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Assessment of residential drinking water quality in Maadi, Egypt

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The American University in Cairo
School of Science and Engineering
Interdisciplinary Engineering Programs

Assessment of Residential Drinking Water Quality in Maadi, Egypt

by

Sally Fadel Naguib El Komos

A thesis submitted in partial fulfillment of the requirements for the degree of:

Master of Science in Engineering

With specialization in:

Environmental Engineering

Under the supervision of:

Dr. Edward H. Smith
Professor, Construction Engineering Department

Spring 2004

The American University in Cairo
School of Sciences and Engineering

Assessment of Residential Drinking Water Quality in Maadi, Egypt

A Thesis Submitted by
Sally Fadel El Komos

to the Interdisciplinary Engineering Programs

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Dedication

I dedicate my work to my beloved family: Thanks to my father, Dr. Fadel El Komos, for his guidance and experienced advice, which helped me acquire the essential tools for good research and professional work. Thanks to my mother, my brothers and sister for their constant support and encouragement.

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Abstract

Safe water is the outcome of successful treatment and distribution practices. Water quality monitoring is essential for the assessment of the compliance of water quality with health-based standards at different stages of treatment/transportation. In the present study, tap water quality was evaluated in Maadi, a residential community of Greater Cairo (Egypt). Additionally, due to the growing number of residents using point of use water treatment units as a supplementary treatment for their tap water, the performance of three commonly used POU devices for removal of water contaminants was evaluated as part of the study program. Six sampling events were carried out during the period September 2001 to June 2003. Samples and water quality data were collected at the Nile water source as well as the water treatment plants (Dar-El-Salaam and Maadi WTPs) and 20 residences in Maadi of which 19 had a POU device in place.

Water quality analyses for conventional parameters (total chlorine, pH, alkalinity, hardness, turbidity, total dissolved solids, lead, iron, copper, arsenic, nitrate, fluoride, coliform bacteria, and total trihalomethanes) showed no violation of standard limits, except for the occasional occurrence of high TTHM levels up to 112 μg/L, thus exceeding the 80 μg/L USEPA limit and the 100 μg/L Egyptian limit. Water quality data exhibited no clear seasonal pattern for the studied parameters during the period of analysis. Due to the nature of the compounds and their implications for human health, emphasis was given to chlorine and TTHM reactions. Modeling was performed with the aim of investigating the kinetics and possible simulation of chlorine and TTHM concentrations in the distribution system studied on the basis of a first order reaction model, including an application of EPANET modeling software. The effect of wall reaction in the decay of residual chlorine was higher than the contribution of bulk water interactions, a finding consistent with water quality modeling research in developed countries. The contribution of wall effects to THM formation, while small compared to bulk interactions, suggests the presence of THM precursors in the form of organic matter adsorbed on the pipe walls. A study was made to investigate the effect of the system hydraulics (residence time, pipe type and diameter, velocity) on chlorine decay, recommending the application of different simulations of chlorine concentration in the distribution system during the design period, in order to choose the hydraulic conditions that would allow optimum chlorine and TTHM kinetics.

GAC POU-WTUs were found to be effective for removal of THMs and chlorine, with almost no removal of inorganic salts. RO POU-WTUs were effective for removal of THMs, chlorine and inorganic salts. POU distillation units were effective for removal of chlorine, inorganic salts, and removed almost 50% of the TTHMs.

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1. INTRODUCTION

1.1 Importance of Drinking Water Quality Research

Due to its importance for all living organisms and its direct effect on health and metabolism, safe water is an essential need to be furnished wherever required for drinking, agricultural and all other uses. This makes continuous water quality research a must, to encounter and control all the factors affecting water from its source until it reaches the consumers.

1.2 Statement of the Problem

The last century has witnessed extensive efforts regarding water quality research. Standards have been set for different water quality parameters. Early research was focused on water treatment processes (in water treatment plants) and ways to make the water quality meet the standards. More recent is the research on water distribution systems and their effect on water quality. Distribution systems have generally been designed to ensure hydraulic reliability, especially with respect to water quantity and fire flow pressure. These design criteria result in large amounts of storage and extended residence times, which affects the water quality in the systems (Clark *et al.*, 1995). Researchers working on distribution systems have tried to track and model the evolution of the different water quality parameters in the system.

Considerable research still needs to be done in this area in order to more accurately model and predict the water quality reaching the consumers. For example, more work is needed to understand the effect of different factors (*e.g.*, pH, temperature, pipeline materials and diameters, residence times, *etc.*) on the decay/formation of key water quality parameters and the relationship between these parameters; as, for example, the chlorine residual and chlorination by-products.

On the other hand, as the water quality in the distribution system has not always been satisfactory, consumers have turned to point-of-use water treatment units (POU-WTUs) in order to amend deficiencies in water quality parameters; for example, high chlorine residuals, solids, hardness, heavy metals, disinfection by-products, *etc.* More research needs to be done in this area, in particular to study the factors affecting the performance of these units.

1.3 Objectives

The objectives of this study are the following.

- Evaluate tap water quality in a residential area in Greater Cairo and its compliance with water quality standards.
- Determine whether there are notable seasonal variations in water quality parameters.
- Evaluate the effectiveness of POU-WTUs for removal of various water contaminants.
- Assess the impacts of the distribution system on the water quality parameters of interest.

Based on the findings of the early stage of the research, this latter objective focused on investigating the impact of the distribution system on chlorine and chlorination by-products.

1.4 Plan of Work

An experimental program was organized to collect data on water quality in Maadi, a community of Greater Cairo in which a growing number of residents are using POU devices. Data was collected at Dar El Salaam Water Treatment

Plant (WTP), the major WTP feeding the study area; Maadi WTP; Nile River in the vicinity of the Dar El Salaam WTP intake; and 20 residences in Maadi, Digla and new Maadi. In the case of residences, samples were collected from both tap water and effluent from the POU device. Six sampling events were carried out in the period September 2001 to June 2003.

Water quality analyses were conducted for regulated parameters including temperature, total chlorine, pH, alkalinity, hardness, turbidity, total dissolved solids, heavy metals (lead, iron, copper, arsenic), nitrate, fluoride, coliform bacteria, and total trihalomethanes.

Maps representing the distribution system network in the study area have been prepared. Water quality data collected was elaborated to achieve the objectives of the study using various tools including statistical and comparative analyses. Existing modeling approaches were examined as to their utility for describing the impact of the distribution system on water quality parameters, in particular the decay of residual chlorine and the formation of chlorination by-products.

2. LITERATURE REVIEW

Drinking water quality monitoring is essential for the assessment of the compliance of water quality with health-based standards. Water samples are commonly taken from the water treatment plants as well as from points in the distribution system and they are analyzed for various parameters depending on the nature of source water and the treatment and distribution system conditions. This chapter provides an overview of the water quality parameters that are conventionally analyzed in most water systems, together with Egyptian and international standards for these parameters.

Special interest is given in this review to our current understanding of transformations in water quality as it moves through the distribution system. Due to the nature of the compounds and their implications for human health, emphasis is given to chlorine and chlorination by-products reactions, including an overview of the modeling approaches developed for the evolution of these compounds in water distribution systems.

Finally, as the distributed water may sometimes be deemed unsatisfactory by consumers, some are choosing to further treat their water using POU units. An overview of three commonly used POU treatment units is provided in this chapter.

2.1 Drinking Water Quality Constituents

2.1.1 Physical and Aggregate Properties

Temperature

Water temperature is a primary measurement in all water quality analyses.

"The most desirable drinking waters are consistently cool and do not have temperature changes of more than a few degrees", (USEPA, 1992). The rate of all chemical reactions increases with temperature, with the exception of enzymatically catalyzed reactions where the catalytic properties are reduced above a certain temperature (Snoeyink and Jenkins, 1980). Common problems that may occur due to high water temperature in water systems are the enhancement of the growth of microorganisms and the increase in corrosion rates.

Total Dissolved Solids (TDS)

Total dissolved solids (TDS) is the term used to describe inorganic salts such as sodium, calcium, magnesium, potassium, chlorides and sulfates, and small amounts of organic matter that are dissolved in water. TDS are normally present in natural water sources. They may also originate from sewage and industrial wastewater discharges to the raw water source (WHO, 1998).

High levels of TDS (>1200 mg/L) may result in a bad taste of the water, scaling in water pipes, heaters, boilers, and household appliances. Low levels of TDS cause a flat, insipid taste. Extremely low TDS levels may result in corrosion of water supply systems, as this condition is normally related to low levels of metallic carbonate ions that form a protective coating on pipe walls (WHO, 1998). On the other hand, high TDS levels are related to high levels of ion concentration in water, which may enhance electrochemical corrosion due to high conductivity and high flow of electrons (Mays, 2000).

Turbidity

Turbidity in water is caused by the presence of suspended matter such as clay, silt, organic and inorganic matter fines, and micron-sized organisms. It may also derive from colloidal iron particles scoured from the pipes or the resuspension of sediment in the distribution system. Turbidity is a measure of the water's ability to scatter and absorb light rather than transmitting it directly (WHO, 1998; Standard Methods, 1992; USEPA, 1992).

In addition to imparting undesirable taste and/or odor or aesthetic rejection due to the lack of clarity or rust in water, turbidity may also be associated with adverse health effects. Harmful organic and inorganic substances may adsorb to suspended particles. Moreover, suspended solids or adsorbed materials may exert a chlorine demand, thus protecting pathogenic organisms from the effects of disinfectants (WHO, 1998). Finally, nutrients adsorbed onto particulate surfaces in turbid water may stimulate the growth of disease-causing microorganisms such as viruses, parasites, and some bacteria in distribution systems (USEPA, 2002; WHO, 1998).

Alkalinity

Alkalinity is a measure of the capacity of water to neutralize strong acid (and resist change in pH). Alkalinity in natural waters mainly results from the presence of bases such as carbonate, bicarbonate, and hydroxyl ions, in addition to other bases (often present with smaller concentrations) such as silicates, borates, ammonia, phosphates and organic bases (Snoeyink and Jenkins 1980). The presence of alkalinity is determined by standard methods involving various indicator solutions and titration with strong acid. It is expressed in milligrams of calcium carbonate equivalent per liter (Standard Methods, 1992, USEPA, 1992).

Alkalinity provides water with buffering capacity to neutralize acid additions. It also contributes in the formation of a metallic carbonate coating (due to the presence of carbonate ions) that protects the pipe wall, and can help prevent calcium dissolution from cement-lined and asbestos-cement pipes (Mays, 2000).

Hardness

Water hardness is a measure of the capacity of water to precipitate soap (hard water retards the cleaning action of soap and detergents). Hard water and soft water are relative terms; however, water with total hardness less than 75 mg/L as calcium carbonate is considered soft, and above 300 mg/L is considered very hard (USEPA, 1992). In common practice, total hardness is defined as the sum of calcium and magnesium concentrations, both expressed as milligrams of calcium carbonate equivalent per liter (Standard Methods, 1992; USEPA, 1992). Water hardness may originate from natural sources such as sedimentary rocks, in addition to industrial wastewater discharges to raw water sources. It may also derive from the dissolution of calcium from cement-lined pipes or asbestos-cement pipes (WHO, 1998).

Water hardness is divided into two general classifications: carbonate hardness (called temporary because, when water is heated, bicarbonates break into solid particles that stick to the heated surface), and non-carbonate hardness (called permanent as it is not removed by heating water) (Standard Methods, 1992; USEPA, 1992).

Very hard water causes an undesirable taste. It is also associated with scale deposition in the distribution system (under high temperature conditions) or in kettles, heating coils and cooking utensils. On the other hand, soft water may cause the corrosion of pipes; it may also have an adverse effect on mineral balance (WHO, 1998).

2.1.2 Inorganic Non Metallic Constituents

Chlorine (Residual)

Chlorine is widely used in water treatment as an oxidizing agent and as a disinfectant. Chlorine applied to water undergoes hydrolysis to form free chlorine, which is responsible for the disinfecting action of chlorine, (Kastl *et al.*, 2002). Free chlorine reacts with water constituents such as nitrogen and ammonia to form combined chlorine. The sum of free and combined chlorine represents the total chlorine concentration. Chlorine residual is the concentration of chlorine (free/total) in the water distributed to consumers as it leaves the treatment plant, or at points in the distribution system. To keep the water biologically safe in the distribution system, chlorine residuals need to be maintained at a certain level until water reaches the consumers [0.2 mg/L (USEPA, 2002); 0.5 mg/L (WHO, 1998)]. High chlorine residuals (above 4.0 mg/L) at the consumer's tap may cause adverse health effects such as eye/nose irritation and stomach discomfort (USEPA, 2002). Chlorine residuals above 1.0 mg/L may cause undesirable taste and odor (WHO, 1998). The reaction of chlorine with organic matter present in water produces by-products that may have carcinogenic impacts on human health, for example trihalomethanes and haloacetic acids (Singer *et al*, 1995; USEPA, 1999).

Fluoride

Fluoride occurrence in water may be due to its presence in raw water sources, the erosion of natural deposits, or the discharge from fertilizer and aluminum factories. It may also be added to treated water in order to promote strong teeth (WHO, 1998). Fluoride is a health benefit with low concentrations (0.5-0.9 mg/L), as it provides protection against dental cavities. Higher fluoride concentrations may cause bone disease (pain and tenderness of the bones) and discoloration of teeth (WHO, 1998; USEPA, 2002).

pH

pH, or hydrogen ion activity, refers to the intensity of the acidic or basic character of a solution at a given temperature. The pH of a solution is the negative common logarithm of the hydrogen ion activity in moles per liter: $pH = -\log \{H^+\}$. In dilute solutions, the hydrogen ion activity is approximately equal to the hydrogen ion concentration. Solutions with pH = 7 are said to be neutral. Solutions with pH below 7.0 are referred to as acid solutions. Solutions with pH above 7.0 are referred to as basic or alkaline solutions (Standard Methods, 1992; Snoeyink and Jenkins 1980). Extreme values of pH may result from accidental spills, treatment breakdowns, and insufficiently cured cement mortar pipe linings (WHO, 1998).

pH is a major parameter at all stages of water treatment, especially clarification and disinfection. For effective disinfection with chlorine, pH should preferably be less than 8.0 (WHO, 1998; Droste, 1997). pH above 10 may cause eye irritation and skin disorders as well as the swelling of hair fibres. pH below 6 becomes problematic in public water supplies, contributing to degradation of tooth enamel, eye irritation, or worse. Low pH levels also enhance the corrosion of pipes in water distribution systems. The optimum pH required to prevent corrosion varies in different supplies according to the composition of the water and the nature of the construction materials used in the distribution system, but it is often in the range 6.5–9.5, (WHO, 1998; Mays, 2000).

Nitrate

Nitrates are nitrogen-oxygen chemical units, which combine with various organic and inorganic compounds. Inorganic sources of nitrates are mainly fertilizers, while organic sources mainly consist in human sewage and animal manure. Once taken into the body, nitrates are converted into nitrites (USEPA, 2002). The toxicity of nitrate to humans is mainly attributable to its reduction to nitrite. The major biological effect of nitrite in humans is its involvement in the oxidation of normal haemoglobin (Hb) to methaemoglobin (metHb), which is unable to transport oxygen to the tissues. Symptoms include shortness of breath and blueness of the skin (USEPA, 2002; WHO, 1998).

2.1.3 Inorganic Metallic Constituents

Calcium

The presence of calcium in water supply systems may result from natural sources such as sedimentary rocks, calcium dissolution from cement-lined pipes or asbestos-cement pipes, or from industrial activities (including food products) (WHO, 1998). Calcium has a direct contribution in the formation of water hardness and alkalinity (described above). As mentioned earlier, small concentrations of calcium carbonate combat corrosion of metal pipes by laying down a protective coating. High levels of calcium salts may cause scale deposition in cooking utensils, water pipes, and boilers including water distillation devices.

Magnesium

Magnesium is normally present in water due to natural sources such as sedimentary rocks or industrial products such as food. As for calcium, magnesium is a major contributor to water hardness. High levels of magnesium salts contribute to the formation of scale in water pipes and utensils.

Lead

Lead occurrence in drinking water is generally due to the erosion of natural deposits, or the corrosion of plumbing materials in the water distribution system (lead service lines, lead solders and fluxes used to connect copper pipes) and household plumbing systems (USEPA, 2002).

Lead accumulation in the body may cause adverse effects even with exposure to very small amounts. It may delay the physical and mental development in babies and young children, and cause deficits in the attention span, hearing and learning abilities of children. Chronic exposure to lead has been linked to cerebrovascular and kidney disease. Lead has the potential to cause cancer from a lifetime exposure at levels above the action level of 0.015 mg/L (USEPA, 2002).

Iron

Iron generally occurs in drinking water due to iron coagulants used in water treatment. It may also originate from the corrosion of steel and cast iron pipes during water distribution.

Iron is an essential element in human nutrition. Estimates of the minimum daily requirement for iron range from about 10 to 50 mg/day; however high levels of iron (higher than 2 mg/liter) may cause adverse health effects. High iron levels may also stimulate the growth of "iron bacteria" in pipes and may cause the staining of laundry and plumbing fixtures (WHO, 1998).

Copper

Copper generally occurs in drinking water due to the corrosion of copper pipes used for interior plumbing of residences, in addition to smelting operations and municipal incineration. In some cases, copper is a component of additives to drinking water used to control the growth of algae (USEPA, 2002).

Copper is an essential nutrient, required by the body in very small amounts. However, at high doses, copper has been shown to cause stomach and intestinal distress, liver and kidney damage, and anemia.

Arsenic

Arsenic generally occurs in drinking water due to volcanic activity in contact with source waters, erosion of rocks and minerals, forest fires, agricultural applications, mining, smelting, and industrial wastes (arsenic is used in paints, drugs, dyes, soaps, metals and semi-conductors).

Adverse health effects of arsenic include cancerous effects such as skin, bladder, lung, kidney, nasal passages, liver and prostate cancer, and non-cancerous effects such as cardiovascular, pulmonary, immunological, and neurological (USEPA, 2002).

2.1.4 Organic Constituents

Natural Organic Matter (NOM) and its Surrogate Parameters

Natural organic matter (NOM) is a complex mixture of organic material present in raw water sources. It can be divided into humic and nonhumic fractions. The humic fraction is characterized as more hydrophobic and includes humic and fulvic acids. The nonhumic fraction includes hydrophilic acids, proteins, lipids, amino acids, and hydrocarbons (Parsons, 2003; Owen *et al.*, 1995). NOM may originate from the degradation of organic residues of plants and animals by the action of microbes in soils of the watershed. Watersheds are those land areas that catch rain (or snow) and drain to specific streams, rivers, lakes, or to groundwater (USEPA, 2003). NOM may also derive from biota such as algae and bacteria present in the water body. (Aiken and Cotsaris, 1995; OMRI, 1997).

NOM exerts a role in the mobilization of hydrophobic organic species, metals and radionuclides, resulting in the migration of some contaminants that would otherwise be considered virtually immobile in water supply systems. It also reacts with disinfectants to form disinfection by-products (Aiken and Cotsaris, 1995). It may also be responsible for bacterial regrowth in water distribution systems, which is mainly related to the presence of biodegradable organic carbon (Owen *et al.,* 1995).

NOM is commonly characterized by surrogate parameters on the basis of

- its organic content: total organic carbon, or dissolved organic carbon,
- its ability to absorb UV light (at a wave length of 254 nm [UV-254]). This ability is mainly associated with aromatic structures present in humic substances (Parsons, 2003;USEPA, 1999; Owen *et al.*, 1995),
- or its potential to form trihalomethanes (THM-FP) (USEPA, 1999; Owen *et al.*, 1995; Standard Methods, 1992).

Total Organic Carbon (TOC)

Carbon present in water is divided into two main categories:

- Inorganic carbon (IC), referring to carbonate, bicarbonate, and dissolved carbon dioxide,
- Total organic carbon (TOC), referring to all carbon atoms covalently bonded to organic molecules. The dissolved portion of TOC is called dissolved organic carbon (DOC), defined as the fraction of TOC that passes through a 0.45-μm filter (Standard Methods, 1992; USEPA, 1999). In most natural waters, DOC represents almost 90% of the TOC (USEPA, 1999; Aiken and Cotsaris, 1995). As mentioned above, TOC and DOC are commonly used as surrogate parameters for the natural organic matter present in water. Other

fractions of TOC that are present in water are the volatile organic carbon (VOC), referred to as purgeable organic carbon removed from aqueous solutions by gas stripping under specified conditions, and the nonpurgeable organic carbon (NPOC), which is not removed by gas stripping.

Trihalomethane Formation Potential (THM-FP)

THM-FP is another surrogate parameter for NOM in raw water (Owen *et al.*, 1995; Standard Methods, 1992). It is determined by a standardized experiment that is performed on raw water, in which THM-FP represents the concentration of THMs in a sample of raw water buffered at pH 7.0, containing an excess free chlorine residual of 1 to 5 mg/L after being held 7 days at 25^{0}C (Standard Methods, 1992). Other THM-FP tests may be performed on raw water or distributed water to simulate certain conditions (Standard Methods, 1992). It is to be noted that there are other formation potential tests (set by sources other than *Standard Methods*, 1992) which are performed with shorter incubation times.

Total Trihalomethanes (TTHM)

Trihalomethanes are produced in water treatment systems as well as in distribution systems as a result of the reaction of chlorine with organic materials, especially dissolved natural organic matter, present in water. A detailed description of the trihalomethanes formation reactions is described later in this chapter. Trihalomethanes have the general form CHX_3, X being chlorine, bromine or iodine (Snoeyink and Jenkins 1980). The most common trihalomethanes are chloroform ($CHCl_3$), bromodichloromethane ($CHCl_2Br$), dibromochloromethane($CHClBr_2$), and bromoform ($CHBr_3$). The sum of these four species is called total trihalomethanes.

Trihalomethanes have been found to be carcinogenic. They may also cause liver, kidney, and central nervous system problems.

2.1.5 Microbiological Contaminants

Waterborne disease can result from microbiological organisms such as bacteria, protozoa and viruses. Some algae and helminthes (worms) may also cause disease. Symptoms of waterborne disease may include diarrhea, cramps, nausea, and jaundice.

Total Coliforms

Total coliforms are common bacteria present in the environment. Although they are not harmful in themselves, their presence in water is generally a result of a problem with water treatment or with the distribution system, and indicates that the water may be contaminated with potentially harmful organisms. Thereby, total coliforms are used as pollution indicators. They also indicate the probable occurrence of contamination with sewage and the probable presence of fecal coliforms. However, some protozoa such as Giardia and Cryptosporidium are able to withstand treatment which removes total coliforms, which makes total coliforms not a perfect sole indicator of the biological safety of water.

Fecal Coliforms

Fecal coliforms are bacteria that are associated with human or animal wastes. The presence of fecal coliforms in drinking water is a strong indication of contact with sewage or animal waste contamination. A commonly used indicator of fecal contamination is Escherichia coli (E. coli), which is a type of fecal coliform bacteria commonly found in the intestines of animals and humans.

2.2 Drinking Water Quality Standards

Table 2.1 presents standards for drinking water quality parameters of interest, as published by:

- The Egyptian Ministry of Health (1995),
- the United States Environmental Protection Agency (USEPA), July 2002,
- and the United Nations World Health Organization (WHO), 1998.

Table 2.1: Egyptian, WHO, and USEPA standards for drinking water quality parameters of interest

Analysis	Units	Egyptian Criteria	WHO Criteria	USEPA Criteria
pH	pH	6.5-9.2^{a}	6.5-8.5	6.5-8.5
TDS	mg/L	1,200	1,000	500
Turbidity	NTU	5	5	0.3 -1.0 ^{i}
Alkalinity	mg/L	-	-	-
Total Hardness	mg/L as $CaCO_3$	500	500	-
Calcium	mg/L	200	-	-
Magnesium	mg/L	150	-	-
Iron	mg/L	0.3	0.3	0.3
Lead	mg/L	0.05	0.01	0.015^{k}
Copper	mg/L	1.0	2.0 (P) 1.0^{c}	Action Level = 1.3^{k} NSDWRs = 1.0^{g}
Arsenic	mg/L	0.05	0.01	0.01
Chlorine residual	mg/L	-	0.6 – 1.0^{c,d} max = 5	MRDL = 4.0$^{h, L}$
Nitrate	mg/L	10 (as Nitrogen)	50 (as nitrate)	10 (as Nitrogen)
Fluoride	mg/L	0.8	1.5	MCL = 4.0 NSDWRs = 2.0^{g}
Total coliforms	No./100 mL	5%^{b}	Nil^{e}	5% positive/month ^{j}
Fecal coliforms	No./100 mL	Nil	Nil	Nil
TTHMs	µg/L	100	-^{f}	80

(NTU): Nephelolometric Turbidity Units.

a: pH should preferably not exceed 8.5.

b: Total coliform bacteria must not be present in 95% of samples taken throughout any 12-month period. No sample should contain more than 3 colonies/100 ml, and this number is not allowed to appear in two consecutive samples from the same source.

(P): Provisional guideline value. This term is used for constituents for which there is some evidence of a potential hazard but where the available information on health effects is limited; or where an uncertainty factor greater than 1000 has been used in the derivation of the tolerable daily intake (TDI).

c: Levels for substances in drinking-water that may give rise to complaints from consumers.

d: For effective disinfection, there should be a residual concentration of free chlorine of = 0.5 mg/L after at least 30 minutes contact time at pH <8.0

e: Total coliform bacteria must not be detectable in any 100-ml sample. In the case of large supplies, where sufficient samples are examined, it must not be present in 95% of samples taken throughout any 12-month period.

f: Trihalomethanes: The sum of the ratio of the concentration of each THM to its respective guideline value (shown in Table 2.2) should not exceed 1.0

Table 2.2: WHO guideline value for each of the TTHM species

THM species	Guideline value (ppb)
Bromoform	100
dibromochloromethane	100
bromodichloromethane	60
Chloroform	200

g: National Secondary Drinking Water Regulations (NSDWR) or secondary standards are non-enforceable guidelines regulating contaminants that may cause cosmetic effects (such as skin or tooth discoloration) or aesthetic effects (such as taste, odor, or color) in drinking water.

h: Maximum Residual Disinfectant Level (MRDL) - The highest level of a disinfectant allowed in drinking water.

i: At no time can turbidity (cloudiness of water) go above 5 NTU. Systems that filter must ensure that the turbidity go no higher than 1 NTU (0.3 NTU for conventional or direct filtration) in at least 95% of the daily samples in any month.

j: More than 5.0% samples total coliform-positive in a month. (For water systems that collect fewer than 40 routine samples per month, no more than one sample can be total coliform-positive per month). Every sample that has total coliform must be analyzed for either fecal coliforms or E. coli. If two consecutive TC-positive samples occur, and one is also positive for E.coli fecal coliforms, system has an acute MCL violation. For Community Water Systems, the number of samples to be analyzed is based on minimum service population.

k: Lead and copper are regulated by a treatment technique that requires systems to control the corrosiveness of their water. If more than 10% of tap water samples exceed the action level, water systems must take additional steps. For copper, the action level is 1.3 mg/L, and for lead it is 0.015 mg/L. However, National Secondary Drinking water Regulations have set the guideline value for copper as 1.0 mg/L

L: USEPA regulatory limit for minimum free chlorine residual in the point of entry to the distribution system (on a continuous basis) is 0.2 mg/L (Mays, 2000).

2.3 Drinking Water Quality Data

Table 2.3 presents a sample of water quality analysis in Alabama, U.S., 2001 (CCRs, 2002) and in Nepean supply system, Oct. 2002 to Sept. 2003 (Sydney Water, 2003). Table 2.3 indicates the large spectrum of chlorine concentration.

In Saudi Arabia, municipal water quality analysis showed no total coliform detection, with average values for TDS of 119 mg/L, pH 7.2, total hardness 186 mg/L, Ca^{+2} 40 mg/L, Mg^{+2} 19.7 mg/L, and fluoride levels of 0.6 mg/L (Abdel Magid, 1997).

In Hong Kong, THM analysis in tap water samples resulted in an average of 60.58 ppb TTHM, with the highest species detected being $CHCl_3$, followed by $CHCl_2Br$, $CHClBr_2$, and almost no detection of $CHBr_3$ (Yu and Cheng, 1999).

In Egypt, research conducted on Nile water in Greater Cairo showed levels of 15 NTU for turbidity, with pH of 7.8, TDS of 270 mg/L, total alkalinity of 112 mg/L as $CaCO_3$, and total hardness of 110 mg/L as $CaCO_3$ (El-Dib, and Ali, 1995). Lead concentrations were found to have an average of 29.6 ± 8.74 ppb in Nile water and 9.93 ± 0.5 ppb in the finished water at four treatment plants in Greater Cairo (Mohamed *et al.*, 1998). THMs evaluation in three water treatment plants in Greater Cairo showed levels of 19.05 to 52.5 ppb. An example of these analyses is presented in Table 2.4, indicating that the highest detection is for $CHCl_3$, followed by $CHCl_2Br$, $CHClBr_2$, and $CHBr_3$. THMs evaluation in five main districts in Alexandria indicated THMs speciation that is comparable to that in Cairo (Hassan *et al.*, 1996) (Table 2.5).

Table 2.3: Residential water quality data in Alabama, U.S. and Nepean supply system, Australia

Analysis	Units	Alabama	Nepean supply system
pH	pH	7.5	7.2 - 9.0 7.7 average
TDS	mg/L	126	
Turbidity	NTU	0.07 highest monthly average	0.09 - 3.15 0.2 average
TOC	mg/L	0.84 – 3.0 1.8 average	
Alkalinity	mg/L as $CaCO_3$	70	
Total Hardness		74.2	
Calcium	mg/L	24.3	
Mg	mg/L	3.29	
Iron	mg/L	ND	<0.005 - 1.01 0.025 average
Lead	mg/L	ND - 0.00439 ND 90th percentile	
Copper	mg/L	ND - 0.023 0.003 90th percentile	
Arsenic	mg/L	ND	
Total Chlorine residual	mg/L	2.6 - 3.5	Free chlorine 0.58 average Monochloramine 0.07 average
Nitrate	mg/L	ND	
Fluoride	mg/L	ND	0.9 - 1.2 1.1 average
Total coliforms	No./100 mL	ND	ND
Fecal coliforms	No./100 mL		ND
TTHMs	µg/L	62	34 - 144 70 average
$CHBr_3$	µg/L	ND	
$CHClBr_2$	µg/L	3.25 average	
$CHCl_2Br$	µg/L	7.92 average	
$CHCl_3$	µg/L	42.82 average	

Table 2.4: Mean values of THMs in water treatment plant effluent in Cairo (Dec. 1991; May, 1992; Feb. 1993) (El-Shahat *et al.*, 1998)

Date	WTP	THMs species (ppb)				
		$CHCl_3$	$CHCl_2Br$	$CHClBr_2$	$CHBr_3$	TTHM
Dec. 1991	Mostorod	31.70	6.70	2.20	ND	40.60
	Tebbin	33.30	6.70	2.30	ND	42.30
	Rod El-Farag	28.40	7.30	1.50	ND	37.20
May, 1992	Mostorod	21.1	20.50	7.40	ND	51.00
	Tebbin	18.40	14.80	7.15	ND	40.35
	Rod El-Farag	16.40	16.90	6.90	ND	40.20
Feb. 1993	Mostorod	11.54	7.80	3.88	ND	23.25
	Tebbin	12.80	9.10	4.30	3.30	29.50
	Rod El-Farag	12.20	10.10	3.60	3.20	32.10

Table 2.5: Mean values of THMs in five main districts in Alexandria, Egypt. (Hassan *et al.*, 1996)

Sampling Location	THMs species (ppb)				
	$CHCl_3$	$CHCl_2Br$	$CHClBr_2$	$CHBr_3$	TTHM
1	16.4	16.7	12	1.8	46.9
2	17	15	9.8	1	42.8
3	6.3	5.2	6.1	0.7	18.3
4	15.6	19.4	14.8	1.7	51.5
5	28.3	23.4	14.2	1.4	67.3

2.4 Water Quality Transformations in Distribution Systems

Several physical, chemical, and biological transformations occur in water as it flows through the distribution system. These transformations normally take place in two phases (Mays, 2000):

- Reactions in the bulk water phase (bulk reactions) that take place in the water volume that is not in contact with the pipe wall.
- Pipe wall reactions occurring between the water constituents and the materials originating from the pipe wall, such as released iron or biofilm slime made up of microorganisms growing on the inner surface of the pipe.

The processes responsible for these transformations are: (1) the decay of disinfection residuals; (2) the formation / increase in disinfection by-products; (3) internal corrosion; and, (4) the formation of biofilm (Mays, 2000). Disinfection residuals are subject to substantial losses (decay) in distribution systems (a detailed description of chlorine residual decay reactions is presented later in this chapter). Monitoring disinfectant residuals throughout the system enables utilities to understand the reasons for disinfectant loss and to design mitigating measures such as:

- using a more stable disinfectant (such as chloramines),
- undertaking a program of pipe replacement, flushing, or relining,

- decreasing residence time in the system by changing operational conditions such as storage and pumping,
- using booster chlorination,
- reducing the total organic carbon (TOC) in the water through changes in the treatment processes at the water treatment plants (Mays, 2000).

The disinfectant decay in distribution systems is often associated with the formation of disinfection by-products (DBPs), as will be discussed later in this chapter. The effect of the different disinfectants on the formation of DBPs has been studied by a number of researchers. It was noted that the use of chloramines as a disinfectant allowed a remarkable reduction in DBPs formation when compared with the use of chlorine. Chlorine dioxide was also found to reduce DBPs, however its principal by-products of concern are the inorganic species chlorite and chlorate. Ozone, which would be used only in primary disinfection, produces aldehydes, organic acids, bromoform and bromate. In addition, some ozonation by-products can encourage the biofilm growth in the distribution system (Mays, 2000; Lykins et al., 1994a).

Internal corrosion is the deterioration of the inside wall or wall lining of a pipe due to reactions with water flowing in the pipe. One of the impacts of corrosion is the leaching of toxic metals to water (lead, copper, *etc*). Corrosion is a function of the pipe material and lining, and the nature of water in the system. It is important to monitor metals concentrations in distribution systems, in order to be able to depict corrosion in its early stages. Mitigating measures suggested for corrosion control include:

- switching to pipe materials that are better resistant to corrosion,
- adjusting pH and alkalinity,
- applying a protective lining to the pipe (Mays, 2000).

Zones of the distribution system that are characterized by low water velocity or corrosion represent an environment that encourages the growth of microorganisms, forming a biofilm at the pipe walls (Mays, 2000). Biofilm growth in the distribution system has several potentially negative impacts. In particular, it provides a nutrient rich environment for pathogens and it can serve as site for the accumulation of chlorine demanding materials that contribute to the decay of residual chlorine and the formation of DBPs.

Mitigation measures for controlling biofilm formation include replacing old pipe sections and proper adjustment of water chemistry parameters such as pH and alkalinity so as to inhibit corrosion (Mays, 2000).

2.5 Drinking Water Disinfection

Disinfectants/oxidants are used in drinking water treatment in order to achieve various targets including:

- Enhance coagulation and filtration of treated water. This is mainly achieved by the oxidation of pathogens into more polar forms, the oxidation of metals ions into insoluble complexes, or by changing the structure and size of suspended particles (USEPA, 1999).
- Meet water quality biological criteria. For instance, in the U.S., water treatment processes, including disinfection, are required to achieve at least a 3-log (99.9%) removal and/or inactivation of Giardia cysts, and a 4-log (99.99%) removal and/or inactivation of viruses, before water is delivered to the first consumer (Singer, 1994; USEPA, 1990).
- Oxidation of specific compounds (*i.e.*, taste and odor causing compounds, in addition to iron, and manganese).
- Protect drinking water from being polluted in the distribution systems via line breaks, cross-connections, or unforeseen occurrences, *i.e.* provide

residual protection of the distribution system against the regrowth of microorganisms (USEPA, 1999; Singer, 1994).

Disinfectants may be applied in three steps: (1) Primary disinfection, which is the application of a disinfectant during treatment processes, to destroy or inactivate pathogens and enhance coagulation and filtration processes; (2) secondary disinfection (or post-disinfection), which refers to the application of a disinfectant to the treated water before its delivery to the distribution system, to maintain disinfectant residuals in the system; however the application of chlorine or chloramines in primary disinfection may sometimes be enough to provide the needed disinfectant residual with no need for secondary disinfection; and (3) booster disinfection, which refers to the addition of a disinfectant in selected points of the distribution system. Booster disinfection can be used to control the formation of DBPs, by optimizing the location of the point of disinfection to achieve full microbial inactivation with less DBPs formation (Mays, 2000; Clark, 1994).

The most commonly used disinfectants/oxidants are chlorine, chlorine dioxide, chloramines (formed by the reaction of chlorine with ammonia as will be shown later in this chapter), ozone, and potassium permanganate (USEPA, 1999). Chlorine is the most widely used disinfectant (Kastl *et al.*, 2002; Abd-El-Shafy and Grünwald, 2000; USEPA, 1999; Clark *et al.*, 1998). Table 2.6 presents the usage of the above listed disinfectants in both primary and secondary disinfection of surface water systems in the U.S. (USEPA, 1999). It is to be noted that the sum of disinfectant usage in Table 2.6 is less than 100%, as there are other disinfection practices which are not listed in Table 2.6. Table 2.7 presents the advantages, disadvantages, and major uses of various disinfectants.

Table 2.6: Disinfectant usage in surface water systems in the U.S.

Disinfectant	Primary disinfection	Post-disinfection
Chlorine	63.8%	67.5%
Chloramines	3.1%	8.1%
Chlorine dioxide	6.3%	1.6%
Ozone	0.9%	
Potassium permanganate	16.0%	

Table 2.7: Advantages, disadvantages, and major uses of various disinfectants.

Disinfectant	Advantages	Disadvantages	Major use
Chlorine[a]	- Generally very effective in the removal of most bacteria and viruses	- Formation of high THMs. - weak removal of Cryptosporidium.	Primary, secondary, and booster disinfection
Chloramines [b]	- Less reactive with organics; less THMs formation - stable monochloramine residual (compared to free chlorine or chlorine dioxide), i.e. better protection against bacterial regrowth. - monochloramine residual has higher capacity to penetrate and control biofilm.	- Relatively weak disinfectants for virus and protozoa inactivation. - Increased production of cyanogen chloride. - Chloramines will not oxidize iron and manganese.	Secondary disinfection (due to its low efficiency with virus and protozoa, and its high stability in distribution systems)
Chlorine [c] dioxide	Effective for removal of Cryptosporidium	Formation of chlorite and chlorate ions	Primary disinfection (lower DBPs occur when ClO_2 is mixed with chlorine)
Ozone[d]	Highest effectiveness for removal of Cryptosporidium	Formation of aldehydes, organic acids, bromoform and bromate (even at trace levels of bromide ion)	Primary disinfection
Potassium permanganate	- Oxidation of iron and manganese; - oxidation of taste and odor compound; - control of biofilm; - reduction of THM and HAA formation.	It is undesirable to maintain a residual of $KMnO_4$ as it gives water a pink color.	Primary disinfection.

a: Rittman and Tarquin, 2002; USEPA, 2001.

b: Basson et al., 2002; Schajnoha, 2002; USEPA, 1999.

c: Rittman and Tarquin, 2002; USEPA, 1999.

d: Rittman and Tarquin, 2002;Mays, 2000; USEPA, 1999; Lykins et al., 1994.

e: USEPA, 1999; Vella and Ficek, 2002

2.5.1 Kinetics of Disinfection

The kinetics of disinfection of microorganisms can be described by a first-order model developed by Chick (1908) and modified by Watson (1908), as follows:

$$\ln(N/N_o) = -\alpha C^n t,$$

where N is the number of microorganisms at time t, N_0 is the number of microorganisms at time zero, C is the concentration of disinfectant, t is the contact time, n is a dilution coefficient (that is commonly assumed to be 1, although it should be experimentally verified), and α is an inactivation constant (Droste, 1997; Clark, 1995). The general principle in this law is that inactivation is increased with the increase in disinfectant concentration C and/or contact time t.

A control parameter that is frequently used for disinfection is the (C x t) value (the product of disinfectant concentration (mg/L) and contact time (min) measured at peak hourly flow). Contact time is measured from the point of disinfectant application to the first consumer. The (C x t) value depends on the effectiveness of the disinfectants to remove different types of organisms. For example, chlorine is considered to have relatively low effectiveness against some protozoa, such as Cryptosporidium, but is generally very effective in the removal of most bacteria and viruses (Droste, 1997; USEPA, 2001).

2.5.2 Disinfectant Residuals in Water Entering the Distribution Systems

To keep the water biologically safe in the distribution systems, residual disinfectant doses need to be maintained at a certain level until water reaches the consumers. Characklis (1981) stated that a minimum of 0.5 mg/L free chlorine residual concentration needed to be maintained to prevent the formation of biofilm (Abd-El-Sahfy and Grűnwald, 2000). The U.S. regulatory limit for minimum free chlorine residual in the point of entry to the distribution system (on a continuous basis) is 0.2 mg/L (Mays, 2000). The maximum level for chlorine residual was set by the USEPA to a value of 4 mg/L (USEPA, 2002). WHO regulatory limits are 0.5 mg/L for the minimum free chlorine residual and 5 mg/L as maximum chlorine residual (WHO, 1998). In Egypt, there are no published limits for chlorine residual concentration; however evaluation of chlorine residual is based on the USEPA regulatory limits.

2.6 Chlorine Reactions in Water

Chlorine is a relatively unstable chemical, which reacts with various organic and inorganic compounds (Abd El-Shafy and Grűnwald, 2000), causing the consumption (decay) of chlorine residuals and reducing chlorine disinfecting capability.

2.6.1 Reactions of Chlorine With Natural Organic Matter (NOM)

The hydrolysis of chlorine in water yields the formation of hypochlorous acid which, in turn, reacts with the components that make up the chlorine demand to from halogenated DBPs. These two processes are given by the following reactions.

$$Cl_2 + H_2O \rightarrow HOCl + H^+ + Cl^-$$

$$\mathrm{HOCl} + \mathrm{Cl}_{\mathrm{demand}} \rightarrow \mathrm{DBPs}$$

HOCl dissociates into OCl^- and H^+, depending on the pH of water. At pH 5.0 and below, almost all chlorine is in the form of HOCl. At pH 10.0 and above, almost all chlorine is in the form of OCl^-. HOCl disinfection abilities are about 80-200 times as much as OCl^- (Droste, 1997); that is why it is recommended to perform disinfection at pH below 8.0 (WHO, 1998). For most waters, the reactions of chlorine with NOM make up the majority of the chlorine demand, (Clark *et al.* 1998).

The presence of the bromide ion in source water yields the formation of brominated DBPs. This is due to the oxidation of Br- into hypobromous (HOBr) acid by the action of dissolved chlorine. HOBr then reacts with the NOM to form brominated DBPs (Singer, 1994).

$$\mathrm{HOCl} + \mathrm{Br}^- + \mathrm{NOM} \rightarrow \text{brominated DBPs}$$

Halogenated organic by-products are divided into several classifications of compounds.

- *Trihalomethanes* (TTHM): Chloroform, Bromodichloromethane, Dibromochloromethane, and Bromoform.
- *Haloacetic acids** (HAA9): Monochloroacetic acid, Dichloroacetic acid, Trichloroacetic acid, Monobromoacetic acid, Dibromoacetic acid, Tribromoacetic acid, Bromochloroacetic acid, Bromodichloroacetic acid, and Dibromochloroacetic acid.
- *Haloacetonitriles:* Dichloroacetronitrile, Bromochloroacetonitrile, Dibromoacetonitrile, Trichloroacetonitrile, and Tribromoacetonitrile.
- *Haloketones:* 1,1-Dichloropropanone, 1,1,1-Trichloropropanone
- *Chlorophenols:* 2-Chlorophenol, 2,4-Dichlorophenol, 2,4,6-Trichlorophenol
- *Halopicrins:* Chloropicrin and Bromopicrin.
- *Chloral Hydrate*

- *Cyanogen Halides:* Cyanogen chloride and Cyanogen bromide.
- *N-Organochloramines*
- *MX* (3-Chloro-4-(dichloromethyl)-5-hydroxy-2(5 H)-furanone)

*: Although reported as acids, these species are actually present in water as the deprotonated anions.

(USEPA, 1999; Singer, 1994)

TTHMs may constitute as little as 20 to 40% of the total organic halogenated by-products of chlorination (Mays, 2000; Abd El-Shafy and Grünwald, 2000). Recent research has shown that the concentrations of HAA9 (sum of the nine HAA species) in water are almost equal to TTHM (sum of the four THM species) (Singer, 2002). In Alabama (2001), TTHM average was 62 ppb while total HAA was 50 ppb (CCR, 2002). The ratio of HAA to TTHM depends on water alkalinity and pH. Low alkalinity water chlorinated under moderately acidic pH conditions (pH 6 to 6.5) tends to have higher HAA to THM ratios (Singer, 2002).

2.6.2 Reactions of Chlorine With Inorganic Compounds

Chlorine reacts with various inorganic compounds, exerting a portion of the chlorine demand in water.

Oxidation of Iron and Manganese

Chlorine oxidizes soluble ferrous (2+) iron into insoluble ferric (3+) and soluble manganese (2+) into the insoluble (4+) form (Droste, 1997). The precise chemical composition of the precipitate depends on the nature of the water, temperature, and pH.

Chlorine Reaction With Ammonia:

Hypochlorous acid resulting from the hydrolysis of chlorine reacts with ammonia to produce different species of chloramines (referred to as combined chlorine residuals) such as:

$NH_3 + HOCl \rightarrow NH_2Cl + H_2O$ (monochloramine)

$NH_2Cl + HOCl \rightarrow NHCl_2 + H_2O$ (dichloramine)

$NHCl_2 + HOCl \rightarrow NCl_3 + H_2O$ (nitrogen trichloride or trichloramine)

The distribution of the three chloramines species is a function of pH, (Droste, 1997; USEPA, 1999). As described previously, chloramination provides good and stable residual protection against bacterial regrowth in the water system, while decreasing the formation of THMs. THMs were found to be more largely formed by free chlorine than by chloramines (Abd El-Shafy and Grűnwald, 2000; Villanova et al., 1997).

2.7 Chlorine Decay in Water Distribution Systems

Chlorine decay in water distribution systems is dominated by the following concepts and reactions:

- Chlorine reactions with organic and inorganic compounds in bulk water.
- Chlorine reactions with biofilm at the pipe wall (Clark, 1998).

  It was noticed that smaller-diameter pipes had higher chlorine consumption than larger-diameter pipes. This can be justified by the fact that smaller-diameter pipes offer a larger wall surface area per unit of flow volume for reaction to occur (Clark *et al.*, 1995).
- Chlorine consumption by the corrosion processes.

  A study made by Sharp et al. (1991) showed that unlined cast-iron pipes have higher chlorine consumption than polyvinyl chloride (PVC) pipes, (Clark *et al.*, 1995).

- Mass transfer of chlorine between the bulk phase and the pipe wall (Clark, 1998).

These concepts represent the chlorine decay due to bulk and wall phases. Both bulk and wall decay reactions depend on the nature of the water (organic/inorganic constituents) and distribution system (pipe material, diameter, length, *etc*), and hydraulic conditions (velocity, residence time). Detailed description of chlorine decay reactions in bulk and wall phases will be discussed later in this chapter.

2.8 Factors Affecting The Formation of DBPs

NOM Type and Level

Water sources vary widely in their NOM concentrations. The type of organic precursors and their concentrations affect the yield of DBPs. For example, high levels of DBPs are associated with high levels of aromatic carbon content (Aiken and Cotsaris, 1995). As mentioned previously, NOM is commonly characterized by surrogate parameters such as TOC, DOC, UV-254, and THM-FP. For example, in a study of different approaches for the removal of NOM, surrogates used for NOM were DOC (resulting of a 1.0-μm filtration followed by a 0.22-μm filtration and measured using a TOC instrument); UV absorbance (measured with a UV-visible spectrophotometer with a 1-cm quartz cell at a single wavelength of 254nm); and THM-FP (based on Cl_2-to-DOC=3:1 mg/mg, 96 h incubation time, pH 7, and 20°C) (Owen *et al.*, 1995). In general, a higher available NOM will result in the formation of more DBPs if enough residual free chlorine is available (Villanova *et al.*, 1997).

Chlorine Doses

Several studies have shown that the formation of DBPs increases with the increase in chlorine concentration (USEPA, 2001). Low to moderate correlation between TTHM formation and prechlorination dose was obtained by Villanova *et al.* (1997), and Abdullah and bin Ramli (2003).

Reaction (or Residence) Time

A longer reaction time (in pipes or storage tanks) generally increases both chlorine demand and DBPs formation, as more time is given to bulk and wall reactions to occur in the presence of available chlorine residual, in addition to higher formation of biofilm associated with low velocities and long residence times (Mays, 2000). In a study by Abd El-Shafy and Grünwald (2000), the relation between TTHM formation and the residence time was exponential with a correlation coefficient of $R^2 = 0.91$, as the formation of TTHM was expressed by the first order equation

$$TTHM_t = TTHM_o e^{kt} \qquad (2\text{-}1)$$

where $TTHM_t$ = TTHM concentration at time t, $TTHM_o$ = the initial TTHM concentration, k = the overall formation rate constant in $time^{-1}$, and t = residence time.

pH

For chlorine reactions, pH variations have been found to have little effect on chlorine demand. However, the sum of HAA6 and HAA9 (the sum of six or nine of the [HAA] species) was found to decrease with the increase in pH (Singer, 2002; USEPA, 2001). On the other hand, it was found that TTHMs increase when pH increases (Singer, 2002; Abd El-Shafy and Grünwald, 2000; Clark *et al.*, 1995).

Temperature

Both chlorine demand and DBPs increase when temperature increases, due to the increase in the reaction rate constant (Kastl et al., 2002; USEPA, 2001). However, it was found that a sharp decrease in chloroform concentration occurs when temperature increases over 17.3°C, which may be due to the volatility of THMs (Abd El-Shafy and Grünwald, 2000; Villanova *et al.*, 1997).

Bromide

The increase in bromide ion concentration increases the formation of brominated compounds (Singer, 1994). Examples of the occurrence of brominated THMs ($CHCl_2Br$, $CHClBr_2$ and $CHBr_3$) were presented in Section 2.3. THM analyses in Cairo (Mohamed *et al.*, 1998) and Alexandria, Egypt (Hassan *et al.*, 1996) showed that the occurrence of $CHCl_3$ was highest among THM species, followed by $CHCl_2Br$, $CHClBr_2$, and $CHBr_3$.

Organic Nitrogen Concentration

Organic nitrogen reacts with chlorine to form nitrogen containing DBPs such as the haloacetonitriles, halopicrins, and cyanogen halides (USEPA, 1999).

2.9 Modeling the Decay of Chlorine Residuals and TTHM Formation

A number of researchers have investigated the modeling of chlorine decay and TTHM formation in drinking water. In one of the earliest studies, the following equation was used to describe chlorine decay (Feben and Taras, 1951)

$$D_t = D_1 \, t^n \qquad (2\text{-}2)$$

where D_t = the chlorine consumed at time t (hr), D_1 = the chlorine consumed after 1 hour, and the constant n is characteristic of the source water. D_1 and n are to be determined on the basis of laboratory experiments using the actual source water (Clark and Sivaganesan., 2002).

Qualls and Johnson (1983) developed a model that described the short-term reaction of free chlorine with fulvic acids during the first 5 minutes. This model was originally developed for cooling water systems, but was further applied to disinfection of natural waters. They described the chlorine decay by the sum of two first order equations:

$$-dC/dt = k_1[Cl][F_1] + k_2[Cl][F_2] \qquad (2\text{-}3)$$

where $k_1[Cl][F_1]$ represents the rapid decay within the first 30 seconds,

$k_2[Cl][F_2]$ represents a slower decay from 30 seconds to 5 minutes,

$[Cl]$ is the free residual chlorine,

k_1 and k_2 are rate constants for the fast and slow reactions respectively,

and $[F_1]$ and $[F_2]$ are the concentrations of reactive species of the fulvic acids for the fast and slow reactions respectively (USEPA, 2001).

Trussell and Umphers (1978) described chlorine decay rate by a first order equation as follows (USEPA, 2001):

$$d[Cl_2]/dt = -k_1[Cl_2][C] \qquad (2\text{-}4)$$

where k_1 is the rate constant for the reaction between free chlorine and organic matter, and C is the concentration of organic matter. The formation of TTHMs was considered as first order with respect to chlorine residual and m^{th} order with respect to organic matter as follows:

$$d[TTHM]/dt = k_2[Cl_2][C]^m \qquad (2\text{-}5)$$

where m = the order of the reaction with respect to the organic matter concentration, and k_2 is the overall rate constant for the reaction of the formation of TTHMs (USEPA, 2001).

Lamia *et al.* (2002), investigated three models for chlorine decay:

- First order model,
- Second order model,
- Parallel first-order model,

They concluded that the parallel first-order decay model equation gave the best results. In this model, two types of agents are assumed to be responsible for chlorine decay. These agents are the fast reducing agents (exerting initial decay) and the slow reducing agents (exerting long – term decay) (Lamia *et al.*, 2002). The parallel first-order decay model was also used by Kastl *et al.* (2002) for prediction of the chlorine concentration profile in bulk drinking water distribution systems. Their application of the model will be discussed later in this chapter.

The first-order decay model was applied by the USEPA in a number of projects (Clark, 1988). Chlorine decay was described by the following reaction:

$$C = C_o e^{-kt} \tag{2-6}$$

where C = the chlorine concentration at time t, C_o = the initial chlorine concentration, and k = the decay rate in time^{-1}. Case studies of the application of this model will be discussed later in this chapter. The first-order decay model has demonstrated good success in describing chlorine decay. It has the advantage of being easily implemented using simple laboratory experiments.

Clark (1998) developed his model for chlorine decay on the basis of the first-order decay approach. He expressed the reaction of HOCl with the components that make up the chlorine demand to form chlorination by-products:

$$\mathrm{HOCl} + \mathrm{Cl}_{\mathrm{demand}} \rightarrow \mathrm{DBPs}$$

by the following balanced equation:

$$aA + bB \rightarrow pP$$

where A and B = the reacting substances, P = the product of the reaction and a, b and p are the stoichiometric coefficients for A, B and P, respectively. The decay rate was modeled as first order with respect to individual components:

$$dC_A/dt = -k_A C_A C_B \tag{2-7}$$

$$dC_B/dt = -k_B C_A C_B \tag{2-8}$$

$$dC_P/dt = k_P C_A C_B \tag{2-9}$$

where k_A, k_B, and k_P are reaction rate constants, and $k_A/a = k_B/b = k_P/p$. The mathematic elaboration of these equations yielded the following equation for free chlorine concentration at time t:

$$C_A = \frac{C_{A0}(1-K)}{1-Ke^{-ut}} \tag{2-10}$$

where C_A = concentration of free chlorine in mg/L at time t, C_{Ao} = initial free chorine concentration in mg/L, u = the reaction rate in time^{-1}, and K = dimensionless parameter. Clark then expressed the reaction rate u by the following equation:

$$u = M\ (1-K) \tag{2-11}$$

where

$$M = k_A b C_{Ao}/a \tag{2-12}$$

and $M > 0$. He also developed an equation for THM formation, expressed as:

$$TTHM = D(C_{A0} - (\frac{C_{A0}(1-K)}{1-Ke^{-ut}})) \tag{2-13}$$

where D = dimensionless parameter. The parameters M, K, and D were estimated for various data sets generated from bench scale chlorination experiments using humic acids in solution at various levels of:

- Chlorine doses (mg/L)
- TOC (mg/L)

- pH
- Temperature (°C)
- Reaction times

The model developed by Clark (1998) can be used in other case studies if M, K, and D are estimated for the specific water quality data as noted above.

2.10 Water Quality Modeling in Distribution Systems

2.10.1 The Development and Use of Hydraulic Models

Hardy Cross (1936) proposed the use of mathematical methods for flow analyses in water networks. These methods were then computerized in the 1950s and 1960s, and became widely available in the 1970s and 1980s (Mays, 2000). Steady state and time varying operating conditions were incorporated in the hydraulic models, which were referred to as extended-period simulation (EPS) models. Extended-period simulation allows the modeler to capture the effects that variations in customer demands and tank water levels have on system performance (Mays, 2000).

Hydraulic modeling of water networks is required to provide flow information essential to perform successful water quality modeling. Hydraulic and water quality modeling can either be performed simultaneously or consecutively (Mays, 2000).

2.10.2 Steady State Water Quality Models

Steady state water quality modeling was suggested by Wood (1980) to investigate the spatial pattern of water quality in a distribution system resulting from sources of different quality. He made an extension to a steady state hydraulic model to incorporate and solve a set of simultaneous equations at each node in the network (*i.e.* where two or more pipes meet). His attempts were then elaborated by other researchers. Although steady state approaches provided useful tools for the modeling of water quality in distribution systems, it was recognized through case studies that dynamic models could do a better job of incorporating chemical kinetics and the impacts of associated parameters on chemical transformations in the distribution system (Mays, 2000; Clark, 1992).

2.10.3 Dynamic Water Quality Model

The USEPA and the North Penn Water Authority (NPWA) in Pennsylvania (USA) conducted a project aiming to investigate the feasibility of developing water quality and chlorine decay models (USEPA, 2001). A steady-state flow-tracing model was developed to predict travel time estimates for water from the source to any point in the system, and the contribution of flow from a given source (in a multi-source system) to any node in the system (Clark et al., 1991; Clark, 1992)

The Dynamic Water Quality Model (DWQM) was developed to provide both time and spatial predictions of water quality. This model makes use of "external" hydraulic models, such as the model developed by the U.S. Army Corps of Engineers which contains provisions for both steady-state and extended period hydraulic simulation (Clark et al., 1991; Clark, 1992). Modeling conditions were changed sequentially by time period, *i.e.* during a certain time period, all external forces affecting water quality were assumed to remain constant (*e.g.*, demand, well pumpage, tank head). The model was calibrated and validated on the basis of field data from the local system (Clark *et al.*, 1991; Clark, 1992; USEPA, 2001; Mays, 2000).

Projects were conducted in other areas in order to validate the concepts and models developed in the North Penn Study, especially the DWQM. In one of these projects, a sampling program was carried out in 1991, for a community in Connecticut, USA, which received its water from a freshwater lake (Mays, 2000). Water was treated with chlorination, filtration, and the addition of a phosphate–corrosion inhibitor. A pump station was used to pump the water from the treatment facility to the service area. Storage was provided in an overhead tank. The DWQM was applied and validated within the service area. The model was run for different operating conditions where the pumps lifting water from the treatment facility were turned on and off. Fluoride was used as a conservative tracer for the movement of flow through the system. It was noticed that, when the pumps were on, the fluoridated water was pumped into the system. When the pumps were off (the system was fed from the tank), the system received water that had reached an equilibrium concentration of fluoride, which illustrated the importance of studying the effect of storage tanks and pumping conditions on the water quality. Chlorine residuals were also monitored at different sites in the system. A chlorine decay rate was introduced to the model in order to estimate chlorine residual concentrations and to compare the model results to the actual concentrations. Chlorine decay was assumed to follow the first order reaction $C = C_0e^{-kt}$. A bench scale study was conducted in which chlorine decay rate constant for this water was calculated to be 0.55 day^{-1}. This decay rate constant represented the chlorine decay in the bulk water. When this bulk decay rate constant was used in the model calculations, it was found to under-estimate the chlorine demand. The actual system demand was two to three times higher than that estimated by the bulk rate alone. This difference was assumed to be due to the effect of the pipe wall demand, in particular biofilm growth and points of corrosion.

This case study shows the importance of capturing the effects of water storage and pumping on different water quality parameters. It also reinforces the idea that chlorine decay in distribution systems needs to be modeled as the combination of decay in both bulk and wall phases.

2.10.4 EPANET Modeling Software

EPANET was developed by the Water Supply and Water Resources Division (WSWRD) (formerly the Drinking Water Research Division) of the U.S. Environmental Protection Agency's National Risk Management Research Laboratory (EPANET, 2002). The program was introduced in 1992 (USEPA, 2001). It represents a third generation of water quality models developed by the WSWRD that has the ability of performing dynamic water quality simulations in complex pipe networks. EPANET is based on the extended-period simulation approach to solving hydraulic behavior of a network and incorporating principles of mass transfer to model water quality. The important assumptions with regards to mass transfer include:

- Mass conservation within differential lengths of pipe.
- Water entering a pipe junction is completely and instantaneously mixed.
- Appropriate kinetic expressions describing the decay or formation of the substance are used (Mays, 2000)

This model has the advantage of handling systems of any size. Friction head loss in pipelines is computed using the Hazen-Williams, Darcy-Weisbach, or Chezy-Manning formulas. Minor head losses for bends, fittings, etc, are also included. Constant and variable speed pumps, various types of valves, and storage tanks can be modeled using EPANET. The program allows for time-varying concentration or mass inputs at any location in the network. Other features of the model that render it useful for water quality analysis are that it can:

- Model the dynamic movement and fate of a reactive material as it grows (e.g., a disinfection by-product) or decays (e.g., chlorine residual).
- Model the dynamic movement of a non-reactive tracer through the distribution system.
- Model the age of water as it moves through the system. This can help to overcome inaccuracies in the description of the kinetics of water quality processes (Kastl et el., 2001).
- Model reactions both in the bulk flow and at the pipe wall.
- Track the percent of flow from a given node going to all other nodes over time.
- Model storage tanks as being either completely mixed, plug flow, or two-compartment reactors (EPANET, 2002).

Modeling Chlorine Decay and TTHM Formation Using EPANET

The chlorine decay model was introduced into the EPANET program by Rossman *et al.*, 1994. The model is mass-transfer based and it considers first order decay reactions of free chlorine in bulk flow and at the pipe wall, as represented by the following equations (Clark, 1995; EPANET, 2002).

$$\frac{dC}{dt} = -k_b C - \frac{k_w k_f C}{r_h (k_w + k_f)} \tag{2-14}$$

or
$$k_{ov} = k_b + \frac{k_w k_f}{r_h (k_w + k_f)} \tag{2-15}$$

where C is the concentration of free chlorine (mg/L), k_b is the bulk reaction rate constant determined independently from laboratory experiments (day^{-1}), k_w is the wall reaction rate constant obtained by trial and error in the EPANET model (m/day), k_f is the mass transfer coefficient between the bulk and wall phases (m/day), k_{ov} is the overall reaction rate constant (day^{-1}), r_h is the hydraulic radius = pipe radius/2 (m),

$k_f = Sh(D/d)$

Sh = Sherwood number (dimensionless) = $0.0149\ R^{0.88}Sc^{1/3}$ for $R \geq 2300$

$$Sh = 3.65 + \frac{0.0668(d/L)R(Sc)}{1 + 0.04[(d/L)R(Sc)]^{2/3}}, \text{ for } R < 2300,$$

L = pipe length, (m); d= pipe diameter (m)

R = Reynolds number = Ud/v,

U = flow velocity (m/day),

v = kinematic viscosity of water = 0.09 m^2/day (Clark 1995; Droste 1997)

Sc = Schmidt number (dimensionless) = v/D

D = molecular diffusivity in water at 20°C (m^2/day).

As shown in the above equations, the overall decay rate constant (k_{ov}) in this model is a function of:

- The bulk decay rate constant.
- The wall decay rate constant.
- The mass transfer coefficient.
- The molecular diffusivity in water.
- The water's kinematic viscosity.
- The pipe radius (Clark, 1995)

In order to model substances other than chlorine using EPANET, the user needs to enter appropriate values for the relative diffusivity of the substance with respect to chlorine, and the bulk reaction rate constant of the specific substance (for the first order kinetics model).

Validation of the model using data from various case studies showed that model predictions were most accurate at locations where the hydraulic conditions were well characterized, which highlights the importance of having accurate hydraulic information before running the model (USEPA, 2001).

As mentioned above, the bulk reaction rate constant is determined independently in the laboratory. The wall reaction rate constant can be varied within a range that is influenced by reaction rate-limiting and mass transfer rate-limiting conditions. In addition to their being case-specific, reaction rate constants are temperature dependent. In some studies, they were doubled for every 10°C rise in temperature (Rossman, 1997). Typical values for bulk chlorine decay rate constant are in the range of 0.1 to 1.5 (day^{-1}) (Rossman, 1998; Hart, 2002); however, values of 0.08 and 17.7 day^{-1} were reported in some case studies (Rossman, 1998). Typical values for wall chlorine decay rate constant are in the range of 0.08 m/day to 0.31 m/day (Rossman, 1998). The overall chlorine decay rate depends on the nature of the system and especially pipe material; in systems including unlined iron pipes, it was reported to be about an order of magnitude higher than bulk decay rate constant (Rossman et al. 2001; Clark, 1995).

THM growth can be modeled using first-order saturation kinetics, on the basis of the limiting concentration (maximum THM level) that the water can produce, given a long enough holding time. This is expressed by the following equation (EPANET 2002):

$$\frac{dTTHM}{dt} = k_b(C_m - C) \tag{2-16}$$

where C_m = ultimate formation of TTHM. K_b is the slope of the plot of $\ln\frac{(C_m - Co)}{(C_m - Ct)}$ versus time, where C_t is concentration after time t and Co is concentration at time 0. Typical values for bulk TTHM formation rate constant are in the range of 0.5 to 1.5 day^{-1} (Rossman, 2003).

EPANET models the wall reaction as single-species first order decay/formation, only incorporating mass transfer between bulk and wall phases, while neglecting the TTHM formation due to reaction of chlorine with organic material attached to pipe walls, which may be about 15% of the total THM formation in the system (Rossman *et al.*, 2001). This presents one of the deficiencies of EPANET 2.0 in modeling THM formation (EPANET, 2002).

2.10.5 Case Study - U.S.A.

A study was conducted by the USEPA in collaboration with the North Marin water district in California to investigate the various factors affecting chlorine decay and TTHM formation in the district water distribution system (Clark *et al.*, 1994). The district derives its water from two sources: Stafford Lake and the North Marin Aqueduct. The aqueduct is a year – round source, but the Stafford Lake is used only in warm summer months. Stafford Lake water had a high humic acid content (high organic matter) and was subject to conventional treatment and high prechlorination doses (from 5.5 to 6.0 mg/L), producing a residual chlorine of 0.5 mg/L in the treated water entering the distribution system. The potential for formation of THMs in Stafford Lake was high. The North Marin Aqueduct had low organic matter and hence low THM formation potential. It also delivered water with 0.5 mg/L residual chlorine to the distribution system.

To predict the chlorine demand at the various sampling points in the system, a first order decay relationship was assumed. EPANET was used to simulate the system hydraulics and transport of the chlorine residual. Based on bulk demand calculations, the first order decay coefficients or bulk demand for the Stafford Lake and the North Marin Aqueduct were found to be 0.31 and 0.03 day^{-1} respectively. The system was simulated using the bulk demand for the two sources and trial and error was used to estimate the wall demand for four sections of the network, based on actual chlorine residuals. High values of k_w were assumed for old pipes in the network as well as for sections that were totally served by the worst water source (Stafford Lake). The calibration resulted in a wide range of wall reaction constants (from 0.3 m/day to 6 m/day) (Clark et al. 1995).

2.10.6 Case Study - Czech Republic

A research was conducted on the water distribution system in South Bohemia, in the Czech Republic (Abd-El-Shafy and Grűnwald, 2000). The water was derived from the Rimov surface water reservoir and treated with coagulation, flocculation, sedimentation, filtration, and disinfection (post-chlorination). The treated water was carried to the consumers in steel pipelines of 80 km lengths without any type of coating. Chlorine dose added during disinfection was about 1.25 mg/L and reached 0.75 mg/L while leaving the treatment plant. This concentration continued to decrease along the distribution system until it reached zero at the end of the pipelines.

The chlorine consumption in the system was assumed to follow the first order decay equation given previously (Equation (2.6)):

$$C_t = C_o e^{-k_t t}$$

Total chlorine concentrations were measured in samples taken throughout the system. These measurements were used to calculate the overall chlorine decay constant, k_t, which had a mean value of 0.3 day^{-1}. This decay rate was used to calibrate the water quality model in the EPANET program. The overall chlorine decay constant (k_t) was divided to a water bulk chlorine decay constant (k_b = 0.1 day^{-1}) and a wall chlorine decay constant (k_w = 0.35 m/day). The correlation between the predicted and measured concentrations showed the success of the model calibration.

As for the TTHMs, it was found that the treatment plant contributed with only 45% of the TTHMs found at the end of the pipelines, while the rest was formed in the pipelines. The TTHM formation in the distribution system was expressed by a first order equation (Equation (2.1) given previously)

$$TTHM_t = TTHM_o e^{kt}$$

where $TTHM_t$ = TTHM concentration at time t, $TTHM_o$ = the initial TTHM concentration (*i.e.*, the water leaving the treatment plant), and k = the coefficient of first order increase which was determined to be 0.08 day^{-1}.

A linear relationship was found between TTHM concentration at any place in the pipeline (μg/L) and the cumulative decay of chlorine residual (ΣDCl) from the beginning of the pipeline to this place (mg/L):

$$TTHM = A + B * \Sigma DCl \qquad (2\text{-}17)$$

where A and B are constant coefficients that are calibrated according to TTHM formed in the treatment plant (before entry to the distribution system) and other water characteristics. In this case study, it was found that A = 4.25 μg/L and B = 10.31μg/mg, with the degree of correlation given by $R^2 = 0.913$.

2.10.7 Case Study – Australia

A research project was conducted by Kastl, Fisher, Jegatheesan, Chandy and Clarkson (2002) in order to predict chlorine and THM concentration profiles in bulk drinking water distribution systems from laboratory data. A model was developed for chlorine decay and TTHM formation and was applied to two case studies: (1) the Melbourne's Greenvale – Sydenham distribution system, which was fed with unfiltered water from Greenvale reservoir and subject to chlorination as its only treatment, and (2) the Sydney Water's Nepean system fed with water from Nepean reservoir, treated with ferric chloride, filtered and chlorinated.

The decay of free and combined chlorine was expressed by two sets of parallel first order reactions, in bulk and wall phases:

$$Cl + FRA \rightarrow Cl^{-} + f\,THM + inert$$

$$Cl + SRA \rightarrow Cl^{-} + s\,THM + inert$$

$$Cl + FNA \rightarrow CCl + inert$$

$$Cl + SNA \rightarrow CCl + inert$$

$$CCl \rightarrow inert$$

where: Cl refers to free chlorine, FRA refers to Fast Reducing Agent, SRA refers to Slow Reducing Agent, FNA refers to Fast Nitrogen Agent, SNA refers to Slow Nitrogen Agent, CCl refers to combined chlorine, THM refers to trihalomethanes, and the stoichiometric coefficients f and s represent THMs formed (in mg) by the reaction of 1mg Cl with FRA and SRA, respectively. The chlorine decay rate for each reaction was expressed by:

$$dC_i/dt = -k_j C_i C_j \qquad (2\text{-}18)$$

where i refers to chlorine, j to a given organic compound, and k refers to the reaction constant. The effect of temperature was introduced in the reaction constants. The total reaction rate was related to the bulk reaction rate as follows:

$$r_t = r_b*(1+d_{eq}/d) \qquad (2\text{-}19)$$

where: r_t is the total (bulk plus pipe wall) reaction rate [mg/L/h], r_b is the bulk reaction rate [mg/L/h], d_{eq} is the equivalent diameter constant which characterizes relative reactivity of the surface [m], and d is the pipe diameter [m].

An important assumption was that the bulk water reactions and pipe wall reactions are of the same type (same type of parallel first order reactions). The rate of the formation of THMs is related to the chlorine decay rate by the following equation:

$$dTHM/dt = s \text{ (or f) } *dC/dt$$

The coefficients f and s are usually very close and lie in the range 0.02 - 0.08 mg-THM/mg-Cl.

For each case study, samples were taken from the distribution system and chlorine decay laboratory tests were made using water entering the systems. Water quality modeling software AQUASIM was used to fit the parameters in the equations described above for various initial chlorine doses and temperatures. AQUASIM is suitable for parameter evaluation of laboratory experiments and for the simulation of simplified drinking water distribution systems. The major simplification in the application of AQUASIM consists in using only main pipes and average flow, as the program does not provide flow control options available in drinking water network modeling software such as EPANET. (Kastl et al., 2001)

The results of the chlorine decay and THM models were then compared to the results of actual measurements. A good correlation of measured and predicted results indicated the effectiveness of the model. It was also noted that in both case studies, chlorine decay was dominated by bulk water reactions. This was justified by the relatively "clean inert" system and large diameter pipes. As noted by Lamia *et al.* (2002), this parallel first-order decay model is recommended for its ability of expressing slow and fast decay reactions.

2.11 POU-WTUs

Point-of-Use Water Treatment Units are devices connected to a specific tap in order to provide additional treatment for drinking water. It is important to note the difference between Point-of-Use Water Treatment Units and Point-of-

Entry Water Treatment Units, which refer to devices that are applied at the water supply point to the residence in order to provide additional treatment of all water entering the residence, (USEPA, 1992; Droste, 1997)

A variety of POU treatment units are available for household use, either for the removal of potentially harmful contaminants or for the improvement of the aesthetic aspects of water. The choice of a specific type of unit is normally on the basis of:

- Water quality considerations: the type and concentration of the contaminant to be removed,
- ease of installation, operation, and maintenance of the treatment unit,
- and economic considerations: cost of implementation, maintenance and operation of the unit.

2.11.1 Activated Carbon POU-WTUs

Activated Carbon is the most widely used POU system for household water treatment due to the ease of its installation and maintenance, and because operating costs are usually limited to filter replacement (Lykins *et al.*, 1994b). Activated carbon is a broad-scale adsorbent of numerous types of dissolved substances depending on the source of carbon material. Activated carbon is prepared form carbonaceous material such as wood, coal, lignite, or coconut shells. The raw carbon source is slowly heated in the absence of air to liberate carbon from its associated atoms. This step is called carbonization. Activation of the carbon is done by passing oxidizing hot gases through the carbon at extremely high temperatures (315-925°C). The activation process produces the pores that result in high adsorptive properties. (Droste, 1997).

Fixed bed granular activated carbon (GAC) (referred to as GAC-FBA - fixed bed adsorber) is the most commonly used activated carbon configuration for POU units, as it provides a cost effective and viable technique for the removal of organic contaminants (Kuennen *et al.*, 1989). However, other configurations of carbon are sometimes used (although infrequently) such as pressed block, powdered activated carbon (PAC), briquettes of powdered activated carbon, and precoat filters (Lykins *et al*, 1989)

Factors affecting the Performance of Activated Carbon POU-Units

The performance of an activated carbon POU-unit depends on various factors which include:

- physical properties of the activated carbon. The type of raw carbon material and the method of activation have an important effect on the adsorption of different types of contaminants, due to the influence that raw material and activation have on pore size and distribution. The adsorption of contaminants is best when pores are barely large enough to admit the contaminant molecule (Seeling *et al.*, 1992; Droste, 1997).
- chemical properties of the activated carbon. Depending on the nature of the carbon and its activation process, the filter surface may sometimes interact chemically with organic molecules. The amount of oxygen and hydrogen associated with the pore surfaces exerts an important role in the determination of the type of contaminant adsorbed to the activated carbon. For example chloroform is adsorbed best by activated carbon that has the least amount of oxygen associated with the pore surfaces (Seeling *et al.*, 1992).
- chemical composition and concentration of the contaminant. The performance of the filter varies for different chemicals and concentrations. It is also to be noted that the occurrence of multiple contaminants in the water to be treated reduces the capacity of the activated carbon filter, especially for the less strongly adsorbed compounds. This is due to the fact that the

various compounds compete for the available sites in the carbon (Lykins *et al.*, 1994b).

- flow rate or time of contact of the water with carbon. Increasing contact time allows better removal of contaminants. This is normally done by increasing the amount of activated carbon in the filter or reducing the flow rate of water through the filter (Seeling *et al.*, 1992).

Contaminants Removed by Activated Carbon POU-WTUs

Activated carbon POU units are mostly used for the removal of chlorine and organic contaminants from water. It is to be noted that by removing excess chlorine and organics, these units play a role in the improvement of aesthetic characteristics (taste, odor, and color) of the water. Lead and other heavy metals are removed only by a very specific type of AC filter (Seeling *et al.*, 1992). The efficiency of an activated carbon system made of a GAC bed followed by a powdered activated carbon precoat filter was investigated by Regunathan *et al.* (1983). Precoat filrers are characterized by the presence of a finely powdered carbon medium applied to the influent side of the barrier portion of the filter, which increases the efficiency of the filter to remove particulates (Regunathan *et al.*, 1983). Table 2.8 presents the efficiency of the tested activated carbon system (granular-precoat device) for removal of contaminants (Regunathan *et al.*, 1983).

Kuennen *et al.* (1989) performed studies on the adsorption of 14 VOCs (volatile organic contaminants), using a POU-GAC-FBA with a rated life of approximately 1,680 bed volumes and empty bed contact time (EBCT) of 4-5 seconds. Single solute and 14 component studies were performed. The performance of the unit for removal of the tested compounds was considered high, as, even for the compounds that were least retained by GAC (1,2-dichloroethane and chloroform), the initial breakthrough did not occur until

5,000 bed volumes were fed (which is quite higher than the rated life of the filter). It was also shown in other studies on this POU-GAC-FBA that the initial removal of NOM was 50-80%, which then decreased to 10-20% over the rated life of the filter (Kuennen *et al.*, 1989).

Table 2.8: Efficiency of activated carbon system (granular-precoat device) for removal of contaminants (Regunathan *et al.*, 1983)

Contaminant	Influent concentration	% Removal
TTHMs (μg/L)	150-675	85-100
$CHCl_3$ (μg/L)	200-470	85-100
Nonpurgeable TOC (mg/L)	2.5	25-75
Nonpurgeable total organic halogen (μg/L)	100	75-100
Carbon tetrachloride (μg/L)	15-44	90-100
Turbidity (tu*)	3-5	95-100

* AC fine test dust, AC Spark Plug Div., General Motors Corp., Flint, Mich.

Bacterial Growth in Activated Carbon POU-WTUs

Activated Carbon POU-WTUs may be subject to bacterial growth if they are not maintained properly. This is actually due to the removal of chlorine residuals and the presence of nutrients, since the organic chemicals that are adsorbed to the activated carbon constitute a source of food for various types of bacteria (Regunathan *et al.*, 1983)

Cost of Activated Carbon POU-WTUs

As mentioned above, the cost of a carbon filter system is limited to initial and replacement costs of the unit, namely:

- Cost of purchasing
- Replacement cost of the filter cartridge.

Table 2.9 presents the initial and replacement cost of carbon filter units in the Egyptian market (Metito, 2003; Radwania, 2003) as well as values published in a U.S.A. study (Seeling *et al.*, 1992). As given in Table 2.9, the replacement cost of the filter is almost 25% of the purchasing cost.

Table 2.9: Initial and replacement cost of an activated carbon unit, Egypt (2003), U.S.A. Study (Seeling *et al.*, 1992).

Cost Item	**Egypt (2003)**	**U.S.A. study (1992)**
Purchasing cost	~ 200 L.E. ~ $ 32	$50 to $375
Carbon cartridge replacement cost	50 L.E – 60 L.E. $8 – $ 9.6	$8 to $69

2.11.2 Reverse Osmosis POU-WTUs

Reverse osmosis treatment is based on the concept of forcing the water through a membrane by applying pressure in excess of the osmotic pressure of the compounds that are dissolved in water (Droste, 1997). The membrane is generally made of a thin, pliable polymer that can be made of a polyamide resin, cellulose acetate, thin film composite, or tri-acetate (Lykins *et al.*, 1994b).

Most Reverse Osmosis units are equipped with a carbon filter prior to or after the reverse osmosis treatment. When used as a post-filter, the role of the carbon filter is to remove undesirable taste and any residual organics from the treated water. A pre-filter is normally needed in order to remove suspended particles to extend the life of the membrane. The pre-filter may consist of a sediment filter or a carbon filter, which will be recommended if the water contains chlorine concentrations that could damage the membrane. As most RO units supply treated water at very low rates, the RO system is usually equipped with a pressurized storage tank to provide a suitable supply (Seeling *et al.*, 1992).

Factors Affecting the Performance of RO systems

The removal efficiency of RO systems depends on the nature of the contaminant and its concentration, the type of the RO membrane, pre-filters and post-filters, and the water pressure.

Contaminants Removed by Reverse Osmosis POU-WTUs:

RO systems have shown good success in the removal of inorganic ions (90-98% removal), turbidity, bacteria, and viruses (Lykins *et al.*, 1994b; Lykins *et al.*, 1989). The removal of organic compounds by the RO membrane alone depends on the molecular weight of these compounds. RO membranes have been successful in the removal of organic compounds with molecular weights above 200 (98-99% removal). Organic compounds with molecular weights below 200 have had lower or no removal (Lykins *et al.*, 1994b). The efficiency of the RO system for removal of THMs was investigated by Regunathan *et al.*, (1983). It was shown that RO membrane alone is inefficient for removal of THMs; however, when equipped with a prefilter and two GAC adsorbers (which is a common design for commercial RO systems), the efficiency of the system for removal of THMs was 90 –100 %. Table 2.10 presents the efficiency of the tested RO system (RO-Carbon Device) for removal of contaminants (Regunathan *et al.*, 1983). Table 2.11 presents typical ranges for contaminant rejection with RO systems (Kamrin *et al.*, 1990)

Table 2.10: Efficiency of RO system (RO-Carbon Device) for removal of contaminants (Regunathan *et al.*, 1983)

Contaminant	Influent concentration	% Removal
TTHMs (μg/L)	770	90-100%
$CHCl_3$ (μg/L)	400	90-100%
$CHClBr_2$ (μg/L)	130	95-100%
$CHCl_2Br$ (μg/L)	90	95-100%
$CHBr_3$ (μg/L)	140	95-100%
Carbon tetrachloride (μg/L)	20	95-100%
TDS (mg/L)	1275	88%
TOC (mg/L)	10-12	99-100%
Nitrate (mg/L)	100	40%
Fluoride (mg/L)	100	85%
Lead (μg/L)	2100	72%

Table 2.11: Ranges for contaminant removal using RO system (Kamrin et al., 1990)

Contaminant	% Removal
Sodium	87-93%
Calcium	80-97%
Magnesium	80-98%
Iron	90-98%
Cadmium	96-98%
Lead	96-98%
Nitrate	83-92%
Organic Halides	83-92%
Trihalomethanes	65-99%
Chlorine	13-91%
Total Dissolved Solids	95-99%

As for the activated carbon POU unit, the efficiency of the RO system is affected by the presence of multiple contaminants; the rejection rate (or rejection percentage) for each contaminant may be reduced or one of the contaminants may be reduced in preference to some other contaminant(s) (Seeling *et al.*, 1992).

Disadvantages of Reverse Osmosis POU-WTUs

The main disadvantage of RO units is the low recovery rate of the volume of water consumed for treatment. They recover only 10 to 30 percent of the water entering the system (Lykins *et al.*, 1994b). The remainder is contaminated with the rejected compounds and is discharged directly to the house drains, exerting a load on the drainage system (Seeling *et al.*, 1992).

Cost of Reverse Osmosis POU-WTUs

- Initial and replacement costs of the System:
  - Cost of purchasing the RO system.
  - Replacement cost of the membranes.
  - Replacement cost of the filter cartridges.
  - Replacement cost of the storage tank.

- Operating and Maintenance Costs: electricity to pump the water is the only significant operating cost.

Table 2.12 presents the initial and replacement cost of RO systems in the Egyptian market (Metito, 2003; Radwania, 2003) as well as values published in a U.S.A. study (Seelig *et al.*, 1992).

Table 2.12: Initial and Replacement Cost of an RO System, Egypt (2003), U.S.A. Study (Seeling *et al.*, 1992)

Cost Item	**Egypt (2003)**	**U.S.A. study (1992)**
Purchasing Cost	1800 L.E – 3,500 L.E. $288 - $560	$300 to $3,000
Membrane replacement cost	300 L.E – 400 L.E. $48 - 64$	$100 to $200
Carbon cartridge replacement cost	50 L.E – 60 L.E. $8 - $9.6	~ 50 L.E.
Sediment cartridge replacement cost	15 L.E.- 25 L.E. $2.4 - $4	
Storage tank replacement cost	350 L.E. $56	

2.11.3 POU-Distillation Units

Distillers or POU-Distillation Units are based on the simple process of heating water to evaporation and then condensing the steam by passing it into a cooling section. The condensed water is then collected into a storage container. When water boils it leaves impurities behind in the boiling chamber.

Contaminants Removed by POU-Distillation Units

Distillers are most commonly used for removing inorganic compounds such as heavy metals, nitrates, sodium, calcium, magnesium, dissolved solids, *etc.* They are also efficient in the removal of particulates, in addition to most bacteria and viruses. Microorganisms that are not killed by distillation are removed form water as the steam rises from the tank. (Lykins *et al.*, 1994b)

Removal of organic compounds by the distillation process is a function of chemical characteristics of the compound such as water solubility and boiling point. It has been found that organic compounds having a boiling point greater than that of the water are efficiently removed by distillers, because they do not evaporate at temperatures used in distillation. On the other hand, other compounds such as volatile organic contaminants (VOCs) evaporate during the distillation process. These compounds are often vented to the atmosphere in order to prevent them from being carried into the condensed water (Seeling *et al.*, 1992; Lykins *et al.*, 1989).

Disadvantages of distillers

- Distillers have small capacities and require high electrical energy.
- Distillation normally removes most or all of the alkalinity of the water. This results in lowering the pH, sometimes to values exceeding the lower limits of the guidelines values for pH. Low pH is also a factor that increases the possibility of corrosion in the distillation unit.
- Distilled water has a bland taste, because the dissolved minerals that give water a pleasant taste are removed.
- Although bacteria are removed by distillation, they may recolonize on the cooling coils during inactive periods (Seelig *et al.*, 1992).

Maintenance of distillers

Minerals and other residues accumulate in the boiling chamber as water is evaporated. These minerals and compounds need to be removed on regular basis. This is normally done using organic acids. The distiller is filled with the proper acid mixture to approximately 1/2 inch above the mineral line. The acid solution is left in the distiller for the proper amount of time and then it is discarded and the distiller is rinsed. The handling and disposal of this acid wash is another disadvantage of these units in household use.

Cost of Distillation Units:

- Initial cost of the system: cost of purchasing the unit.
- Operating Costs: which consists in the cost of the electricity needed to function the distillation unit.

Table 2.13 presents the initial cost of distillation units in the Egyptian market (Metito, 2003; Radwania, 2003) as well as values published in a U.S.A. study (Seeling *et al.*, 1992).

Table 2.13: Initial cost of distillation units, Egypt (2003), U.S.A. Study (Seeling *et al.*, 1992)

Cost Item	**Egypt (2003)**	**U.S.A. study (1992)**
Purchasing Cost	1000 L.E – 13,000 L.E. $160 - $ 2080	$200 to $1,500

3. EXPERIMENTAL WORK

A work plan was developed to assess the various aspects of drinking water quality as it leaves the municipal water treatment plant to the consumer's tap, including the effect of residential POU treatment units. The work plan was divided into two main parts. The first part consisted in compiling a detailed description of the study area, including a map representing the water distribution system delivering water to the sampling sites. The second part consisted in sampling and analyzing water samples from Nile River water in the vicinity of the water treatment plant, Dar-El-Salaam WTP, and the 20 residential sites in Maadi, of which 19 sites contained a POU-treatment unit. Six sampling events were conducted over a 20-month period (Sept. 2001 to June 2003). A bench scale experiment was also conducted to examine the chlorine decay rate and TTHM formation rate used in the assessment of the effect of the distribution system on chlorine residuals and TTHMs.

3.1 Description of the Study Area

This study was conducted in residential areas in Maadi, mainly in Digla (nine sampling locations) and Maadi-proper (nine locations), with two additional locations in New Maadi. The main sources feeding these areas with water are Dar-El-Salaam water treatment plant (more recently implemented plant, age less than 20 years), and Maadi water treatment plant (older plant), both plants obtaining their raw water from the Nile River.

Detailed maps for sampling locations in the study area are presented in Appendix A, together with the water distribution network serving the area. There were a few changes in actual sampling locations over the study period in

Digla and Maadi-proper, due to participants moving their residence. This was taken into account in the statistical interpretation of the results.

Dar-El-Salaam WTP produces a discharge of about 1.08×10^6 m^3/day, while Maadi WTP produces about 7×10^4 m^3/day. Dar-El-Salaam WTP is considered as the main plant feeding the system, as it produces about 93.5% of the water delivered to the study area. Four main pipes exit Dar-El-Salam WTP, two of them carrying water to the study area. These two pipes deliver water to the Maadi area by branching into pipes of smaller diameter, and reinforcing the old network fed by Maadi WTP through by-pass connections.

3.2 Water Quality Sampling and Analysis

Water samples were collected form the following locations

- The effluent of Dar-El-Salaam WTP,
- Nile River in the vicinity of the WTP,
- residential locations in Digla (9 locations, all having POU-treatment units),
- residential locations in Maadi-proper (9 locations, all but one having POU-treatment units),
- and residential locations in New Maadi (2 locations, both having POU-treatment units).

Residential sampling locations have been chosen in a way to include a variety of POU units, namely three types of units: carbon filters, reverse osmosis units, and distillers. Residential sampling locations in Maadi-proper and Digla have also been distributed as to offer a good spatial coverage of these two areas.

Data for Maadi WTP was obtained from the Central Water Quality Laboratory-General Organization for Greater Cairo Water Supply. Six sampling events were conducted on the following dates: (October – 2001, January – 2002, May – 2002, September – 2002, February – 2003, and May – 2003)

Water samples were analyzed for conventional water quality parameters comprising:

- Physical and Aggregate Properties: Temperature, Total Dissolved Solids (TDS), Turbidity, Alkalinity, and Hardness.
- Inorganic non metallic constituents: Total Chlorine (Residual), Fluoride, pH, and Nitrate.
- Inorganic metallic constituents: Calcium, Magnesium, Lead, Iron, Copper, and Arsenic.
- Organic constituents: Total Trihalomethanes (for all samples except the Nile River). Trihalomethane Formation Potential (THM-FP) was determined for the Nile samples. THM-FP is the concentration of THMs in a sample of raw Nile River water buffered at pH 7.0, containing an excess free chlorine residual of 1 to 5 mg/L after being held 7 days at 25^0C. This test is useful in estimating THM precursors in raw water (*Standard Methods,* 1992). Total Organic Carbon (TOC) data for the Nile River and the WTP was obtained from the Central Water Quality Laboratory.
- Microbial Contaminants: Total Coliforms and Fecal Coliforms

3.2.1 Sampling Procedures (Collection, Handling and Preservation)

Sampling including water samples collection, handling and preservation was conducted according to *Standard Methods* (1992).

Sampling equipment

At each sampling location, one set of containers was used to collect samples from tap water (or river water for the Nile) and another set for the effluent of the POU treatment unit (whenever applicable).

The set of containers was designed on the basis of the volume of sample needed for each water parameter to be analyzed. Table 3.1 presents the type and minimum volume required for each parameter, and the actual sampling container used for each group of parameters.

Sampling provisions for certain parameters

For bacteriological sampling (*i.e.*, total and fecal coliforms), the tap was sterilized with a disinfectant before sampling in order to block the effect of contamination resulting from the tap itself. Sodium thiosulfate (a reducing agent) was added to the sample for dechlorination in order to make the sample represent the biological conditions at the time of sampling.

For total trihalomethanes (TTHM), sodium thiosulfate was added for dechlorination. No head-space was allowed in the sample container for TTHM. No reducing agent was added to Trihalomethane Formation Potential (TFP) samples and no head-space was allowed in the container.

Sampling preservation and storage

Two ice chests were used for immediate storage of samples prior to delivery to the laboratory. Samples were analyzed as soon as possible after collection. Preservation and storage were conducted according to criteria given by *Standard Methods* (1992) and presented in Table 3.1.

Table 3.1: Container type and size, preservation, and storage criteria.

Parameter	Container* (specified)	Minimum Sample size (ml)	Sampling container actually used	Preservation*	Maximum* Storage Recommended / Regulatory
Temperature	On-site analysis				
pH	P	-	500 ml P	Analyze immediately	2 h / Stat
TDS	P	-		Refrigerate	7d/2 – 7d
Turbidity	P	-		Refrigerate	24 h / 48 h
Alkalinity	P	100 ml		Refrigerate	24 h / 14 d
Chlorine residual	P, G	50 ml		Analyze immediately	0.5 h / stat
Fluoride	P	50 ml		None Required	28 d / 28 d
Nitrate	P, G	50 ml		Refrigerate	48 h
Total Hardness	P	100 ml		Add HNO_3 to pH < 2	3 Months / 6 Months
Calcium	P	100 ml		Add HNO_3 to pH < 2	3 Months / 6 Months
Magnesium	By Calculation				
Iron	P, (rinsed with 1:1 HNO_3)	100 ml	500 ml P, (rinsed with 1:1 HNO_3)	Add HNO_3 to pH < 2	3 Months / 6 Months
Lead	P, (rinsed with 1:1 HNO_3)	100 ml		Add HNO_3 to pH < 2	3 Months / 6 Months
Copper	P, (rinsed with 1:1 HNO_3)	100 ml		Add HNO_3 to pH < 2	3 Months / 6 Months
Arsenic	P, (rinsed with 1:1 HNO_3)	100 ml		Add HNO_3 to pH < 2	3 Months / 6 Months
Total Colifroms	PP, G, (sterilized)	100 ml	500 ml G, sterilized	Refrigerate	6 h
Total Colifroms	PP, G, (sterilized)	100 ml		Refrigerate	6 h
Total Trihalomethanes (TTHM)	Amber Glass with septum-seal screw cap, (dried at 105°C for 1h)	40 ml	45 ml Amber G., sept.-seal screw cap, (105°,1h)	Refrigerate	14 d
THM-FP	Amber Glass, septum-seal screw cap, (105°C for 1h)	1L	1 L Amber G., sept.-seal screw cap, (105°,1h)	Refrigerate	7 d

*: Standard Methods (1992).

TDS: Total Dissolved Solids

P: Polyethylene, PP: Polypropylene, G: Glass

Stat.: no storage allowed.

3.2.2 Water Quality analyses

Analysis Methods and Devices

Laboratory analyses of samples for the water quality parameters listed (other than the temperature, which was measured on site and the TOC data, which was obtained from the Central Laboratory) were according to Standard Methods (1992) and Hach (1993). Table 3.2 presents the methods used for the water quality parameter analysis and the measurement devices used, in addition to the optimum concentration range for measuring the concentration of a specific water quality parameter using the corresponding device.

Quality Assurance and Quality Control

Quality Assurance and Quality Control (QA/QC) procedures were applied to the analyses in terms of:

- Duplicate analysis for samples. (For large numbers of samples, duplicates were analyzed for randomly chosen samples)
- Laboratory Control Standard (Calibration Check Standard): A standard used to determine the state of calibration of an instrument between periodic recalibrations.
- Laboratory reagent Blank: An aliquot of reagent water that is analyzed in the same way as all the standards and samples, to determine if analytes or other interferences are present in the lab- environment, reagent, or the apparatus.

QA/QC tests performed in the laboratory present a good means to evaluate the accuracy and reliability of measurements. A summary of QA/QC testing for THMs and trace metals (arsenic, lead and copper) performed in the laboratory is presented in Table 3.3 and Table 3.4 respectively.

The tests were performed using standards that were prepared with certain concentrations (test standards). Seven measurements were performed for each standard. The mean and standard deviation (SD) of the measurements were calculated. Precision (%RSD) is a measure of how far an individual determination may be from the mean of replicate measurements and is calculated as the % ratio of SD to the mean. Low values for %RSD indicate high precision of measurements. Percent Recovery represents the percent ratio of the mean of measured replicates to the true (prepared) concentration. Values of % Recovery that are closer to 100% indicate the accuracy of measurements.

Upper Control Limit (UCL) and Lower control limit (LCL) are statistical means used for monitoring the performance of an analytical instrument. If all of the replicates lie between the UCL and LCL, this indicates high accuracy of measurements. UCL and LCL are calculated as follows:

$$\mathrm{UCL} = \mu + \frac{3.\mathrm{SD}}{\sqrt{\mathrm{N}}} \tag{3-1}$$

$$\mathrm{LCL} = \mu - \frac{3.\mathrm{SD}}{\sqrt{\mathrm{N}}} \tag{3-2}$$

where μ = mean, SD = standard deviation, and N = number of replicates.

MDL (Method Detection Limit) represents the constituent concentration that, when processed through the complete method, produces a signal with 99% probability that is different from the blank. MDL is calculated as follows:

$$\mathrm{MDL} = t_{(n-1,\, 1-\alpha=0.99)} . \mathrm{SD} \tag{3-3}$$

where SD = Standard Deviation, $t_{(n-1,\, 1-\alpha=0.99)}$ = Student's t-value for one-sided 99% confidence level.

Table 3.2: Methods and devices for water quality parameter analysis

Parameter	Analytical method #	Measurement Device	Modifications/ Notes	Optimum Concentration Range
pH	SM[a] #4500-H^+ B Hach Method[b] # 8156	Hach One Laboratory pH meter		1.99 to 19.99pH[f]
TDS	Hach Method[b] # 8160	Hach Conductivity / TDS meter		0–199.9mg/L[e] 0-1.999 g/L[e]
Turbidity	Hach Method[b] # 8237**	DR/2000 Spectrophotometer		0 to 450 FTU**,[b]
Alkalinity	SM[a]#2320 B		Titrimetric method	
Total Hardness	SM[a]#2340 C		EDTA Titrimetric method	
Calcium	SM[a]#3500-Ca D		EDTA Titrimetric method	
Magnesium	SM[a]#3500-Mg E		Calculation Method	
Iron	SM[a]#3500-Fe B SM[a]#3113-AAS[c]	atomic spectroscopy[c]	Digestion by SM # 3030	5–100 µg/L[a]
Lead	SM[a]#3500-Pb B SM[a]#3113-AAS[c]	atomic spectroscopy[c]	Digestion by SM # 3030	5–100 µg/L[a]
Copper	SM[a]#3500-Cu B SM[a]#3113-AAS[c]	atomic spectroscopy[c]	Digestion by SM # 3030	5–100 µg/L[a]
Arsenic	SM[a]#3500-As B SM[a]#3113-AAS[c]	atomic spectroscopy[c]	Digestion by SM # 3030	5–100 µg/L[a]
Total Chlorine residual	SM[a]#4500-Cl G Hach Method[b] # 8167	DR/2000 Spectrophotometer		0-2.00mg/L[b]
Free Chlorine	SM[a]#4500-Cl G Hach Method[b] # 8021	DR/2000 Spectrophotometer		0 to 2.00 mg/L[b]
Fluoride	Hach Method[b] # 8029	DR/2000[g]		0-2.00mg/L[b]
Nitrate	Hach Method[b] # 8192	DR/2000[g]		0-0.40 mg/L[b] as $NO3^-$-N
Total Colifroms	SM[a]#9222 B		Membrane Filter procedure	
Total Colifroms	SM[a]#9222 D		Membrane Filter procedure	
TTHM	SM[a]#6232 B	GC[d]		0.5-200µg/L[a]
TFP	SM[a]#5710 B	GC[d]		0.5-200µg/L[a]

TDS: Total Dissolved Solids

TTHM: Total Trihalomethanes.

TFP: Trihalomethane Formation Potential.

**: Absorptometric Method: Adapted from FWPCA *Methods for Chemical Analysis of Water and Wastes*, 275 (1969)
This procedure is calibrated using Formazin Turbidity Standards and the measurements are in terms of Foramzin turbidity Units (FTU). "This test cannot be used for EPA reporting purposes but can be used for day to day in plant monitoring. [A Formazin turbidity Unit (FTU) is equivalent to a Nephelometric Turbidity Unit (NTU) when readings are made on a nephelometer]", (*DR/2000 Spectrophotometer Handbook*)

a: *Standard Methods* (1992).

b: *Hach* (1993).

c: Atomic Absorption Spectrophotometer, Perkin Elmer SIMMA 6000 with Zeeman Furnace module and auto sampler, for low level metal analysis and controlled with software.

d: Perkin Elmer Gas Chromatograph equipped with electron capture detector, split/splitless injector and controlled with software (Turbochrom 4, version 4.1)

e: *Model 44600 Hach Conductivity/TDS meter Manual.*

f: *Hach One Laboratory pH meter Instruction Manual.*

g: DR/2000 Spectrophotometer.

Table 3.3: QA/QC testing for THMs

THM compound	**$CHCl_3$ (µg/L)**	**$CHCl_2Br$ (µg/L)**	**$CHClBr_2$ (µg/L)**	**$CHBr_3$ (µg/L)**
Test Standard	12	2	2	2
Mean of 7 analyses	11.887	2.046	1.897	1.923
Standard deviation	0.372	0.046	0.112	0.043
Precision (%RSD)	3.1	2.3	5.9	2.2
% Recovery	99.1	102.3	94.9	96.2
Upper Control Limit	12.32	2.10	2.03	1.97
Lower control Limit	11.467	1.994	1.774	1.874
Method Detection Limit	1.168	0.145	0.353	0.135

Table 3.4: QA/QC testing for trace metals

Trace Metal	**Arsenic (µg/L)**	**Copper (µg/L)**	**Lead (µg/L)**
Test Standard	5.00	15.00	5.00
Mean of 7 analyses	4.909	16.250	5.420
Standard deviation	0.592	0.539	0.356
Precision (%RSD)	12.1	3.3	6.6
% Recovery	98.2	108.3	108.4
Upper Control Limit	5.59	16.87	5.83
Lower control Limit	4.239	15.639	5.017
Method Detection Limit	1.859	1.695	1.335

The results of the tests described in Tables 3.3 and 3.4 indicate low values for %RSD, in addition to % recovery values that are close to 100%. Additionally, 82.1% of the measurements for THMs and 64.29% of the measurements for trace metals were between UCL and LCL.

3.3 Determination of the Bulk Chlorine Decay Rate Constant and Bulk TTHM Formation Rate Constant in the Distribution System

Bulk water reaction rate constants for the decay of chlorine residual and TTHM formation are commonly determined from laboratory tests (Rossman *et al.*, 2001). In the present study, a laboratory experiment was performed in order to estimate the bulk reaction kinetics of chlorine and TTHM. Samples were collected from effluent of Dar-El-Salaam WTP, the main plant feeding water to the study area. A set of amber glass bottles were filled with water from the effluent of the WTP, sealed with Teflon caps with no headspace allowed. Bottles were placed on a shaker at 60 rpm (to represent the average bulk water velocity in pipes), at a temperature between 20 and 22 °C. Samples were taken at the beginning of the test (time = 0) and at regular close intervals during the first seven hours (the actual residence time in the system does not exceed 4 hours as will be noted in Chapter 4), and at 24-hr intervals for seven days, with additional samples taken in a number of days. Samples were analyzed for free and total chlorine residual and TTHM. The first order chlorine decay rate constant, k_b, was then determined as the slope of $\ln(C_t/C_o)$ versus time. The first order rate constant, k_b, for the formation of TTHM was determined as the slope of the plot of $\ln\frac{(C_m - C_o)}{(C_m - C_t)}$ versus time, where C_t is concentration after time t, C_o is concentration at time 0, and C_m = ultimate formation concentration of

TTHM. Detailed results of the experiment and calculation of k_b and C_m are presented in Chapter 4.

In this experiment, the TTHM level at time = zero (in the effluent of the WTP) was 45.1 µg/L, while the average TTHM level in the effluent of the WTP over the study period (Sept. 2001 to June 2003) was 70.4 µg/L. It was assumed that the same formation rate applies to both TTHM levels. For the determination of the input value of the ultimate TTHM formation concentration required by EPANET modeling software, the results of another experiment performed in the same laboratory (Emam, 2003) & starting with a TTHM level of 77.0 µg/L (more comparable to the average TTHM at the WTP over the study period) were used. This experiment was performed by filling a set of amber glass bottles with tap water; Teflon cap seals were used with no head space allowed. Bottles were placed on a shaker at 165 ± 5 rpm. Samples were taken at the beginning of the test (time = 0) and every 24 hours over a seven day period, and they were analyzed for total chlorine residual and TTHM. Results of the experiment and calculation of C_m are presented in Chapter 4.

4. RESULTS AND DISCUSSION

In this chapter, various water quality aspects are elaborated based on water quality data obtained from the present study. The compliance of water quality with health-based standards and the possible existence of spatial or seasonal patterns in the variation of the water quality parameters are evaluated. Considerable interest is given to the relation between chlorine levels (initial/residual concentrations) and total trihalomethanes, including an attempt to model the kinetics of chlorine decay and total trihalomethanes formation in the water distribution network studied.

Due to the relatively low discharge of Maadi WTP compared to Dar-El-Salaam WTP and the low observed differences in water quality effluent of the two plants, water quality data for Dar-El-Salaam plant effluent has been used to represent the water entering the distribution system in the study area. However, for the modeling of chlorine and TTHM using EPANET modeling software, the results from both plants were introduced to the model (namely, the chlorine and TTHM concentrations).

The efficiency of point of use (POU) water treatment units used at residential sampling locations is also studied in this chapter.

4.1 Characteristics of Raw and Tap Water Quality

Water quality data from six events for raw Nile water, Dar-El-Salaam WTP effluent and the 20 residential locations are interpreted using statistical and analytical analyses in the following sections. It is to be noted that parameter measurements reported as not detected, (ND) or under detection limits, (UDL) were counted as zero. In addition, arsenic was measured for Events 1 and 2 only, as the results of these two events showed no detection of arsenic in any sample. Also, the method used for nitrate measurements in Event 1 and for Digla measurements only in Event 2 was overestimating nitrate levels (high range nitrate method, Hach (1993)). It was found that the low range nitrate method (Hach, 1993) was more accurate for the given samples; this method was further used for all other samples in Event 2 through Event 6 and measurements for Event 1 were discarded. This is responsible for the relatively high nitrate level in Event 2 for Digla with respect to other values, which is presented in Figure 4.17.

4.1.1 Raw Water Quality

Average raw water quality for the parameters measured over the six sampling events are presented in Table 4.1, together with standard deviations and maximum and minimum values for each parameter. High standard deviations relative to means in total and fecal coliforms reflect the variation of biological conditions within the 6 events. Total coliforms in Event 1 were an order of magnitude higher than the other events, which partially justifies the large standard deviation. However, the mean value for total coliforms in Events 2 through 6 is 1300 colonies/100 ml, with a standard deviation of 748, which is also high. These variations are also expressed in Figure 4.2, which also shows that they did not have a seasonal pattern during the study period.

Figure 4.1 and Figure 4.2 present histograms depicting relative variations of 12 of the studied water quality parameters throughout the 6 events; each parameter being expressed by the ratio of the parameter value for the event to the maximum value for the 6 events. This presentation can be used to investigate the seasonal variations in water quality and relative variations between the different parameters. Iron levels also had a high standard deviation, with the highest levels during Events 4 and 6. Also, from Figure 4.2, iron variations are not considered to be seasonal during the study period.

In this work, an effort was made to detect seasonal variations in water quality by sampling three times throughout the year. Moreover, temperature changes are known to have an effect on reaction kinetics which, in turn, affect the quality of raw and distributed water.

Table 4.2 presents average raw water quality for summer and winter, with summer (considered to be May thru October) being represented by sampling events 1, 3, 4, and 6, and winter (November thru April) represented by sampling events 2 and 5. Table 4.2 indicates no substantive difference in water quality between summer and winter for the studied parameters except that TOC and alkalinity exerted a slight increase in winter. Variations in iron and coliform levels have been shown to be due to individual events as mentioned previously. For total coliforms, the summer average was calculated for Events 3, 4 and 6, with the exclusion of Event 1 in which total coliforms were an order of magnitude higher than the other events; however, the standard deviation for Events 3, 4 and 6 was still high as mentioned previously in this section. For the rest of the parameters, it can be deduced from Table 4.2, Figure 4.1 and Figure 4.2 that, in spite of the variations in parameter concentration over the study period, there was no clear seasonal pattern in these variations. This conclusion can only be applied for the period of study (18 months from October 2001 to May 2003) and for the studied parameters. A discussion was made with water resources officials in order to understand the reason for variations in raw water quality parameters. The discussion revealed that such variations are due to operational conditions in the Nile barrages, for example, the opening of gates to allow certain flow velocities.

Table 4.1: Mean, Max., and Min. concentrations in Nile raw water (means represent the average of 6 events)

Analysis	Units	Nile Raw Water (Mean Conc.)	Min	Max
pH	pH	8.23 ± 0.21	7.97	8.50
TDS	mg/L	181 ± 20	160	207
Turbidity	FTU	10 ± 2	8	12
Total Alkalinity	mg/L as CaCO$_3$	148 ± 31	122	208
Total Hardness		106 ± 14	88	122
Calcium	mg/L	24 ± 4	21	31
Mg	mg/L	11 ± 4	8	18
Iron	mg/L	0.388 ± 0.234	0.147	0.730
Lead	mg/L	0.000 ± 0.001	0.000	0.0018
Copper	mg/L	0.000 ± 0.000	0.000	0.000
Arsenic	mg/L	0.000 ± 0.000	0.000	0.000
Nitrate (as Nitrogen)	mg/L	0.04 ± 0.01	0.02	0.05
Fluoride	mg/L	0.38 ± 0.04	0.31	0.43
Total coliforms	/100 mL	3750 ± 6038	700	16000
Fecal coliforms	/100 mL	448 ± 227	200	640
TOC	mg/L	3.26 ± 0.38	2.80	3.73

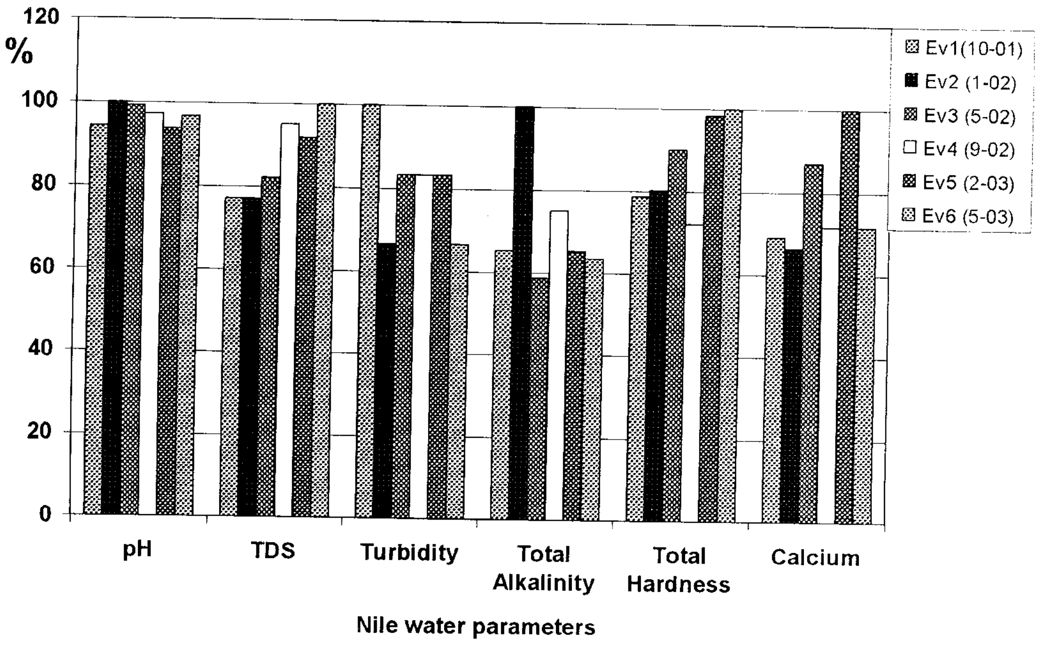

Figure 4.1: Relative variations in raw water quality. Each parameter is presented as percent of the maximum measurement of 6 events.

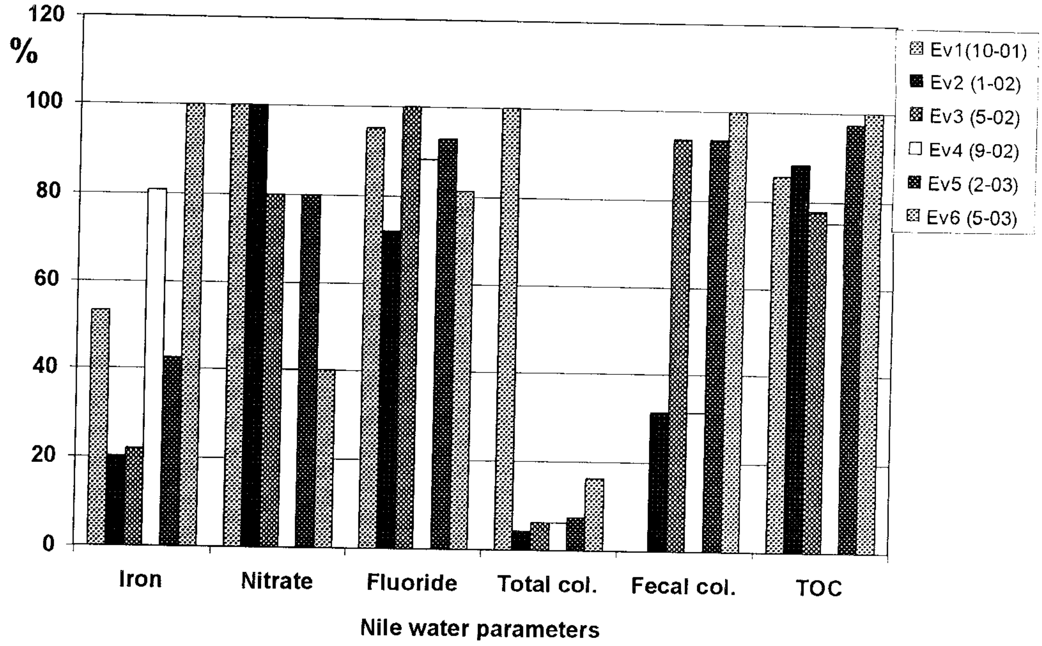

Figure 4.2: Relative variations in raw water quality. Each parameter is presented as percent of the maximum measurement of 6 events (Fecal coliforms presented for events 2 through 6 only)

Table 4.2: Nile raw water quality for summer (Events 1, 3, 4, and 6) versus winter (Events 2, 5)

Analysis	**Units**	**Nile Raw Water**	
		Summer	**Winter**
pH	pH	8.23 ± 0.17	8.24 ± 0.37
TDS	mg/L	184 ± 22	175 ± 21
Turbidity	FTU	10 ± 2	9 ± 1
Total Alkalinity	mg/L as	137 ± 14	172 ± 51
Total Hardness	$CaCO_3$	104 ± 15	109 ± 16
Calcium	mg/L	23 ± 3	26 ± 7
Mg	mg/L	12 ± 5	11 ± 1
Iron	mg/L	0.467 ± 0.248	0.228 ± 0.115
Lead	mg/L	0.000 ± 0.001	0.000 ± 0.000
Copper	mg/L	0.000 ± 0.000	0.000 ± 0.000
Arsenic	mg/L	0.000 ± 0.000	0.000 ± 0.000
Nitrate (as Nitrogen)	mg/L	0.03 ± 0.02	0.05 ± 0.01
Fluoride	mg/L	0.39 ± 0.03	0.36 ± 0.06
Total coliforms	/100 mL	1533 ± 924	950 ± 354
Fecal coliforms	/100 mL	480 ± 243	400 ± 283
TOC	mg/L	3.16 ± 0.42	3.47 ± 0.24

Table 4.3 presents five experiments of the Trihalomethane Formation Potential (THM-FP) for Nile raw water. Three of the experiments can be used to investigate the effect of the initial chlorine dose on the TTHM formation.

Table 4.3: Trihalomethane Formation Potential (THM-FP) for raw water

		TOC Raw Water	Initial chlorine dose	free chlorine residual after 7 days	THM-FP
Experiment	**Sampling Period**	**mg/L**	**mg/L**	**mg/L**	**µg/L**
1	Jan-02	3.3	16	1.68	203
2	May-02	2.9	18.9	1.9	229
			23.6	4.7	277
			28.3	6	307
3	Sep-02	2.8	10	1.2	139
4	May-03	3.73	10	1	198
			14	2.4	210
			16	3.68	259
5	Sep-03	3	10	1	150
			12.7	2.4	159

For the experiment to be representative of TTHM formation potential, the levels for free chlorine residual after 7 days should be between 1.0 mg/L and 5.0 mg/L (*Standard Methods,* 1992). For Experiment 2, free chlorine residual after 7 days was 6 mg/L which is higher than the 5 mg/L boundary set by the *Standard Methods*. However, this experiment was used to investigate the correlation between initial chlorine dose and THM-FP. It is clear from Table 4.3 that a higher chlorine dose generates higher THM-FP. This is even better expressed in Figures 4.3 and 4.4, showing a linear correlation between initial chlorine dose and THM-FP, with R^2 values of 0.98 for Experiment 2 and 0.75 for Experiment 4.

It is to be noted that THM-FP experiments are conducted at controlled temperature (25°C) and pH buffered at 7.0. These conditions block the effect of temperature and pH variation on the formation of TTHMs.

Figure 4.5 expresses the relation between THM-FP and TOC levels in raw water, using data from Experiments 3, 4, and 5, for the initial chlorine dose of

10 mg/L, showing a linear correlation with $R^2 = 0.999$. This high degree of correlation between TOC and THM-FP is consistent with the use of both of them as surrogate parameters for the natural organic matter present in water (Owen *et al.*, 1995), which is also the primary goal of the THM-FP experiment (*Standard Methods*, 1992).

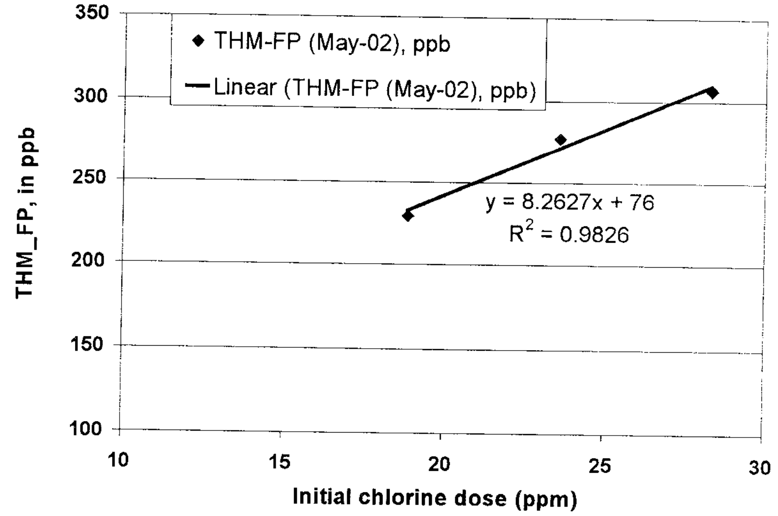

Figure 4.3: Initial chlorine dose vs. THM-FP for experiment 2 (May-02)

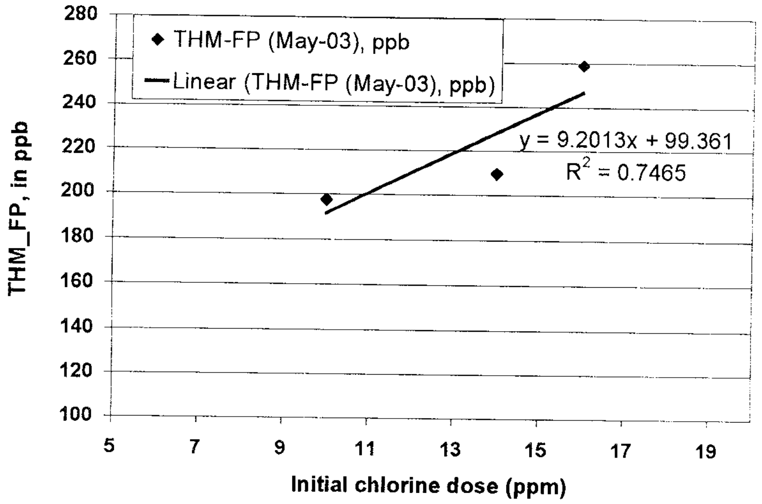

Figure 4.4: Initial chlorine dose vs. THM-FP for experiment 4 (May-03)

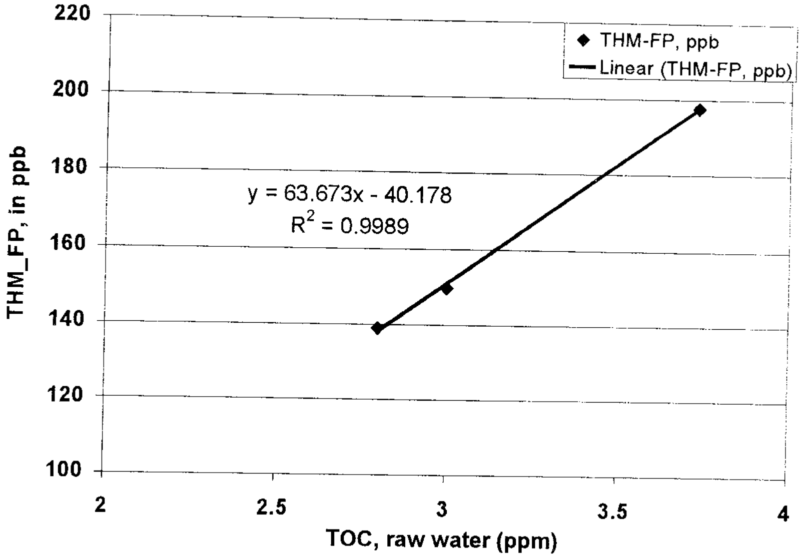

Figure 4.5: Correlation of THM-FP with TOC for an initial chlorine dose in the level of 10 mg/L

4.1.2 WTP Effluent Quality.

Table 4.4 presents a comparison between mean concentrations (for most of the studied parameters) at Dar-El-Salaam WTP (6 events) and Maadi WTP (5 events only; except total chlorine, TOC, and TTHM for 6 events). Table 4.4 indicates that water quality at both plants is relatively close. This observation, in addition to the relatively low discharge of Maadi WTP compared to Dar-El-Salaam WTP, led to using Dar-El-Salaam WTP effluent to represent the water entering the distribution system in the study area except that, for the modeling of chlorine and TTHM using EPANET modeling software, the results from both plants were introduced to the model (namely, the chlorine and TTHM concentrations).

Water samples have been collected from the Dar EL Salaam WTP effluent, once per sampling event. These samples are considered to represent the whole period (2 to 3 weeks) of sampling for all parameters, except total chlorine and TTHM. This is due to the fact that operational conditions for the chlorination process can vary over the sampling period, causing variations in chlorine residuals and TTHM in the effluent of the WTP. To account for this effect, data for these two parameters was collected in a second day during each event (data was collected from the Central Water Quality Laboratory). Averages for chlorine and TTHM values of the two days were calculated and used for the statistical and analytical analyses. Table 4.5 presents statistical means, standard deviations, maximum and minimum values for the parameters studied for the WTP effluent over the six sampling events.

Evaluation of the compliance of water quality effluent of the WTP with Egyptian and international guidelines (WHO and USEPA) was the first interpretation of these results. It was noticed that water quality complies with the standard criteria for all parameters except for the occurrence of two TTHM values higher than the USEPA criteria of 80 ppb. These values are shown in Table 4.6.

Histograms in Figure 4.6, 4.7, and 4.8 present relative variations of 13 of the studied water quality parameters throughout the 6 events (12 parameters for WTP effluent, in addition to TOC in raw water); each parameter being expressed by the ratio of the parameter value for the event to the maximum value for the 6 events. These histograms, as well as Table 4.7, indicate that summer and winter values are almost identical for all studied parameters except for TTHMs, TOC, and TDS. Table 4.7 and Figure 4.8 indicate that TTHMs had slightly higher levels in summer than in winter. However, this was not reflected in residential samples (Table 4.12) and it was not related to TOC (or organic matter) levels, which decreased in summer for both the raw Nile water and the effluent of the WTP (Section 4.1.1, Table 4.7). Table 4.7 also indicates a slight increase in TDS levels in winter, which was also noted in residential locations (Table 4.12). However, this observation has not been the same for TDS levels in Nile water, which exerted a slight decrease in winter (Table 4.2). Based on these observations, only variation on TOC levels may be regarded as seasonal.

associated with the decrease in TOC values for summer versus winter seasons. As was the case for the Nile raw water, iron levels at the WTP have also had a high standard deviation (Table 4.5, Table 4.7). In spite of the correlation found between TOC of the raw water and TTHM formation potential for the same initial chlorine dose, the correlation between actual values for TOC and TTHM the WTP is not that obvious.

Table 4.8 presents the % TOC removal at the WTP throughout the study period, indicating non-uniformity in TOC removal. There is no guideline value for the TOC concentration. However, the Disinfectants and Disinfection Byproducts Rule in the U.S.A. contains a guideline criteria for the required % removal of TOC: for a source water quality having an alkalinity greater than 120 mg/L and TOC levels in the range of 2.0 - 4.0 mg/L, 15% TOC removal is required (USEPA, 2001b). Table 4.8 shows that % TOC removal at the WTP was less than this required value in a number of events over the period of study.

Table 4.4: Mean concentrations at the effluent of Dar-El-Salaam WTP (6 events) and Maadi WTP (5 events except total chlorine, TOC, and TTHM for 6 events)

Analysis	Units	Dar-El-Salaam WTP	Maadi WTP	Egyptian Criteria	WHO Criteria	U.S. EPA Criteria
Temp.	^0C	25 ± 4	22 ± 5			
PH	pH	7.28±0.18	7.51±0.23	6.5-9.2	6.5-8.5	6.5-8.5
TDS	mg/L	190 ± 24	no data	1,200	1,000	500
Turbidity	FTU	1 ± 1	no data	5	5	0.3 - 1.0
Alkalinity	mg/L as $CaCO_3$	127 ± 10	114 ± 8	-	-	-
Total Hardness		103±15	120±11	500	500	-
Calcium	mg/L	21 ± 1	29 ± 2	200	-	-
Mg	mg/L	12 ± 3	11 ± 1	150	-	-
Iron	mg/L	0.044±0.05	0.066±0.148	0.3	0.3	0.3
Lead	mg/L	0.000±0.001	0.002±0.001	0.05	0.01	0.015
Copper	mg/L	0.000±0.000	0.000±0.000	1.0	1.0	1.0
Total Chlorine	mg/L	1.96±0.31	1.92±0.40	-	5	4.0
Nitrate (as N)	mg/L	0.04±0.01	0.00±0.00	10 (as N)	50 (as nitrate)	10 (as N)
Fluoride	mg/L	0.38±0.10	0.40±0.08	0.8	1.5	2.0
Total coliforms	/100 mL	0±0	no data	5%	Nil	5%/month
Fecal coliforms	/100 mL	0±0	no data	Nil	Nil	Nil
TTHMs	µg/L	70.4±15.5	73.8±8.4	100	-	80
TOC	mg/L	2.95±0.54	2.93±0.42			

Table 4.5: Mean, Max., and Min. concentrations at Dar-El-Salaam WTP effluent for 6 Events

Analysis	Units	WTP Effluent (Mean)	Min	Max	Egyptian Criteria	WHO Criteria	U.S. EPA Criteria
Temp.	^{0}C	25 ±4	17	30			
PH	pH	7.28 ±0.18	7.06	7.55	6.5-9.2	6.5-8.5	6.5-8.5
TDS	mg/L	190 ±24	160	212	1,200	1,000	500
Turbidity	FTU	1 ±1	0	1	5	5	0.3 -1.0
Alkalinity	mg/L	127 ±10	116	142	-	-	-
Total Hardness	mg/L as $CaCO_3$	103 ±15	86	124	500	500	-
Calcium	mg/L	21 ±1	19	23	200	-	-
Mg	mg/L	12 ±3	7	16	150	-	-
Iron	mg/L	0.044 ±0.05	0.000	0.123	0.3	0.3	0.3
Lead	mg/L	0.000 ±0.001	0.000	0.0017	0.05	0.01	0.015
Copper	mg/L	0.000 ±0.00	0.000	0.000	1.0	1.0	1.0
Arsenic	mg/L	0.000 ±0.00	0.000	0.000	0.05	0.01	0.01
Total Chlorine	mg/L	1.96 ±0.31	1.3	2.1	-	5	4.0
Nitrate (as Nitrogen)	mg/L	0.04 ±0.01	0.02	0.05	10 (as Nitrogen)	50 (as nitrate)	10 (as Nitrogen)
Fluoride	mg/L	0.38 ±0.10	0.28	0.54	0.8	1.5	2.0
Total coliforms	No./100 mL	0 ±0	0	0	5%	Nil	5% /month
Fecal coliforms	No./100 mL	0 ±0	0	0	Nil	Nil	Nil
TTHMs	µg/L	70.4 ±15.5	51.1	91.2	100	-	80
TOC	mg/L	2.95 ±0.54	2.30	3.62			

Table 4.6: Occurrence of values exceeding water quality standards at Dar-El-Salaam WTP

Analysis	Units	Event	Value	Egypt. Criteria	WHO Criteria	U.S. EPA Criteria
TTHMs	µg/L	3	85.3	100	-	80
		4	91.2	100	-	80

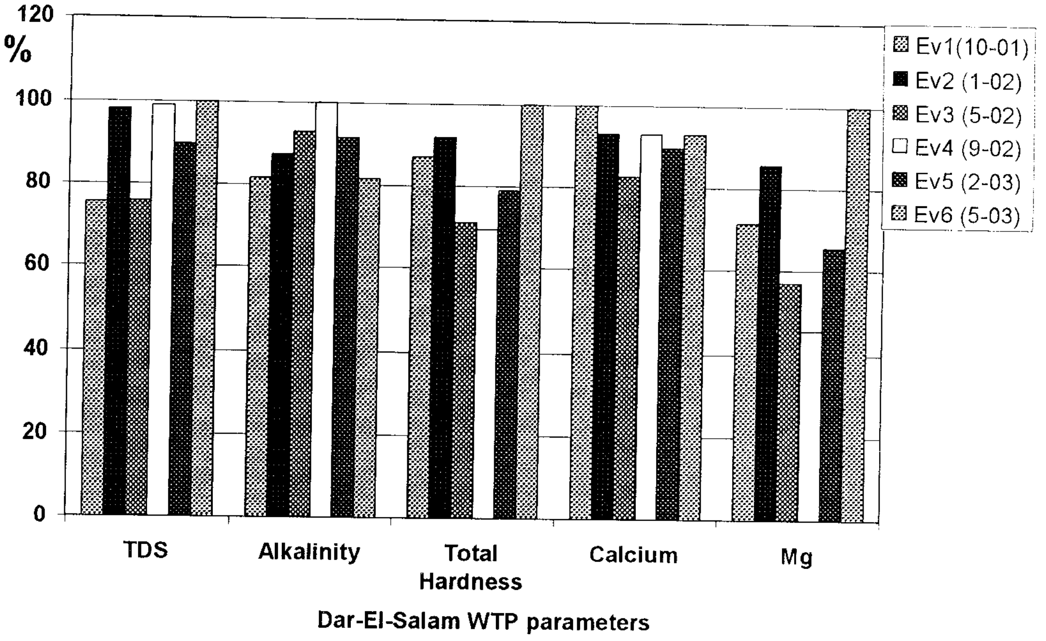

Figure 4.6: Relative variations in Dar-El-Salaam WTP effluent water quality. Each parameter is presented as percent of the maximum measurement of 6 events.

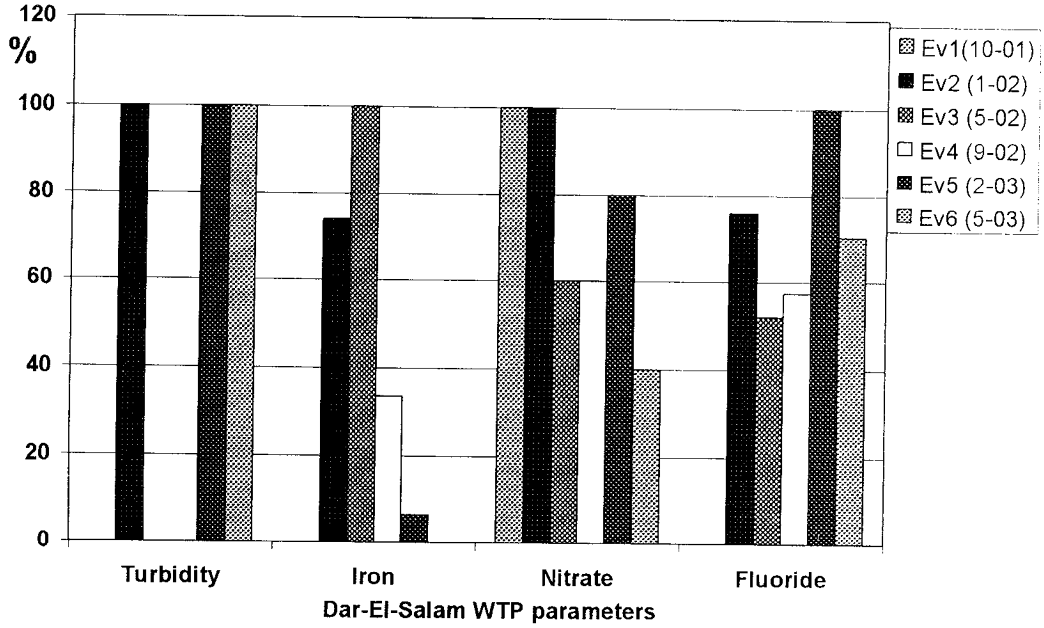

Figure 4.7: Relative variations in Dar-El-Salaam WTP effluent water quality. Each parameter is presented as percent of the maximum measurement of 6 events.

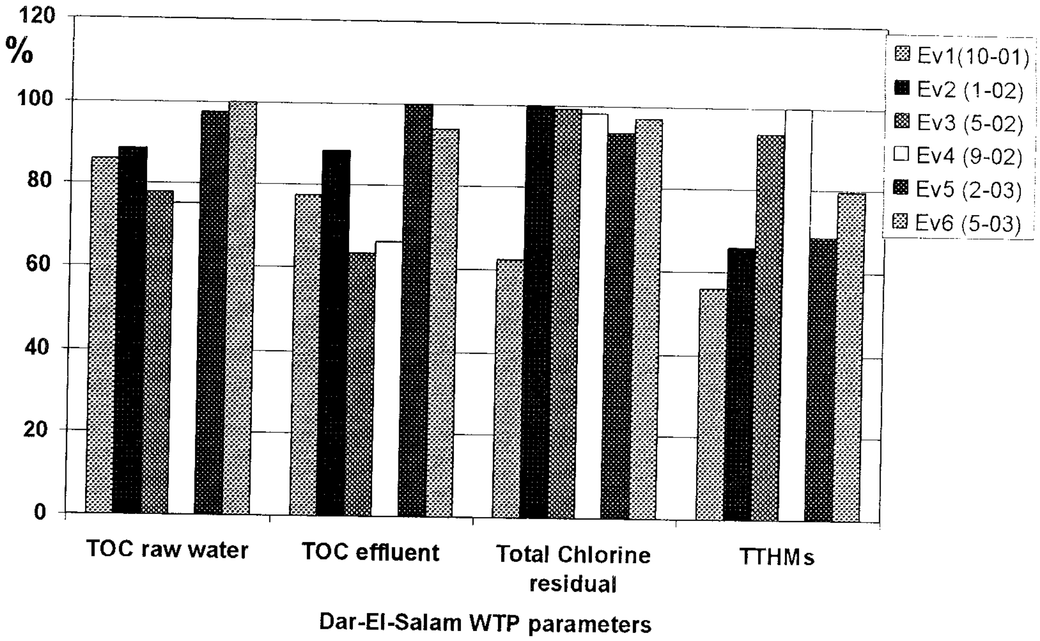

Figure 4.8: Relative variations in Dar-El-Salaam WTP effluent water quality & TOC in raw water. Each parameter is presented as percent of the maximum measurement of 6 events.

Table 4.7:Dar El-Salaam WTP water quality for summer (Events 1, 3, 4, and 6) versus winter (Events 2, 5)

Analysis Temp.	Units ^{0}C	Dar El-Salaam WTP	
		Summer	Winter
pH	pH	7.30 ± 0.18	7.25 ± 0.26
TDS	mg/L	186 ± 29	199 ± 13
Turbidity	FTU	0 ± 1	1 ± 0
Alkalinity	mg/L as CaCO$_3$	127 ± 13	127 ± 4
Total Hardness		102 ± 18	106 ± 11
Iron	mg/L	0.041 ± 0.058	0.050 ± 0.059
Total Chlorine	mg/L	1.91 ± 0.38	2.07 ± 0.10
TTHMs	µg/L	75.02 ± 17.77	61.19 ± 1.64
TOC raw water	mg/L	3.16 ± 0.42	3.47 ± 0.24
TOC effluent	mg/L	2.73 ± 0.5	3.41 ± 0.3

Table 4.8: % TOC removal in Dar-El-Salaam WTP:

	Pre-chlorination	Post-chlorination	TOC raw water	TOC WTP effluent	TOC removal at WTP
Event	**mg/L**	**mg/L**	**mg/L**	**mg/L**	**%**
Ev. 1	7	0	3.2	2.8	13
Ev. 2	6	0	3.3	3.2	3
Ev. 3	6	0	2.9	2.3	21
Ev. 4	6	0	2.8	2.4	14
Ev. 5	6	0	3.64	3.62	1
Ev. 6	6	0	3.73	3.4	9

4.1.3 Residential tap water quality.

As mentioned previously, 20 residential taps were sampled: 9 in Digla, 9 in Maadi-proper, and 2 in New Maadi. It is to be noted that having only two sampling sites in New Maadi contributes to the high standard deviations in this area. Table 4.9 presents statistical means and standard deviations for the parameters studied over the six sampling events (the standard deviation here is the deviation in means for individual events). This statistical approach was chosen in order to dampen the bias of individual measurements that could be due to individual storage at each location. Table 4.10 presents maximum and minimum values for each parameter over the study period, except for the Arsenic, which was measured in two events only.

Evaluation of the compliance of water quality in the three areas (Digla, Maadi proper, and New Maadi) with Egyptian and international (WHO and USEPA) standards showed that water quality complies with the standard criteria for all parameters, except for occurrences of TTHM, turbidity, lead, and one case of fecal coliforms exceeding standard criteria. These values are shown in

Table 4.11. Elevated lead values in Event 3 could partially be due to experimental problems. In fact, some lead contamination was detected in the nitric acid used in the sample preparation step for extracting the metals prior to analysis. This lot was discarded and no contamination problems were observed in subsequent events.

Figures 4.9 thru 4.18 present histograms for the average concentration of 10 parameters (over the 6 events) for each of the study areas: Digla, Maadi proper, and New Maadi. The parameters presented in these figures are: pH, TDS, turbidity, alkalinity, total chlorine residual, TTHM, total hardness, fluoride, nitrate, and iron.

Table 4.12 presents average residential tap water quality for summer and winter seasons, of all residential locations in Digla, Maadi-proper, and New Maadi. It indicates that TDS levels are higher in winter than in summer, with a slight increase in total hardness. As mentioned in Section 4.1.2, TDS variations in residential locations did not correlate to variations in TDS levels in the Nile water (Table 4.2), which leads to the conclusion that these variations need to be investigated more closely in terms of the reason for their occurrence. Other parameters such as pH, turbidity, total chlorine residual, TTHMs, fluoride, nitrate, and iron, have had similar values for summer and winter. This is also illustrated in Figures 4.9 through 4.18, which emphasize the interpretation that, in spite of the variation of these parameters between successive events, these variations are not clearly related with a seasonal pattern.

Table 4.9: Mean concentrations in tap water for 6 events

Analysis	Units	Digla	Maadi proper	New Maadi	Egyptian Criteria	WHO Criteria	U.S. EPA Criteria
Temperature	^{0}C	23 ±5	24 ±5	23 ±4			
pH	pH	7.52 ±0.15	7.43 ±0.13	7.45 ±0.35	6.5-9.2	6.5-8.5	6.5-8.5
TDS	mg/L	177 ±22	177 ±22	178 ±22	1,200	1,000	500
Turbidity	FTU	2 ±2	1 ±1	1 ±1	5	5	0.3 -1.0
Alkalinity	mg/L	121 ±13	120 ±12	123 ±11	-	-	-
Total Hardness	mg/L as $CaCO_3$	94 ±13	91 ±10	94 ±10	500	500	-
Calcium	mg/L	21 ±2	20 ±2	21 ±2	200	-	-
Mg	mg/L	10 ±2	10 ±2	10 ±2	150	-	-
Iron	mg/L	0.046 ±0.033	0.052 ±0.044	0.075 ±0.081	0.3	0.3	0.3
Lead	mg/L	0.007 ±0.009	0.011 ±0.020	0.011 ±0.025	0.05	0.01	0.015
Copper	mg/L	0.001 ±0.002	0.001 ±0.001	0.009 ±0.021	1.0	1.0	1.0
Total Chlorine	mg/L	1.65 ±0.67	1.63 ±0.61	1.67 ±0.91	-	5	4.0
Nitrate	mg/L	0.04 ±0.03	0.03 ±0.01	0.03 ±0.01	10 (as Nitrogen)	50 (as nitrate)	10 (as Nitrogen)
Fluoride	mg/L	0.48 ±0.06	0.44 ±0.07	0.38 ±0.09	0.8	1.5	2.0
Total coliforms	No./100 mL	0 ±0	0 ±0	0 ±0	5%	Nil	5% positive/month
Fecal coliforms	No./100 mL	0 ±0	0 ±0	0 ±0	Nil	Nil	Nil
TTHMs	µg/L	73 ±18	73 ±21	84 ±16	100	-	80

Table 4.10: Max. & Min. concentrations in tap water for 6 Events

Analysis	Units	Digla		Maadi		New Maadi		Egypt. Criteria	WHO Criteria	U.S. EPA Criteria
		Min	Max	Min	Max	Min	Max			
Temp.	^0C	15	30	11	32	16	28			
PH	pH	6.84	7.91	6.89	8.2	7	8.07	6.5-9.2	6.5-8.5	6.5-8.5
TDS	mg/L	106	224	149	223	157	215	1,200	1,000	500
Turbidity	FTU	0	4	0	3	0	2	5	5	0.3 -1.0
Alkalinity	mg/L as $CaCO_3$	95	164	98	146	102	134	-	-	-
Total Hardness		76	128	80	116	82	116	500	500	-
Calcium	mg/L	16	27	16	25	17	24	200	-	-
Mg	mg/L	6	17	7	15	8	15	150	-	-
Iron	mg/L	0	0.2467	0	0.2172	0	0.1989	0.3	0.3	0.3
Lead	mg/L	0	0.0709	0	0.0711	0	0.0708	0.05	0.01	0.015
Copper	mg/L	0	0.04742	0	0.041	0	0.1048	1.0	1.0	1.0
Arsenic	mg/L	0	0	0	0	0	0	0.05	0.01	0.01
Chlorine	mg/L	0.22	4.85	0.46	3.75	0.38	3.3	-	5.0	4.0
Nitrate	mg/L	0.02	0.13	0	0.13	0	0.06	10 (as Nitrogen)	50 (as nitrate)	10 (as Nitrogen)
Fluoride	mg/L	0.2	0.71	0.3	0.66	0.25	0.54	0.8	1.5	2.0
Total col.	/100 ml	0	1	0	3	0	1	5%	Nil	5% /month
Fecal col.	/100 ml	0	4	0	1	0	1	Nil	Nil	Nil
TTHMs	μg/L	22.3	106.7	35.9	111.6	42.3	112.2	100	-	80

Table 4.11: Occurrence of values exceeding water quality standards

Analysis	Units	Event	#of cases	Locations	Range	EPA	WHO
Turbidity	FTU	1	17	2, 3, 4, 5, 6, 8, 9,10, 11, 12, 13, 14,15, 16, 17, 18, 19	2 to 4	0.3 to 1.0	5
		2	5	2, 6, 10, 13, 14	2 to 4		5
		3	4	3, 6, 7, 18	2 to 4		5
		4	8	1, 7, 9, 10, 16, 17,19, 20	2		5
		5	1	12	2		5
		6	7	1, 2, 3, 5, 6, 13, 18	2		5
Lead	mg/L	1	5	1, 6, 16, 17, 18	0.018-0.04	0.015	0.01
		2	3	3, 7, 16	0.016-0.06	0.015	0.01
		3	16	4, 5, 6, 8, 9,10,11, 12,13,14,15, 16, 17, 18,19,20	0.012-0.071	0.015	0.01
		5	1	9	0.022	0.015	0.01
TTHMs	μg/L	1	2	19,20	94.8-106.7	80	-
		2	1	11	82.69	80	-
		3	10	2,5,6,7,10,15,17,18,19,20	81 to 112.2	80	-
		4	3	2,17,19	81.7-104.1	80	-
		5	13	6,8,9,10,11,12,13,14,15,16,18,19,20	84 to 111.6	80	-
		6	16	1,2,3,4,5,6,8,10,11,12,14, 15,16,17,18,20	87.9-109.5	80	-
Fecal col.	/100 ml	1	1	6	1	Nil	Nil
		2	1	19	1	Nil	Nil
		6	1	4	4	Nil	Nil

Table 4.12: Average residential tap water quality for summer (Events 1, 3, 4, and 6) versus winter (Events 2, 5)

Analysis	Units	Average of residential locations	
		Summer	Winter
pH	pH	7.48 ± 0.24	7.44 ± 0.25
TDS	mg/L	165 ± 13	201 ± 18
Turbidity	FTU	1 ± 1	1 ± 1
Alkalinity	mg/L as $CaCO_3$	118 ± 16	129 ± 12
Total Hardness	mg/L as $CaCO_3$	87 ± 6	105 ± 10
Calcium	mg/L	20 ± 2	21 ± 2
Mg	mg/L	9 ± 1	12 ± 2
Total Chlorine	mg/L	1.67 ± 0.99	1.59 ± 0.76
TTHMs	µg/L	75.27 ± 23.16	73.43 ± 17.21

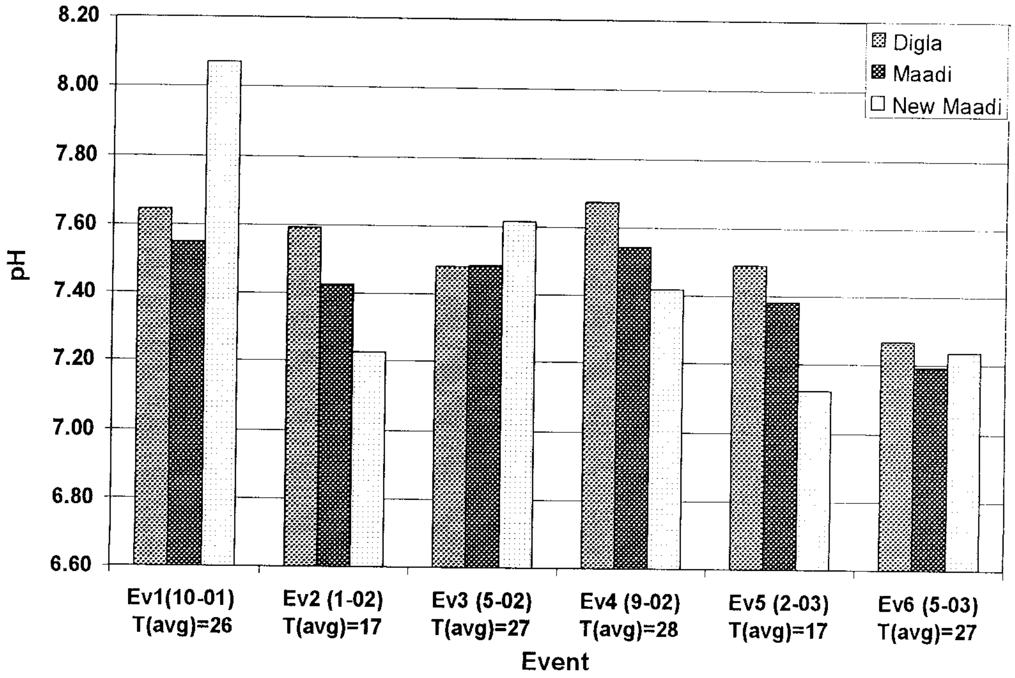

Figure 4.9: Average pH values for the 6 events (T. is in °C)

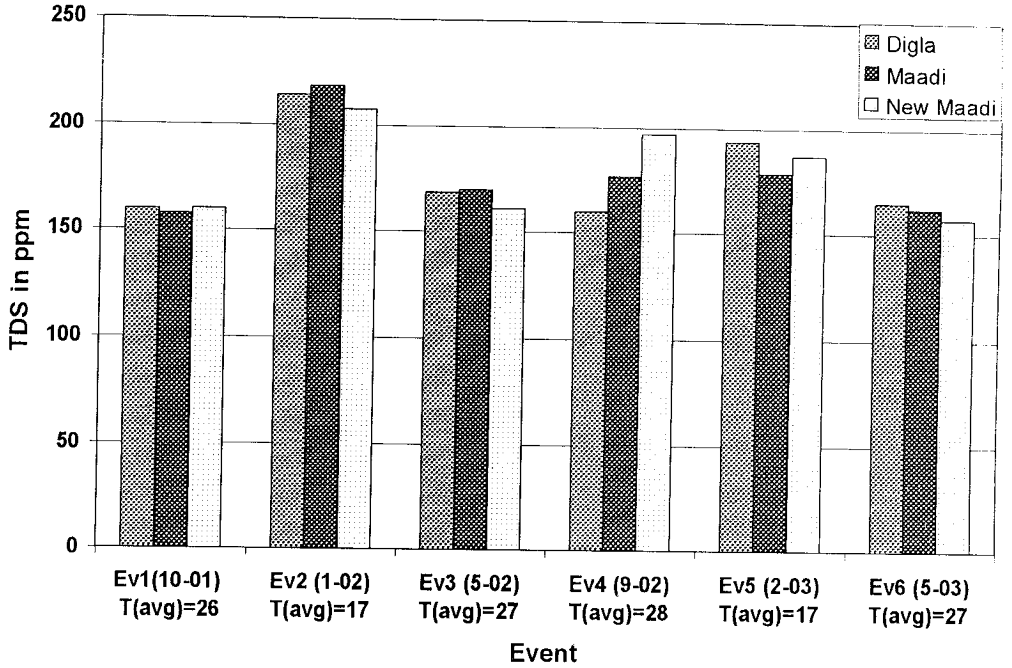

Figure 4.10: Average TDS values for the 6 events (T. is in °C)

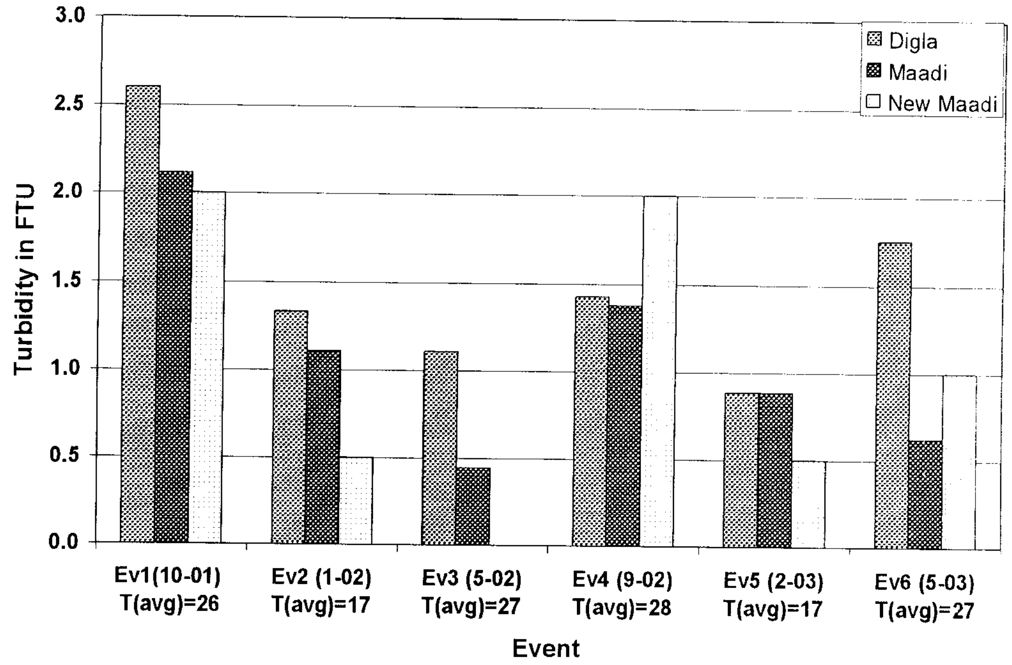

Figure 4.11: Average Turbidity values for the 6 events (T. is in °C)

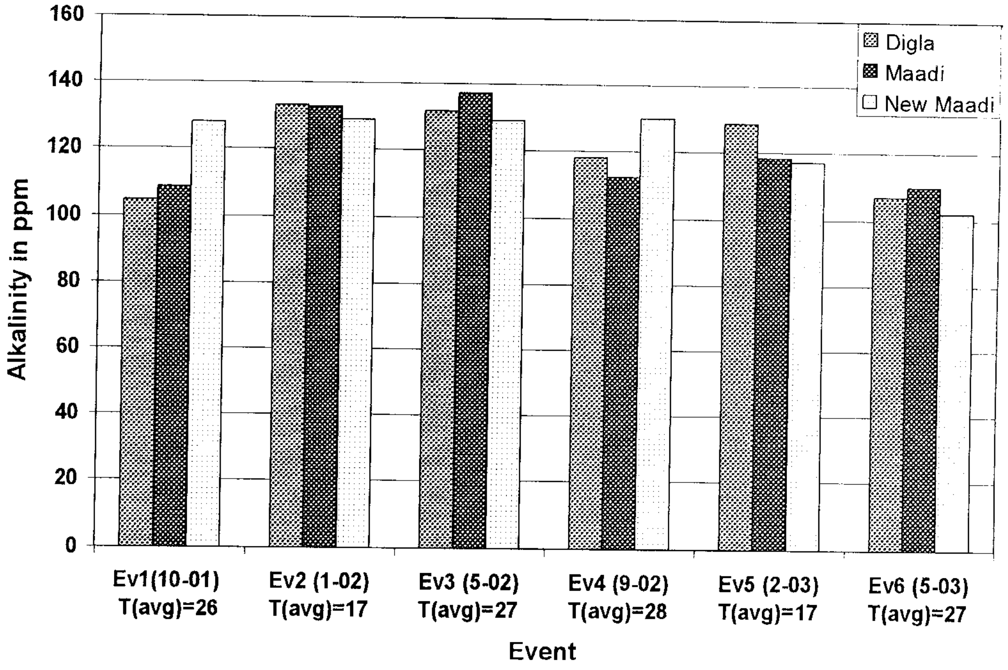

Figure 4.12: Average Alkalinity values for the 6 events (T. is in °C)

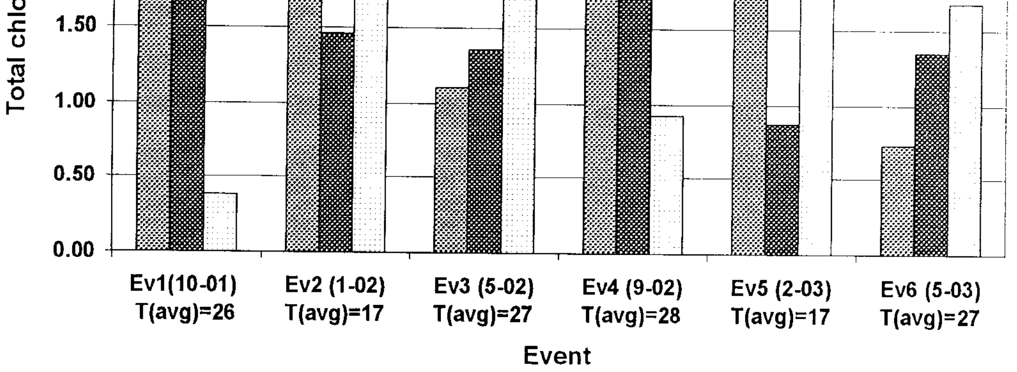

Figure 4.13: Average Total Chlorine Residual values for the 6 events (T. is in °C)

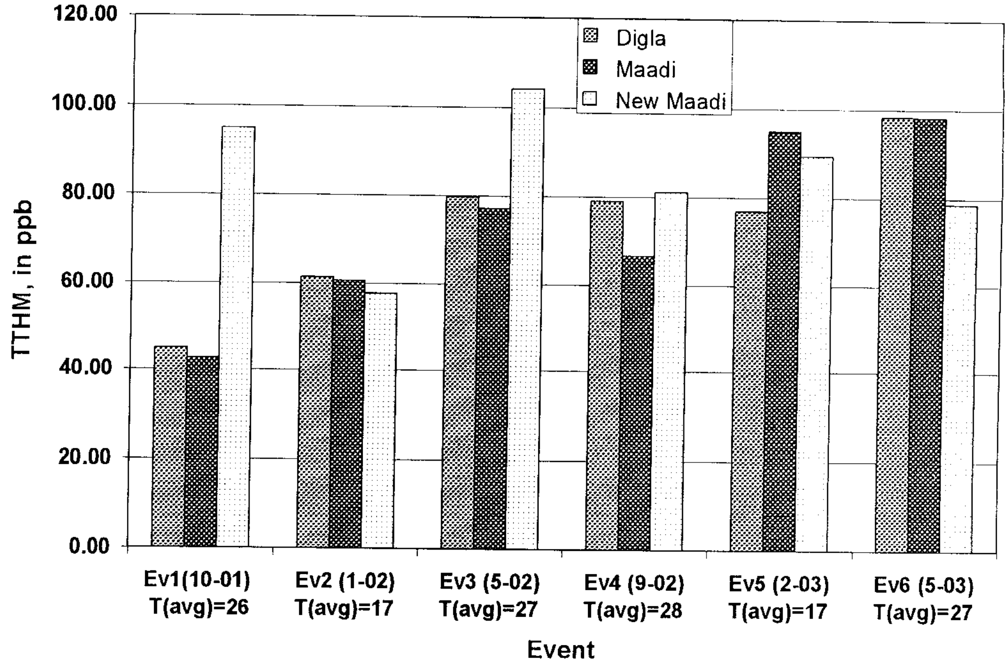

Figure 4.14: Average TTHM values for the 6 events (T. is in °C)

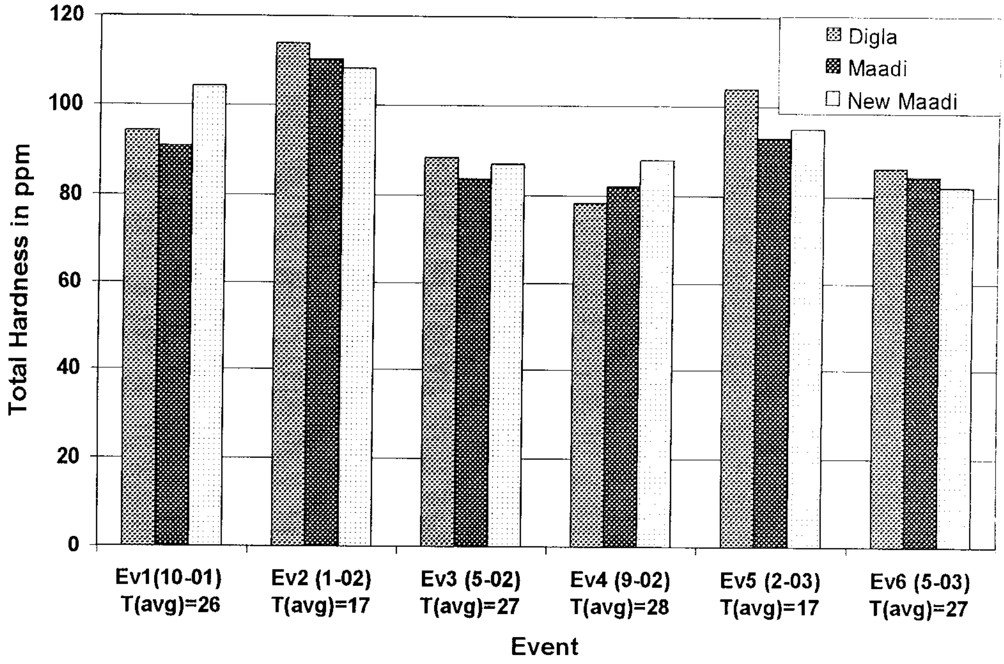

Figure 4.15: Average Total Hardness values for the 6 events (T. is in °C)

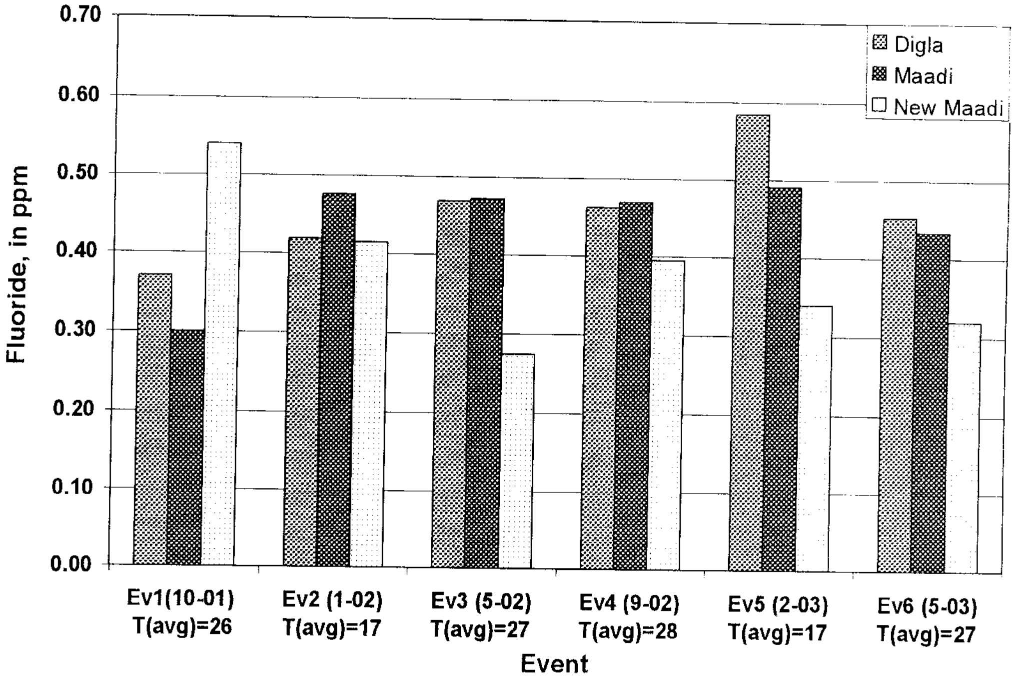

Figure 4.16: Average Fluoride values for the 6 events (T. is in °C)

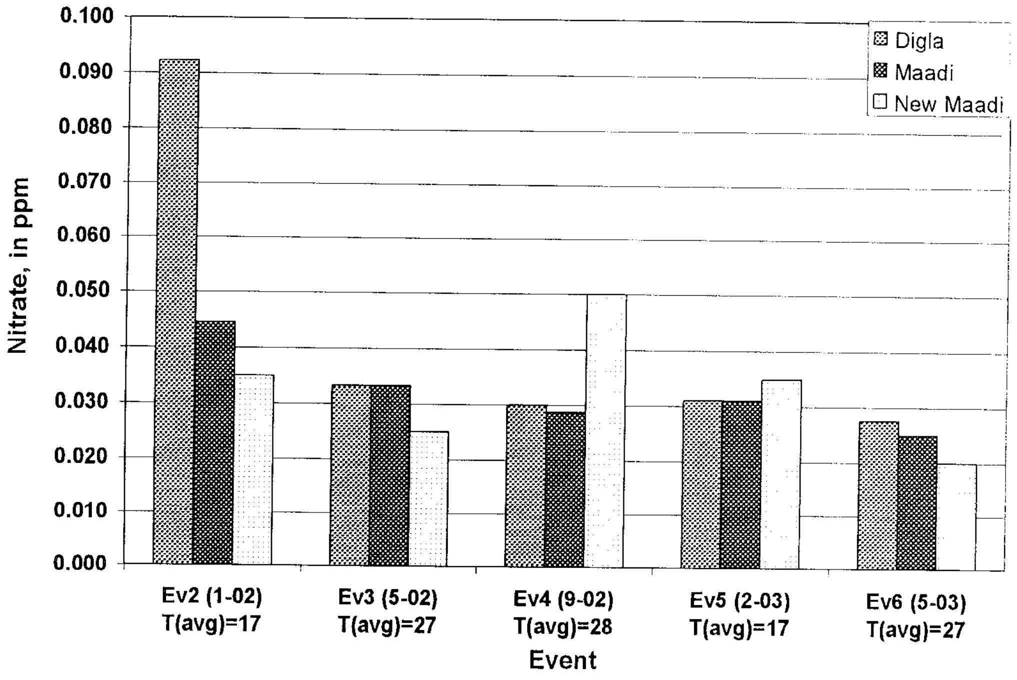

Figure 4.17: Average Nitrate values for Events 2 through 6 (T. is in °C)

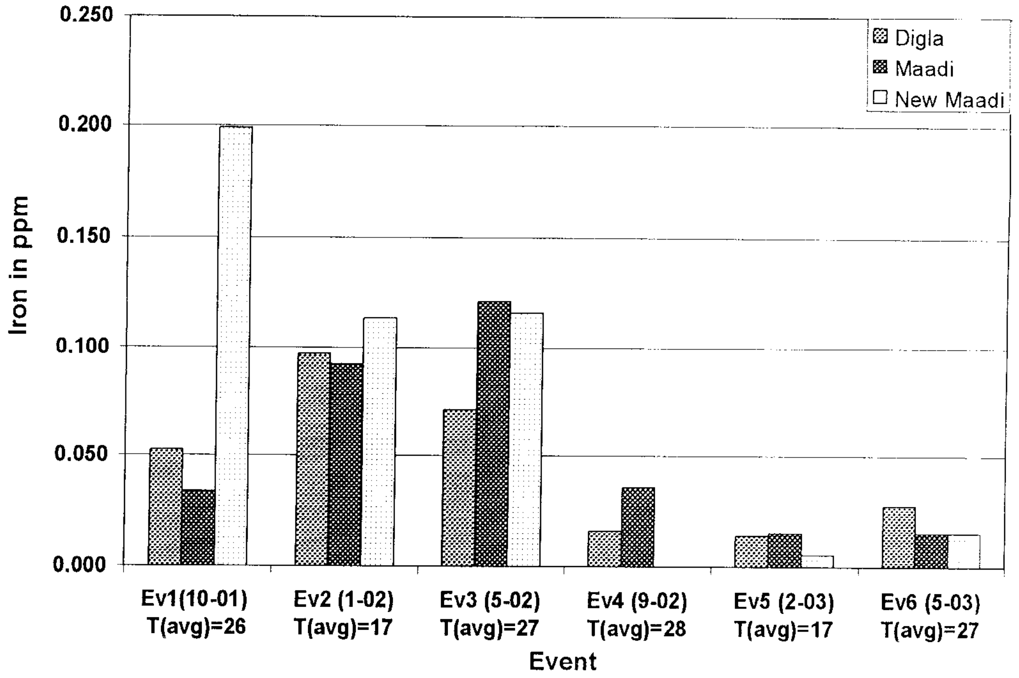

Figure 4.18: Average Iron values for the 6 events (T. is in °C)

4.2 Effect of the distribution system on water quality parameters

In this section, the changes in water quality parameters as a result of transport through the distribution system are investigated. Table 4.13, 4.14, and 4.15 present a comparison (for each event) between the effluent water quality of Dar-El-Salaam WTP and tap water quality in Digla, Maadi proper, and New Maadi, respectively. However, as mentioned earlier, sampling was conducted at the WTP on a different day than the tap locations in the residential areas. Operating conditions at the WTP are subject to variations that can produce substantial variation in results, especially for chlorine and TTHMs (as shown in Table 4.8: variation in TOC removal efficiency). Also, based on discussions with engineers in the water distribution system office, it was concluded that operating conditions in the distribution system are subject to irregular variations, mainly due to the fact that the water distribution network is an integral network, in which flows are adjusted to fulfill the demand of the served areas. Therefore, a comparison of the average water quality parameters over the 6 events would be useful to dampen the effect of such variations in operating conditions, as presented in Table 4.16.

Table 4.13, 4.14, and 4.15 depict some variations in the following water quality parameters: turbidity, iron, lead, copper, chlorine residuals, and TTHM. However, due to the variation in operating conditions mentioned above, the pattern of the variation of water quality is not very clear in these tables.

Table 4.16 gives a better indication of the variation of these parameters.

- Minimal increase occurred in turbidity, iron, lead, and copper levels. The most likely reason for this slight increase is the possible corrosion in pipes.
- Total chlorine levels were found to be lower in residential locations than in the WTP effluent. This suggests a decay in chlorine residual, which was also expected based on literature review, (Clark *et al.*, 1998; Abd-El-Shafy and Grűnwald, 2000; Kastl *et al.* 2002).
- TTHM levels exhibit some increase from the WTP to the residential locations. This also conforms to previous research (Clark *et al.*, 1998; Abd-El-Shafy and Grűnwald, 2000; Kastl *et al.* 2002). However, the dependence of the studied system on local versus municipal storage results in lower residence times, meaning less overall conversion, which is positive in a way that less TTHMs are allowed to be formed in the distribution system and chlorine levels are kept as well at moderate levels on the average of 1.65 mg/L.

Table 4.13: Comparison of water quality in Digla with the effluent of Dar-El-Salaam WTP for each event

Analysis	Units	Ev. 1 D.S. WTP	Ev. 1 Digla	Ev. 2 D.S. WTP	Ev. 2 Digla	Ev. 3 D.S. WTP	Ev. 3 Digla	Ev. 4 D.S. WTP	Ev. 4 Digla	Ev. 5 D.S. WTP	Ev. 5 Digla	Ev. 6 D.S. WTP	Ev. 6 Digla
Temp	^{0}C	27	26 ±3	17	17 ±2	30	25 ±1	25	28 ±1	26	16 ±1	26	27 ±2
pH	pH	7.2	7.65 ±0.20	7.06	7.59 ±0.21	7.31	7.48 ±0.12	7.15	7.67 ±0.12	7.43	7.49 ±0.15	7.55	7.27 ±0.18
TDS	mg/L	160	160 ±5	208	214 ±5	161	168 ±8	210	160 ±26	190	194 ±17	212	165 ±7
Turbidity	FTU	0	3 ±1	1	1 ±1	0	1 ±1	0	1 ±1	1	1 ±0	1	2 ±0
Alkalinity	mg/L as CaCO3	116	105 ±12	124	133 ±6	132	132 ±5	142	118 ±22	130	128 ±7	116	107 ±3
Total Hardness		108	94 ±6	114	114 ±6	88	88 ±4	86	78 ±5	98	104 ±9	124	86 ±2
Calcium	mg/L	23	22 ±2	22	24 ±1	19	20 ±2	22	20 ±1	21	21 ±1	22	17 ±1
Mg	mg/L	12	9 ±1	14	13 ±2	10	9 ±1	8	7 ±1	11	12 ±2	17	11 ±1
Iron	mg/L	0.000	0.052 ±0.079	0.091	0.097 ±0.045	0.123	0.071 ±0.019	0.041	0.016 ±0.020	0.008	0.014 ±0.021	0.000	0.028 ±0.015
Lead	mg/L	0.000	0.007 ±0.015	0.000	0.010 ±0.020	0.000	0.023 ±0.027	0.000	0.000 ±0.000	0.000	0.000 ±0.000	0.002	0.000 ±0.000
Copper	mg/L	0.000	0.005 ±0.016	0.000	0.000 ±0.000	0.000	0.000 ±0.000	0.000	0.000 ±0.000	0.000	0.000 ±0.000	0.000	0.000 ±0.000
Arsenic	mg/L	0.000	0.000 ±0.000	0.000	0.000 ±0.000								
Total Chlorine	mg/L	1.34	2.64 ±1.59	2.14	1.79 ±0.59	2.13	1.11 ±0.55	2.10	1.77 ±0.54	2.00	1.87 ±0.91	2.08	0.73 ±0.31
Nitrate (as N)	mg/L	0.05		0.05	0.09 ±0.04	0.03	0.03 ±0.01	0.03	0.03 ±0.00	0.04	0.03 ±0.01	0.02	0.03 ±0.01
Fluoride	mg/L			0.41	0.42 ±0.11	0.28	0.47 ±0.11	0.31	0.46 ±0.07	0.54	0.58 ±0.09	0.38	0.45 ±0.11
Total col.	/100 mL	0	0 ±0	0	0 ±0	0	0 ±0	0	0 ±0	0	0 ±0	0	0 ±0
Fecal col.	/100 mL	0	0 ±0	0	0 ±0	0	0 ±0	0	0 ±0	0	0 ±0	0	1 ±1
TTHMs	µg/L	51.1	44.8 ±25.6	60.0	61.3 ±8.3	85.3	79.7 ±8.3	91.2	79.0 ±12.0	62.4	76.9 ±11.6	72.5	98.4 ±10.0

Table 4.14: Comparison of water quality in Maadi proper with the effluent of Dar-El-Salaam WTP for each event

Analysis	Units	Ev. 1 D.S. WTP	Ev. 1 Maadi	Ev. 2 D.S. WTP	Ev. 2 Maadi	Ev. 3 D.S. WTP	Ev. 3 Maadi	Ev. 4 D.S. WTP	Ev. 4 Maadi	Ev. 5 D.S. WTP	Ev. 5 Maadi	Ev. 6 D.S. WTP	Ev. 6 Maadi
temp	^{0}C	27	27 ±2	17	16 ±3	30	26 ±1	25	28 ±2	26	18 ±3	26	27 ±2
pH	pH	7.2	7.55 ±0.15	7.06	7.42 ±0.19	7.31	7.48 ±0.19	7.15	7.54 ±0.09	7.43	7.38 ±0.34	7.55	7.19 ±0.32
TDS	mg/L	160	157 ±9	208	219 ±3	161	169 ±7	210	177 ±11	190	178 ±9	212	162 ±4
Turbidity	FTU	0	2 ±1	1	1 ±1	0	0 ±1	0	1 ±1	1	1 ±1	1	1 ±1
Alkalinity	mg/L as $CaCO_3$	116	108 ±5	124	133 ±9	132	137 ±4	142	112 ±12	130	118 ±5	116	110 ±1
Total Hardness		108	91 ±5	114	110 ±5	88	84 ±4	86	82 ±2	98	93 ±4	124	84 ±1
Calcium	mg/L	23	20 ±1	22	22 ±2	19	17 ±2	22	20 ±1	21	20 ±1	22	18 ±1
Mg	mg/L	12	10 ±1	14	13 ±1	10	10 ±1	8	8 ±1	11	11 ±1	17	9 ±1
Iron	mg/L	0.000	0.033 ±0.063	0.091	0.092 ±0.090	0.123	0.121 ±0.037	0.041	0.036 ±0.054	0.008	0.015 ±0.011	0.000	0.015 ±0.008
Lead	mg/L	0.000	0.008 ±0.013	0.000	0.002 ±0.005	0.000	0.052 ±0.009	0.000	0.000 ±0.000	0.000	0.005 ±0.008	0.002	0.000 ±0.000
Copper	mg/L	0.000	0.000 ±0.000	0.000	0.005 ±0.014	0.000	0.000 ±0.000	0.000	0.000 ±0.000	0.000	0.000 ±0.000	0.000	0.000 ±0.000
Arsenic	mg/L	0.000	0.000 ±0.000	0.000	0.000 ±0.000								
Total Chlorine	mg/L	1.34	2.51 ±1.21	2.14	1.46 ±0.34	2.13	1.36 ±0.48	2.10	2.25 ±0.27	2.00	0.87 ±0.35	2.08	1.35 ±0.25
Nitrate (as Nitrogen)	mg/L	0.05		0.05	0.04 ±0.04	0.03	0.03 ±0.01	0.03	0.03 ±0.00	0.04	0.03 ±0.01	0.02	0.03 ±0.01
Fluoride	mg/L			0.41	0.48 ±0.09	0.28	0.47 ±0.12	0.31	0.47 ±0.09	0.54	0.49 ±0.10	0.38	0.43 ±0.05
Total col.	/100 mL	0	0 ±1	0	0 ±0	0	0 ±0	0	0 ±0	0	0 ±0	0	0 ±0
Fecal col.	/100 mL	0	0 ±0	0	0 ±0	0	0 ±0	0	0 ±0	0	0 ±0	0	0 ±0
TTHMs	µg/L	51.1	42.6 ±5.1	60.0	60.5 ±9.3	85.3	77.1 ±15.6	91.2	66.6 ±6.7	62.4	94.9 ±9.0	72.5	98.2 ±7.0

Table 4.15: Comparison of water quality in New Maadi with the effluent of Dar-El-Salaam WTP for each event

Analysis	Units	Ev. 1	Ev. 1	Ev. 2	Ev. 2	Ev. 3	Ev. 3	Ev. 4	Ev. 4	Ev. 5	Ev. 5	Ev. 6	Ev. 6
		D.S. WTP	**New Maadi**	**D.S. WTP**	**New Maadi**	**D.S. WTP**	**New Maadi**	**D.S. WTP**	**New Maadi**	**D.S. WTP**	**New Maadi**	**D.S. WTP**	**New Maadi**
temp	^{0}C	27	25	17	18 ±0	30	28 ±0	25	26 ±0	26	18 ±2	26	26 ±1
pH	pH	7.2	8.07	7.06	7.23 ±0.25	7.31	7.62 ±0.12	7.15	7.42 ±0.01	7.43	7.12 ±0.17	7.55	7.24 ±0.05
TDS	mg/L	160	160	208	208 ±11	161	161 ±2	210	197 ±1	190	187 ±3	212	157 ±0
Turbidity	FTU	0	2	1	1 ±1	0	0 ±0	0	2 ±0	1	1 ±1	1	1 ±0
Alkalini-ty	mg/L as CaCO3	116	128	124	129 ±7	132	129 ±4	142	130 ±0	130	117 ±1	116	102 ±0
Total Hardness		108	104	114	108 ±11	88	87 ±1	86	88 ±0	98	95 ±4	124	82 ±0
Calcium	mg/L	23	24	22	22 ±0	19	19 ±0	22	21 ±1	21	21 ±1	22	17 ±0
Mg	mg/L	12	11	14	13 ±3	10	9 ±0	8	8 ±0	11	10 ±0	17	10 ±0
Iron	mg/L	0.000	0.199	0.091	0.113 ±0.098	0.123	0.116 ±0.000	0.041	0.000 ±0.000	0.008	0.006 ±0.008	0.000	0.015 ±0.012
Lead	mg/L	0.000	0.000	0.000	0.003 ±0.005	0.000	0.061 ±0.013	0.000	0.000 ±0.000	0.000	0.000 ±0.000	0.002	0.000 ±0.000
Copper	mg/L	0.000	0.000	0.000	0.000 ±0.000	0.000	0.000 ±0.000	0.000	0.052 ±0.074	0.000	0.000 ±0.000	0.000	0.000 ±0.000
Arsenic	mg/L	0.000	0.000	0.000	0.000 ±0.000								
Total Chlorine	mg/L	1.34	0.38	2.14	2.92 ±0.54	2.13	2.22 ±0.48	2.10	0.92 ±0.34	2.00	1.89 ±1.23	2.08	1.68 ±0.26
Nitrate (as Nitrogen)	mg/L	0.05		0.05	0.04 ±0.01	0.03	0.03 ±0.01	0.03	0.05 ±0.01	0.04	0.04 ±0.01	0.02	0.02 ±0.00
Fluoride	mg/L		0.54	0.41	0.42 ±0.09	0.28	0.28 ±0.04	0.31	0.40 ±0.21	0.54	0.34 ±0.06	0.38	0.32 ±0.03
Total col.	/100 mL	0	0 ±0	0	1 ±1	0	0 ±0	0	0 ±0	0	0 ±0	0	0 ±0
Fecal col.	/100 mL	0	0 ±0	0	1 ±1	0	0 ±0	0	0 ±0	0	0 ±0	0	0 ±0
TTHMs	µg/L	51.1	94.8	60.0	57.8 ±21.8	85.3	104.0 ±11.7	91.2	81.1 ±30.3	62.4	89.5 ±5.0	72.5	78.7 ±18.0

Table 4.16: Comparison of water quality in the three study areas with the effluent of Dar-El-Salaam WTP, averaged for the 6 events

Analysis	Units	WTP Effluent (Mean)	Digla*	Maadi*	New Maadi*
Temp.	^{0}C	25 ±4	23 ±5	24 ±5	23 ±4
pH	pH	7.28 ±0.18	7.52 ±0.15	7.43 ±0.13	7.45 ±0.35
TDS	mg/L	190 ±24	177 ±22	177 ±22	178 ±22
Turbidity	FTU	1 ±1	2 ±2	1 ±1	1 ±1
Alkalinity	mg/L	127 ±10	121 ±13	120 ±12	123 ±11
Total Hardness	mg/L as CaCO$_3$	103 ±15	94 ±13	91 ±10	94 ±10
Calcium	mg/L	21 ±1	21 ±2	20 ±2	21 ±2
Mg	mg/L	12 ±3	10 ±2	10 ±2	10 ±2
Iron	mg/L	0.044 ±0.05	0.046 ±0.033	0.052 ±0.044	0.075 ±0.081
Lead	mg/L	0.000 ±0.00	0.007 ±0.009	0.011 ±0.020	0.011 ±0.025
Copper	mg/L	0.000 ±0.00	0.001 ±0.002	0.001 ±0.001	0.009 ±0.021
Arsenic	mg/L	0.000 ±0.00			
Total Chlorine	mg/L	1.96 ±0.31	1.65 ±0.67	1.63 ±0.61	1.67 ±0.91
Nitrate (as Nitrogen)	mg/L	0.04 ±0.01	0.04 ±0.03	0.03 ±0.01	0.03 ±0.01
Fluoride	mg/L	0.38 ±0.10	0.48 ±0.06	0.44 ±0.07	0.38 ±0.09
Total coliforms	No./100 mL	0	0 ±0	0 ±0	0 ±0
Fecal coliforms	No./100 mL	0	0 ±0	0 ±0	0 ±0
TTHMs	µg/L	70.4 ±15.5	73 ±18	73 ±21	84 ±16

* The standard deviation here is the deviation in means for individual events. This statistical approach was chosen in order to dampen the bias of individual measurements that could be due to individual storage at each location.

4.3 Relation between chlorine residuals and TTHM in the distribution system

Previous research has shown that the decay in chlorine residuals is associated with an increase in TTHM levels (Clark *et al.*, 1998; Abd-El-Shafy and Grünwald, 2000; Kastl *et al.* 2002). This relationship was investigated for the data for chlorine residuals and TTHMs in the distribution system studied. This relation is expressed in Figure 4.19 through 4.21. The correlation between actual concentrations of total chlorine residual and TTHMs was somewhat evident for Digla (R^2 = 0.819) and Maadi-proper (R^2 = 0.657), but not as clear for New Maadi (R^2 = 0.199). This may well be due to the fact that we only had two sampling locations in New Maadi versus 9 locations in each of Digla and Maadi-proper. If in fact such a correlation is established for a system, then an obvious application would be for a utility to carefully monitor and control chlorine residuals in treatment plant effluents as one of the best-practice control measures for TTHM levels in tap water

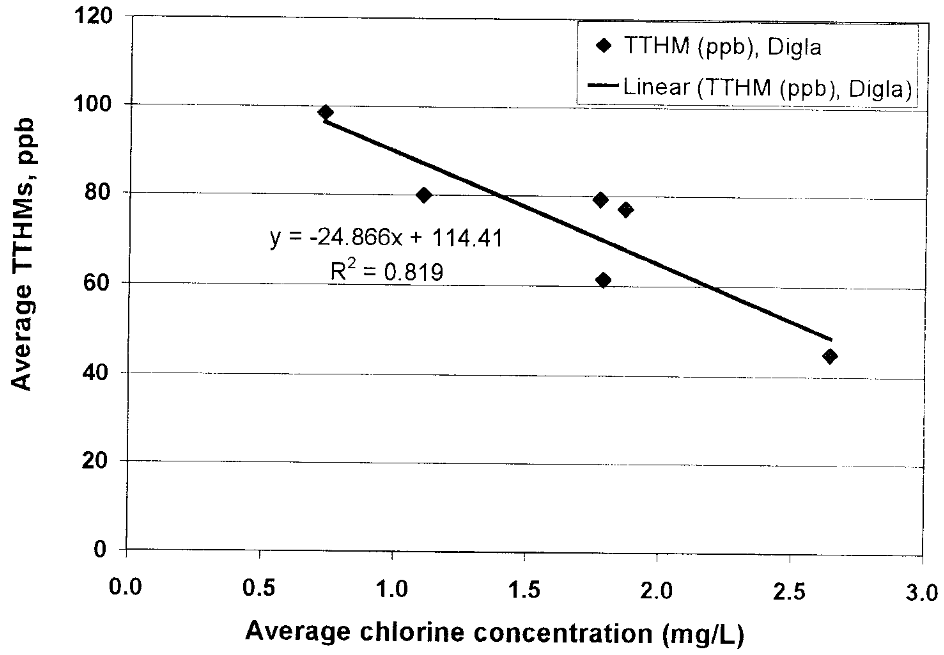

Figure 4.19: Correlation between total chlorine residual and TTHM levels for Digla.

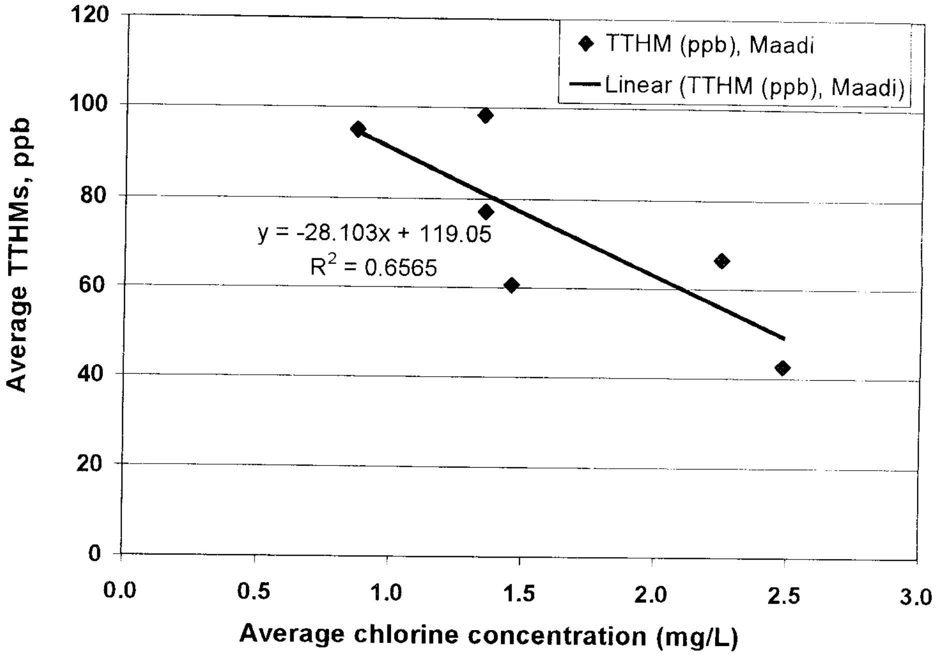

Figure 4.20: Correlation between total chlorine residual and TTHM levels for Maadi proper.

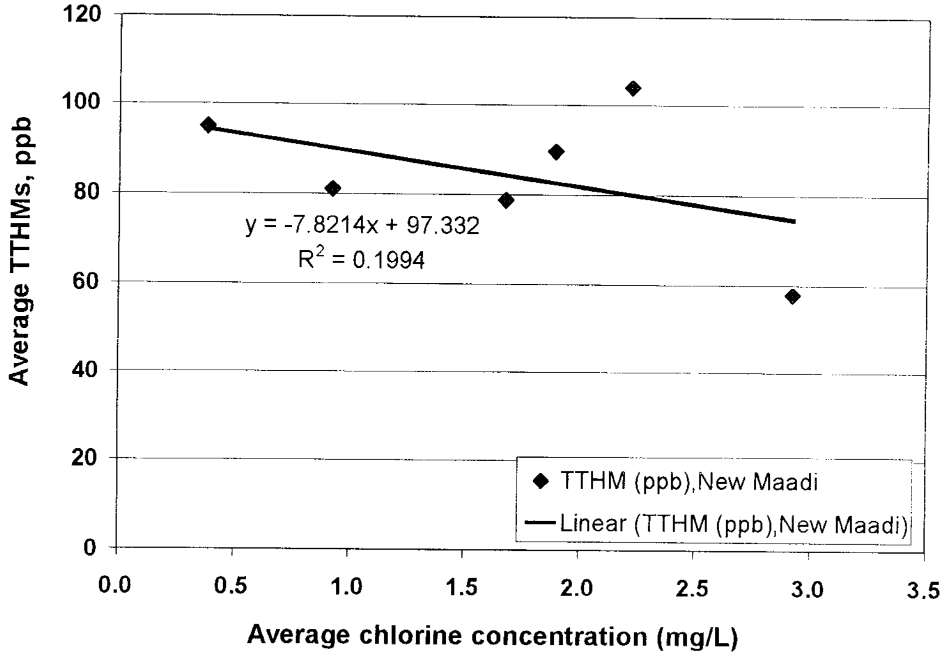

Figure 4.21: Correlation between total chlorine residual and TTHM levels for New Maadi.

4.4 Modeling Chlorine Residuals and TTHM Formation in the Distribution System

In a number of cases, TTHMs have shown higher levels than the allowable by standard criteria (Table 4.11). They are also associated with decay in the chlorine residual. Hence, it is important to model both chlorine residuals and TTHM in the distribution system, in order to:

- understand the reaction kinetics of both chlorine and TTHM in the distribution system,
- simulate or even predict the concentration profile of each of these two parameters in the distribution system,
- determine the optimum conditions at the water treatment plant effluent: *i.e.*, optimum chlorine dose that results in acceptable TTHM formation with no violation in the microbial safety of the system.

In this study, modeling of the system was limited to the first objective: understanding the kinetics of chlorine and TTHM variations in the distribution system. This is due to the fact that available data about the water distribution network did not allow satisfactory simulation of the network in its actual integral form.

Three approaches have been used in this study to investigate the kinetics of chlorine residuals and TTHMs. The first approach is based on the direct application of the first order model equation to chlorine residual and TTHM data collected from sampling locations; this approach does not distinguish between bulk and wall reactions, as it studies the overall reaction in the system. The second approach investigates the bulk reactions separately using the first order model. The third approach studies both bulk and wall reaction in the system by the application of EPANET modeling software.

4.4.1 Estimating the Kinetics of Chlorine Residuals and TTHMs in the Distribution System Using Model Equations

As mentioned in Chapter Two, the parallel first-order model has been found to give the best approximation of chlorine decay and TTHM formation in distribution systems (Lamia *et al.*, 2002; Kastl *et al.*, 2002). However, this model was not applicable to the present study, as the study did not investigate the ratios and kinetics of each of the fast and slow reducing agents in the natural organic material present in water.

The first order model was used by researchers in expressing the kinetics of chlorine decay (Rossman *et al.*, 2001; Clark *et al.*, 1995) and TTHM formation (Abd-El-Shafy and Grűnwald, 2000). In this study, the first order model was used to determine the overall chlorine decay rate constant and TTHM formation rate constant, based on the following data:

- Average value (over the 6 events) for total chlorine residual / TTHM in the effluent of Dar-El-Salaam WTP.
- Average value (over the 6 events) for total chlorine residual / TTHM in the Digla area. Two locations out of 11 were expected to depend completely on local storage and they were excluded from the calibration. Extremely high or low individual measurements were also excluded from the calibration (two measurements out of 47 for each of chlorine and TTHMs).

Overall chlorine decay rate

The equation used to express the chlorine decay rate is

$$C_t = C_o e^{-kt} \qquad (4.1)$$

Where C_t = the chlorine concentration at time t. For example, average value (over the 6 events) for chlorine residual in the Digla area = 1.63 mg/L.

C_o = the initial chlorine concentration. The average value (over the 6 events) for chlorine residual in the effluent of Dar-El-Salaam WTP = 1.96 mg/L.

k = the overall decay rate constant in time^{-1}

t = travel time of water from the WTP to an arbitrary point in Digla. Time was calculated on the basis of the estimated velocity of the water and the travel distance. Travel distance ~ 5350 m (this value was estimated as the total distance traveled by water in pipes from Dar-El-Salaam WTP to an arbitrary point in Digla, using the best available map of the distribution system).

Table 4.17 shows different estimations of overall decay rate constant based on different estimates of the average velocity in the distribution system. It is clear from the Table 4.17 that the decay rate constant calculation from field data needs an accurate estimate of the flow velocity. It should also be noted that the use of a larger number of measurements, (and not only measurements at the WTP and at the destination point) would help to give a better estimate of the decay rate constant.

Table 4.17: Overall chlorine decay rate constant for different estimates of the average velocity in the distribution system

v (m/s)	t (min)	t (day)	k (day-1)
0.75	118.89	0.0826	2.233
1	89.17	0.0619	2.977
1.25	71.33	0.0495	3.722
1.5	59.44	0.0413	4.466
1.75	50.95	0.0354	5.210

Overall TTHM formation rate

The equation used to express the TTHM formation rate is also a simple first order rate model.

$$TTHM_t = TTHM_o e^{kt} \qquad (4.2)$$

Where $TTHM_t$ = TTHM concentration at time t. For example, average value (over the 6 events) for TTHM in the Digla area = 73.42 ppb.

$TTHM_o$ = the initial TTHM concentration. The average value (over the 6 events) for TTHM in the effluent of Dar-El-Salaam WTP = 70.41 ppb,

k = the overall formation rate constant in $time^{-1}$

t = travel time of water from the WTP to an arbitrary point in Digla. As in the previous example, travel distance ~ 5350 m

Table 4.18 shows different estimations of overall formation rate constant based on different estimates of the average velocity in the distribution system.

Table 4.18: Overall TTHM formation rate constant for different estimates of the average velocity in the distribution system

v (m/s)	t (min)	t (day)	k (day^{-1})
0.75	118.89	0.0826	0.507
1	89.17	0.0619	0.677
1.25	71.33	0.0495	0.846
1.5	59.44	0.0413	1.015
1.75	50.95	0.0354	1.184

4.4.2 Bulk Water Kinetics of Chlorine Decay and TTHM Formation

Bulk water reaction rate constants for the decay of chlorine residual and TTHM formation are commonly determined from laboratory experiments (*e.g.*, bottle tests) (Rossman *et al.*, 2001). A bottle test (described in Chapter 3) was performed using water samples from the effluent of Dar-El-Salaam WTP in order to estimate the bulk reaction kinetics of chlorine residual and TTHM. Measurements for free and total chlorine residual during the experiment duration (seven days) have had a clear decay pattern, while TTHM data have had an asymptotic increase (Data from the experiment are presented in Table 4.19). The decay rate of free chlorine was found to be higher than that of total chlorine, which also complies with findings from previous research (Hart, 2002). It is to be noted that *Total Chlorine = Free Chlorine + Combined Chlorine*: in most cases, the combined chlorine is chloramines. However, Egyptian water utilities do not currently practice chloramination; this leads to little contribution of combined chlorine in tap water samples. In fact, the ratio of free chlorine to total chlorine in tap water was found to be around 0.8 (Table B-7, Appendix B).

Due to the absence of municipal storage in the area of study, the residence time in the network (from the WTP to Digla) does not exceed 4 hours, as will also be noted from the EPANET output (Section 4.4.3.9). Therefore, short term bulk reaction kinetics were determined on the basis of data for the first 4 hours in Table 4.19. The first order chlorine decay rate constant, k_b, was determined as the slope of $\ln(C_t/C_o)$ versus time, where C_t is concentration after time t and C_o is concentration at time 0 (Figure 4.22). Based on Figure 4.22, the bulk decay coefficient is 0.52 day^{-1} for total chlorine and 1.42 day^{-1} for free chlorine, during the first 4 hours. In the present study, modeling of chlorine residuals was mainly oriented towards total chlorine. Hence, the first order bulk decay rate constant used in the modeling of total chlorine residual was taken as 0.52 day^{-1}.

Table 4.19: Chlorine decay and TTHM formation data

Time (hr)	Time (day)	Free Chlorine (mg/L)	Total Chlorine (mg/L)	TTHM (μg/L)
0.00	0.00	3.00	3.30	45.1
1.00	0.04	2.72	3.18	45.7
1.50	0.06	2.64	3.16	46.4
2.50	0.10	2.60	3.08	49.3
4.00	0.17	2.32	3.02	50.1
4.50	0.19	2.20	2.96	54.0
5.75	0.24	2.08	2.94	59.0
7.00	0.29	2.00	2.72	56.9
24.00	1.00	1.52	2.06	71.5
32.00	1.33	1.48	1.97	69.1
48.00	2.00	1.16	1.68	70.1
56.00	2.33	1.00	1.50	87.5
72.00	3.00	0.87	1.34	94.5
80.00	3.33	0.82	1.29	98.3
96.00	4.00	0.75	1.20	109.7
120.00	5.00	0.65	1.08	121.0
128.00	5.33	0.56	0.94	111.7
144.00	6.00	0.44	0.83	99.8
168.00	7.00	0.20	0.40	103.8

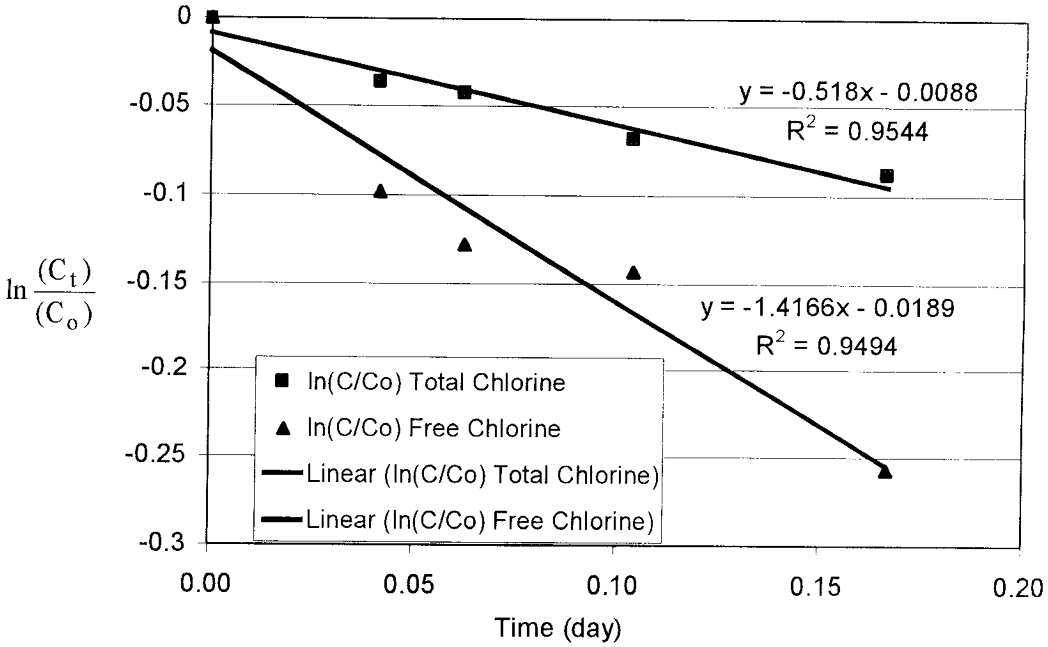

Figure 4.22: Determination of bulk chlorine decay rate constant.

The formation rate constant of TTHM was determined on the basis of laboratory data from the same experiment, using the first-order saturation growth equation based on the concept of allowing TTHM to grow to a maximum (or saturation) concentration (EPANET, 2002). This is expressed by the following equation:

$$\frac{dTTHM}{dt} = k_b(C_m - C)$$

where C_m = maximum formation concentration of TTHM. Figure 4.23 presents TTHM data (μg/L) plotted versus time (day). Fitting TTHM data results in an asymptotic curve with a maximum TTHM concentration of C_m = 118 μg/L. k_b is the slope of the plot of $\ln\frac{(C_m - C_o)}{(C_m - C_t)}$ versus time (Figure 4.24) for data points during the first 4 hours, where C_t is concentration after time t, C_o is concentration at time 0, and C_m = 118 μg/L. Based on Figure 4.24, the bulk TTHM formation rate constant ~ 0.48 day^{-1}.

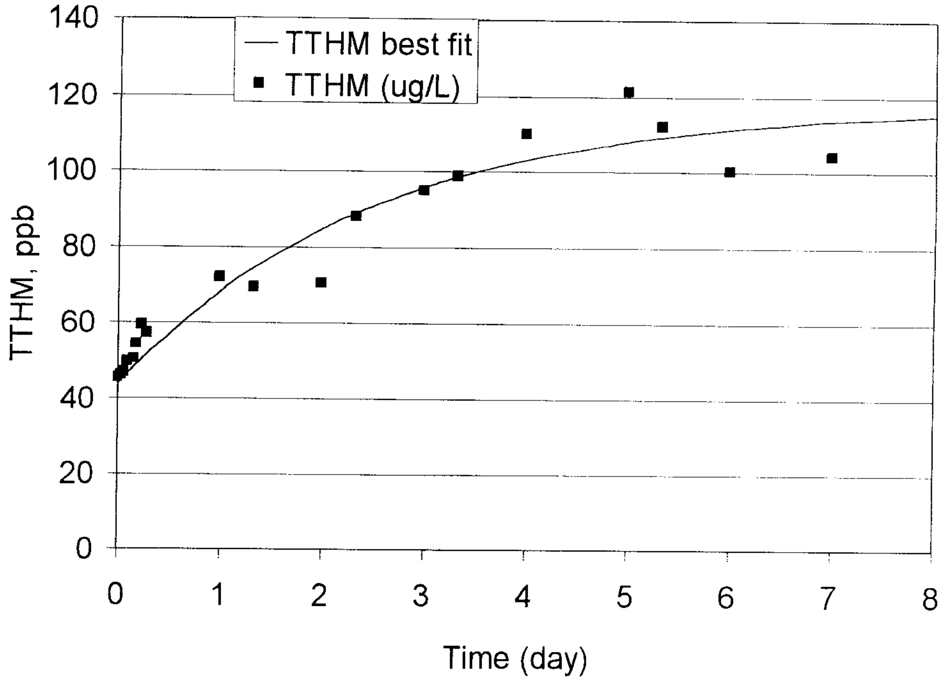

Figure 4.23: TTHM data versus time.

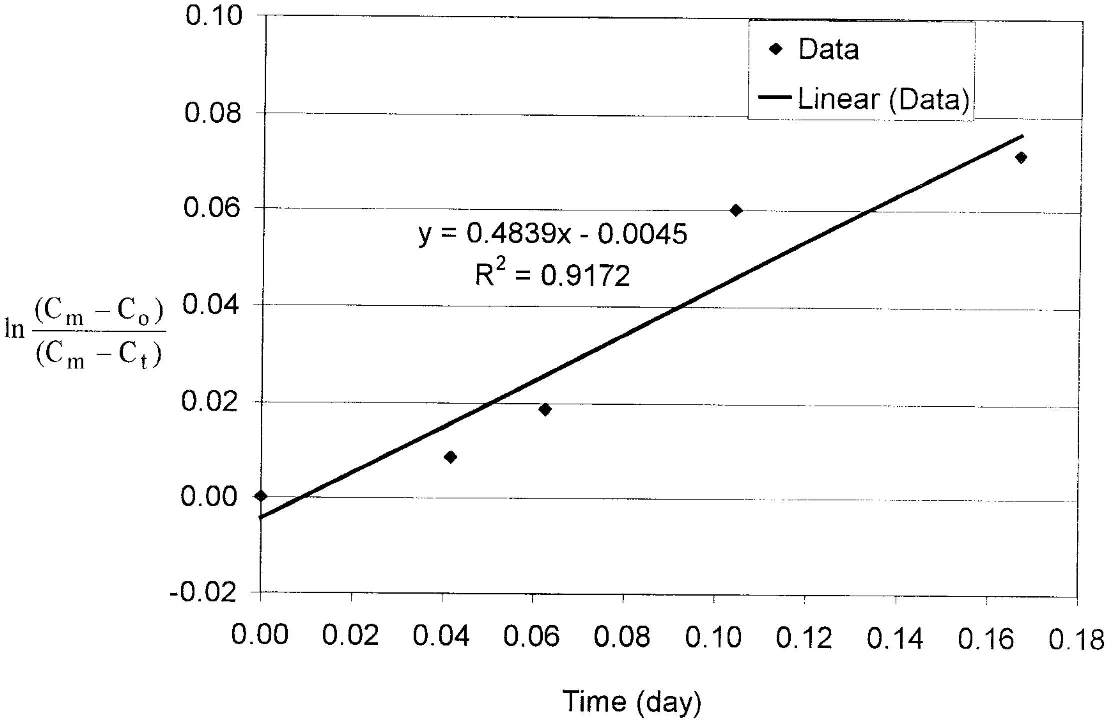

Figure 4.24: Determination of bulk TTHM formation rate constant.

It should be noted here that, in this experiment, the TTHM level at time = zero (in the effluent of the WTP) was 45.1 μg/L, while the average TTHM level in the effluent of the WTP over the study period (Sept. 2001 to June 2003) was 70.4 μg/L. It was assumed that the same formation rate applies to both TTHM levels. For the determination of the ultimate TTHM formation concentration required by EPANET, the results of another experiment performed in the same laboratory (Emam, 2003) & starting with a TTHM level of 77.0 μg/L (more comparable to the 70.4 μg/L TTHM average at the WTP) were used. Data for total chlorine and TTHM from the latter experiment are presented in Table 4.20, depicting an increase in TTHM to an asymptote of 134 μg/L (as presented in Figure 4.25). This value was used in the EPANET model as an estimate of the ultimate TTHM concentration.

Table 4.20: Total Chlorine and TTHM data (Emam, 2003)

Time (day)	Total Chlorine (mg/L)	TTHM (μg/L)
0	2.28	77.0
1	1.68	90.8
2	1.40	92.6
3	1.06	118.9
4	0.86	121.3
5	0.75	138.7
6	0.66	116.6
7	0.08	127.9

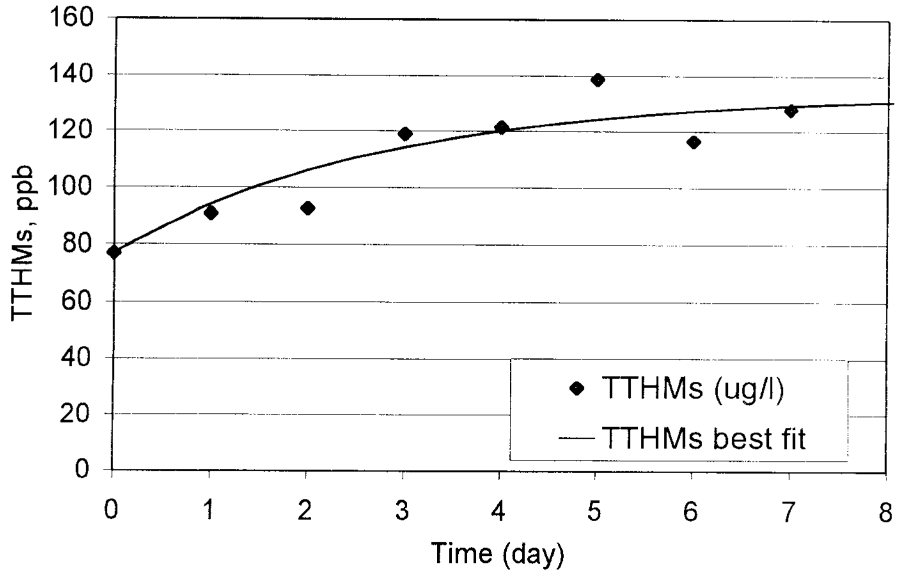

Figure 4.25: TTHM data versus time (Emam, 2003)

Typical values for bulk chlorine decay rate constant are in the range of 0.1 to 1.5 day^{-1} (Rossman, 1998; Hart, 2002); however values of 0.08 and 17.7 day^{-1} were reported in some case studies (Rossman, 1998). Typical values for bulk TTHM formation rate constant are in the range of 0.5 to 1.5 (day^{-1}) (Rossman, 2003). In addition to their being case-specific, reaction rate constants are temperature dependent. In some previous studies, they were doubled for every 10°C rise in temperature (Rossman, 1997). The bulk reaction rate constants estimated in the present study for chlorine (0.52 day^{-1} for total chlorine and 1.42 day^{-1} for free chlorine) and TTHM (0.48 day^{-1}) agree with typical ranges for both parameters. In a case study with source water TOC levels of 2.1 mg/L, pH of 8.0, initial free chlorine levels (in the water entering the distribution system simulator) ranging from 2.5 to 6.0 mg/L and temperature controlled between 21°C and 23°C, bulk free chlorine decay rate constant (calculated from a 24 hr bottle test) was found to range from 0.16 to 0.5 day^{-1} (Rossman *et al*., 2001). Overall chlorine decay rate constant was in the level of 3.7 day^{-1} (24 hr test for the distribution system simulator, which consisted of a 27-m long loop of 150 mm unlined ductile iron pipe). In the present study, the bulk decay rate constant for free chlorine over 24 hrs was found to be 0.62 day^{-1} (the slope of $\ln(C_t/C_o)$ versus time for free chlorine data of the first 24 hrs in Table 4.16), which is close to the values obtained by Rossman *et al* (2001). In a case study from the Czech Republic, described in Section 2.10.6 in Chapter 2 (Abd-El-Shafy and Grünwald, 2000), the total chlorine concentration in the water entering the distribution system was 0.75 mg/L, water was carried in uncoated steel pipes of about 80 km length, average temperature was 7.3°C, and average pH was 7.95, the residence time in the system was about 10 days (due to the low velocity in the pipes and the presence of large reservoirs), the overall decay rate constant for total chlorine (calculated from field data) was 0.3 day^{-1}, the bulk decay rate constant for total chlorine was 0.1 day^{-1}, and the wall decay rate constant was found to be 0.35 m/day (obtained by the calibration of the EPANET model, as will also be done in the present study). In the present study, the bulk decay rate constant for total chlorine over 7 days (using the 7 days data in Table 4.19) was found to be 0.25 day^{-1}, which is comparable to that obtained by Abd-El-Shafy and Grűnwald (2000). It was also noted in the present study that the ratio of the overall total chlorine decay rate constant (2.2-5.2 day^{-1} depending on average velocity) to the bulk constant (0.52 day^{-1}) ranges from 4 to 10 times. This has also been reported by other researchers, especially for systems including unlined iron or steel pipes which were found to exert higher chlorine demand than other materials such as PVC (Rossman *et al.*, 2001; Abd-El-Shafy and Grűnwald, 2000; Clark *et al.*, 1995). This is not the case for TTHM, where the ratio of overall TTHM formation rate constant (0.5-1.2 day^{-1} depending on average velocity) to bulk formation rate constant (0.48 day^{-1}) was found to range from ~1.04 to ~ 2.5. Rossman *et al* (2001) found that wall reactions contribute about 15% of the overall TTHM formation over a 24 hour period. These findings imply higher contribution of wall reactions for chlorine decay than for TTHM formation, consistent with the findings in this research. This can be justified by the fact that not all of the chlorine decay results in THM formation (THM formation depends on THM precursors, mainly organic material, attached at the pipe wall and not only on available chlorine concentration).

4.4.3 Modeling Chlorine Residuals and TTHM in The Distribution System Using EPANET Modeling Software

EPANET modeling software was used to represent the water distribution network, including both Maadi and Dar-El-Salaam WTPs. This software has been selected over other models for its ability to express and analyze hydraulics for large water networks and to efficiently model water quality reactions both in the bulk flow and at the pipe wall (Section 2.10.4). A presentation of the study area with the distribution system pipes and sampling locations is shown in Map A-1 in Appendix A. A presentation of the network with the junction identifications is shown in Map B-1 in Appendix B. Each water treatment plant was represented as a reservoir. The head at the reservoir was set equal to an estimated value of the water elevation at the WTP exit, which is one meter higher than the ground level at the WTP. Reservoirs coordinates and elevations are shown in Table B-1 in Appendix B. A more detailed discussion of the setup and implementation of the model is given in the following.

4.4.3.1 Water treatment plants presentation in the model

Dar-El-Salaam WTP

Daily production

- approximately 1.08 $x10^6$ m^3/day.

Pumps

- Pump head H = 75m for a discharge Q = $86.4x10^3$ m^3/day per pump.

Pipes carrying treated water

- 1400 mm pipe feeding the study area, served by 4 pumps connected in parallel.

  This pipe carries $2.57x10^5$ m^3/day according to EPANET calculations.
- 1400 mm pipe feeding the study area, served by 3 pumps connected in parallel.

  This pipe carries $1.97x10^5$ m^3/day according to EPANET calculations.
- 1600 mm pipe NOT feeding the study area.
- 1000 mm pipe NOT feeding the study area.

Water quality at the WTP

- Average chlorine and TTHM concentrations over the six events were used for the model calibration.
- Chlorine concentration at Dar-El- Salaam WTP was assigned = 1.96 ppm
- TTHM concentration at Dar-El- Salaam WTP was assigned = 70.41 ppb

Maadi WTP

Daily production

- approximately 70 x 10^3 m3/day

Pumps

- Pump head H = 65 m for a discharge Q = 300L/s = 25.92 x10^3 m^3/day per pump.

Pipes carrying treated water

- 800 mm pipe serving the study area.

  This pipe carries 3.38x10^4 m^3/day according to EPANET calculations.
- 400 mm pipe serving the study area.

  This pipe carries 1.47x10^4 m^3/day according to EPANET calculations.
- 200 mm pipe NOT serving the study area.
- 200 mm pipe NOT serving the study area.
- 200 mm pipe NOT serving the study area.

The WTP feeds all pipes through a pump chamber, by 3 pumps connected in parallel.

Water quality at the WTP

- Average chlorine and TTHM concentrations over the six events were used for the model calibration.
- Chlorine concentration at Maadi WTP was assigned = 1.92 ppm.
- TTHM concentration at Maadi WTP was assigned = 73.80 ppb

Dar-El-Salaam WTP was connected to two pipes of 1400mm diameter each.

It was connected to the first pipe by 4 pumps (all pumps having the same pump curve with: H=75 m, Q= 86400 m^3/day). It was also connected to the other 1400 mm pipe by 3 pumps having the same pump curve.

For Maadi WTP, the pump chamber was assigned as a junction that was connected from one side to Maadi WTP (by 3 pumps all having the same pump curve: H=65 m, Q= 25.92 x10^3 m^3/day) and from the other side to the 5 pipes fed by Maadi WTP.

4.4.3.2 Network junctions

Junctions were assigned at various points in the network, to represent:

- Inflection points in pipes, intersection of connected pipes. and by-pass connections.
- Points representing water demand for the surrounding locations in the study area.
- Points representing demand for pipes feeding locations outside the study area.

Junctions were assigned to represent actual demand locations as much as possible. However, in some cases junctions represented arbitrary points to express the demand of a certain area using the best estimates that could be done based on the relatively limited available data about the distribution system. Junction description, coordinates, elevations, and water demand assigned to each junction are presented in Tables B-1 and B-2 in Appendix B.

4.4.3.3 Water demand estimation

Water demand was estimated on the basis of data for Cairo in which total water consumption is comprised of: 90 L/capita/day for domestic use, 40 L/capita/day for industrial use, 30 L/capita/day for commercial use. 20 L/capita/day for public uses: (irrigation of gardens, flushing of streets and fire protection), and 20 L/capita/day to account for losses in the water system (Abdel Shafy, Ezzat, 2000).

However, due to the presence of a large number of gardens and green areas in the study area, the estimated demand for public uses was assumed to be 45 L/capita/day. Also due to the fact that the network in the study area is a relatively older one, losses in the system were estimated as 50 L/capita/day. Hence, the total water demand was estimated as 255 L/capita/day.

Population density was calculated on the basis of the 1996 Population Housing and Establishment Census, (CAPMAS, 1996). According to this source, a population of 70,415 capita was calculated for Maadi, without including Digla and New Maadi. Population annual growth rate from year 1996 to year 2003 was estimated as 1.62% (CAPMAS, 2003). Calculations are shown in Appendix B and the population in year 2003 was estimated as 78,400 capita. Dividing the population by the area assigned in the Census resulted in a population density of ~ 0.0513 capita/m^2 (calculations are shown in Spreadsheet B-1 in Appendix B). This population density was assumed to be applicable to Digla as well as Maadi-proper.

The total demand for Old Maadi including Maadi-proper and Maadi-El Khabiri ~ 5.96 x10^4 m^3/day; therefore, the sum of the demands at the junctions serving these areas is set at 5.96 x10^4 m^3/day. The total demand for Digla ~ 1.16 x10^3 m^3/day; therefore, the sum of the demands at the junctions serving this area is set at 1.16 x10^4 m^3/day. (a more detailed presentation is shown in Tables B-1 and B-2 in Appendix B).

4.4.3.4 Initial chlorine and TTHM concentrations

Initial concentrations, *i.e.*, at the beginning of the simulation period, were assumed throughout the network as follows:

- measured values were assigned to the nodes where measurements were made.

- interpolation (by eye) was used to assign values to other locations (EPANET, 2002). Initial chlorine concentrations at all nodes are presented in Table B-3 in Appendix B, where time 0:00 represents midnight. Initial TTHM concentrations at all nodes are presented in Table B-4 in Appendix B.

4.4.3.5 Time of simulation

A 24-hour simulation was performed. In fact, water demand is expected to vary over the 24 hours. However, due to the limited information about actual demands and to the fact that modeling was performed for chlorine and TTHM concentrations averaged over a long period (6 events), the estimated water demand was assumed to be constant over the 24 hour simulation period. The simulation was assigned to start at midnight and calibration of the model was made at time = 12:00 noon (which is the average time of field sampling).

4.4.3.6 Headloss formula and pipe material used for each diameter

Hazen Williams headloss formula was assigned to be used in the EPANET model. Pipe material used for each diameter is presented in the Table 4.21 (Maadi Water Distribution System Office, 2003), together with Hazen Williams roughness coefficients (C) (EPANET, 2002)

Table 4.21: Pipe material used for each diameter and Hazen Williams roughness coefficient (C)

Diameter mm	Pipe Material	Roughness coef. C unitless
1400	Ductile Iron (lined with a cement layer)	130
1200	Ductile Iron (lined with a cement layer)	130
800	Cast Iron (unlined)	130
400	Steel (unlined)	140
300	Asbestos	130
200	Asbestos	130
150	Asbestos	130
100	Asbestos	130
80	Iron (unlined)	130

4.4.3.7 Reaction parameters input to EPANET

- **Bulk decay/formation coefficients and limiting concentrations**

Bulk decay/formation coefficients and limiting concentrations were assigned on the basis of the experimental bottle test data elaborated in Section 4.4.2. The bulk chlorine decay rate constant was set to 0.52 day^{-1}. Chlorine limiting concentration was assigned to be equal to zero. In other words, there is no limit to the extent of chlorine decay. The bulk TTHM formation rate constant was set to 0.48 day^{-1} according to the experimental rate. TTHM limiting concentration was set to 134 μg/L.

- **Relative diffusivity of TTHM with respect to chlorine:**

EPANET model is calibrated for chlorine diffusivity (in water) at 20°C which is equal to 1.2×10^{-5} cm^2/s. For substances other than chlorine, EPANET model requires the relative diffusivity (in water) of the modeled substance with respect to chlorine diffusivity at 20^{0}C. This input value is needed to model wall reactions. Chlorine diffusivity in water = 1.44×10^{-5} cm^2/s at 25°C (Spectrum Laboratories, 2003). TTHM species diffusivities at 25°C are shown in Table 4.22, as given by Risk Assessment Information System (2003).

Table 4.22: Diffusivity in water of TTHM species at 25°C

Chemical	**Diffusivity in Water (cm^2/s)**
Bromodichloromethane	1.06E-05
Bromoform	1.03E-05
Chloroform	1.00E-05
Dibromochloromethane	1.05E-05

The average diffusivity of these species = 1.04×10^{-5} cm^2/s. The relative diffusivity of TTHM with respect to chlorine (required by the model) was assumed to be equal to the ratio of this value to chlorine diffusivity, which gives 0.72. The ratio of TTHM diffusivity to chlorine diffusivity was assumed to be the same at 20° and 25°C.

4.4.3.8 Estimation of wall reaction constants using EPANET

As mentioned previously, bulk reaction coefficients (constants) were estimated by a laboratory experiment. Wall reaction coefficients were estimated using trial and error in the EPANET model. The model was calibrated with measured concentrations at certain points in Digla, averaged over the six events (extremely high or low individual measurements were excluded from the calibration, as described previously). From this calibration exercise, the chlorine wall decay rate constant was found to be ~ 0.26 m/day. This value is close to the range of typical values (0.25 ft/day to 1.0 ft/day, or ~ 0.08 m/day to 0.31 m/day) mentioned by Rossman (1998). Results of the calibration are presented in Table 4.23.

As stated in Chapter Two, EPANET uses a single-species model for wall reactions. It simulates each constituent (chlorine, TTHM) separately, and hence does not incorporate the reaction of chlorine with organic material at the pipe wall. It is not common to model wall reactions for TTHM using EPANET (Rossman, 2003). Other attempts were able to model TTHM formation more accurately by using AQUASIM modeling software to fit the laboratory determined kinetic parameters in the parallel first order equations for the simultaneous reactions of chlorine decay and TTHM formation (Kastl *et al.*, 2002).

In the present study, TTHM was modeled using EPANET for two cases:

- Using bulk reactions only. Results of the calibration are presented in Table 4.24.

- Incorporating wall reactions, for the sake of comparison. TTHM wall formation rate constant was found by trial and error to be ~ 0.027 m/day. Results of the calibration are presented in Table 4.25.

Comparison of measured versus computed chlorine and TTHM concentration is presented in Table 4.23, 4.24, and 4.25. Calibration (by trial and error of the wall reaction coefficient) was made in a way to get the mean observed and computed concentrations (of all calibration points) as close as possible. It is to be noted that the precision of measurement for chlorine is ~ 0.02 mg/L, and for TTHM precision is about 3.5% (weighed average for the four TTHM species according to QA/QC test performed in the laboratory, described in Chapter Three), which were taken into account for the calculation of the % error. Table 4.24 presents the measured versus computed TTHM concentrations without incorporating wall reaction. Table 4.24 indicates that using bulk reactions alone underestimates TTHM concentration. This conforms to the findings of Rossman *et al* (2001) that the TTHM formation due to overall reaction is higher than that due to bulk reactions only.

Table 4.23: Calibration statistics for chlorine

Location	Observed	Computed	Error	% Error
	ppm	ppm	ppm	
13-2	1.33	1.70	0.37	26.3
142	1.67	1.56	0.11	5.2
25	1.60	1.68	0.08	3.7
152	1.82	1.52	0.30	15.4
3	1.33	1.66	0.33	23.3
12-2	1.97	1.60	0.37	17.8
10	1.70	1.70	0.00	1.2
Mean	1.63	1.63		

Table 4.24: Calibration statistics for TTHM, without incorporating wall reactions

Location	Observed	Computed	Error	% Error
	ppb	ppb	ppb	
13-2	75.31	72.10	3.21	0.8
13-2	75.31	72.10	3.21	0.8
142	64.25	72.34	8.09	9.1
25	64.40	72.10	7.70	8.5
152	85.8	72.42	13.38	12.1
3	68.94	72.21	3.27	1.2
182	79.33	72.45	6.88	5.2
12-2	66.64	72.24	5.60	4.9
10	80.83	72.10	8.73	7.3
Mean	73.42	72.23		

Table 4.25: Calibration statistics for TTHM, incorporating wall reactions

Location	Observed	Computed	Error	% Error
	ppb	ppb	ppb	
13-2	75.31	72.92	2.39	0.3
13-2	75.31	72.92	2.39	0.3
142	64.25	73.87	9.62	11.5
25	64.4	73.01	8.61	9.9
152	85.8	74.14	11.66	10.1
3	68.94	73.21	4.27	2.7
182	79.33	74.07	5.26	3.1
12-2	66.64	73.56	6.92	6.9
10	80.83	72.92	7.91	6.3
Mean	73.42	73.40		

4.4.3.9 Discussion and results: EPANET modeling

It was noted that the concentration of both chlorine and TTHM (with/without wall reactions) becomes constant at time = 4 hours. This is illustrated in Tables B-3, B-4 and B-5 in Appendix B. This can be justified by the fact that WTPs are assigned in the model to produce a constant water flow with a constant concentration of chorine and TTHM. It is also related to the fact that there is no storage assigned in the model, as there is no municipal storage in the study area, and local storage was not taken into account in the model. However, the omission of local storage would not affect the kinetic estimation remarkably because calibration was made using residences in non-elevated floors.

Mass-transfer-based model used by EPANET

EPANET uses a mass-transfer-based model for the estimation of the reaction rate (decay/growth) of substances. The general expression for the reactions applied to chlorine and TTHM within bulk and wall phases (note limitation for TTHM, which will be described in this same paragraph) can be represented by the following Equations (4.3) through (4.9) (EPANET, 2002). For chlorine, these equations mainly represent the decay rate of free chlorine in pipes (Clark *et al.*, 1995). However, the present study of chlorine decay was made for total chlorine, as field data was all for total chlorine. Furthermore, as discussed earlier, because of current practice in Cairo water supply (no chloramination), there is little contribution of combined chlorine in tap water samples. Modeling total chlorine using EPANET was previously done by other researchers (*e.g.*, Abd-El-Shafy and Grünwald, 2000). As mentioned previously, EPANET is not commonly used for modeling TTHM formation at pipe walls. A closer look at Equations (4.3) through (4.9) justifies this matter, as these equations model the wall reaction as single-species first order decay/formation, only incorporating mass transfer between bulk and wall phases, while neglecting the TTHM formation due to reaction of chlorine with organic material attached to pipe walls.

For the simple first order bulk reaction kinetics (applied to chlorine decay), the bulk reaction rate is presented as follows

$$R_b = k_b C \qquad (4.3)$$

For the first order saturation growth bulk kinetics (applied to TTHM), the bulk reaction rate is presented as follows

$$R_b = k_b (C_m - C) \tag{4.4}$$

Wall reaction rate is represented using first order kinetics by

$$R_w = \frac{k_w k_f C}{r_h (k_w + k_f)} \tag{4.5}$$

The overall reaction rate is the sum of bulk and wall reaction rates

$$R_{ov} = R_b + R_w \tag{4.6}$$

$$K_f = Sh(D/d) \tag{4.7}$$

$$Sh = \text{Sherwood number} = 0.0149\ R^{0.88} Sc^{1/3} \text{for } R \geq 2300 \tag{4.8}$$

$$Sh = 3.65 + \frac{0.0668(d/L)R(Sc)}{1 + 0.04[(d/L)R(Sc)]^{2/3}},\ \text{for } R < 2300, \tag{4.9}$$

In the above equations:

- L = pipe length, (m).
- r_h is the hydraulic radius = pipe radius/2 (m)
- Sc = Schmidt number = v/D
- C is the concentration of chlorine or TTHM (ppm or ppb)
- C_m = maximum formation concentration of TTHM (ppb).
- R_b is the bulk reaction rate (mass/volume/time).
- R_w is the wall reaction rate (mass/volume/time).
- R_{ov} is the overall reaction rate (mass/volume/time)
- k_b is the bulk reaction rate constant calculated from the laboratory experiment (day^{-1})
- k_w is the wall reaction rate constant obtained by try and error in the EPANET model (m/day).
- k_f is the mass transfer coefficient between the bulk and wall phases (m/day)

- D = molecular diffusivity in water at 20°C (m^2/day). (described above in this chapter).
- d= pipe diameter (m)
- R = Reynolds number = Ud/v, where U = flow velocity (m/day) and v = kinematic viscosity of water = 0.087 m^2/day (Clark 1995; Droste 1997)

Sample of EPANET results and comparison with results from Equations (4.1) and (4.2) (Section 4.4.1)

EPANET output includes the concentration of the modeled substance at nodes/pipes, in addition to the overall reaction rate (R_{ov}) (mg/L/day). For the purpose of comparison with results from Section 4.4.1, the overall reaction rate constant for pipes is calculated by dividing R_{ov} (mg/L/day) for the pipe by the concentration of the substance (mg/L) in the pipe. Table 4.26 presents overall reaction rate constants in pipes carrying water from Dar-El-Salaam WTP to Junction 10 in Digla. For the present modeling conditions, all pipes were subject to turbulent flow (Table B-6, Appendix B). Therefore, the calculation of reaction coefficients was independent of the pipe length (See Equations (4.7) through (4.9)). Table 4.26 also indicates that pipes of smaller diameter are subject to higher overall reaction coefficients, for cases considering bulk and wall reactions. The physical explanation of this matter is that smaller pipe diameters offer a larger wall surface area per unit flow volume for wall reactions to occur (Clark *et al.*, 1995). Table 4.26 also indicates that for a certain pipe diameter, the overall reaction coefficient increases with the increase in velocity. This also conforms to previous findings (Clark *et al*, 1995). However, for the calculation of TTHM without incorporating wall reaction, the overall formation coefficient was found to be constant = 0.48 day $^{-1}$, which is equal to the bulk coefficient input to the model (Section 4.4.3.7); therefore, k_{ov} was not related to pipe diameter or to velocity by any specific pattern, which is also clear from Equations (4.3) through (4.9) indicating that the relation between the overall reaction rate and velocity and/or diameter is mainly due to the fact that the calculation of R_w is partially based on these two factors. This will be investigated in more details later in this chapter.

Table 4.27 presents overall reaction rate constants calculated for the same concentrations computed by EPANET (1.70 mg/L chlorine, 72.10 μg/L TTHM from bulk reactions only, and 72.92 μg/L TTHM from bulk + wall reactions, as shown in Table 4.23, 4.24, and 4.25 respectively), using the first order equation scheme (Equations (4.1) and (4.2) as in Section 4.4.1). As shown, average reaction coefficients in Table 4.26 are comparable to those calculated by the first order equation in Table 4.27. The results of Section 4.4.1 and Table 4.27 give a good primary idea of the reaction rate constants.

Table 4.26: Overall reaction coefficients in pipes carrying water from Dar-El-Salaam WTP to junction 10 (EPANET result)

Pipe		**Length (m)**	**Diameter (mm)**	**Velocity (m/s)**	**Chlorine (Bulk+ wall) k_{ov} (day^{-1})**	**TTHM (Bulk only) k_{ov} (day^{-1})**	**TTHM (Bulk+ wall) k_{ov} (day^{-1})**
Start node*	**Start node***						
14to12begin	14012	2464	1400	1.47	1.21	0.48	0.51
14012	12club	643	1200	2.01	1.35	0.48	0.51
12club	12port	522	1200	2.01	1.34	0.48	0.51
12port	F	865	1200	2.01	1.35	0.48	0.51
F	D	35	800	2.17	1.76	0.48	0.55
D	C	39	800	0.57	1.65	0.48	0.55
C	B	21	400	1.95	3.01	0.48	0.68
B	L	170	400	1.76	3.00	0.48	0.68
L	N	36	300	0.36	3.41	0.48	0.76
N	Digla-Chamber	325	150	0.82	6.89	0.48	1.12
Digla-Chamber	H	130	150	2	7.19	0.48	1.13
H	10	102	150	1.02	6.96	0.48	1.12
Average		**NA**	**NA**	**1.51**	**3.26**	**0.48**	**0.72**

* according to Map B-1, Appendix B.

Table 4.27: Overall reaction coefficients for different estimates of the average velocity using the first order equation (as in Section 4.4.1) from Dar-El-Salaam WTP to junction 10

v (m/s)	t (min)	t (day)	Chlorine	TTHM (Bulk only)	TTHM(Bulk + wall)
			k ov(day^{-1})	k ov (day^{-1})	k ov (day^{-1})
0.75	118.89	0.0826	1.72	0.29	0.42
1	89.17	0.0619	2.30	0.38	0.57
1.25	71.33	0.0495	2.87	0.48	0.71
1.5	59.44	0.0413	3.45	0.58	0.85
1.75	50.95	0.0354	4.02	0.67	0.99

Ratio of the contribution of bulk reactions to wall reactions

For chlorine, bulk reactions contributed almost 38 % of the total average of reaction rates in the network, while wall reactions contributed almost 62 % (Figure B-1, Appendix B). In contrast, for TTHM, bulk reactions contributed almost 82 % of the total average of reaction rates in the network, while wall reactions contributed almost 18 % (Figure B-2, Appendix B). These findings agree with those made in Section 4.4.2, by the comparison of bulk and overall reaction constants for chlorine residuals and TTHMs, which revealed that the ratio of the overall constant to the bulk constant ranges from 4 to 10 times for total chlorine and from ~1.04 to ~ 2.5 for TTHMs, which is also consistent with the findings of Rossman *et al* (2001) and Clark *et al* (1995).

In spite of the fact that EPANET modeling of TTHM wall reactions does not reflect the actual reaction at the wall (it neglects the formation of TTHM due to reaction between chlorine and organic matter accumulated at the pipe wall), it still emphasizes the previous findings that: (1) the contribution of wall reactions in the formation of TTHM is less than the contribution of wall reactions in the decay of chlorine residual, but (2) wall interactions may represent a significant contribution to TTHM formation in distribution systems.

Chlorine and TTHM profiles

Chlorine and TTHM profiles were drawn for water trajectories from Dar-El-Salaam WTP, as it was shown by the model that Maadi WTP does not feed water to Digla, according to the present modeling conditions.

- The first trajectory starts from the beginning of the main line of 1400 mm (leaving Dar-El-Salaam WTP) and then passes through Junction (14012) (Map B-1, Appendix B). This is the main line feeding water to Digla (Figures B-4 and B-5 for chlorine and TTHM (without incorporating wall reactions) respectively, Appendix B).
- The second trajectory starts from the beginning of the other main line of 1400 mm and passes through Junction (1401) (Map B-1, Appendix B). This line does not feed water to Digla under the present conditions. This is clear from the non-continuity in the profile shape (Figures B-3 and B-6 for chlorine and TTHM (without incorporating wall reactions) respectively, Appendix B)

As expected, profiles for chlorine displayed a decay trend, and those for TTHM showed a growth trend.

Sensitivity of EPANET calculated chlorine concentration to bulk and wall reaction coefficients

Sensitivity of EPANET calculated concentrations to bulk and wall reaction coefficients was made by tracking chlorine concentration at Junction 10, while changing the bulk reaction coefficient for fixed values of the wall reaction coefficient, and vice-versa. Tables 4.28 and 4.29 present % variations in chlorine at junction 10 versus % variations in each of k_b and k_w, respectively.

Table 4.28: Sensitivity of chlorine concentration (by EPANET) to variations in k_b

Absolute value of (k_b)	Chlorine conc. at junction 10	% variation in abs(k_b)	% variation in chlorine conc. at 10
day^{-1}	mg/L		
0.00	1.76	-100	3.5
0.10	1.75	-80	2.9
0.21	1.74	-60	2.4
0.31	1.73	-40	1.8
0.42	1.71	-20	0.6
0.52	1.7	0	0.0
0.62	1.69	20	-0.6
0.73	1.68	40	-1.2
0.83	1.68	60	-1.2
0.94	1.67	80	-1.8
1.04	1.66	100	-2.4
1.56	1.61	200	-5.3
2.08	1.57	300	-7.6
2.60	1.52	400	-10.6

Table 4.29: Sensitivity of chlorine concentration (by EPANET) to variations in k_w

Absolute value of (k_w)	Chlorine conc. at Junction 10	% variation in abs(k_w)	% variation in chlorine conc. at 10
m/day	mg/L		
0.00	1.90	-100	11.8
0.05	1.86	-80	9.4
0.10	1.82	-60	7.1
0.16	1.77	-40	4.1
0.21	1.74	-20	2.4
0.26	1.70	0	0.0
0.31	1.67	20	-1.8
0.36	1.64	40	-3.5
0.42	1.61	60	-5.3
0.47	1.58	80	-7.1
0.52	1.56	100	-8.2
0.78	1.43	200	-15.9
1.04	1.32	300	-22.4
1.30	1.22	400	-28.2

Tables 4.28 and 4.29 indicate that chlorine concentration is considerably more sensitive to changes in k_w than k_b. This can be associated with the fact that the contribution of wall reactions in chlorine decay is higher than the contribution of bulk reactions (as mentioned in previous discussions).

Sensitivity of EPANET calculated TTHM concentration to bulk reaction coefficient

Table 4.30 presents % variations in TTHM concentration at Junction 10 versus % variations in the bulk reaction coefficient, k_b. Table 4.30 indicates that the sensitivity of TTHM concentration to variations in bulk TTHM formation coefficient is very close to the sensitivity of chlorine concentration to variations in bulk chlorine decay coefficient. Table 4.31 presents % variations in TTHM concentration at junction 10 vs. % variations in the maximum concentration, C_m. From Tables 4.30 and 4.31, it can be concluded that TTHM concentration estimated using this modeling approach is more sensitive to the maximum concentration, C_m, than to the bulk formation coefficient, k_b.

Table 4.30: Sensitivity of TTHM concentration (by EPANET) to variations in k_b

k_b	TTHM conc. at Junction 10	% variation in k_b	% variation in TTHM conc. at 10
day^{-1}	µg/L		
0.00	70.41	-100	-2.34
0.10	70.77	-80	-1.84
0.19	71.08	-60	-1.41
0.29	71.44	-40	-0.92
0.38	71.75	-20	-0.49
0.48	72.10	0	0.00
0.58	72.45	20	0.49
0.67	72.76	40	0.92
0.77	73.10	60	1.39
0.86	73.41	80	1.82
0.96	73.75	100	2.29
1.44	75.36	200	4.52
1.92	76.93	300	6.70
2.40	78.46	400	8.82

Table 4.31: Sensitivity of TTHM concentration (by EPANET) to variations in the maximum concentration (C_m)

C_m	TTHM conc. at Junction 10	% variation in C_m	% variation in TTHM conc. at 10
μg/L	μg/L		
13	70.41	-90	-2.34
27	70.41	-80	-2.34
54	70.41	-60	-2.34
80	70.67	-40	-1.98
107	71.38	-20	-1.00
134	72.10	0	0.00
161	72.82	20	1.00
188	73.54	40	2.00
214	74.23	60	2.95
241	74.95	80	3.95
268	75.67	100	4.95
402	79.23	200	9.89
536	82.80	300	14.84
670	86.36	400	19.78

Variation of the overall reaction coefficient with pipe diameter and flow velocity, for chlorine calculations

Figure 4.27 presents the variation of k_{ov} (as calculated by EPANET) with different pipe diameters and velocities. Figure 4.27 indicates that k_{ov} increases for smaller pipe diameters, as mentioned previously in this chapter.

Variation of the overall chlorine decay coefficient with pipe diameter and velocity has been investigated by means of the previously described Equations (4.3) through (4.9). Before using Equations (4.3) through (4.9) for this investigation, they were examined in terms of their compliance with k_{ov} values resulting from EPANET. k_{ov} calculated for all pipes using Equations (4.3) through (4.9) and EPANET showed satisfactory similarity. This comparison is presented in Table B-6, Appendix B. The application of Equations (4.3) through (4.9) for different values of the velocity in a pipe of 400mm diameter shows that

k_{ov} tends to become asymptotic to a constant value as velocity increases (Figure 4.28); in other words, for a certain pipe diameter, the increase in velocity is associated with a diminishing increase in overall decay coefficient. This is also consistent with the findings of Clark *et al* (1995). However, one should be careful in interpreting the relation between velocity and chlorine decay rate. It is a complex relation in which two concepts occur simultaneously.

- Low velocity increases residence time in pipes, causing chlorine decay to increase, as more time is given to bulk and wall reactions in addition to higher formation of biofilm associated with low velocities and long residence times (Mays, 2000). One should note that the first order model used in this approach relates concentration to time (=distance/velocity) and is expressed by Equation (4.1): ($C_t = C_o e^{-kt}$), implying a higher decay of chlorine with increasing residence time.
- High velocity increases the overall reaction rate constant, presented as k in Equation 4.1.

Figure 4.29 presents the overall chlorine decay constant calculated by Equations (4.3) through (4.9), for different values of pipe diameter, for a velocity of 1.0 m/sec. Figure 4.29 indicates that, for a fixed flow velocity, the increase in pipe diameter is associated with a diminishing decrease in decay coefficient. These results conform to previous finding by Clark *et al.* (1995). However, in actual systems, diameters affect the flow velocity, rendering the relation between reaction rate and diameter not independent of velocity and residence time.

For decision makers in the design of the system hydraulics, it is important to investigate the complex relation between time, velocity, and diameter in order to optimize the system for the best kinetics available.

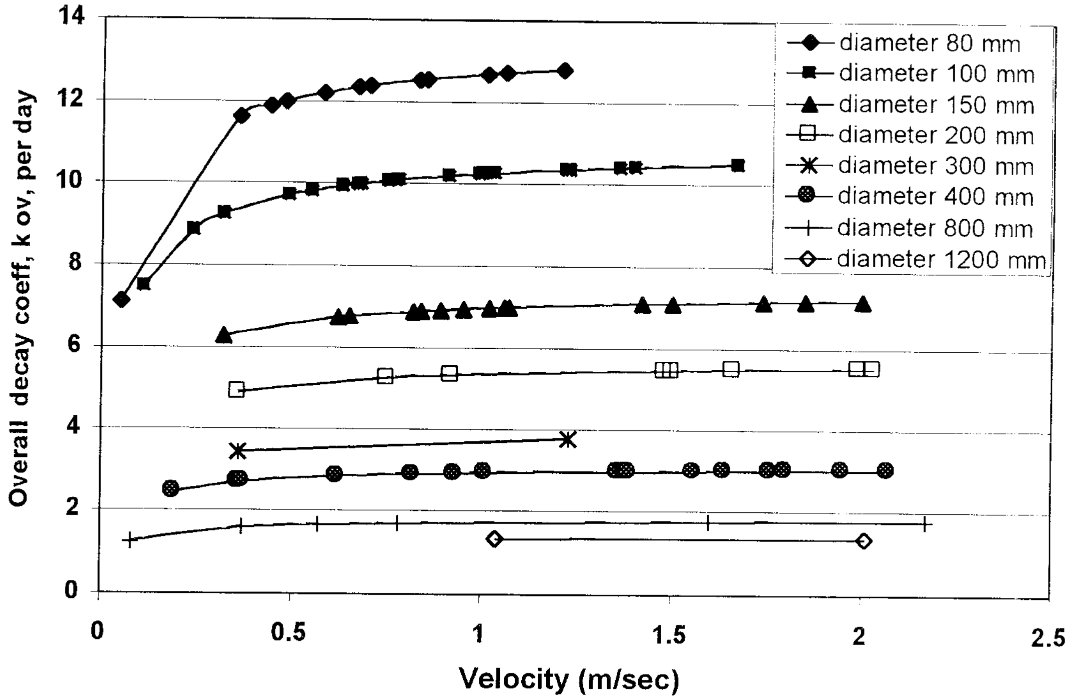

Figure 4.27: Overall chlorine decay coefficient versus pipe diameter and velocity

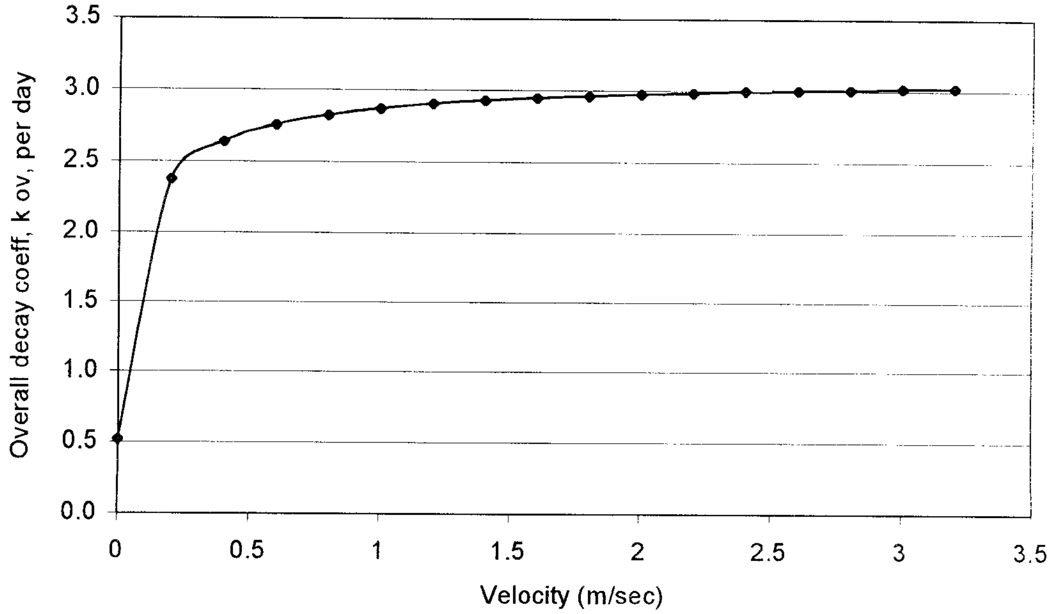

Figure 4.28: Overall chlorine decay coefficient versus velocity for a 400 mm diameter pipe

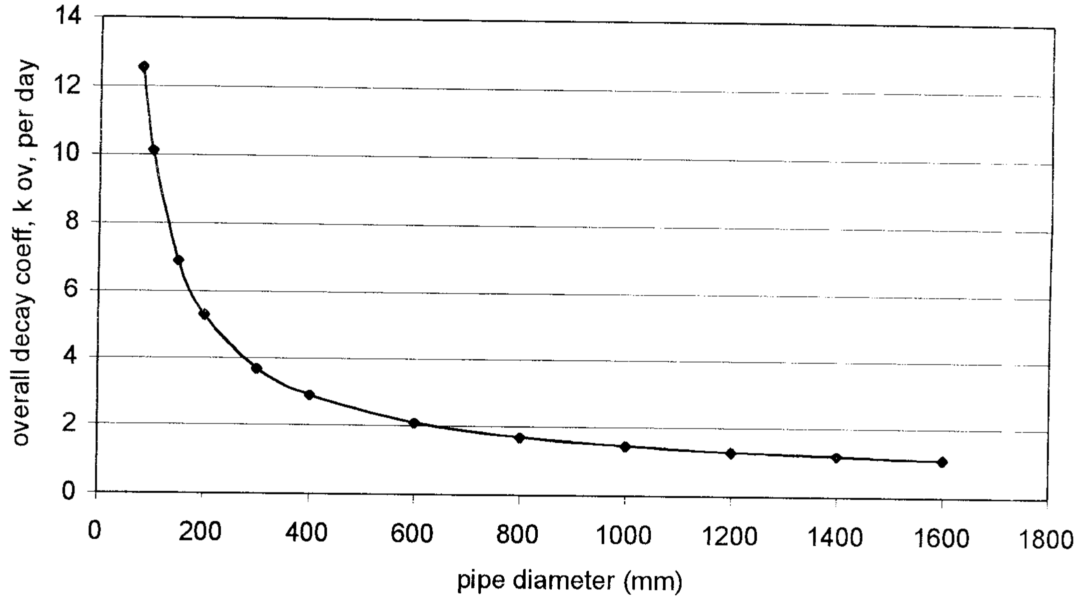

Figure 4.29: Overall chlorine decay coefficient versus pipe diameter for a velocity of 1 m/sec

As a result of the modeling efforts for chlorine and TTHMs in the present study, it can be deduced that a substantial portion of the decay of chlorine residuals in distribution systems occurs at the pipe wall. Pipe material, diameters, flow velocities and residence times are key factors decision makers should investigate for network design. In the present study, a large portion of the pipes consisted of unlined iron and steel, which contributed to the overall chlorine decay constant being up to an order of magnitude higher than the bulk decay constant. As mentioned previously, pipe diameters, flow velocities, and residence times should be studied together for optimum chlorine kinetics.

The contribution of wall effects to THM formation, while small compared to bulk interactions, suggests the presence of THM precursors in the form of organic matter adsorbed on the pipe walls. Although successful for estimating chlorine residuals in the distribution system, EPANET modeling of TTHM formation in the bulk phase only underestimated TTHM levels in the system. Other modeling approaches would be more successful if they properly incorporate TTHM formation in both bulk and wall phases including TTHM formation due to the reaction of chlorine with organics accumulated (adsorbed) at pipe walls.

4.5 Effectiveness of POU-WTUs

An important objective of this study was to evaluate the effectiveness of POU-WTUs for removal of various water contaminants. Three types of POU-WTUs were sampled during the study period (September 2001 to June 2003) at the studied residential locations:

- Granular Activated Carbon (GAC) adsorber (9 units were sampled)
- Reverse Osmosis (RO) units (6 units were sampled)
- Distillation Units (5 units were sampled).

The results of water quality analysis in the studied residential locations for the tap versus device-treated water are used to evaluate the effectiveness of the three types of POU units.

Individual operation / maintenance of a given unit were not always uniform. Also different product types were used in sampled residences. Therefore, the comparison of average performance of the devices becomes difficult.

This study evaluates the performance the three types of units in two ways:

- Evaluation of the performance of one device each of a GAC adsorber, RO unit, and distillation unit operated continuously and trouble-free over the duration of the study period. These selected devices are used as typical examples for the comparison of the performance of the three types of units.
- Minimum and maximum % removal of contaminants for a number of sampled units, as a way to ascertain the presence of trends in the performance of the units with respect to various contaminants; *i.e.*, which contaminants are not generally removed by a certain type of unit, and which

are removed with different percentages - even under different operating conditions and different product types.

4.5.1 Granular Activated Carbon (GAC) Adsorbers

Nine GAC units were sampled over the period of study. One of these units was part of a three-stage unit including a sediment filter and ion exchange column. This unit was not included in the evaluation of GAC devices. Another GAC unit was sampled for two events only, which was also not included in the evaluation of GAC devices.

Table 4.32 presents the performance of a selected well operated GAC unit (Unit 1 in Table 4.33). Mean values (for the six events) for tap and device-treated water quality are presented, in addition to the average of percent removals of contaminants for the six sampling events. Table 4.33 presents the average % removal of contaminants for seven devices, with the maximum and minimum removal for the seven devices.

Total and fecal coliform growth was detected in two GAC units (location 3 in Event 4 and location 19 in Events 1 and 2) up to 2 colonies/ 100 ml, which were also found not to be well operated based on discussions with the users.

Table 4.32 shows that even for the well operated unit, standard deviations of the % removals are high relative to the average values of % removals. This is also emphasized in Table 4.33 by the wide range of performance for the different sampled units. However, the general performance of GAC units can be described as:

- Almost no removal of TDS, alkalinity, hardness, calcium and magnesium.
- Good removal of turbidity, depending on product type and operating conditions.
- Probable removal of iron and lead, for specific product types.
- Good removal of chlorine and TTHM if well operated.

- Some removal of nitrate and fluoride, depending on product type and operating conditions.

Table 4.32: Performance of selected GAC POU unit for six events (except arsenic for 2 events only)

Analysis	Units	Tap Water	Treated water	Average % Removal	Egyptian Criteria	WHO Criteria	U.S. EPA Criteria
pH	pH	7.60	7.60	NA	6.5-9.2	6.5-8.5	6.5-8.5
TDS	mg/L	182	179	3 ± 3	1,200	1,000	500
Turbidity	FTU	1	1	13 ± 25	5	5	0.3 -1.0
Alkalinity	mg/L as $CaCO_3$	120	120	1 ± 2	-	-	-
Total hardness		97	95	2 ± 3	500	500	-
Calcium	mg/L	20	20	0 ± 0	200	-	-
Mg	mg/L	11	11	6 ± 8	150	-	-
Iron	mg/L	0.050	0.059	26 ±43	0.3	0.3	0.3
Lead	mg/L	0.004	0.000	100 (only one value)	0.05	0.01	0.015
Copper	mg/L	0.000	0.000	None in tap	1.0	1.0	1.0
Arsenic	mg/L	0.000	0.000	None in tap	0.05	0.01	0.01
Chlorine	mg/L	1.30	0.16	77 ± 22	-	5	4.0
Nitrate	mg/L	0.05	0.03	43 ± 46	10 (as Nitrogen)	50 (as nitrate)	10 (as Nitrogen)
Fluoride	mg/L	0.47	0.45	9 ± 10	0.8	1.5	2.0
Total coliforms	/100 ml	0	0		5%	Nil	5% / month
Fecal col.	/100ml	0	0		Nil	Nil	Nil
TTHMs	µg/L	67	20	74 ± 16	100	-	80

Table 4.33: Performance of seven GAC POU units

Parameter	Units	% Removal for each unit averaged over all events							Range of %Removal
		Unit/Number of events							
		1/6	2/6	3/3	4/3	5/6	6/6	7/6	
pH	pH	NA	NA	NA	NA	NA	NA	NA	NA
TDS	mg/L	3	3	3	0	1	1	1	0 - 3
Turbidity	FTU	13	38	50	50	13	0	67	0 - 67
Alkalinity	mg/L CaCO3	1	2	4	1	1	1	0	0 - 4
Total Hardness	mg/L CaCO3	2	1	1	1	2	3	4	1 - 4
Calcium	mg/L	0	3	2	0	2	3	3	0 - 3
Mg	mg/L	6	1	0	1	3	4	5	0 - 6
Iron	mg/L	26	60	0	41	37	36	47	0 - 60
Lead	mg/L	100		36		33	33		0 - 100
Chlorine	mg/L	77	92	92	51	67	16	75	16 - 92
Nitrate	mg/L	43	38	67	0	11	17	0	0 - 67
Fluoride	mg/L	9	22	3	20	17	13	21	3 - 22
TTHMs	µg/L	74	32	12	31	47	13	11	11 - 74

4.5.2 Reverse Osmosis Units

All reverse osmosis units include pre-treatment and post-treatment sediment (and sometimes GAC) filter attachments. The whole package is classified as the RO unit. Six RO units were sampled, of which five had interrupted service for one or more events during the sampling period. Table 4.34 presents the performance of a selected uninterrupted RO unit (Unit 3 in Table 4.35). Table 4.35 presents the average % removal of contaminants for six individual devices.

As in the case of the GAC units, the performance of RO units varies widely depending on the product type and operation / maintenance conditions. It is to be noted that total and fecal coliform growth was also detected in a number of RO units: location 6 in Events 1, 2, 3 and 4 up to 2 colonies/100 ml, and location 4 in Events 2 and 3 up to 35 total coliform colonies/100 ml and 4 fecal coliform colonies/100 ml; these units, especially the latter one, were found to face problems with their performance, and maintenance was suggested to the owners of these units, which was also successful to control bacterial growth.

The general performance of RO units can be described as:

- Good removal of TDS, alkalinity, hardness, calcium and magnesium.
- Good removal of turbidity, depending on operating conditions.
- Some removal of iron, depending on operating conditions.
- Probable removal of lead, depending on operating conditions.
- Almost complete removal of chlorine and TTHM.
- Some removal of nitrate and good removal of fluoride.

Table 4.34: Performance of selected RO POU unit for six events (except arsenic for 2 events only)

Analysis	Units	Tap Water	Treated water	Average % Removal	Egyptian Criteria	WHO Criteria	U.S. EPA Criteria
pH	pH	7.59	7.34	NA	6.5-9.2	6.5-8.5	6.5-8.5
TDS	mg/L	174	16	91 ± 3	1,200	1,000	500
Turbidity	FTU	2	1	67 ± 41	5	5	0.3 -1.0
Alkalinity	mg/L	117	16	87 ± 5	-	-	-
Total Hardness	as CaCO$_3$	92	6	94 ± 6	500	500	-
Calcium	mg/L	21	0	98 ± 2	200	-	-
Mg	mg/L	10	1	90 ± 10	150	-	-
Iron	mg/L	0.069	0.033	53 ± 39	0.3	0.3	0.3
Lead	mg/L	0.006	0.010	0 ± 0	0.05	0.01	0.015
Copper	mg/L	0.000	0.000	None in tap	1.0	1.0	1.0
Arsenic	mg/L	0.000	0.000	None in tap	0.05	0.01	0.01
Total Chlorine	mg/L	0.83	0.02	97 ± 3	-	5	4.0
Nitrate	mg/L	0.05	0.03	22 ± 40	10 (as Nitrogen)	50 (as nitrate)	10 (as Nitrogen)
Fluoride	mg/L	0.58	0.16	72 ± 19	0.8	1.5	2.0
Total coliforms	/100 ml	0	1	Coliform growth	5%	Nil	5% / month
Fecal coliforms	/100 ml	0	1	Coliform growth	Nil	Nil	Nil
TTHMs	µg/L	79.3	23.5	67 ± 39	100	-	80

Table 4.35: Performance of Six RO Units

Parameter	Units	% Removal for each unit averaged over all events						Range of %Removal
		Unit/Number of events						
		1/3	1/5	3/6	4/4	5/3	6/3	
pH	pH	NA	NA	NA	NA	NA	NA	NA
TDS	mg/L	56	62	91	87	56	88	56 - 91
Turbidity	FTU	63	53	67	89	75		53 - 89
Alkalinity	mg/L CaCO3	29	62	87	76	80	83	29 - 87
Total Hardness	mg/L CaCO3	52	73	94	91	88	96	52 - 96
Calcium	mg/L	63	80	98	96	97	99	63 - 99
Mg	mg/L	37	64	90	83	80	92	37 - 92
Iron	mg/L	33	67	53	0	7	50	0 - 67
Lead	mg/L	0	0	0	0	7	0	0 - 7
Chlorine	mg/L	99	98	97	99	99	99	97 - 99
Nitrate	mg/L	67	20	22	56	67	33	20 - 67
Fluoride	mg/L	14	35	72	60	53	76	14 - 76
TTHMs	µg/L	88	2	67	86	83	86	2 - 88

4.5.3 POU Distillation Units

Table 4.36 presents the performance of a selected continuously operated distillation unit (Unit 1 in Table 4.37). Table 4.37 presents the average % removal of contaminants for five individual devices.

It is to be noted that the occurrence of high levels of lead was not uncommon in the water effluent of distillation units. This was often associated with lowered pH (sometimes even lower than the allowable standard limits) and elevated iron levels. Low pH in distilled water can be justified by the fact that distillers remove almost all of the water alkalinity. Low pH values normally encourage the occurrence of spot corrosion of iron, steel and lead connections in distillers.

Total and fecal coliform growth was detected in one distillation unit: location 12 in Event 3, up to 2 colonies/100 ml, which could be due to the post-treatment storage unit associated with distillers.

The general performance of distillation units can be described as:

- Almost complete removal of TDS, alkalinity, hardness, calcium and magnesium.
- Good removal of turbidity, depending on maintenance/corrosion.
- Probable removal of iron, depending on maintenance/corrosion.
- Almost no removal of lead, depending on maintenance/corrosion.
- Almost complete removal of chlorine.
- Some removal of nitrate and good removal of fluoride.
- Almost 50% removal of TTHM

Table 4.36: Performance of selected distillation unit for six events (except arsenic for 2 events only)

Analysis	Units	Tap Water	Treated water	Average % Removal	Egyptian Criteria	WHO Criteria	U.S. EPA Criteria
pH	pH	7.60	6.70	NA	6.5-9.2	6.5-8.5	6.5-8.5
TDS	mg/L	174	3	98 ± 0	1,200	1,000	500
Turbidity	FTU	1	0	60 ± 55	5	5	0.3 – 1.0
Alkalinity	mg/L $CaCO_3$	119	7	94 ± 3	-	-	-
Total Hardness		90	0	100 ± 1	500	500	-
Calcium	mg/L	20	0	100 ± 1	200	-	-
Mg	mg/L	10	0	99 ± 1	150	-	-
Iron	mg/L	0.065	0.093	25 ± 43	0.3	0.3	0.3
Lead	mg/L	0.016	0.023	0 ± 0	0.05	0.01	0.015
Copper	mg/L	0.000	0.000	None in tap	1.0	1.0	1.0
Arsenic	mg/L	0.000	0.000	None in tap	0.05	0.01	0.01
Total Chlorine	mg/L	2.05	0.01	99 ± 0	-	5	4.0
Nitrate	mg/L	0.07	0.02	38 ± 49	10 (as Nitrogen)	50 (as nitrate)	10 (as Nitrogen)
Fluoride	mg/L	0.43	0.06	85 ± 17	0.8	1.5	2.0
Total coliforms	/100 ml	0	0		5%	Nil	5% / month
Fecal coliforms	/100 ml	0	0		Nil	Nil	Nil
TTHMs	µg/L	69.4	38.4	51 ± 20	100	-	80

Table 4.37: Performance of five distillation units

Parameter	Units	% Removal for each unit averaged over all events					Range of %Removal
		Unit/Number of events					
		1/6	2/5	3/6	4/4	5/6	
pH	pH	NA	NA	NA	NA	NA	NA
TDS	mg/L	98	98	99	99	97	97 - 99
Turbidity	FTU	60	88	92	33	60	33 – 92
Alkalinity	mg/L CaCO3	94	95	96	95	96	94 - 96
Total Hardness	mg/L CaCO3	100	100	100	100	100	100
Calcium	mg/L	100	100	100	100	99	99 - 100
Mg	mg/L	99	99	99	99	100	99 - 100
Iron	mg/L	25	43	66	50	61	25 - 66
Lead	mg/L	0	11	0	0	18	0 – 18
Chlorine	mg/L	99	99	99	99	98	98 - 99
Nitrate	mg/L	38	40	25	46	8	8 - 46
Fluoride	mg/L	85	81	70	88	87	70 - 88
TTHMs	µg/L	51	36	59	60	46	36 - 60

4.5.4 Comparison of the effectiveness of the three types of units

Table 4.38 compares the relative performance of GAC, RO, and distillation POU units for a selected unit of each type. Comments on the relative performance of the units are presented in Table 4.39. Regarding the performance of the three types of units for removal of bacterial contamination, no conclusion can be made in this issue from the present study, as a very small percentage (less than 2.5%) of the tap water samples had coliforms in, which is not enough to draw a statistical conclusion.

Table 4.38: Relative Performance of Three Selected POU Units for 6 Events

Analysis	Units	GAC Unit %Removal	RO Unit %Removal	Distillation Unit %Removal
TDS	mg/L	3	91	98
Turbidity	FTU	13	67	60
Alkalinity	mg/L	1	87	94
Total Hardness	mg/L	2	94	100
Calcium	mg/L	0	98	100
Mg	mg/L	6	90	99
Iron	mg/L	26	53	25
Lead	mg/L	100	0	0
Total Chlorine	mg/L	77	97	99
Nitrate	mg/L	43	22	38
Fluoride	mg/L	9	72	85
TTHMs	µg/L	74	67	51

Table 4.39: Comments on the Relative Performance of Three Selected POU Units for Six Events

Contaminant	GAC performance	RO Unit	Distillation Unit
TDS	~ no removal	good removal	~ complete removal
alkalinity	~ no removal	good removal	~ complete removal
hardness, calcium & magnesium.	~ no removal	good removal	~ complete removal
Turbidity	good removal. Depends on filter condition	good removal. Depends on unit condition	good removal. Depends on maintenance/corrosion
Iron	probable removal (specific product types)	some removal. Depends on operating conditions	probable removal. Depends on maintenance/corrosion
Lead	probable removal (specific product types)	probable removal. Depends on operating conditions	~ no removal. Depends on maintenance/corrosion
Chlorine	good removal. Depends on filter condition	~complete removal if well operated	~complete removal
Nitrate	some removal	some removal	some removal
Fluoride	some removal	good removal	good removal
TTHM	good removal. Depends on filter condition	~ complete removal if well operated	~ 50% removal

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Evaluation of residential tap drinking water quality in the community of Maadi (Cairo, Egypt) was performed. The sampling points selected for the study were chosen in part because the users also had an operating POU-WTU in place. Therefore, another objective of this research was to evaluate the performance of three commonly used POU devices for removal of water quality contaminants, including chlorine and TTHM. Finally, an attempt was made to model chlorine decay and TTHM formation in the distribution system because of the health concern that these substances pose. Several important conclusions derive from the present study.

- Residential tap water quality in the study area showed no violation of Egyptian, WHO, or USEPA standards, except for the occasional occurrence of high TTHM levels up to 112 μg/L, thus exceeding the 80 μg/L USEPA limit and the 100 μg/L Egyptian limit. The highest level of TTHM at the WTP was 91 μg/L. TTHMs increased as water was carried form the WTP to residential locations due to the ongoing reaction of chlorine residuals with dissolved organic matter in the distribution system.
- Chlorine levels exert a decrease in the distribution system and TTHMs exert an increase. However, the dependence on local versus municipal storage in the area of study results in lower residence times, meaning less overall conversion, which is positive for TTHMs as less of them are allowed to be formed in the distribution system. For larger systems associated with larger residence times, more reaction will be allowed, which will normally be negative.

- Iron levels (and, to a much lower extent, lead and copper) were found to increase in the water distribution system, reflecting the probable presence of corrosion in pipes or fittings.
- Analyses performed on Nile River water demonstrated a strong relation between THM-FP and TOC, which are both used as surrogate parameters for NOM in water.
- No clear seasonal pattern was detected in water quality in the study area during the period of analysis (September 2001 to June 2003), except for the slight increase in Nile water TOC and alkalinity levels in winter.
- Modeling chlorine decay using EPANET showed successful compliance of measured and observed concentrations, while TTHM formation was under-estimated by EPANET. Other modeling approaches would be more successful if they properly incorporate TTHM formation in both bulk and wall phases including TTHM formation due to the reaction of chlorine with organics accumulated at pipe walls.
- The effect of wall reaction in the decay of residual chlorine, using the first order model, is high relative to the contribution of bulk interactions. This is consistent with water quality modeling research in developed countries.
- Hydraulic conditions have an important effect on the kinetics of chlorine in pipes. For decision makers in the design of the distribution system hydraulics, it is important to investigate the complex relation between time, velocity, and pipe diameter in order to optimize the system for the best kinetics available. Of course, such a study should take into consideration the quality of the water entering the system, especially as regards to the concentration and nature of the dissolved organic matter.
- The performance of the three tested POU-WTUs depends on the maintenance of each individual unit. No conclusion can be made about the performance of

these units as regards to bacterial removal, as a very small percentage (less than 2.5%) of the tap samples had coliforms in, which is not enough to draw a statistical conclusion. There were a few isolated instances of regrowth of a few organisms, especially in systems including post-treatment storage units, namely RO and distillers.

- GAC POU-WTUs were effective for removal of THMs and chlorine, with almost no removal of inorganic salts.
- RO POU-WTUs were effective for removal of THMs, chlorine and inorganic salts.
- POU distillation units were effective for removal of chlorine, inorganic salts, and removed almost 50% of the TTHMs.

5.2 Recommendations

- Monitoring of tap water needs to be carried out in other districts of Cairo in order to determine whether the trends observed in Maadi are uniform for all treatment/distribution systems in the city.
- The kinetics of chlorine residuals and THMs in distribution systems might be better expressed by a parallel first order model which takes into account fast and slow reactions between chlorine and NOM. However, this model was not applicable to the present study, as the study did not investigate the ratios and kinetics of each of the fast and slow reducing agents.
- Based on literature review, more interest needs to be given to haloacetic acid in water samples.
- As noted from the present study, TTHM formed at the WTP (although not exceeding allowable limits) contribute a large portion of TTHM in residential tap water. This implies the need for more research on Nile water quality in order to study the implementation of techniques for reducing DBPs formation

at the WTP (for example, some are based on reducing NOM prior to chlorination, using alternative disinfectants, controlling operating conditions such as pH, *etc*). The challenge here consists in identifying an economically available technique for the present municipal public water supplies. The application of booster chlorination would also be useful in the optimization of both chlorine and TTHM levels in distribution systems.

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APPENDIX A

Map A-1

Water distribution system in the study area