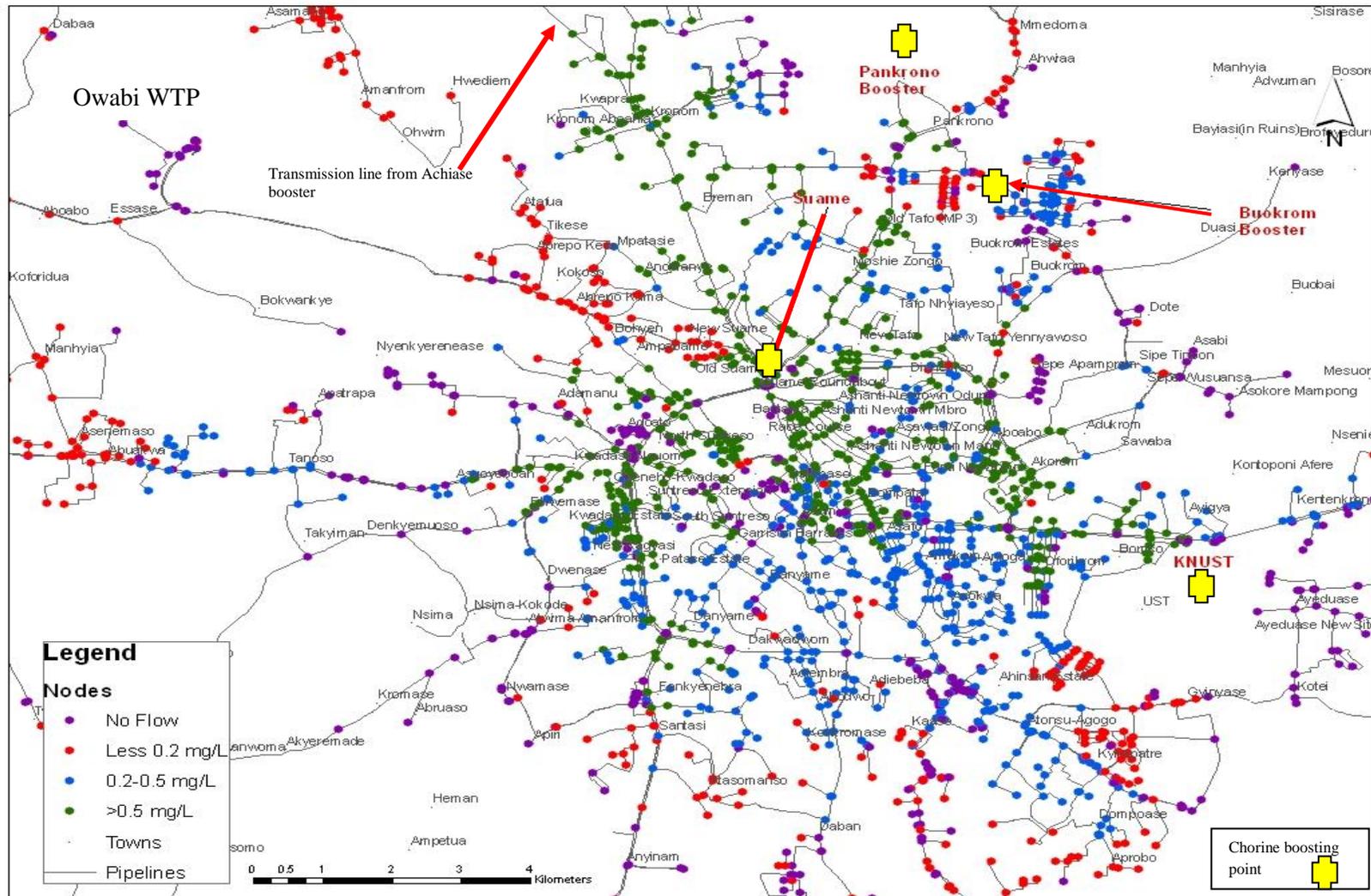
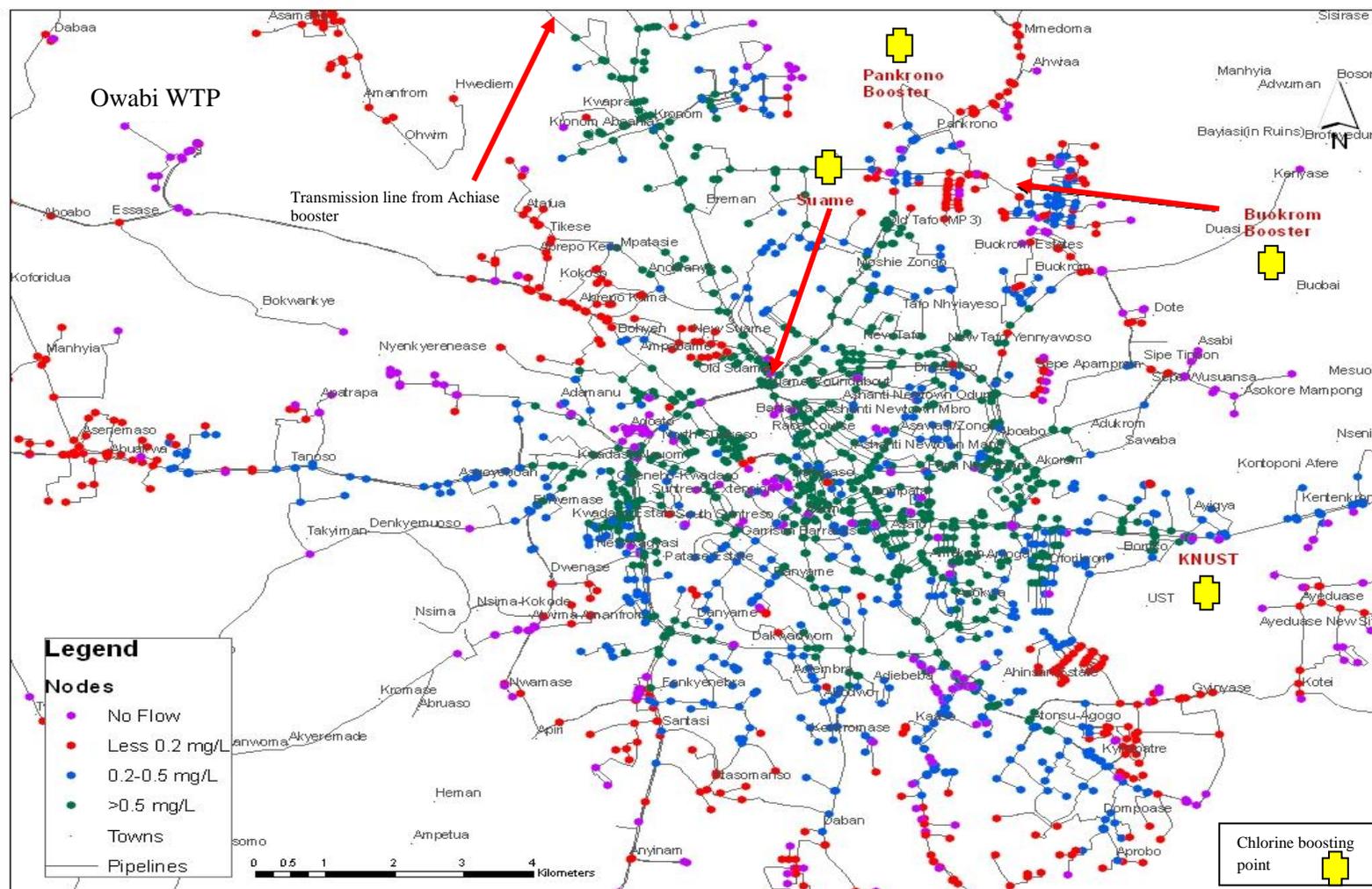


Figure 4 - 32: Nodal free residual chlorine concentrations at 12:00 hours



## CHAPTER 5

### 5.0 CONCLUSIONS AND RECOMMENDATION

#### 5.1 Conclusion

- The bulk decay coefficient ( $K_b$ ) was found to be  $0.053 \text{ hr}^{-1}$  at  $26 \text{ }^\circ\text{C}$ , and within 8 hours, the residual chlorine concentration within the bulk fluid decreases by 32 – 34 % of its initial dose.
- Under the conditions tested, the overall decay coefficients ( $K$ ) for the aged PVC (15 - 20 years), AC (40 – 50 years), and Cast Iron (84 years) were found to be 0.30, 0.24 and  $0.43 \text{ hr}^{-1}$  respectively whereas that of the newer pipes were found to be 0.14, 0.10,  $0.16 \text{ hr}^{-1}$  for PVC, AC and CI pipes respectively. These decay coefficients could be applied to pipe materials within the same age brackets.
- Chlorine decay was predominantly due to pipe wall reactions and the rate was in the order  $\text{CI} > \text{PVC} > \text{AC}$  (pipe material) and  $3 > 6 > 9$  inch (diameter). The decay in C.I pipes was approximately 1.8 and 1.5 times higher than that of PVC and AC pipes respectively.
- After a hydraulic retention time of 6 hours, the water experiences microbial re-growth.
- The wall decay coefficients of the pipe materials vis-à-vis their ages indicate that for every year the  $K_w$  increases at a rate of 0.0125, 0.0038 and  $0.0045 \text{ hr}^{-1}$  for PVC, AC and CI pipes respectively.
- Within a hydraulic retention time of 2 hours, the free residual chlorine recorded in Cast Iron pipes were 65 – 73 % lesser than its initial dose, that of PVC was 55 % lesser and that of the AC pipe was 33 – 55 % lesser than its initial dose. Thus, the CI pipes (84 years old) consume chlorine faster due to encrustations.

- pH of distributed water from the PVC and AC pipes were fairly constant ranging from 6.7 to 7.3, whereas that of the CI pipes decreased over time, ranging from 5.7 to 6.9.
- The Achiase, Pankrono, Buokrom, Suame (reservoirs) and KNUST booster stations are probable chlorine boosting points and free residual chlorine concentrations predicted by the model at these points are 1.0 mg/L, 0.5mg/L, 0.5 mg/L, 0.74 mg/L and 0.5 mg/L respectively.

## **5.2 Recommendation**

- This experiment was done using lower free chlorine concentrations. Thus, GUWL may replicate this study using higher doses to ascertain the decay coefficients obtained. Also, the GUWL may conduct field studies to ascertain the decay coefficients by calibrating the model developed in the Kumasi water distribution system to verify their accuracy.
- The GUWL may conduct further studies to investigate the effects of TOC and NOM's on residual chlorine decay.
- For pipe segments in the distribution system where the chlorine consumption is very high, as in the case of the cast iron pipes, it may be economical to replace such pipes to avoid or reduce the risk of zero residuals so as to meet water quality goals. The GUWL may therefore conduct further studies as to whether it would be economical to replace these pipe lines or rather boost the chlorine levels.
- Also it will be advisable to regularly flush the CI pipes with high pressurized water to remove Fe and Mn deposits.
- The GWUL may consider Pankrono, Buokrom, Suame (reservoirs) and KNUST booster points as points for boosting chlorine.

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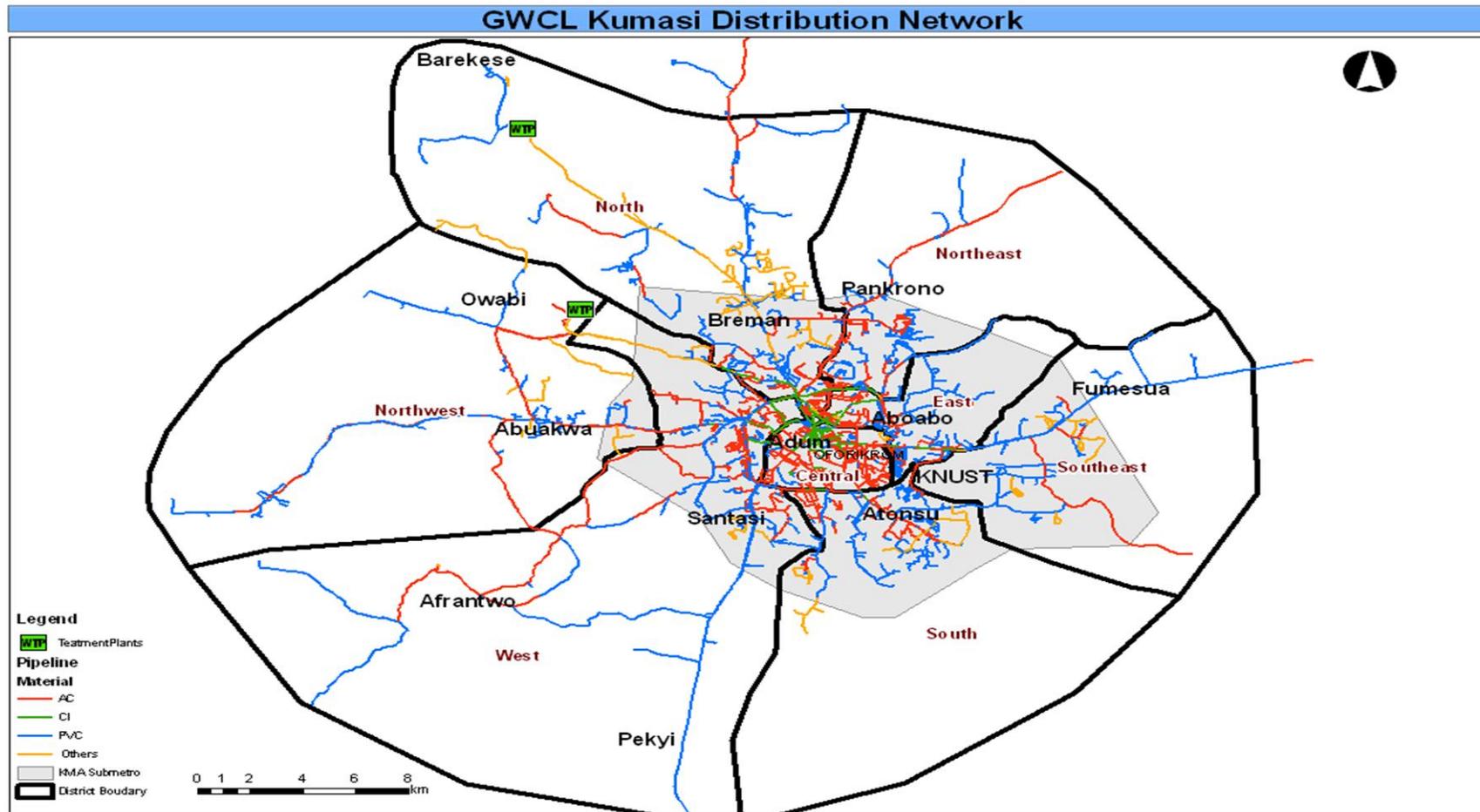
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APPENDICES

Appendix 1: Kumasi Water Distribution Network



Appendix 2: Five (5) year Water Quality Data of the Barekese Final water

| <b>Parameter</b>                  | <b>Average value</b> |
|-----------------------------------|----------------------|
| pH                                | 7.15                 |
| Temperature (°C)                  | 25.63                |
| Colour (Hazen Units)              | 5.27                 |
| Turbidity (NTU)                   | 1.90                 |
| Free Residual Chlorine (mg/L)     | 0.74                 |
| E-coli                            | 0                    |
| Hardness (mgCaCO <sub>3</sub> /L) | 61.93                |
| Chloride (mg/L)                   | 17.95                |
| Total Dissolved Solid (mg/L)      | 65.90                |
| Sulphate (mg/L)                   | 34.69                |
| Aluminium (mg/L)                  | 0.10                 |
| Iron (mg/L)                       | 0.20                 |
| Manganese (mg/L)                  | 0.01                 |
| Fluoride (mg/L)                   | 0.50                 |
| Nitrate (mg/L)                    | 0.19                 |
| Arsenic (mg/L)                    | 0.00                 |
| Nitrite (mg/L)                    | 0.02                 |
| Ammonia (mg/L)                    | 0.00                 |

**SOURCE: Regional Water Quality Division, GUWL, Kumasi**

Appendix 3: Experimental Results for Overall Decay Constant (K) determination for All Pipe Materials under study

| <b>Free Residual Chlorine(mg/L)</b> |                       |       |       |       |                          |
|-------------------------------------|-----------------------|-------|-------|-------|--------------------------|
| Time (hrs)                          | PVC<br>(Control Pipe) | PVC 1 | PVC 2 | PVC 3 | Average<br>(PVC 1, 2,3 ) |
| 0                                   | 0.74                  | 0.76  | 0.74  | 0.73  | 0.74                     |
| 2                                   | 0.52                  | 0.42  | 0.27  | 0.30  | 0.33                     |
| 4                                   | 0.34                  | 0.19  | 0.20  | 0.22  | 0.20                     |
| 6                                   | 0.26                  | 0.13  | 0.15  | 0.13  | 0.14                     |
| 12                                  | 0.13                  | 0.03  | 0.04  | 0.03  | 0.03                     |
| 18                                  | 0.02                  | 0.01  | 0.01  | 0.01  | 0.01                     |
| 24                                  | 0.02                  | 0.01  | 0.01  | 0.01  | 0.01                     |

| <b>Free Residual Chlorine (mg/L)</b> |                      |      |      |      |                          |
|--------------------------------------|----------------------|------|------|------|--------------------------|
| Time (hrs)                           | AC<br>(Control Pipe) | AC1  | AC 2 | AC 3 | Average<br>(AC 1, 2, 3 ) |
| 0                                    | 0.74                 | 0.76 | 0.73 | 0.74 | 0.74                     |
| 2                                    | 0.43                 | 0.29 | 0.32 | 0.39 | 0.33                     |
| 4                                    | 0.33                 | 0.26 | 0.22 | 0.26 | 0.25                     |
| 6                                    | 0.25                 | 0.20 | 0.17 | 0.15 | 0.17                     |
| 12                                   | 0.15                 | 0.08 | 0.08 | 0.07 | 0.08                     |
| 18                                   | 0.10                 | 0.05 | 0.05 | 0.04 | 0.05                     |
| 24                                   | 0.02                 | 0.01 | 0.00 | 0.00 | 0.00                     |

| <b>Free Residual Chlorine (mg/L)</b> |                      |      |      |      |                          |
|--------------------------------------|----------------------|------|------|------|--------------------------|
| Time (hrs)                           | AC<br>(Control Pipe) | AC1  | AC 2 | AC 3 | Average<br>(AC 1, 2, 3 ) |
| 0                                    | 1.44                 | 1.43 | 1.45 | 1.44 | 1.44                     |
| 2                                    | 1.23                 | 0.90 | 1.03 | 0.96 | 0.96                     |
| 4                                    | 0.77                 | 0.74 | 0.79 | 0.74 | 0.76                     |
| 6                                    | 0.51                 | 0.47 | 0.42 | 0.36 | 0.42                     |
| 12                                   | 0.18                 | 0.10 | 0.11 | 0.10 | 0.10                     |
| 18                                   | 0.18                 | 0.07 | 0.09 | 0.09 | 0.08                     |
| 24                                   | 0.17                 | 0.02 | 0.02 | 0.03 | 0.02                     |

| <b>Free Residual Chlorine (mg/L)</b> |                      |      |      |      |                          |
|--------------------------------------|----------------------|------|------|------|--------------------------|
| Time (hrs)                           | CI<br>(Control Pipe) | CI 1 | CI 2 | CI 3 | Average<br>(CI 1, 2, 3 ) |
| 0                                    | 0.74                 | 0.75 | 0.70 | 0.76 | 0.74                     |
| 2                                    | 0.33                 | 0.18 | 0.23 | 0.20 | 0.20                     |
| 4                                    | 0.25                 | 0.13 | 0.12 | 0.12 | 0.12                     |
| 6                                    | 0.19                 | 0.07 | 0.07 | 0.07 | 0.07                     |
| 12                                   | 0.10                 | 0.02 | 0.03 | 0.03 | 0.03                     |
| 18                                   | 0.02                 | 0.02 | 0.02 | 0.01 | 0.02                     |
| 24                                   | 0.01                 | 0.00 | 0.01 | 0.00 | 0.00                     |

| <b>Free Residual Chlorine (mg/L)</b> |                      |      |      |      |                          |
|--------------------------------------|----------------------|------|------|------|--------------------------|
| Time (hrs)                           | CI<br>(Control Pipe) | CI 1 | CI 2 | CI 3 | Average<br>(CI 1, 2, 3 ) |
| 0                                    | 1.44                 | 1.46 | 1.45 | 1.41 | 1.44                     |
| 2                                    | 0.82                 | 0.50 | 0.49 | 0.52 | 0.50                     |
| 4                                    | 0.52                 | 0.25 | 0.25 | 0.22 | 0.24                     |
| 6                                    | 0.39                 | 0.16 | 0.17 | 0.14 | 0.16                     |
| 12                                   | 0.12                 | 0.06 | 0.03 | 0.04 | 0.04                     |
| 18                                   | 0.11                 | 0.02 | 0.02 | 0.02 | 0.02                     |
| 24                                   | 0.08                 | 0.00 | 0.00 | 0.00 | 0.00                     |

Appendix 4: Three pieces of CI pipes (1.0 m each) used in the experimental and the encrustation observed in the pipe lumen



Appendix 5: Three pieces of PVC and AC (1.0 m each) used in the experimental



Appendix 6: Control pipes (PVC, AC, CI) used in the experimental

