DESIGN OF WATER SUPPLY TREATMENT PLANT FOR GOVERNMENT RESIDENTIAL ESTATE AT ILESHA ROAD, AKURE, ONDO - STATE.

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DAFIEWHARE, Avirhorhoraye Henry Etukes.
(CVE/92/4194)

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A thesis submitted to the School of Postgraduate Studies in partial fulfillment of the award of the degree of Master of Engineering (M. Eng) in Civil Engineering (Water Resources and Environmental Engineering Option)

Department of Civil Engineering,
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2005

CERTIFICATION

This is to certify that this research was undertaken by **DAFIEWHARE, Avirhorhoraye Henry Etukes (CVE/92/4194)** and submitted through the Department of Civil Engineering to the School of Postgraduate Studies, Federal University of Technology, Akure, Ondo State in partial fulfillment of the award of the degree of Master of Engineering (M. Eng) in Civil Engineering (Water Resources and Environmental Engineering Option).

11/10/05

Dr. A. M. Oguntuase

Supervisor.

DEDICATION

I dedicate this work to my wonderful and godly wife, precious children and all those that wish me well in life.

ACKNOWLEDGMENT

In particular I thank God Almighty for making me what I was yesterday, I am today and I will be tomorrow. I thank also Dr. A. M. Oguntuase, my supervisor and God-sent motivator, who did every thing to encourage my going through this programme. I thank all my lecturers, colleagues, friends and classmates for their wonderful assistance before, during and after this programme.

I thank my humble, lovely, caring, and blessed sister and very good friend, Sister Oluwafunmilayo Oke for typing part of this work even at very short notices and difficult conditions.

I say special thank you to every member of my family, my senior brother, Professor G. O. Iremiren, his God-given wife, Mrs. A. E. Iremiren, who happened to be my most senior sister and other members of my extended families.

Finally, I thank my wonderful and godly friend and mother, wife, precious children and all those who stood by me throughout the periods of this academic progamme.

To God be the glory.

ABSTRACT

This study examines the present management and control of groundwater, stormwater and stream flow in Government Residential Estate at Ilesha Road- Akure, Ondo State. The samples of these waters were collected for laboratory analysis to determine their level of contamination and the treatment necessary them to human use for domestic water supply. The results of the laboratory analysis were analyzed and used to design a treatment plant for water supply for the study area.

Inorder to get an efficient conveyance system for the stormwater flow and water supply in the study area, the topography of the area was studied and the spot heights of all the roads in the area were taken and their slopes determined. The results of these measurements and studies were used to design the drain channels and treated water piping networks.

The population of the estate was calculated using the allocated plots and assumed inhabitants of 12, 12, and 8 per plot in the high, medium, and low densities areas respectively. (Note that the "density" here referred to the number of plots that makes a section of the estate.) These information and that of the rainfall depth of the area were used to calculate the various components of a treatment plant.

There are two possible means of convenient water supply to the estate; these are the construction of a treatment plant in the estate or the extension of the public water supply pipe to the estate. However the construction of a treatment plant is suggested because the public water supply is not reliable.

The economic implications of all available systems were assessed to show their financial involvement ant this shows that it will cost ₦144,323,000.00 to provide the present system of water supply by the inhabitants while Government will provide ₦635,000,000.00 for the proposed treatment plant and ₦475,000,000.00 for the public-water distribution system.

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CHAPTER ONE

INTRODUCTION

1.1 General

Water has been in existence since the begimning of creation according to the Holy Book, as narrated in Genesis 1: 1 – 9. Water is obtained from rainfall, surface and ground waters. That man, a social being, deserves a comfortable and habitable environment free of any pollutant is no news at all. The main factor that determines the settlement of man is water. The quality and availability of water determine levels of public health, food production, the productivity of industries, the production of energy and other important aspects of the quality of life. All the water available to man becomes wastewater after use and therefore needs to be properly managed; hence it becomes another problem on its own.

Ilesha Road Government Residential Estate in Akure is one of the residential estates created by the Ondo State Government in 1990. The estate covers an area of about 2.75square kilometers. About 80% of this area is allocated for residential developments while the remaining 20% is for recreational and social developments. The estate plots are bought and developed by individuals. The different levels of development have made it difficult to have a uniform data for the various analysés. Rainwater and groundwater are the only source of domestic water supply in the community. Majority of the residents depend on hand-dug wells while only very few could afford boreholes. The state government has not shown any concrete evidence of developing any area of the estate. The wastewaters generated in this estate can be classified as medium strength polluted. Analysis of some samples show that they contain materials of suspended solids, such as food remnants, leaf, etc. and some dissolved materials. The only method of wastewater disposal and treatment presently in use in the estate are the septic tanks and soakaway systems, which are individually owned.

In this study, it is proposed that stormwater should be harvested, treated and supplied as domestic water supply. A water treatment plant in line with the prevailing technology is envisaged to replace the existing methods. The collection and transportation of stormwater will be through open channel drains. Through the provision of this treatment plant, the pollution of the surface and ground water sources will be drastically reduced and the economic, social and environment conditions of the benefactors will considerably improve.

1.2 Project Background

The creation of Ondo State in 1976 brought about influx of people to Akure, the State capital. Establishment of Government and private or family residential estates was part of the developmental processes of the town. The Ilesha Road Government Residential Estate is one of the Government Estates in Akure. The main features of the Estate are Ilesha Road, through which its name is derived, FUTA Road, and the Ala River tributary flowing within it. The topographical map of the Estate shows that the place is slopy and uneven in landscape. The highest point of the Estate is on the southern side, which is about 345m above the sea level while the lowest point on the northern side, is about

325m above the sea level Surface, storm and wastewaters needed to be collected, treated and the effluent properly disposed off to eliminate any diseases that may arise from the process and to provide an environmentally safe community. This research project is aimed at an optimal design and construction of a treatment plant that minimizes the cost and maximizes the benefits derivable by its stakeholders.

1.3 Project Aim and Objectives

This research project is basically aimed at providing an engineering solution to a pressing human and environmental water problems of Government Residential Area, Ilesha Road, Akure- Ondo State by assessing the water requirement of the estate to design appropriate treatment facilities for the domestic water supply of the estate.

1.3.1 Specific Objectives of the Research:

The specific objectives of the research are to:

i determine the quality of water from the raw water sources in the estate to assess the level of treatment required for the raw water;

ii. assess the existing systems adopted in the collection and conveyance of the raw waters to the point of abstraction, if any;

iii. design the treatment plant facilities, drainage network for raw water collection and mains pipe network for portable water distributions in Government Residential Area, Ilesha Road, Akure- Ondo State;

iv. compare the costs of establishing the independent water system for the community with that of direct supply of potable water from the State's Water Corporation mains and from individual household wells and boreholes.

1.3.2 Expected Contributions to Knowledge:

The study is expected to provide the technical guidelines for the design and construction of small-scale independent water supply systems for small communities and urban estates.

1.4 Justification

Judging from the economic, social, political and public health viewpoints, the relevance of this project to the community can be listed as follows

i. On economic viewpoint, it will be seen that it is more economical to construct a single treatment plant than constructing hundreds of septic and soakaway tanks. The bye product of the treatment plant, i.e. sludge can also generate extra money if sold for agricultural and industrial uses.

ii. Ordinarily, rainwater producing runoff causes erosion of the landscape if not properly controlled and channeled through a network of drainage channels is not good enough. The presence of a water treatment plant will improve the social status of the community and thus attract quick development of the estate.

iii. On political grounds, this kind of project will enhance the self-help philosophy of government. The funds for the construction need not come from the government alone, the people of the community can be asked to contribute a percentage of the cost to assist the government. It gives them a sense of belonging, both in its construction and maintenance.

iv. On public health ground, the community health status will be improved. Less effort will be exerted to obtain good and safe domestic water supply for use. Ground water pollution or contamination during the rains will be reduced. This project will enhance the quality of life of the residents.

v. Professionals involved in the design, construction, operation and maintenance of projects, such as Civil, Mechanical and Electrical Engineers, and Chemists, etc. will be tasked to justify their professional capabilities.

CHAPTER TWO

LITERATURE REVIEW

2.1 Importance of Water

Many authors have written a lot about water and they all agreed that water is the most important natural resource in the world and that without it life cannot exist. Although human life can exist for many days without food, the absence of water for only a few days has fatal consequences. The presence of a safe and reliable source of water is thus an essential prerequisite for the establishment of a stable community. It is therefore not surprising that sources of water are often jealously guarded. According to Tebbutt (1998) the growth of population in many developing countries was such that unless strenuous efforts to increase water supply and sanitation facilities could be made, the percentage of the world's population with satisfactory services would actually decrease in the years to come. A major objective in water quality control work is to reduce the incidence of water-related diseases. This objective depends on the ability to develop water sources to provide an ample supply of water of wholesome quality.

2.2 Establishment of Urban Estates and Residential Areas

Establishment of urban estates and residential areas by various government and private individuals is based on providing houses and lands for the interested individuals or group of people who want inexpensive houses of their own.

(Newcomber, 1983). In these areas some basic facilities, such as public or community facilities, which should be provided include

a. Public facilities like electricity, water supply, telecommunication, roads and drainages.

b. Community facilities such as educational, health and places of worship.

The Federal Government, with the aim of overcoming land problems, introduced the Land Use Decree No. 6 of 1978. The Decree nationalized all land; and the system of conveyance by Lawyers was replaced by Certificate of Occupancy (C of O) to be issued by the Ministries of Lands and Housing in respective states of the Federation.

2.3 Population Forecast

Roberts (1999) states that, behind almost every planning decision lie some assumptions about population. The factors land use planners need to know about population concern the present and the future and, as a guide, the past. The size and composition of a population are decisive factors in deciding what institutions and facilities required to be planned. The characteristics of the population of any area-age, sex, family structure, socio-economic class, level of education and skills, ethnic characteristics or religious beliefs – are crucial, as well as their total numbers and geographical distribution. A major producer of data for population studies is the Central Government. Roberts, (1999) listed mentioned that the methods that can be adopted for population forecasting as direct or indirect approach, trend projection and level use based projection.

In direct technique, forecasts are based on past data and recognized trends which can be extrapolated. Indirect techniques relate forecasts to other indicators which are supplied to the model as independent predictions.

The trend projection exemplifies the macro approach, where it is assumed that past relationships over time for the whole population will continue to hold good. The equation for this is:

$$P_t + P_1 \quad b_0 \qquad \text{.............(2.1)}$$

where P_1 = the base population at time t.

P_t = the population after a time interval of t.

b_0 = constant representing annual growth.

For a polynomial and exponential curves, the equation becomes:

$$P_t = P_1 \, b_{(t)} \qquad \text{.....................(2.2)}$$

where $b_{(t)}$ is the polynomial or exponential expression. A special form of this is the compound interest curve equation, i.e.

$$P_t = P_1(1 + r)^t \qquad \text{.............(2.3)}$$

where r is the average annual rate of growth.

In the land use based projection, the formula for the rate of population change in areas unaffected by redevelopment assumes only two factors and these values can vary by:

a. change in numbers of persons per household (household size)

b. change in numbers of household per dwelling (multi-occupancy), and is

$$X = \left(D \times \frac{H}{D} \times \frac{P}{A} \right) - P^{i} \times 100 \qquad \text{.............(2.4)}$$

where, D = the fixed number of dwellings

H/D = the house fields /dwellings ratio at the end of the period

P/A = the household size at the end of the period

P^{i} = the initial population in the dwelling stock.

The population technique that is chosen to use for any particular population study will depend on the time scale involved, the extent and nature of the study area and the desired end products of the study which will determine what specific facts need to be estimated.

2.4. Hydrology of Rainwater Systems

Amold and Andiran (1991), remark that the hydrology of rainwater collection needs to be approached differently according to whether one is dealing with runoff farming or collection of water from roofs. With the latter projects can be successfully initiated using very uncertain knowledge of rainfall amount, but in runoff farming, a detailed appreciation of rainfall intensity and runoff is essential from the start. Information about rainfall intensities is especially important in the context of soil erosion storms in which intensities exceeding 25mm/hr are very likely to cause erosion on exposed ground surfaces.

2.5. Rainfall and Runoff

Amold and Andiran (1991) states that rainfall is a point in the hydrology circle and when the soil infiltration capacity is higher than rainfall intensity, rainwater reaching the surface of the earth percolates into the ground to form parts of the shallow subsurface, saturated overhead or groundwater flows. Rainfall generation that is responsible for storm channel flow was found to be limited to the channel and riparian areas where saturated or nearly saturated condition eliminate or inhibit losses. Peak runoff decreases as the catchment area increases due to higher time of concentration. Low intensity storms over long period contribute to ground water storage and produce relatively less runoff. High intensity storm area covered by it increases the runoff since the losses like evaporation and infiltration are reduced. Succession of storm increases runoff due to initial wetness of the soil due to rainfall.

2.5.1 Flood Estimation: Many writers have derived formulas for flood estimation for practical uses but the most acceptable ones are

i) Dicken's formula: $Q = 825a^{0.75}$ ……………..(2.5)

where, a = square miles ($Mile^2$) and

Q = discharge, (ft^3/sec)

ii) Fuller method: $Q_m = Q_{AV}(1+0.8\log T)$ ……………..(2.6)

and $Q_{AV} = CA^{0.8}$ ……………..(2.7)

where Q_m = maximum flood, (m^3/sec)

Q_{AV} = average discharge, (m^3/sec)

T = return period. (year)

A = catchment area. (km^2) and

C = runoff coefficient. (dimensionless)

2.5.2 Rational Method:

The rational method has been the most popular in computing peak runoff for small waters sheds despite the fact that it is more than a century old; runoff is related to rainfall intensity by the formula

$$Q = 0.00278CiA \qquad \text{.......................(2.8)}$$

where Q = peak discharge (m^3/sec.)

A = catchment area (ha)

i = rainfall intensity (mm/hour) and

C = runoff coefficient. (dimensionless)

The rainfall intensity duration curve is illustrated in (Fig. 2.1), while (Table 2.1) provides typical values of runoff coefficients. Some assumptions for using this method are that:

i) the area covered does not change during the storm.

ii) the runoff coefficient does not change during the storm volume.

iii) the frequency of peak discharge is the same as the frequency of the rainfall intensity average.

iv) the runoff is maximum when the rainfall intensity lasts as long as the time of concentration is constant.

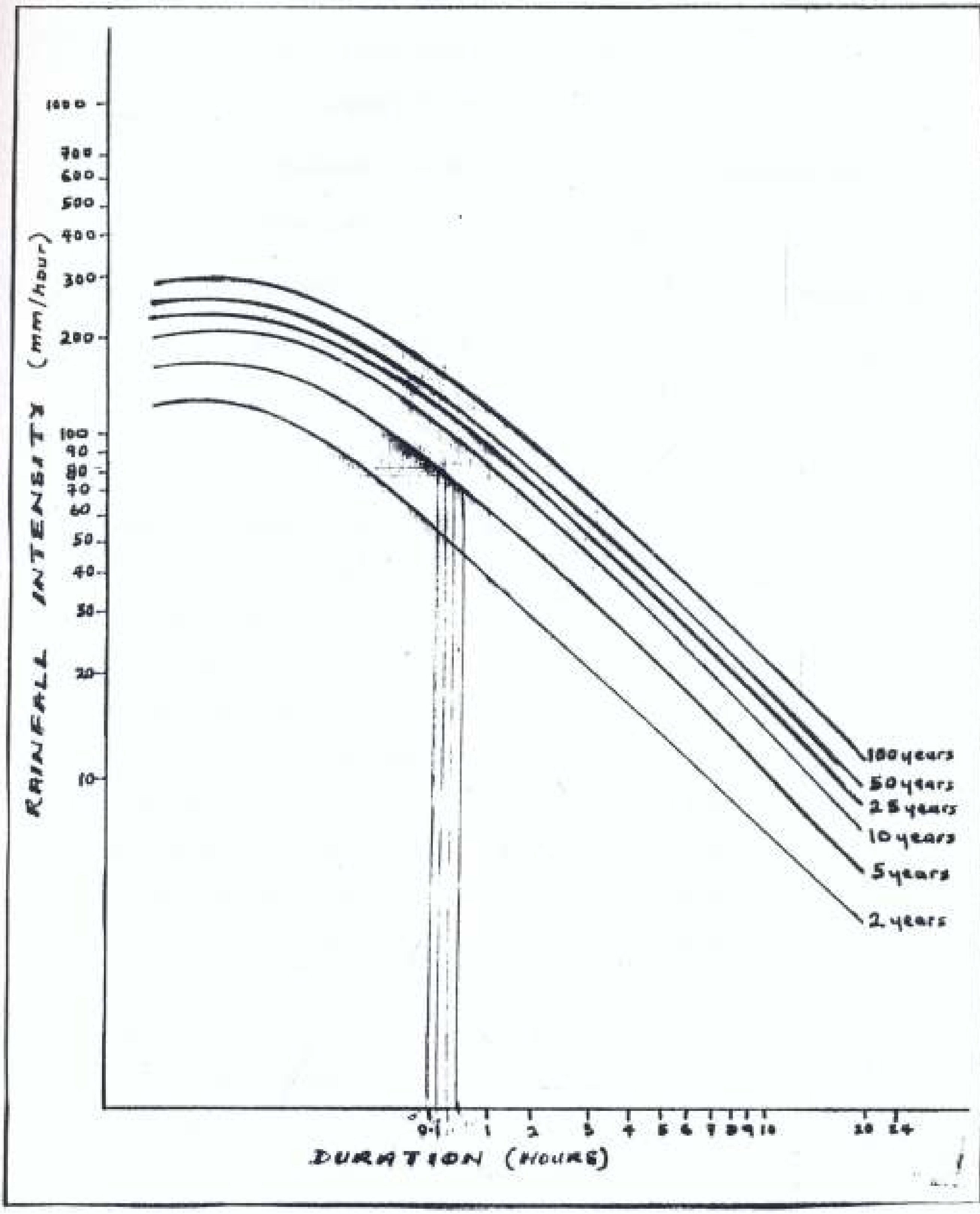


Figure 2.1. Rainfall intensity – duration curve for Akure.

v) the rainfall intensity must be constant for a time interval at least equal to the time of concentration

The time of concentration, Tc for small natural catchments is given by

$$Tc = 0.0663\,L^{0.77} / S^{0.385} \qquad \text{......................(2.9)}$$

where, L = length (m) of the longest diameter for the catchment runoff

S = slope of the catchment (dimensionless)

Tc = the time required for the runoff to become established and flow from the most remote part of the drainage area to the point under design (mins).

Table 2.1 Runoff coefficient, C for various surfaces.

Surfaces	Value of C
Asphalt or concrete	0.85 – 0.95
Parks, farmland, pastures	0.30 – 0.50
Forested area (depending on soil)	0.20 – 0.50
For composite areas	
Simple houses	0.30
Apartment houses (flats)	0.50
Commercial and industrial	0.90
Business areas depending on density	0.70-0.95
Apartment dwelling areas	0.50-0.70
Single family area	0.30-0.50
Parks and playgrounds	0.10-0.25
Paved streets	0.80-0.90
Lawns (depending on surface, slopes and soil characters)	0.10-0.25

(Source: Tchobanogous 1993)

2.6. Importance of Water Planning

Management of water resources is a critically important human activity. The quality and availability of water determine levels of public health, food production, the productivity of industry, the production of energy and other important aspects of the quality of life. Tchobanogous (1993), defines management of water to mean "people's control of water as it passes through its natural cycle with balanced attention to maximizing economics, social and environmental benefits". He also states that effective water planning is sometimes difficult, and overcoming the problems is one of the chief objectives of water planners. Grigg (1985), report that water planning is needed among other reasons for

i. sitting a new water supply reservoir or a treatment plant;

ii. development of integrated, multipurpose development plans for a river basin;

ii. development of the best policies for the regulation of contaminants that are discharged into waterways;

Planning for water resources must also take into account the integration of different purposes and interests if it is to be effective. John Kennedy, a former President of USA, is said to have said, "Anybody who can solve the problems of water will be worthy of two Nobel prizes – one for peace and one for science" (Grigg, 1985).

2.7. Data Collection and Design Criteria

The essence of appropriate technology is that equipment and techniques should be relevant to local resources and needs, to feasible patterns of organization and to the local environment. It is necessary instead to think in such a way of collecting information opinions and innovations from the users of the system as well (Davis and Cornwell,1985). According to the authors, if an introduced technology is to take root in a particular locality what is needed in addition to research, is a dialogue with all stakeholders; and that is what is called "interactive research". The technical assessment begins by seeking out formal rainfall records from weather stations near the project area, then the 'oral qualitative data base' asking people about wet and dry seasons, observing visual impression of rainfall intensity. Many development programmes fail, and much environmental degradation and soil erosion occur because local construction techniques are ignored or undervalued.

2.8 Water Sources

Water is a finite natural resource and in the context of global water use since 1950 many parts of the world are facing growing pressures on their water resources. Brandon (1984), wrote that the science of hydrology is concerned with the assessment of water resources in the hydrological cycle and their management for the optimum results. The earth and its surrounding atmosphere contain large amounts of water and about 7 percent of the earth's mass is made up of water. 95.5 percent of this water occurs as saline seawater, and much of the remaining fresh water is incorporated into the polar icecaps and glaciers. It is this water which takes part in the hydrological cycle and which fixes finite limits on availability. He gave the various sources of water as: Rainwater, Spring, Tube Wells (Bone holes), Hand-dug wells.

2.9 Water Quality and Quality Control

Water is the most important natural resource in the world since without it life cannot exist and most industries could not operate. This has been emphasized by many authors including Adebola (2001). The importance of safe water supply and effective sanitation was recognized centuries ago by several ancient civilizations. A major objective in water quality control work is to reduce the incidence of water-related diseases. According to Montegomery (1985), to gain a true understanding of the nature of a particular sample of water, it is necessary to measure several different properties by undertaking analyses under the broad headings of physical, chemical and biological characteristics. The cost of analytical work can be considerable and thus not all characteristics would be investigated for a particular sample. Table 2.2 gives examples of the properties most likely to be used for various types of sample tests.

2.9.1. Water Characteristics

Water characteristics are both physical and chemical. Some of the principal physical characteristics of water are Total Suspended and Dissolved Solids, Turbidity, Colour, Tastes and Odors, Temperature, Electrical Conductivity and

Table 2:2 Properties most likely to be used for various types of water sample to determine their quality

Characteristics	River Water	Drinking Water	Raw Sewerage	Sewerage Effluent
pH	*	*	*	*
Temperature	*	*	*	
Colour	*	*		
Turbidity	*	*		
Taste		*		
Odour	*	*		
Total Solids	*	*		
Settleable Solids			*	
Suspended Solids			*	*
Conductivity				
Radioactivity	*	*		*
Alkalinity	*	*		*
Acidity	*	*	*	
Hardness	*	*	*	
Dissolved oxygen (DO)	*	*		*
Biochemical oxygen demand (BOD)	*		*	*
Chemical oxygen demand (COD)	*		*	*
Total organic carbon (TOC)	*		*	*
Volatile organic carbon (VOC)	*	*	*	
Assailable organic carbon (AOC)	*	*	*	
Organic nitrogen			*	*
Ammonia nitrogen				*
Nitrite nitrogen	*	*	*	*
Nitrate nitrogen	*	*	*	*
Chloride	*	*		
Phosphate	*		*	*
Synthetic detergent	*		*	*
Bacteriological counts	*	*		

(Source Tchobanogous and Schoreder. 1985).

Radioactivity. The chemical characteristics of water which are quantified in terms of the inorganic and organic constituent are pH, Hardness, Alkalinity, Dissolved oxygen (DO), Chemical Oxygen Demand (COD), Nitrogen, and Chloride. The common tests used to characterize the chemical quality of water are summarized in Table2.3.

Table 2.3 Common analysis used to assess the chemical characteristics of water

Test	Abbreviation and definition	Use of the test results
	Inorganic properties	
pH	pH = log1h/[H^+]	To measure the acidity or basicity of an aqueous solution.
Dissolved cations		
Calcium	Ca^{2+}	
Magnesium	Mg^{2+}	
Potassium	K^+	
Sodium	Na^+	
Dissolved cations		To determine the ionic chemical composition of water and to assess the suitability of water for most alternative uses.
Bicarbonate		
Carbonate	HCO_3^-	
Chloride	Co_3^{2-}	
Hydroxide	Cl^-	
Nitrate	OH^-	
Sulfate	NO_3^-	
Alkalinity	SO_4^{2-}	To measure the capacity of the water to neutralize acids.
Acidity	$\Sigma HCO_3^- + Co_3^{2-} + OH^-$	To measure the amount of basic substance required to neutralize the water.
Carbon dioxide	CO_2	To assess the corrosiveness of water and the dosage requirements where chemical treatment is to be used; an be used to estimate pH if the bicarbonate concentration is known.
Hardness	Σ multivalent cations	To measure the soap-consuming capacity and scale-forming tendencies of water.
Conductivity	µmhos (microomhos) /cm (at 25°C)	To estimate the total dissolved solids or check on the results of a complete water analysis [total dissolved solids (TDS) in mg/L = 0.55 to 0.7 * conductivity value of sample in µmhos/cm]
Specific inorganic constituents		To assess the suitability of a water for use as a public water supply
	Organic properties	
Total organic carbon (TOC)		To measure the amount of organic material in a supply source.
Total organic halogen (TOX)		To determine the presence of halogenated compounds
Specific organic constituents		To assess the suitability of a water for use as a public water supply

* Detailed procedures for the tests listed in this table may be found in "Standard Methods for the Examination of Water and Wastewater, "17th ed., American Public Health Association, 1989. + These dissolved cations and anions are the principal ionic species found in most natural waters. ! µS(micro siemens)/cm in SI units. (Salvato. 1982). (Source: Montegomery, et al. 1985).

The most common microorganisms in water are bacteria, algae, fungi and protozoa. (Tchobanogous and Schroeder, 1985). Biological characteristics of water, also called Microorganisms are commonly present in surface waters, but they are usually absent from most groundwater because of the filtering action of the aquifers. The types of microorganism that may be found in water are classified as encaryotes, eubacteria or archaebacteria, depending on whether the cell contains a true nucleus. The primary process of surface water treatment is chemical clarification by coagulation, sedimentation, and filtration.

2.9.2 Water Processing

The objective of municipal water treatment is to provide a potable supply that is chemically and microbiologically safe for human consumption. For domestic uses, treated water must be aesthetically acceptable-free from apparent turbidity, color, odor and objectionable taste. Common water sources of municipal supplies are deep wells, shallow wells, rivers, natural lakes and reservoirs. The advantages and disadvantages of surface and ground potable water supplies as summarized by Allavi (1984) are detailed in Table 2.4.

The simplest treatment of disinfections and fluoridation is illustrated in Fig. 2.2. Deep well supplies are chlorinated to provide residual protection against potential contamination in the water distribution system. In the case of shallow wells recharged by surface waters, chlorine both disinfects the ground water and provides residual protection. Pollution and eutrophication are major concerns in

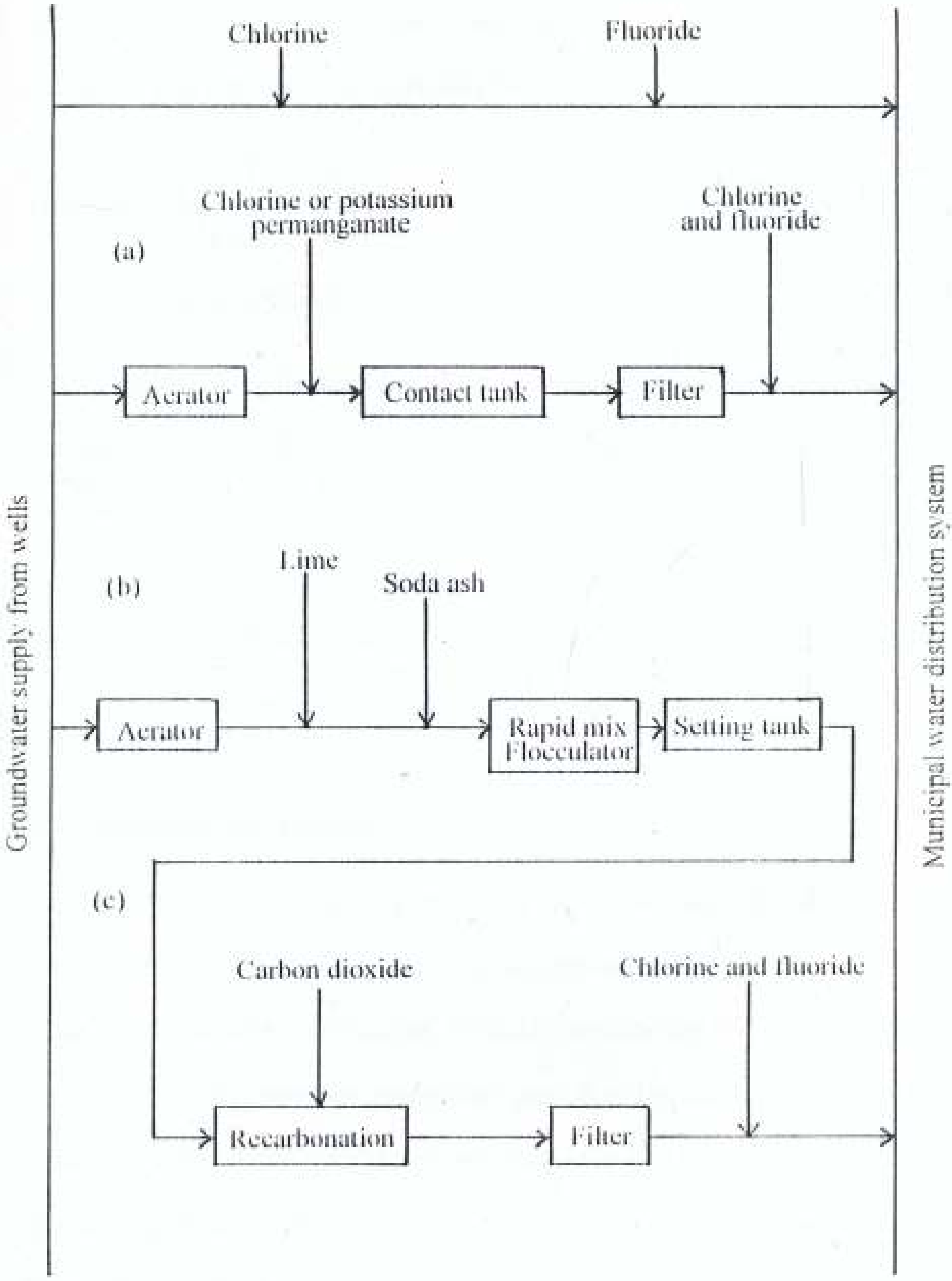


Figure 2.2 Flow diagrams of typical groundwater treatment systems.
(a) Disinfections and fluoridation. (b) Iron and manganese removal.
(c) Precipitation softening. Source, Allavi, (1984).

surface water supplies. Municipal water quality control actually starts with management of the river basin to protect the source of water supply.

Table 2.4 Advantages and disadvantages of surface and ground portable water supplies.

Surface Water		Ground Water	
Advantages	Disadvantages	Advantages	Disadvantages
Availability Visible Cleanable Low Fe and Mn Low H_2S Low hardness	Easily polluted Variable quality High color High turbidity Biological taste and color. High organics High trihalomethane formation potential	Protection Low color Low turbidity Low corrosivity Constant quality Low organics Low trihalomethane formation potential	Not cleanable Inaccessible Low H_2S Low hardness

Source, Allavi (1984)

2.9.3. Sampling and Analysis

To obtain an accurate representation of the composition and nature of water it is first essential to ensure that the sample analyzed is truly representative of the source. (Linsley and Franzini, 1994) Having satisfied this requirement it is then necessary to carry out appropriate analytical procedures suing standard techniques which have been developed specifically for water and wastewater analyses such as: Sampling, Analytical methods: Gravimetric analysis, Volumetric analysis, Colorimetric analysis, Visual method, Instrumental methods, Electrode technique, Automated analysis remote monitoring and sensing

2.10. Water Treatment Plants Design Methods

A water-supply system capable of supplying a sufficient quantity of potable water is a necessity for a modern city. The elements that make up a modern water supply system are detailed in Fig. 2.3.

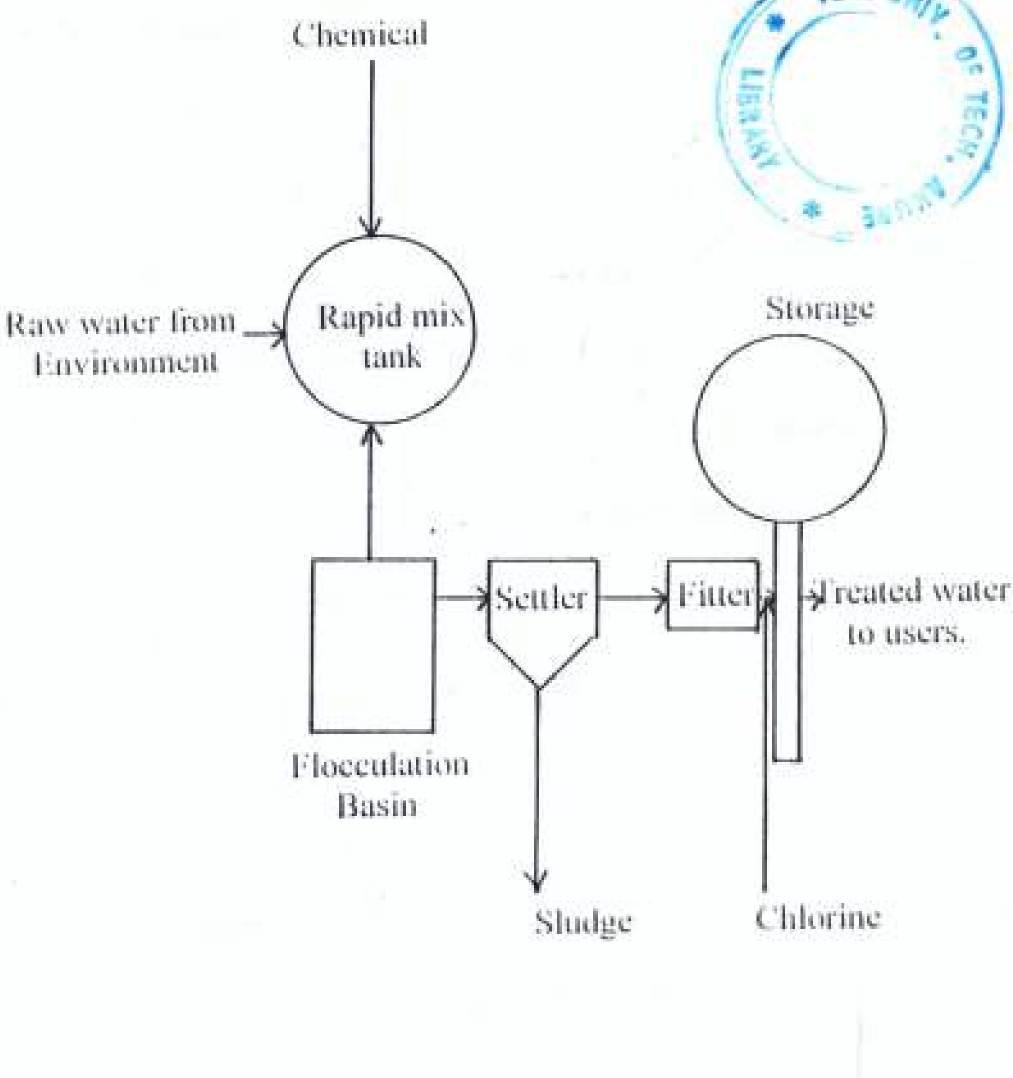


Figure 2.3 Typical process flow diagram of a portable treatment plant. Source. Allavi. (1984.)

The methods used for the treatment of water are related to the contaminants in a given water supply. The principal contaminants of concern in most water supplies are pathogenic bacteria, turbidity and suspended materials, color, tastes and odors, hardness, natural and synthetic organic compounds, and total dissolved solids, selected inorganic constituents. These factors are related primarily to health and aesthetics, (Pontius 1990).

In the development of public water supplies, the quantity and quality of the water are of paramount importance. The quantity is dependent on the population to be served, and the source where the water is to be harnessed. The quantity on the other hand is contingent upon the quality of the raw water and the level of treatment. The range of water treatment and the unit operations and processes involvement are illustrated in Figure 2.4 and Table 2.5. According to Australian Academy of Technological Sciences and Engineering (1988.) the main problem of sewer maintenance is that of keeping the sewer clear of obstructions. In no case should a workman be permitted to enter a sewer until proper tests for the presence of dangerous gases have been made.

2.10.1. Analysis and Design of Treatment Methods

One of the most important parameters used in the analysis and design of water treatment plant facilities is the detention time, the mean time of exposure of liquid to the phenomenon or forces responsible for treatment. That is;

Detention time, D_t = volume / flow rate Q ....(2.10)

Another commonly used parameter, which is related to the detention time, is the surface loading rate that is defined as:

Surface loading rate = flow rate, Q / surface area, As ....(2.11)

Both of these parameters are commonly used in the design of physical operations.

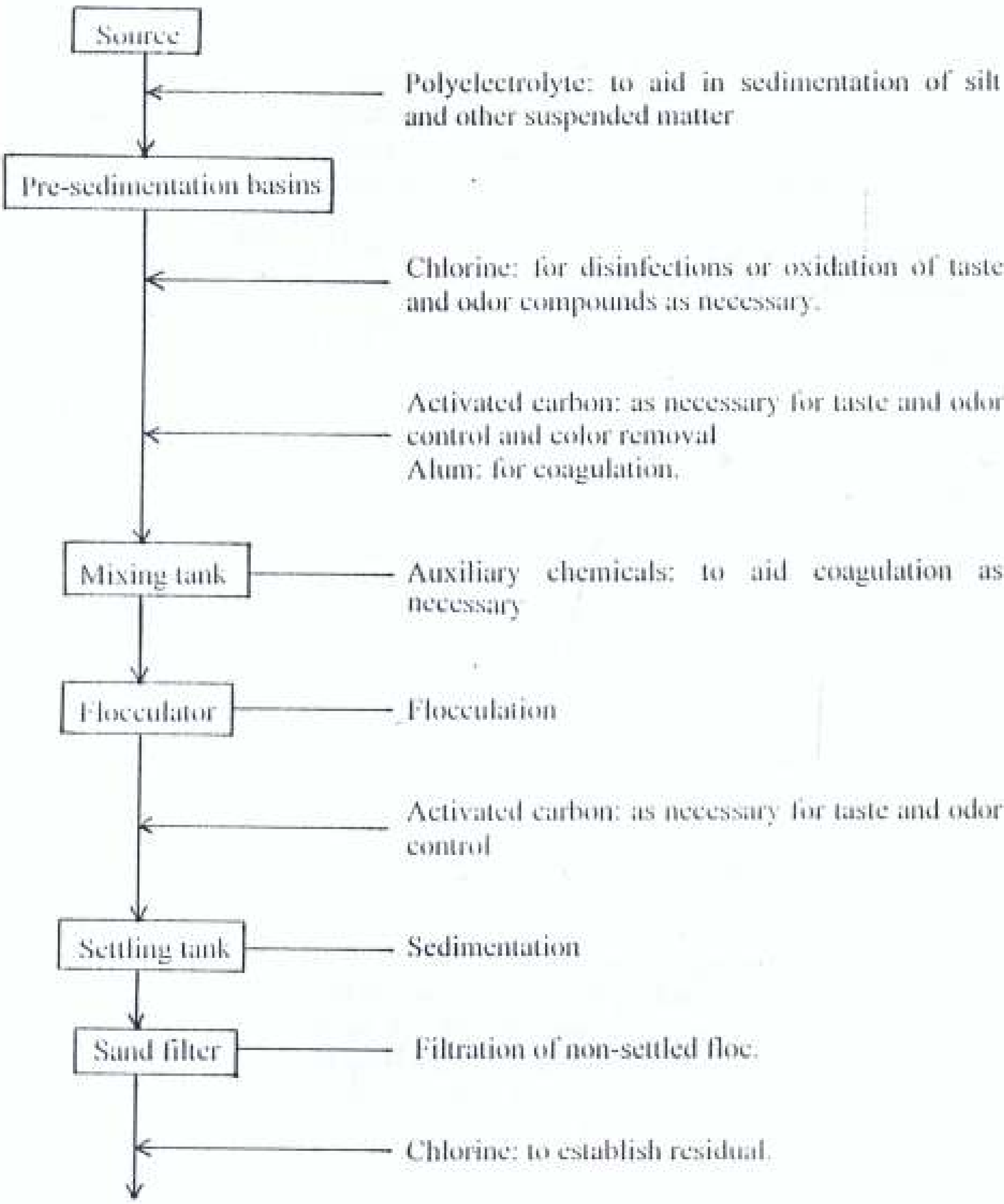


Figure 2.4: Schematics of water treatment plant for chemical-coagulation treatment plant with special provisions for taste and odor and for handling high turbidity. (Tchobanogous and Schoreder, 1985).

Table 2.5: Unit operations and processes and their application in the treatment of water.

Operation or process*	Application
Unit operation	
Screening	Coarse screens are used to protect pumps from floating solids. Fine screens are used to remove floating and suspended materials.
Micro screening	Used to filter out fine impurities such as algae, silt, etc.
Aeration (gas transfer)	Used to add and remove gases that may be under- or supersaturated with respect to the water. Air stripping is used for the removal of VOCs.
Mixing	Used to mix chemicals and gases that may be added for treatment.
Flocculation	Creation of velocity gradients by gentle mixing to promote the aggregation of particle.
Sedimentation	Used to remove particle such as silt and sand or flocculated suspensions.
Filtration	Used to filter the residual solids that remain in water after settling.
Membrane filtration	Used for the removal of natural organic compounds and SOCs. Also used for water softening.
Unit process	
Coagulation	The process of adding and initial mixing of chemicals used for the chemical treatment of water by precipitation.
Disinfection	Used to destroy the pathogenic organisms that may be present in natural water.
Precipitation	The removal of dissolved ionic species such as calcium and magnesium (hardness) by adding chemicals that brings about their precipitation.
Ion exchange	Used for the selection or complete removal of the dissolved cationic and anionic ions in solution.
Adsorption	Used for the removal of a variety of organic compounds such as those responsible for colour, taste, and odors.
Chemical oxidation	Used to oxidize various compounds that may be found in water such as those responsible for taste and odors.

*unit operations and processes are arranged more or less in their order of application in the treatment of water. (Source: Tchobanogous, 1993).

2.10.2. Water Distribution Systems

Extensive distribution systems are needed to deliver water to individual consumers in the required quantity and under a satisfactory pressure. If topographic conditions are favorable, gravity distribution is used, which requires a reservoir at a sufficient elevation above the ground so that water can reach any part of the distribution system with adequate pressure. The consumer is principally interested in the quality of water delivered to the tap, not the quality at the treatment plant. Therefore, water utility operations should be such that quality is not impaired as water flows from the treatment plant through the distribution system to the consumers. Storage and distribution systems should be designed and operated to prevent biological growths, corrosion, and contamination by cross-connections (Wagner and Shamier, 1988). The third basic objective of water treatment is that it be accomplished using facilities with reasonable capital and operating costs. Various alternatives in plant design should be evaluated for cost effectiveness and water quality produced. Alternatives developed should be based upon sound engineering principles and under consideration of flexibility for changing future conditions, emergency situations, operating personnel capabilities and future expansion.

2.10.3 Plant Siting

The selection of the purest drinking water source available should be the first priority. The source, the treatment plant and the distribution system should be thought of as a system. The better the source the less stress will be put on the rest of the system. Once the supply is chosen, its protection should be a priority. A treatment plant site close to power and sewerage is ideal, however availability of land, cost of the land, and taxes that will be incurred on land and plant often dictate where a plant will be located (Sanks.1980). The treatment process should be located above high ground water, but when this is not practicable, the structures must be prevented from floating by providing extra concrete, or by providing positive under drainage of the structures. The ground floor of all building must be located above high flood levels, and equipment and drives must be protected from flooding.

2.10.4 Plant Capacity and Layout

Present and future water consumption is dependent on the population served by the system, water use by residents, commercial and industrial establishments. The historical population and water use trends are first addressed, noting restrictions imposed on water use during droughts or dry seasons, so that the real demand on the system can be established. The treatment facility is normally designed for maximum 24-hr demand, so that an adequate amount of water will be treated and transmitted to the distribution's storage system throughout the year including days when usage is maximum. The design period for process and equipment is often 15 to 20 years because processes do become outmoded and equipment wears out. Structures are normally planned for longer periods, so the planning of a site and of building and civil structures often involves a 40 to 50 year planning period (Kawamura, 1991). Room is provided at the site for either replacing or adding equipment and for adding additional process units.

The preliminary design phase centers on siting the pumping and treatment facilities and laying out the processes and support facilities. The flow of water through the units should be kept as direct as possible and chemical lines should be kept as short as possible, therefore chemical storage and feed facilities should be located adjacent to the point of application. The plant should be a pleasant place to work in. The site should be planned to encourage separation of staff and visitors traffic from heavy truck movement (Metcalf and Eddy Inc., 1991). Signs are posted, where necessary, so that administrative staff and visitors take on route to their parking slots outside administrative areas, while chemical delivery and maintenance vehicles and operating staff go via another route for delivery of chemicals and for maintenance and parking.

Enclosed offices are provided for plant managers and engineers. The control room houses a desk and cubicles, and amenities are provide on operating floors and in maintenance areas. The plant laboratory is designed for conducting the routine testing and analysis necessary for controlling and monitoring the plant, including organic and microbiological tests. Special care is given to the design of counter and floor materials (Montgomery, 1985). When upgrading an existing facility, it is important to schedule construction so that operation is not impaired. Close attention must be paid to construct ability and sequence of operation in the contract documents.

Some of the items to be given proper attention are:

i) The design of process structures and building foundations to allow for simplicity of excavation and foundation engineering.

ii) Temporary operation of an existing plant throughout the construction period

iii) A complete detailed check of the construction documents to cut down on major and minor conflicts in the field.

iv) The contractor requirements for temporary utilities and storage of equipment

v) Training of the plant staff by the engineer and manufacturers

vi) An adequate start up period.

These items are always important but they are especially important when existing facilities are to be upgraded (Murfitt, 2002.)

The preliminary site plans are drawn to a reasonable scale so that the location of process tanks, building and road can be tied down; in addition, the plan show the drainage and landscape patterns and the location of large pipelines. The structure and architectural drawings replicate the shape and size of structures; decisions on building materials are show, and the process/mechanical drawings represent decisions on types and size of equipment. The structural dimensions of major walls and beams are determined.

The site plans represent several decisions, including:

i) The location of access roads and the placement of buildings for the convenience of staff, public visitors, chemical deliveries and vehicular maintenance.

ii) Solids handling

iii) Buffer zones, neighbors

iv) Future expansion

v) Architecture

vi) Geotechnical/Structure elevations

vii) Compactness of the layout.

2.11 Design of Water Treatment Plant Facilities.

According to American Society of Civil Engineers and American Water Works Association (1990), the degree to which water must be treated depends on the raw water quality and the desired quality of the finished water. The most common treatment plants for surface waters are rapid sand filtration plants and lime-soda softening plants. For groundwater's the most common plants are gas stripping and chlorination plants and softening plants, either lime-soda or ion exchange type. Slow sand filtration plants are sometimes used for surface waters.

2.11.1. Design Flowrates and Parameters.

Water treatment plants for municipalities are usually designed for a flowrate that will occur from 10 to 25 years in the future. This requires a population projection for the design period and also an estimate of the water demand per capita for the future (Cembrowicz, 1976). The most accurate way to determine the water demand per capita is by studying long-term water pumpage records. In the treatment of certain surface waters, preliminary treatment such as screening, pre-sedimentation, aeration, adsorption and prechlorination may be required.

2.11.2. Filter Layout, Appurtenances and Details.

The minimum number of filters required is two, however four are preferable. The filters are placed side by side in a row, and the pipe gallery, which contains all the necessary piping, valves, and so on, runs parallel to the filter row. The pipe gallery and operating floor are always enclosed to protect personnel and equipment. A dehumidified pipe gallery will reduce maintenance on controls, valves and other equipment (Walski, 1984). Pressure filters are usually cylindrical in shape and are prefabricated of steel with a maximum diameter of 3.05 to 3.66m and a maximum length of 18.29m. They should be equipped with a slight glass for observation of the bed during backwashing and should have an access manhole for necessary maintenance. Hydraulic removal of the filter media should be provided.

2.11.3. Construction and Maintenance of Sewers

There are many ways in which the actual construction of a sewer system may be accomplished, depending on the soil conditions encountered and the construction equipment available for the job. The important thing is that the finished sewer performs the function for which it is intended with a minimum of maintenance cost. According and whenever a workman enters a sewer, there should be a second person at the surface that can give emergency aid if required.

2.11.4. Pipe Network and Channel Analysis

The Hardy-Cross method is a computational procedure used to accurately design a water distribution network that has many crossover points. Crossover points or nodes illustrate that pipes can operate in more than one loop with many draw off points (Peavy, et. al, 1985). The first step in the design of a water-supply system is to estimate the requirement for water. After deciding on an average per-capital requirement based on the local conditions, the future population of the city must be estimated to determine average total use. Pipe network analysis involves the determination of the pipe flow rates and pressure heads, which satisfy the continuity and energy conversation equations

a. Open Channels

The Manning equation is used to aid in design and operation of open channel for effective results.

The Manning equation is

$$Q = n^{-1} (R^{2/3} AS^{1/2}) \qquad \text{.................. (2.12)}$$

where n	=	coefficient of roughness (dimensionless)	
A	=	channel area	(m^2)
S	=	slope of channel	(dimensionless)
Q	=	discharge	(m^3/sec)
R	=	A/P	(m)
P	=	wetted perimeter	(m)

b. Closed Conduits (Pipe Networks)

In water-distribution systems, the Hazen-Williams formula has been used to determine pipe sizes and friction loss for a given flow rate.

$$Q = 0.849CR^{0.63}S^{0.54} \qquad \text{.................. (2.15)}$$

Flow rate is added to the Hazen-Williams formula when the continuity equation is substituted for velocity. The continuity equation states that flow rate is the product of flow area and velocity, that is:

$$Q = AV \qquad \text{...................... (2.16)}$$

$$R = A/P$$

$$= \pi D^2 / 4\pi D$$

$$= D/4 \qquad \text{.........................(2.17)}$$

Substituting into the Hazen-Williams formula (2.15)

$$Q = 0.278CD^{2.63}S^{0.54} \qquad \text{..................... (2.18)}$$

where	V	=	average velocity.	(m/sec)
	R	=	hydraulic gradient	(dimensionless)
	S	=	hydraulic slope,	(dimensionless)
	h_l	=	frictional loss	(m)
	L	=	length of pipe	(m)
	P	=	wetted perimeter.	(m)
	C	=	friction coefficient	(dimensionless)
	n	=	coefficient of roughness	(dimensionless)
	A	=	cross-sectional area of the flow.	(m^2)

TABLE 2.6 Hazen – Williams's coefficients.

Type of pipe	C
PVC (plastic)	140
Finished wood and masonry	120
Vitrified clay	110
Old cast iron	100
Old riveted steel	95

(Source: Linsley and Franzini, 1994)

2.12. Engineering Economics

The practice of engineering involves many choices among many alternatives designs, procedures, plans and methods. The available alternative courses of action involve different amounts of investment and different prospective receipts and disbursements. To be able to judge whether any proposed course of action will prove to be economical in the long run as compared to other possible alternatives calls for an economy study (Barhour, et. al, 1982). An economy study may be defined as a comparison between alternatives in which the differences between the alternatives are expressed so far as practicable in money terms.

2.12.1 Factors of Engineering Economy

When engineering alternatives involve capital investments for equipment, materials, and labor, the techniques of economic analysis may be used to assist in determining the best alternative. This is based on the comparison of the monetary

Table 2.7 Values of n to be used with the Manning equation.

Surface	Best	Good*	Bad
Uncoated cast-iron pipe	0.012	0.013	0.015
Coated cast-iron pipe	0.011	0.012	0.013
Commercial wrought-iron pipe galvanized	0.013	0.014	0.017
Vitrified sewer pipe	0.010	0.013	0.017
Cement mortar surfaces	0.011	0.012	0.015
Concrete-lined channels	0.012	0.014	0.018
Cement-rubble surface	0.017	0.020	0.030
Canals and ditches			
Earth, straight and uniform	0.017	0.020	0.025
Rock cuts, smooth and uniform	0.025	0.030	0.035
Rock cuts, jagged and irregular	0.035	0.030	0.045
Winding sluggish canals	0.0225	0.025	0.030
Dredged earth channels	0.025	0.0275	0.033
Canals with rough stony edges, weeds on earth banks	0.025	0.030	0.040
Earth bottom, rubble sides	0.028	0.030	0.045
Natural-stream channels			
1. Clean, straight bank, full stage, no rifts or deep pools	0.025	0.0275	0.033
2. Same as (1), but some weeds and stones	0.030	0.033	0.040
3. Winding, some pools and shoals, clean	0.033	0.035	0.045
4. Same as (3), lower stages, more ineffective slope and sections	0.040	0.045	0.055
5. Same as (3), some weeds and stones	0.035	0.040	0.050
6. Same as (4), stony sections	0.045	0.050	0.060
7. Sluggish river reaches, rather weedy or with very deep pools	0.050	0.060	0.080
8. Very weedy reaches	0.075	0.100	0.150

*Frequently used for design. (Source: Linsley and Franzini, 1992)

values of the alternatives selected for implementation. These estimates are obtained from actual cost of the components involved in the alternative methods analyzed with experience from the past, and judgment. The engineering design to accomplish a specific test may be best possible, but if it is not economically competitive, the design should not be implemented. Economic factors for comparison are expressed in standard formulas. (Grant, et al., 1990).

2.12.2. Economic Evaluation

Economic evaluation is aimed at assessing whether the value of the final products exceeds the value of the resources used by the project. The values of the outputs when measured in economic terms are called "the benefits of the project" and the values of the resources used in its construction and maintenance when measured in economic terms are called "the costs of the project" (Dandy and Warner, 1989). In the private sector, benefits and costs are measured by cash flows into and out of the firm, respectively, while for public sector projects, benefits and costs must be considered for society as a whole, that is, benefits and cost are not necessarily associated with cash flow

2.12.3 Discontinued Cash Flow Techniques for Project Evaluation

For projects that have to be financially viable, the cost of the project has to be recovered over a period of time. The interest rate applied to the project cost will depend on the source of the capital. Money from the shareholders' funds is generally cheapest, but not always available. The interest rate applied to the cost of a project may be based on:

i) the interest rate for borrowed money (either a current rate or a forecast rate)

ii) the rate of return earned by the capital already invested in the business, or some similar criterion and a combination of the external and internal rates (Dandy and Warner, 1989.)

Because of interest charges, money that becomes available in the future is worth a proportion of its current value. This money in future years is discounted by a present value factor, to give the present value. This is perhaps easier to understand from the conversion process, namely compounding.

This is expressed as:

$$\text{Value of investment at year, } n = p\,(1+i)^n \qquad \text{.....................(2.19)}$$

The capital recovery factor, which indicates the rate at which any major investment can be recovered over the year, is given by

$$CRF = i\ (1+i)^n / ((1+i)^n - 1) \qquad \text{.............. (2.20)}$$

where, i = the interest rate (%).

n = the number of time intervals (Years.).

Table 2. 8: Standard factor notations

Factor name	Standard	To fund	Given	Factor	Equation	Formula
Single – payment present-worth factor (SPPWF)	(P/F, i%, n)	P	F	(P/F, i%, n)	P= F (P/F, i%, n)	$P = F\left[\frac{1}{(1+i)^n}\right]$
Single – payment compound-amount factor (SPCAF)	(F/P, 1%, n)	F	P	(F/P, i%, n)	F= P (F/P, i%, n)	$F = P\,(1+i)^n$
Uniform-series present factor (USPWF)	(P/A, i%, n)	P	A	(P/A, i%, n)	P= A (P/A, i%, n)	$P = A\left[\frac{(1+i)^n-1}{i(1+i)^n}\right]$
Capital-recovery factor (CRF)	(A/P, i%, n)	A	P	(A/P, i%, n)	A= P (A/P, i%, n)	$A = P\left[\frac{i(1+i)^n}{(1+i)^n-1}\right]$
Sinking fund factor (SFF)	(A/F, i%, n)	A	F	(A/F, i%, n)	A= F (A/F, i%, n)	$A = F\left[\frac{i}{(1+i)^n-1}\right]$
Uniform-series compound-amount factor (USCAF)	(F/A, i%, n)	F	A	(F/A, i%, n)	F= A (F/A, i%, n)	$F = A\left[\frac{(1+i)^n-1}{i}\right]$
Uniform – gradient Present Worth	(P/G, i%, n)	P	G	(P/G, i%, n)	P= G (P/G, i%, n)	$P = G\,\frac{1}{i}\left[\frac{(1+i)^n-1}{i(1+i)^n}-\frac{n}{(1+i)^n}\right]$
Uniform – gradient Annual Series	(G/P, i%, n)	G	P	(G/P, i%, n)	G= P (G/P, i%, n)	$P = G\left[\frac{1}{i}-\frac{n}{(1+i)^n-1}\right]$

CHAPTER THREE

RESEARCH METHODOLOGY

3.1 Present Conditions of the Area's Services

Reconnaissance study of the estate shows that there are a few duplexes and multiple rooms buildings while most the other buildings are blocks of flat. The roofing systems of most of these buildings are with gable roofs and very few-hipped roof.

The main source of domestic water supply to most of the houses is the hand dug well. In the low land areas, water levels of these wells were less than 1.5m below the ground surface while in the high land areas, the water levels are as deep as 10m below the ground level. Generally only very few houses pump their wells while majority fetch directly from their wells by bucket and rope devices. Only few houses however use boreholes as their source of water supply. The roads in the estate have no side drains as all of them were opened-up by the individual residents using them. These roads have to be graded after every raining season, to repair eroded surfaces. The only social amenity provided by government in the area is electricity supplied by the National Electric Power Authority, (NEPA).

3.2 Detailed Surveys of the Estate

A field survey of the Estate was conducted to assess the landscape, type of buildings and materials, shape or type of their roofs, length of conduits and drainages and possible sites for the location of the water treatment plant. Samples of stormwater and streamwater were collected and laboratory analyses carried out on them to determine their physical and chemical characteristics. The results were used to estimate the level of treatment required by the raw water to be potable. The existing system of collection and conveyance of the raw water, from the various houses, streets and open lands was evaluated with a view to designing a new conveyance system to the stream, and locating the point of abstraction of water for treatment.

The water requirement for the Estate was determined from population analysis and projection up to the design period. A water treatment plant that will meet this requirement and the associated sewers, drainage network and the mains pipe network for the Estate was then designed. To determine the economic feasibility of the project, costs of establishing these facilities were estimated, and compared with the costs of direct supply of potable water from the State's Water Corporation and from individual wells and boreholes.

Optimal designs of the treatment plant, drains and conduits require the topographic plan of the area, and spot heights of the estate roads network. Reconnaissance survey was conducted by going through the area occupied by the estate to determine the physical features. These features include the types of buildings and their roofs, number and the depths of hand-dug shallow wells, the types of drainages provided by individual landlords if any, the direction of their flow and runoff effect on existing roads, number of developed and undeveloped plots and the areas affected by the existing streams. The leveling and spot heights of the estate roads were carried out to determine the direction for the drain flow and the best route for water conduits and the distribution system. The location of the proposed treatment plant was also determined.

Other factors which were determined include:

a. The landscape of the area and all its features.

b. The physical conditions of the wells and boreholes in the estate.

c. Number of streams, their types and conditions.

d. The estimated human population of the estate.

e. The social amenities in the area.

3.3 Survey of the Project Area

a. The reconnaissance survey of the whole area was undertaken to know the physical conditions of the area covered by the estate and the types of structures in the Estate.

b. Visits were paid to the responsible Ministry for the estate, that is, the Ministry of Lands to determine what plan it had for the Estate in area of water supply and storm, stream and groundwater (wells and boreholes) treatment.

c. Existing storm, stream and groundwater (wells and boreholes) systems of the area and its immediate environments were studied.

d. Rainfall depths for the Estate were calculated using that of Akure town for a period of three years.

e. Spot levels on the estate roads were taken to determine their relative heights in order to determine the direction of flows for drainages and the placement of the treatment plant of this study.

3.4 Samples Collection and Laboratory Analysis

a. Samples of groundwater (wells and boreholes) from some residence and storm water in the estate were taken to determine their qualities.

b. Samples were also taken from the Ala River tributary the main stream in the area, at the upstream, midstream and downstream during and after the rain.

c. Laboratory examinations were conducted on the collected samples to determine some of their physical, biological and chemical properties.

3.5 Data Analysis

Various data obtained from the surveys and laboratory examinations conducted were analyzed to obtain figures for the design of the various components of the proposed treatment plant.

3.6 Design of Proposed Treatment Plant Component

Based on the data collected, appropriate designs were prepared for the following:

a) the drainage network

b) the raw water impoundment

c) the mixing plant

d) the filtration tanks

e) the sedimentation tanks

f) the treated water storage tanks

g) the pumps, and

h) distribution pipes.

Other components of the treatment plant where not designed for but assumed to be in conformity with the normal treatment plants constructions.

Costs analysis of the proposed facilities was estimated and compared with the current public water supply.

CHAPTER FOUR

DESCRIPTION OF STUDY AREA

4.1 Study Area

Akure town, the capital of Ondo State since its creation on February 3, 1976, is located in the rain forest zone of Nigeria. It has an annual average rainfall of between 2000 – 3000mm and a humid tropical climate with daily temperature of between 20^0C and 30^0C generally through out the year. The town lies between latitude $7^0\,5^1$ and longitude $5^0\,15^1$ and occupies a land area of about 23 km^2. Akure is densely populated with about 320,000 people according to the 1992 census data. The town has two main roads running through it with other smaller ones inter-connecting them from other areas of the town as shown in Fig. 4.1. (Drawing Sheet).

Akure being also the headquarters of Akure South Local Government Area is fast expanding towards Idanre, Ijoka, Oba-Ile , Iju, Ogbese, Ibule, Orita-Obele, Irese, Aule and other surrounding villages.

4.2. Ilesha Road Government Residential Area.

Ilesha Road Government Residential Area, shown in Fig. 4.2 (Drawing Sheet), is located in the southern part of Akure town and occupies an area of land of approximately 2.75 km^2. It is one of the residential estates created by the Ondo State Government since 1980. The estate is professionally planned to have all the social facilities such as schools, hospital, market place, places of worship, etc. It is bounded by Ilesha Road, where it got its name, FUTA Road and some private estates, like Apatapiti, Osokiti, Alaba, etc. It is divided into two parts by Awule Road and one of River Ala tributary.

The topography of the estate, shown in Fig 4.3 (Drawing Sheet), is such that the southern end is hilly. Table 4.1 shows the results of the spot-heights carried out on the estate roads.

The Ala tributary flows from the east to the west and from the northern to the southern part of the estate, with discharges of storm water along its course. This stream is planned to play the important role of being the main source of raw water for this study. The estate presently has 272 low, 285 medium and 371 high density plots. This plan may change due to human and natural considerations. Only about 70% of the total plots are presently being developed. No social facilities have been provided by the authorities concerned to date; instead the places earmarked for them are being redesigned and allocated for residential plots.

4.3. Social Facilities to the Estate

The provision of social facilities to the estate would have been zero percent but for the presence of electricity and two primary schools, one of which is privately owned. There are 928 residential plots, 65 proposed roads, as shown in Fig 4.2 (Drawing Sheet), and yet none of these roads can be considered standard under whatever consideration. The estate does not have a market place, no public schools at all levels or other social amenities like police station, etc. Public water supply system is non-existent. The present source of water supply is groundwater obtained by individuals through shallow hand-dug wells, a few boreholes and some cases rainwater harvesting.

Table 4:1 Value of Spot Heights on Estate Roads.

Roads No	Distance of Roads	Height of Spot A	Height of Spot B	Roads No	Distance of Roads	Height of Spot A	Height of Spot B
1.	250	1.51	3.41	39.	733	0.43	3.12
2.	381	2.93	4.91	40.	147	0.13	2.11
3.	154	1.06	2.55	41.	147	4.12	4.88
4.	785	3.75	6.56	42.	280	1.66	2.11
5.	60	0.26	0.31	43.	541	0.26	2.80
6.	96	0.65	1.10	44.	296	1.15	1.60
7.	223	1.05	1.81	45.	251	1.83	0.58
8.	482	1.01	3.37	46.	170	0.50	1.32
9.	184	0.45	1.98	47.	469	1.87	2.46
10.	360	1.31	2.21	48.	63	1.86	2.15
11.	606	2.36	3.15	49.	370	1.15	4.11
12.	190	0.94	2.88	50.	277	0.63	2.68
13.	230	0.54	4.77	51.	140	0.23	1.89
14.	473	0.85	4.54	52.	102	0.21	0.41
15.	199	1.87	3.50	53.	100	0.66	1.12
16.	224	0.20	1.59	54.	316	1.76	3.21
17.	190	0.72	2.07	55.	653	1.38	2.23
18.	390	2.49	6.77	56.	100	0.32	1.06
19.	800	1.95	9.23	57.	110	0.00	0.91
20.	242	0.48	1.52	58.	580	1.62	6.09
21.	250	0.30	1.03	59.	184	0.15	1.38
22.	892	1.44	7.35	60.	100	0.55	1.43
23.	240	0.72	2.06	61.	272	0.52	0.76
24.	351	0.41	3.71	62.	923	0.25	10.31
25.	100	0.19	0.36	63.	336	2.55	3.49
26.	1,041	1.91	10.24	64.	111	1.03	2.06
27.	64	0.91	1.20	65.	77	1.20	1.91
28.	462	1.46	4.88	66.	98	1.11	1.23
29.	226	0.84	2.99	67.	73	0.34	1.25
30.	202	0.86	1.65	68.	233	0.29	0.64
31.	165	1.29	2.37	69.	80	0.47	0.63
32.	400	1.96	6.00	70.	498	0.68	1.38
33.	88	2.92	2.92	71.	90	0.76	0.82
34.	188	1.77	3.07	72.	90	0.52	0.64
35.	850	0.80	7.94	73.	64	1.78	1.88
36.	530	0.58	4.61	74.	95	1.13	1.26
37.	150	0.89	1.19	75.	49	1.94	2.21
38.	458	1.16	1.98				

4.4 General Climate Conditions

The rainfall pattern in the estate is not different from that of Akure town. Table 4.2 shows the rainfall depth for Akure town. Raining season in the estate, like in

Table 4.2 Yearly Rainfall Value of Akure (mm)

MONTH/YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEPT	OCT	NOV	DEC
1980	0.0	41.8	64.6	129.8	83.0	298.0	104.1	226.0	246.0	179.9	43.6	1.30
1981	0.0	22.4	89.1	253.3	223.3	258.6	148.2	152.7	210.6	55.9	10.1	0.0
1982	10.6	68.1	79.3	221.8	211.8	169.8	180.9	52.2	131.0	269.6	11.0	0.0
1983	0.0	26.2	7.9	161.9	254.5	194.8	88.8	65.9	313.6	142.0	6.0	62.9
1984	0.0	0.0	174.9	189.6	203.0	218.3	151.3	206.6	197.1	166.1	0.5	0.0
1985	0.0	0.0	155.7	146.5	220.9	186.7	231.3	236.7	252.6	99.5	24.3	0.0
1986	12.7	150.2	130.8	41.8	157.5	142.2	100.6	77.5	217.9	82.0	46.6	0.0
1987	7.7	49.1	107.1	94.3	93.1	116.7	158.1	384.4	276.4	113.7	0.0	0.8
1988	18.4	44.1	74.3	182.0	156.9	203.5	193.1	82.6	278.1	229.9	44.7	17.6
1989	0.0	0.0	117.2	59.1	183.7	226.8	199.6	357.4	175.5	111.5	19.1	0.0
1990	4.4	15.5	0.0	158.5	113.6	90.7	370.3	245.8	230.9	213.3	72.4	62.5
1991	1.2	98.6	136.0	223.2	210.2	162.9	463.0	203.6	200.7	152.6	0.0	10.4
1992	0.0	0.0	40.8	107.8	151.1	273.9	265.3	101.7	348.1	194.6	35.6	0.0
1993	0.0	57.0	18.8	84.6	95.1	104.1	122.2	274.1	303.7	95.9	72.5	21.4
1994	11.2	39.4	44.4	97.8	169.9	225.8	147.6	162.8	181.3	157.3	28.0	0.0
1995	0.0	28.4	120.0	196.3	146.4	214.2	268.6	379.6	262.3	87.3	14.3	0.0
1996	0.0	68.0	67.8	218.5	220.0	154.0	106.2	116.2	286.8	111.0	17.6	0.0
1997	33.3	0.0	90.9	191.9	187.0	207.3	218.3	146.7	170.1	204.4	53.2	4.5
1998	0.0	23.7	10.9	122.3	231.1	243.5	154.3	90.4	256.3	273.3	46.8	0.0
1999	20.3	35.7	57.0	117.3	183.5	169.2	98.0	223.0	299.0	120.0	71.6	3.6
2000	44.7	0.0	34.9	207.2	114.9	299.3	215.3	215.9	232.6	137.0	19.6	5.4

(Source: FUTA, 2003))

Akure, is between March and October while the dry season is between November and February. Humidity ranges between 50 to 90% with an estimated mean annual humidity of 72%. The estate is generally slopy and uneven, allowing the storm water to flow into the few streamlets which eventually empty into Ala River tributary, the main stream in the estate.

The runoff from the estate is not properly controlled. It is allowed to flow on its natural course which is eroding the road surfaces. Measurement of the runoff yield was taken in catchments and the result is shown in Table 4.3.

Table 4.3 Estates Runoff Drains Catchments Characteristics Values

Runoff drains catchments characteristics									
Codes	Drain Numbers	Length m	Area m^2	Runoff coefficient	Codes	Drain Numbers	Length m	Area m^2	Runoff coefficient
A1	1	150	16500	0.5	B10	47	48	4320	0.5
A2	2	100	9800	0.5	B11	48	36	3240	0.5
A3	3	80	10240	0.5	B12	49	100	10100	0.5
A4	4	80	7200	0.5	B13	50	100	9200	0.5
A5	5	310	41880	0.5	B14	51	100	11000	0.5
A6	6	251	14339	0.5	B15	52	99	14256	0.5
A7	7	73	8030	0.5	B16	53	224	21280	0.5
A8	8	150	16500	0.5	B17	54	90	8550	0.5
A9	9	158	14694	0.5	B18	55	140	20720	0.5
A10	10	70	8050	0.5	B19	56	50	4850	0.5
A11	11	250	27292	0.5	B20	57	101	10706	0.5
A12	12	135	12960	0.5	B21	58	113	6667	0.5
A13	13	90	5850	0.5	B22	59	150	17700	0.5
A14	14	25	3700	0.5	B23	60	100	9200	0.5
A15	15	130	16770	0.5	B24	61	283	25187	0.5
A16	16	120	15600	0.5	B25	62	250	23500	0.5
A17	17	154	14694	0.5	B26	63	102	10404	0.5
A18	18	78	7722	0.5	B27	64	140	13720	0.5
A19	19	184	14904	0.5	B28	65	280	33600	0.5
A20	20	60	4800	0.5	B29	66	70	2590	0.5
A21	21	110	10100	0.5	B30	67	100	9000	0.5
A22	22	98	8722	0.5	B31	68	240	22080	0.5
A23	23	40	3520	0.5	B32	69	100	9800	0.5
A24	24	96	8544	0.5	B33	70	420	58800	0.5
A25	25	80	6640	0.5	B34	71	100	10000	0.5
A26	26	88	7.656	0.5	B35	72	128	16256	0.5
A27	27	190	15960	0.5	B36	73	197	18443	0.5
A28	28	95	15950	0.5	B37	74	187	16388	0.5
A29	29	92	8280	0.5	C1	75	172	17888	0.5
A30	30	90	7380	0.5	C2	76	190	20140	0.5
A31	31	98	10878	0.5	C3	77	100	11800	0.5
A32	32	64	7650	0.5	C4	78	71	6887	0.5
A33	33	90	9812	0.5	C5	79	140	11800	0.5
A34	34	180	18300	0.5	C6	80	165	16510	0.5
A35	35	70	3200	0.5	C7	81	93	9579	0.5
A36	36	110	8910	0.5	C8	82	86	7654	0.5
A37	37	90	9810	0.5	C7	81	93	9579	0.5
B1	38	100	8900	0.5	C8	82	86	7654	0.5
B2	39	49	5530	0.5	C7	81	93	9579	0.5
B3	40	150	21000	0.5	C8	82	86	7654	0.5
B4	41	190	22040	0.5	C9	83	86	13020	0.5
B5	42	106	10600	0.5	C10	84	64	7654	0.5
B6	43	230	34040	0.5	C11	85	116	10208	0.5
B7	44	200	20200	0.5	C12	86	110	6710	0.5

Table 4.3 (Cont.) Estates Runoff Drains Catchments Characteristics Values

Runoff drains catchments characteristics									
Codes	Drain Numbers	Length	Area	Runoff coefficient	Codes	Drain Numbers	Length	Area	Runoff coefficient
		m	m^2				m	m^2	
B8	45	100	9000	0.5	C13	87	88	8624	0.5
B9	46	197	18464	0.5	C14	88	173	23340	0.5
C15	89	100	11800	0.5	D18	127	110	4070	0.3
C16	90	156	32448	0.5	D19	128	94	5640	0.3
C17	91	86	8428	0.5	D20	129	90	5670	0.3
C18	92	150	31050	0.5	D21	130	100	9400	0.3
C19	93	304	36856	0.5	D22	131	272	25314	0.3
C20	94	147	20850	0.5	D23	132	90	3150	0.3
C21	95	147	20580	0.5	D24	133	110	6314	0.3
C22	96	130	15860	0.5	D25	134	80	9240	0.3
C23	97	150	28350	0.5	D26	135	208	16428	0.3
C24	98	155	24800	0.5	D27	136	100	6800	0.3
C25	99	200	105800	0.5	D28	137	100	7000	0.3
C26	100	186	18224	0.5	D29	138	273	4815	0.3
C27	101	200	15600	0.5	D30	139	98	7840	0.3
C28	102	96	7008	0.5	D31	140	190	16810	0.3
C29	103	72	9218	0.5	D32	141	346	32,137	0.3
C30	104	251	19540	0.5	D33	142	72	750	0.3
C31	105	70	7400	0.5	D34	143	337	18518	0.3
C32	106	63	4851	0.5	D35	144	73	749	0.3
C33	107	100	4851	0.5	D36	145	160	8320	0.3
C34	108	90	9900	0.5	D37	146	79	12640	0.3
C35	109	307	39970	0.5	D38	147	80	5600	0.3
D1	110	730	68580	0.5	D39	148	100	7600	0.3
D2	111	78	4056	0.3	D40	149	102	10404	0.3
D3	112	110	10670	0.3	D41	150	151	10721	0.3
D4	113	150	23250	0.3	D42	151	118	14160	0.3
D5	114	280	22680	0.3	D43	152	85	6970	0.3
D6	115	57	5187	0.3	D44	153	90	3510	0.3
D7	116	120	12120	0.3	D45	154	80	6000	0.3
D8	117	190	16810	0.3	D46	155	78	6970	0.3
D9	118	91	8918	0.3	D47	156	273	7840	0.3
D10	119	150	22500	0.3	D48	157	71	4815	0.3
D11	120	240	12240	0.3	D49	158	76	782	0.3
D12	121	111	7548	0.3	D50	159	133	7581	0.3
D13	122	60	1800	0.3	D51	160	300	71100	0.3
D14	123	77	6776	0.3	D52	161	80	4720	0.3
D15	124	66	3828	0.3	D53	162	70	756	0.3
D16	125	90	7110	0.3	D54	163	100	7600	0.3
D17	126	180	18000	0.3	D55	164	140	20720	0.3

4.5 Water Quality

Water quality in the estate varies from point to point. General indication of the overall water quality in the estate is based on samples collected at some specific locations. Analysis of the samples taken from the Ala steam and other sources are shown in Tables 4.4 to 4.6.

Table 4.4 Physical and Chemical Characteristics of Water Sources in the Study Area.

Laboratory Examinations			Samples Codes	Temperature	Turbidity	Colour	Total Solid	Conductivity	Cadmium
Laboratory Examinations Units				^0C	JTU	^{0}Hazen	mg/l	µS/cm	mg/l
World Health Organization	Highest Desirable Level		A1	28	5	5	500	-	0.005
	Maximum Permissible Level		A2	27	25	50	1500	-	0.01
Samples of Ala River Tributary	During the Rains	Up Stream	B1	26	20	40	800	260	0.006
		Mid Stream	B2	26	20	40	800	260	0.006
		Down Stream	B3.	26	20	40	800	260	0.006
	After the Rains	Up Stream	B4	27	10	15	600	260	0.006
		Mid Stream	B5	27	10	15	600	260	0.006
		Down Stream	B6	27	10	15	600	260	0.006
Samples of Stormwater	Low Density Area		C1	26	24	40	750	240	0.005
	Medium Density Area		C2	26	24	48	850	280	0.005
	High Density Area		C3	27	24	60	1000	330	0.005
Samples of Groundwater	Wells	Low Density Area	D1	27	10	9	100	240	0.002
		Medium Density Area	D2	27	12	9	100	240	0.002
		High Density Area	D3	27	12	9	100	240	0.002
	Boreholes	Low Density Area	D4	26	12	8	90	200	0.002
		Medium Density Area	D5	26	10	8	90	200	0.002
		High Density Area	D6	26	10	8	90	200	0.002

Table 4.5 Physical and Chemical Characteristics of Water Sources in the Study Area

Laboratory Examinations			Samples codes	pH	Magnesium	Nitrogen Nitrate	Nitrogen Nitrite	Total Hardness	Total Alkalinity	Iron
Laboratory Examinations Units					mg/l	mg/l	mg/l	mg/l	mg/l	mg/l
World Health Organization	Highest Desirable Level		A1	7.0	0.05	50	1.0	100	-	0.1
	Maximum Permissible Level		A2	7.5	0.5	100	2.0	500	-	1.0
Samples of Ala River Tributary	During the Rains	Up Stream	B1	7.5	0.13	70	1.50	650	650	0.5
		Mid Stream	B2	7.5	0.13	70	1.50	650	650	0.5
		Down Stream	B3	7.5	0.13	70	1.50	650	650	0.5
	After the Rains	Up Stream	B4	8.0	0.10	65	1.20	550	600	0.3
		Mid Stream	B5	8.0	0.10	65	1.20	550	600	0.3
		Down Stream	B6	8.0	0.10	65	1.20	550	600	0.3
Samples of Stormwater	Low Density Area		C1	8.5	0.15	75	1.70	680	680	0.7
	Medium Density Area		C2	8.5	0.15	75	1.70	680	680	0.7
	High Density Area		C3	8.5	0.15	75	1.70	680	680	0.7
Samples of Groundwater	Wells	Low Density Area	D1	7.3	0.00	65	1.20	170	100	0.1
		Medium Density Area	D2	7.3	0.00	65	1.20	170	100	0.1
		High Density Area	D3	7.3	0.00	65	1.20	170	100	0.1
	Boreholes	Low Density Area	D4	7.5	0.00	60	1.20	100	100	0.0
		Medium Density Area	D5	7.5	0.00	60	1.20	100	100	0.0
		High Density Area	D6	7.5	0.00	60	1.20	100	100	0.0

Table 4.6 Physical and Chemical Characteristics of Water Sources in the Study Area

Laboratory Examinations			Samples Codes	Lead	Zinc	Sulphate	Fluoride	Chloride	Sodium
Laboratory Examinations Units				mg/l	mg/l	mg/l	mg/l	mg/l	mg/l
World Health Organization	Highest Desirable Level		A1	0.05	5	200	1.0	200	170
	Maximum Permissible Level		A2	0.1	65	400	2.0	600	400
Samples of Ala River Tributary	During Rains	Up Stream	B1	0.80	45	250	1.0	500	230
		Mid Stream	B2	0.80	45	250	1.0	500	230
		Down Stream	B3	0.80	45	250	1.0	500	230
	After the Rains	Up Stream	B4	0.50	20	180	1.0	340	190
		Mid Stream	B5	0.50	20	180	1.0	340	190
		Down Stream	B6	0.50	20	180	1.0	340	190
Samples of Stormwater	Low Density Area		C3	0.80	50	300	1.5	460	260
	Medium Density Area		C2	0.80	50	300	1.5	460	260
	High Density Area		C1	0.80	50	300	1.5	460	260
Samples of Groundwater	Wells	Low Density Area	D1	0.00	15	170	0.8	300	150
		Medium Density Area	D2	0.00	15	170	0.8	300	150
		High Density Area	D3	0.00	15	170	0.8	300	150
	Boreholes	Low Density Area	D4	0.00	10	150	0.6	290	100
		Medium Density Area	D5	0.00	10	150	0.6	290	100
		High Density Area	D6	0.00	10	150	0.6	290	100

CHAPTER FIVE

RESULTS AND DESIGN

5.1 Population of the Estate

The population of the estate was calculated to be 17,500 people in 20years time with a growth rate of 3.0% assumed for the community. The growth rate for Akure as a state capital is between 2.6 and 3.0%, according to the 1992 census data. The water consumption rate was taken as 100litres/capita/day.

5.2 Determination of the Treatment Level of Raw Water

In order to determine the treatment level required for the impounded water, the results of the laboratory tests of the collected samples of all the contributing waters from the study area were compared with the World Health Organization (W.H.O.) water quality standard values. The comparison is shown in Figs 5.1A to 5.1S.

The temperature of the water samples are almost the same, and therefore do not appear to be a significant parameter in water treatment. The climate of the region is such that temperature does not vary much throughout the year.

With a desirable level of 5^0Hazen, water of colour level of 60, 40, 15, and even 8^0Hazen, becomes objectionable, even though the maximum permissible level is 50^0Hazen. The high level of colour in the tributary and stormwater are probably due to the leachate of decaying vegetation, and the eroded soils being washed into them. The colour is associated with high turbidity, and can be considerably reduced by the simple filtration process.

The element, cadmium is representative of the heavy metals – inorganic pollutants. The traces probably come from the wastes disposal, hence the relatively higher concentration found in stream and the storm and stream waters. Cadmium in drinking water may be lethal, but in the estate's water supply, the concentration is within the permissible range; and can be reduced further when the sediments are removed.

Concentration of total solids indicates the total amount of organic and inorganic contaminants dissolved in the raw water, which remains as residue after evaporation. Even though the value measured in the stream and storm waters are less than the maximum permissible level, the dissolved solids need to be removed as sediments by coagulating, settling and filtering. It should be noted that the total solids concentration in ground water, that is, well and boreholes, are much lower thus being an indication that the solids is associated with the surfacewater's erosional activities.

The level of conductivity is too low to indicate any saline condition. In any case, the uniform level in all the samples including the groundwater indicates that it is the minimum that can be attained, no matter the treatment provided.

The pH value for water represents the overall balance of a series of equilibrant existing in the solution. . All pH values measured are above 7.0, indicating that the alkaline nature of the raw water is high, and in correlation with the total alkalinity and the total hardness concentrations. The figures also show that the raw water must be "softened" through treatment.

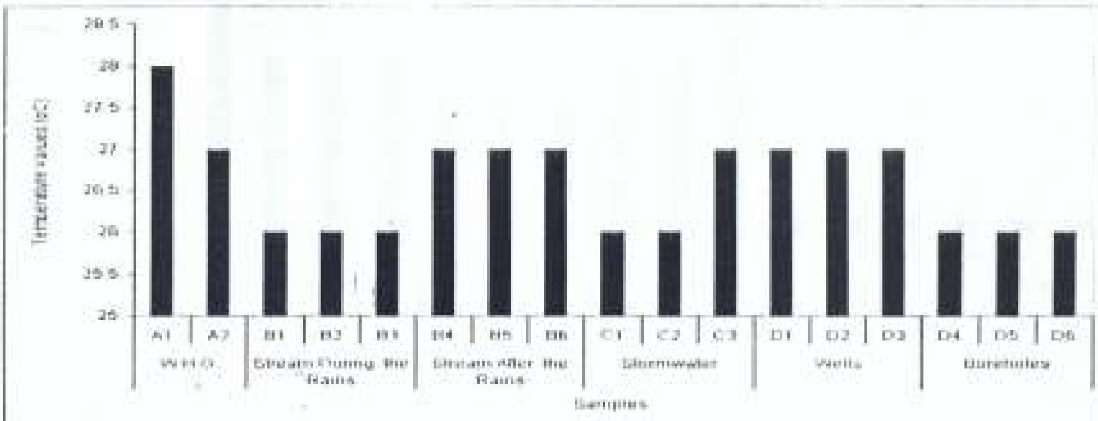


Figure 5.1A: Temperature values compared with W.H.O. values.

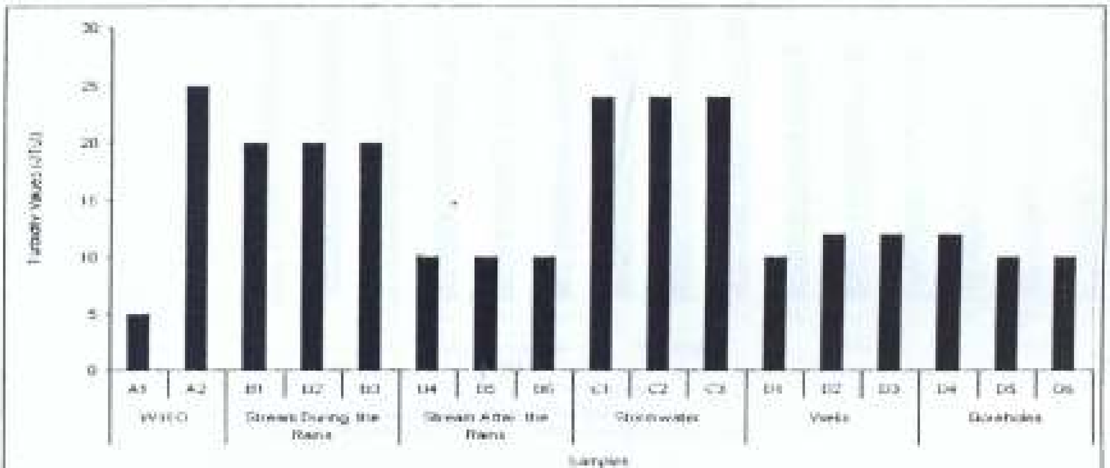


Figure 5.1B: Turbidity values compared with W.H.O. values.

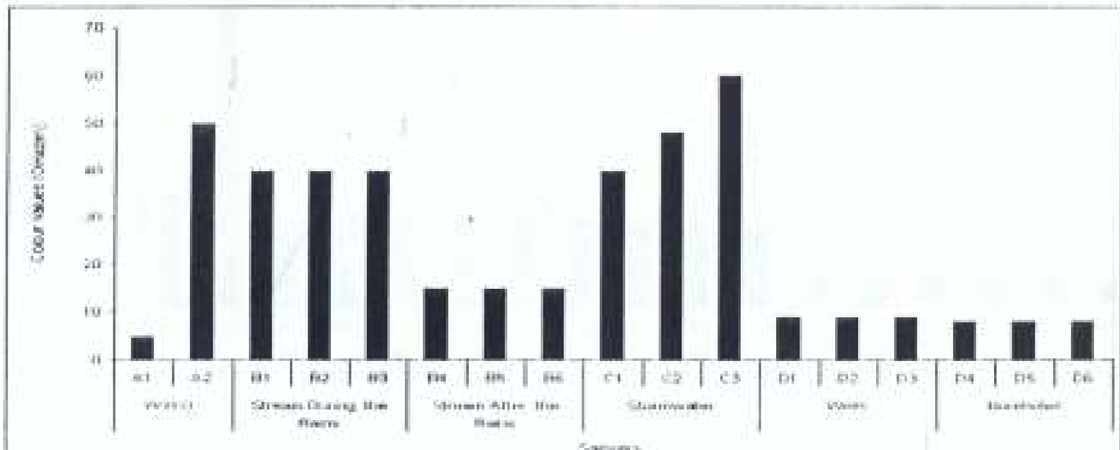


Figure 5.1C: Colour values compared with W.H.O. values.

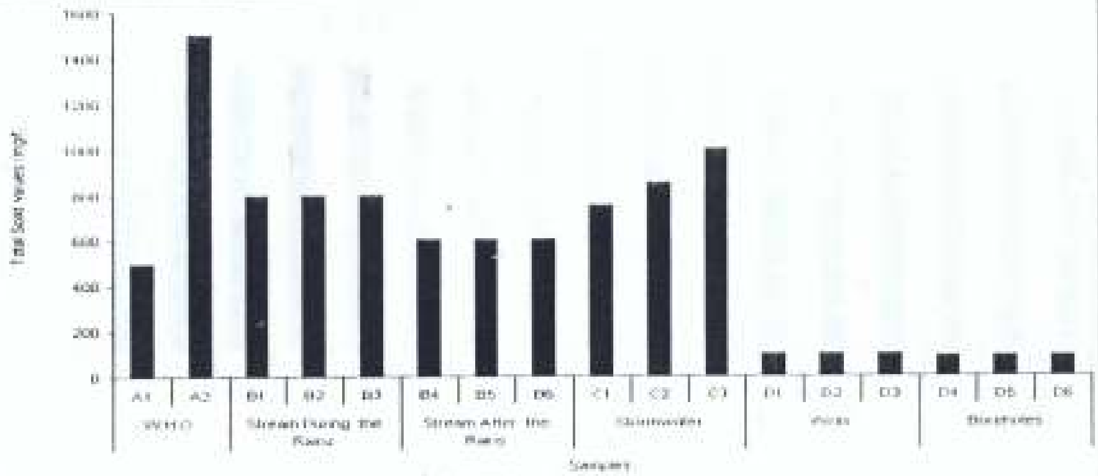


Figure 5.1D: Total Solid values compared with W.H.O. values.

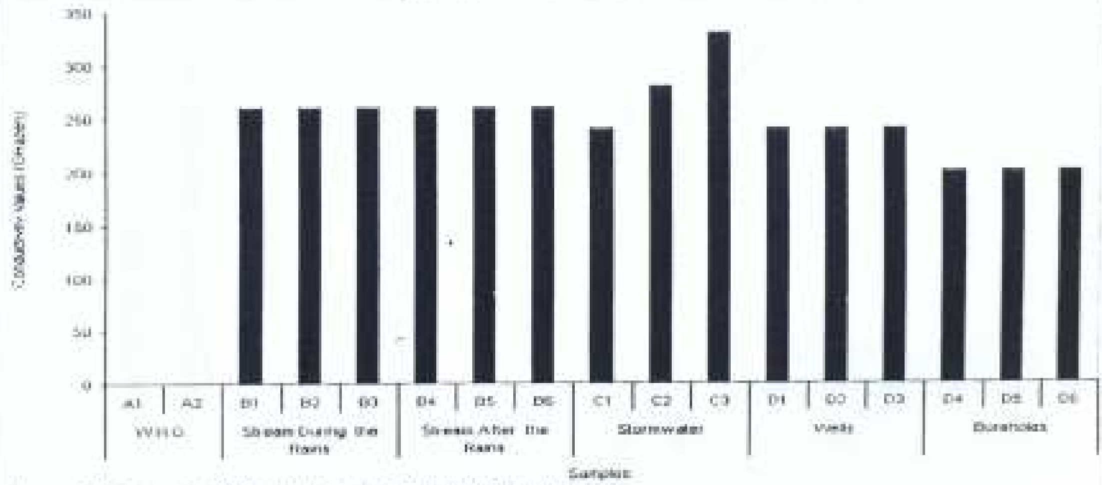


Figure 5.1E: Conductivity values compared with W.H.O. values.

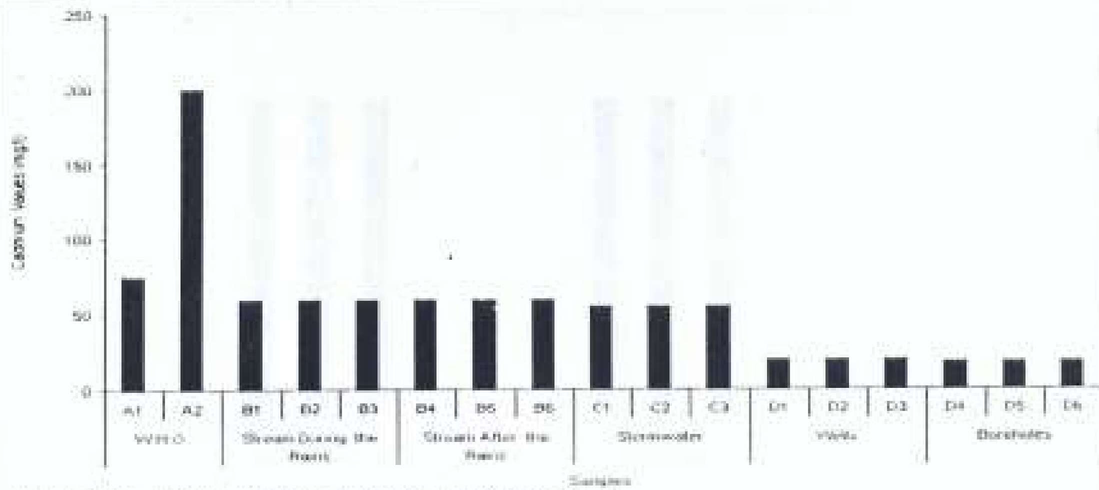


Figure 5.1F: Cadmium values compared with W.H.O. values.

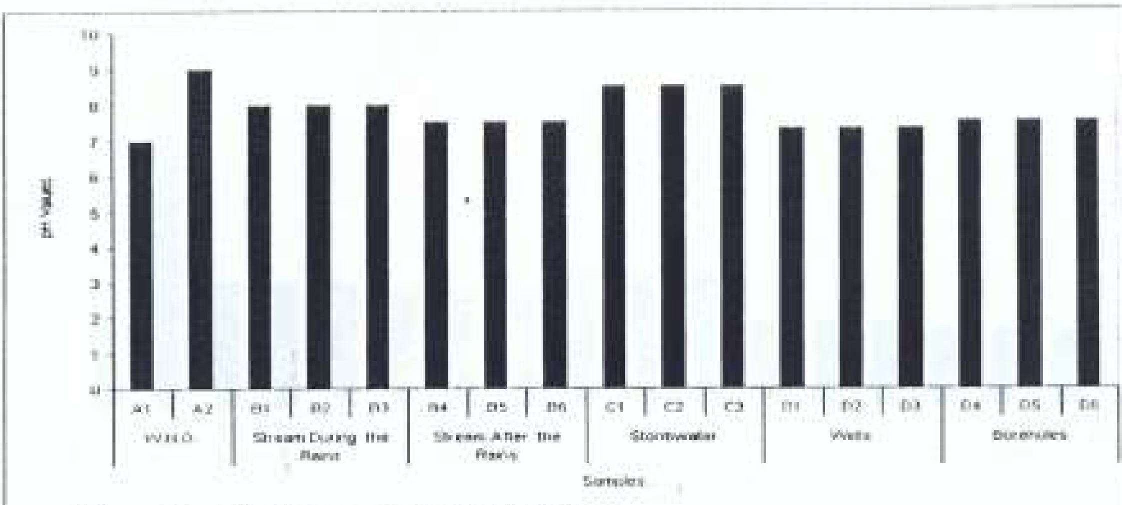


Figure 5.16: pH values compared with W.H.O. values.

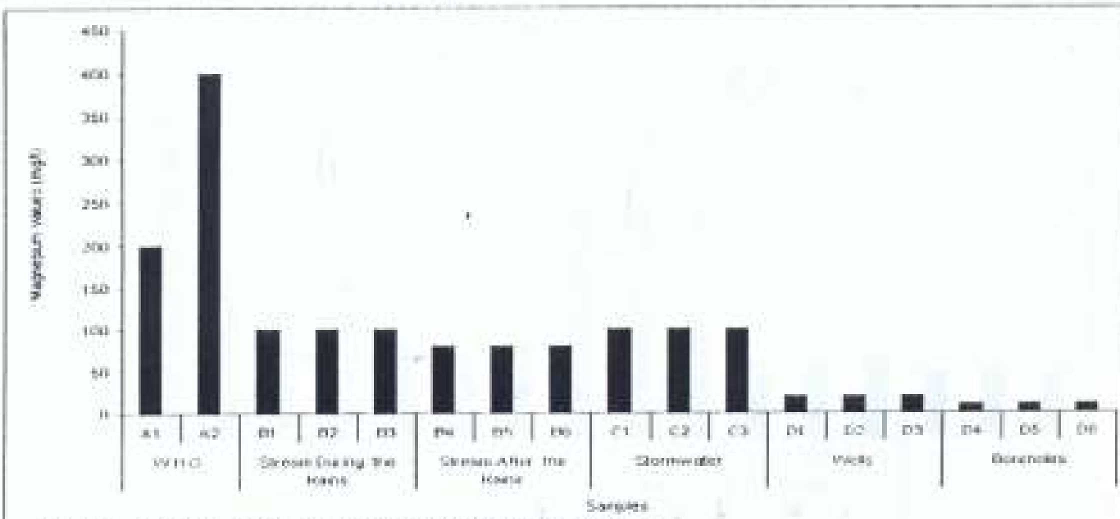


Figure 5.18: Magnesium values compared with W.H.O. values.

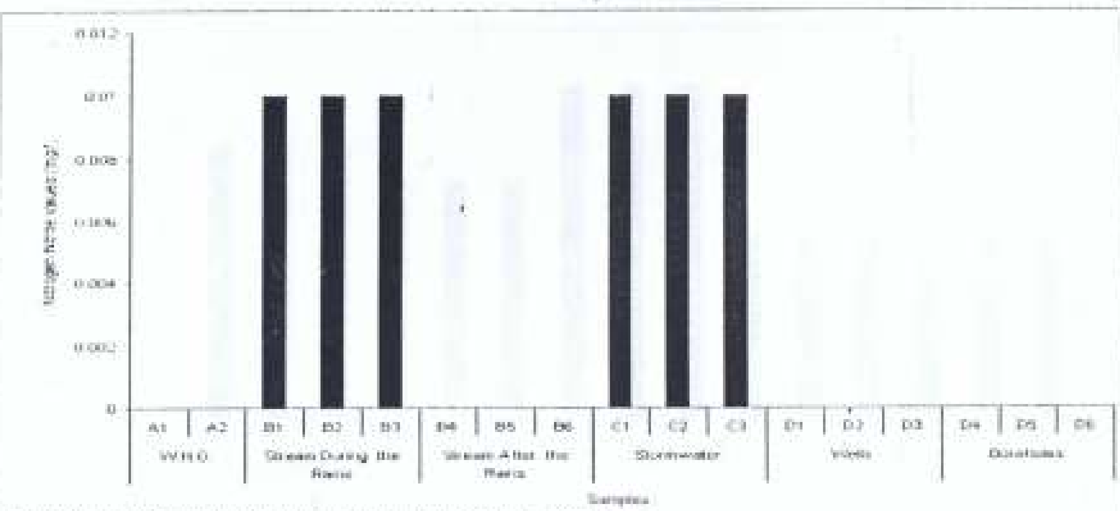


Figure 5.18: Nitrogen Nitrite values compared with W.H.O. values.

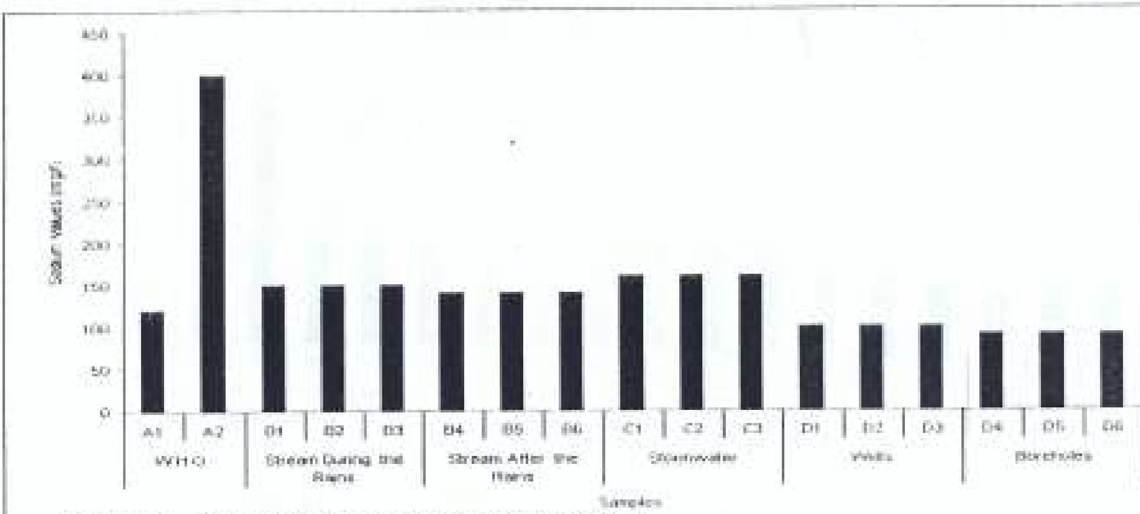


Figure 5.13: Sodium values compared with W.H.O. values.

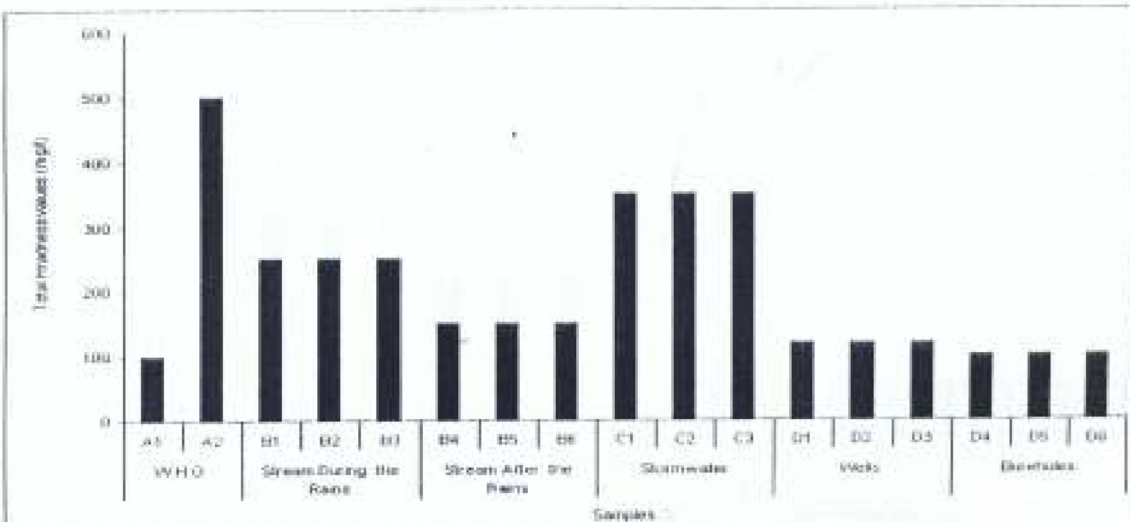


Figure 5.16: Total Hradness values compared with W.H.O. values.

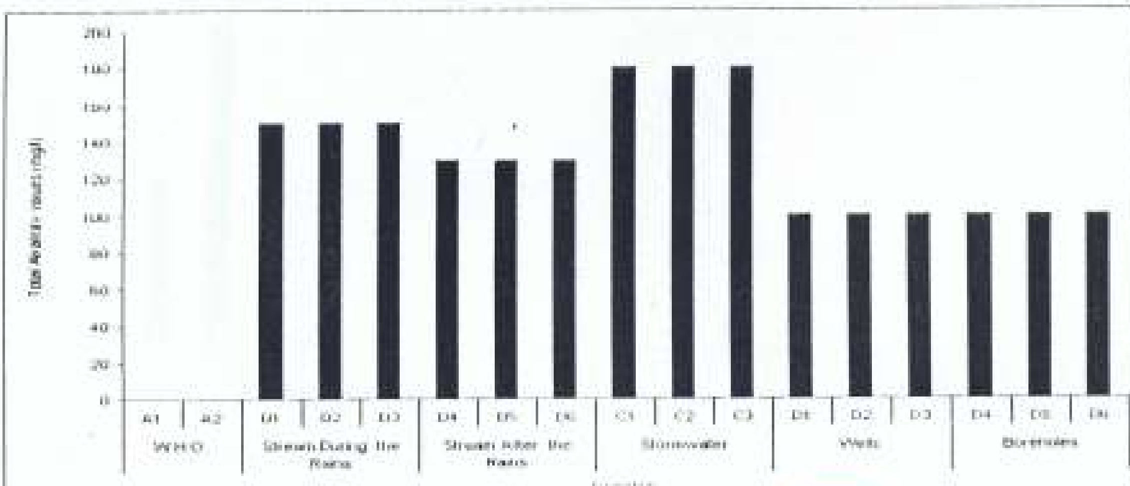


Figure 5.11: Total Alkalinity values compared with W.H.O. values.

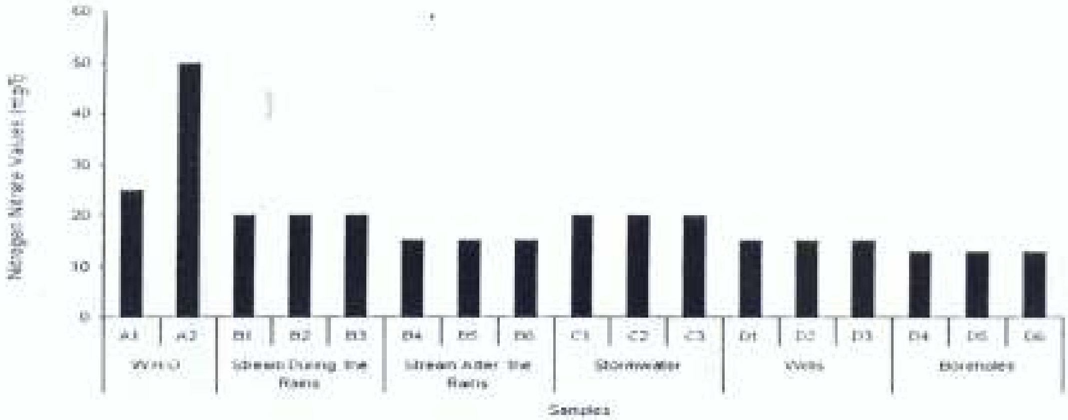


Figure 5.1M: Nitrogen Nitrate values compared with W.H.O. values

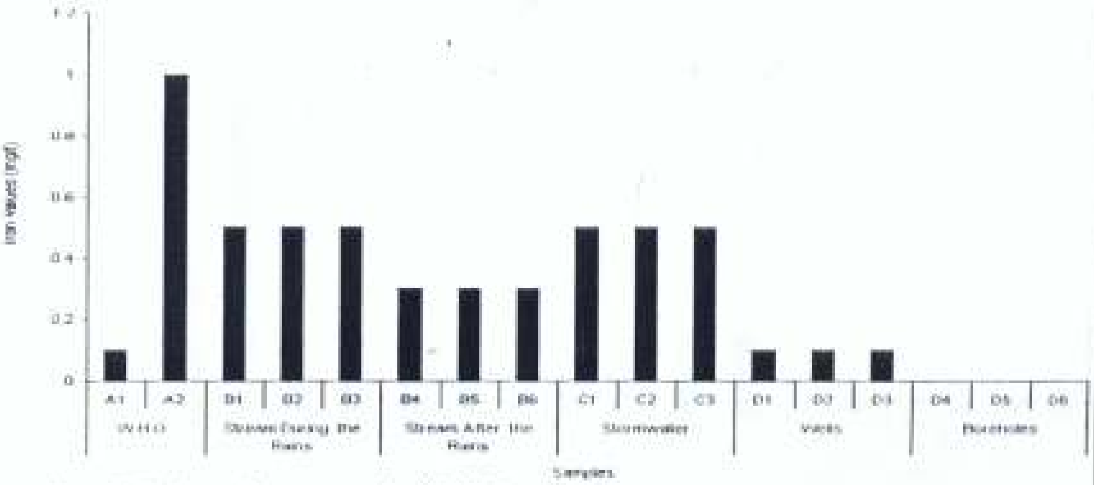


Figure 5.1N: Iron values compared with W.H.O. values.

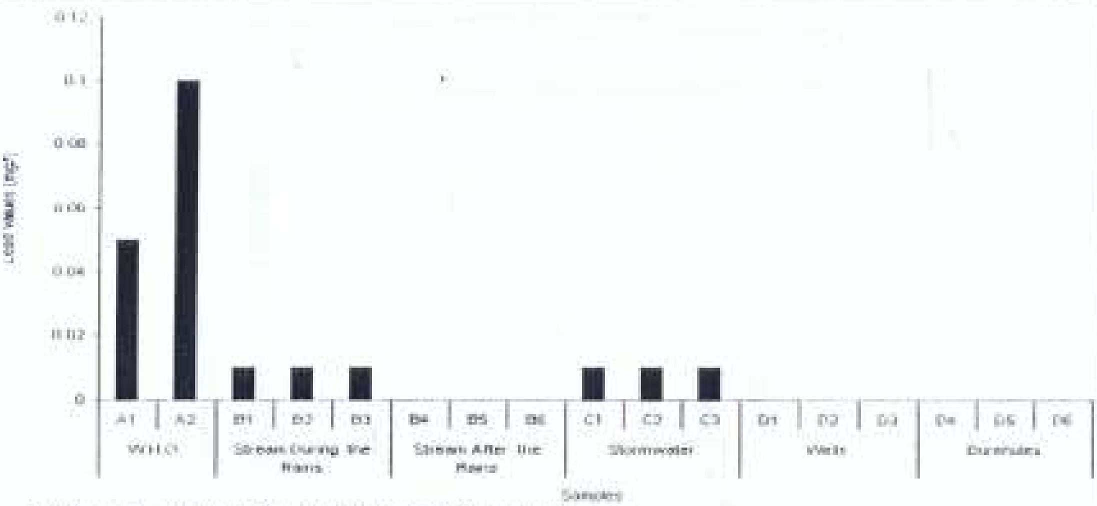


Figure 5.1O: Lead values compared with W.H.O. values.

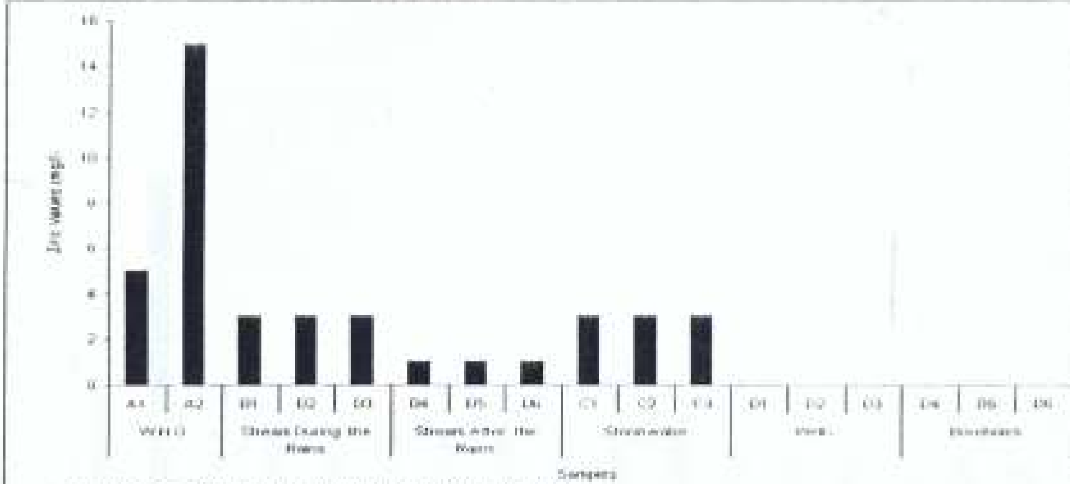


Figure 5.18: Zinc values compared with W.H.O. values.

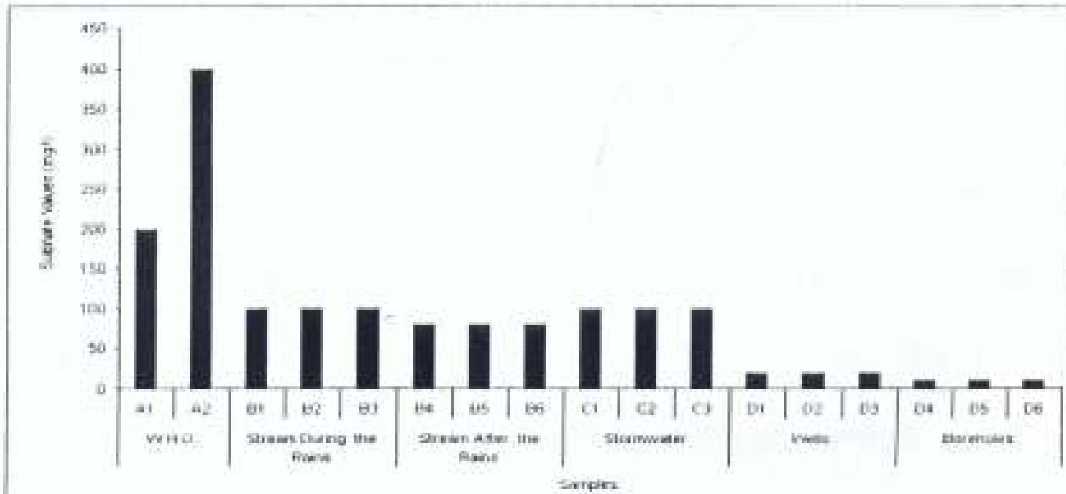


Figure 5.19: Sulphate values compared with W.H.O. values.

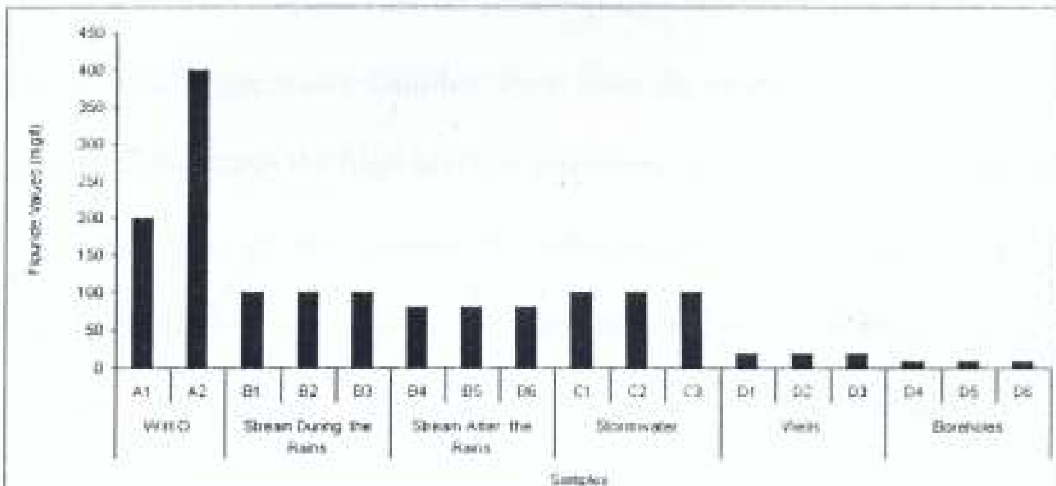


Figure 5.18: Flouride values compared with W.H.O. values.

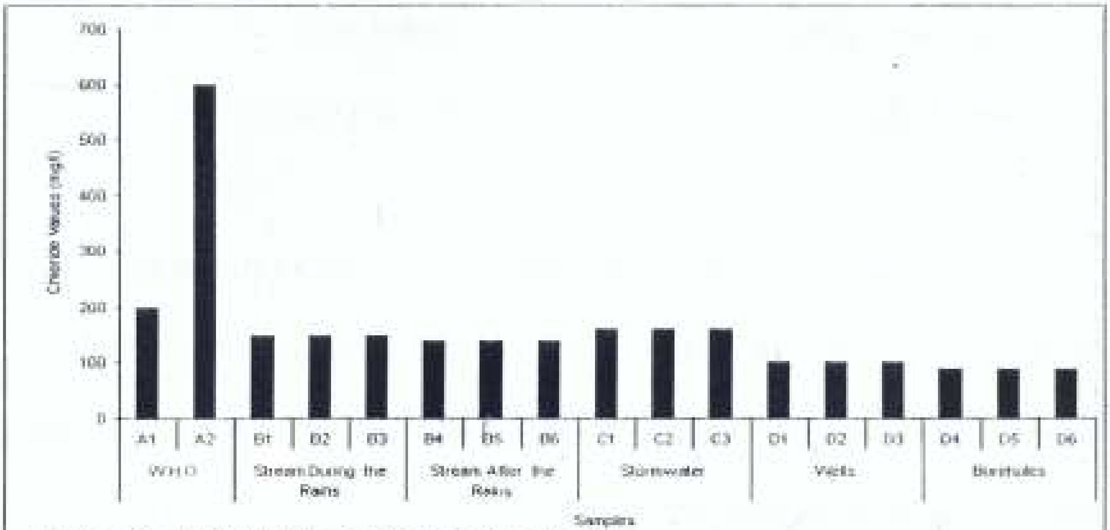


Figure 5.15: Chloride values compared with W.H.O. values.

Iron is present in the water in several forms. Their actual form in the water samples is not defined, but the figures obtained show that the concentration fall within permissible range. Nevertheless, the proposed treatment system will absorb sufficient amount of oxygen through exposure to the atmosphere when it is impounded before treatment.

Any traces of sulphur can be removed by chlorination. The turbidity is expectedly high in the water samples taken from the stream during the rain and stormwater. It indicates the high level of sediment which must be removed from the raw water through the process of sedimentation and filtration. For high turbidity, addition of alum facilitates the fast removal of the floating substances by allowing them to coagulate. The sediments would later be removed by filtration.

In general, the results from analysis of the water samples show that the water can be obtained from the combination of streamwater and stormwater, and this need only to be subjected to simple treatment for potability. Most of the quality parameters fall below the maximum permissible level of the WHO standard for drinking water, while a few meet the highest desired level.

5.3 Stormwater Collection and Raw Water Impoundment

The total yearly and monthly rainfall values for Akure were used for the estate and these are shown in Tables 5.1 and 5.2 and Figs 5.2 and 5.3 respectively. From these it can be seen that the months of January, February, November and December are relatively dry, therefore the impounded water from the previous months will be more relied upon in these months for supply of raw water to the treatment plant. From Table 5.2 and Fig. 5.3, it can be seen that the trend of the yearly rainfall values in Akure and by implication, the estate ranges between 1160 and 1853 mm. This shows that sufficient water can be gotten from rain if it is well collected by impoundment for future use. The impoundment capacity was calculated to be 360,000m^3, so as to have the ability to take water that will be needed for the required period for the proposed treatment plant.

The stream, which have a sectional area of 0.72m^2 and flowing at a velocity of 1.2m/s will supply about 0.864m^3/sec or 75,000m^3 per day. The meaning of this is that the stream alone can supply about 14million m^3 for 186 days and that water need of the treatment plant will be guaranteed. Stormwater, that is, runoff movement, from the estate was calculated (Appendix A) and controlled through the open drainages system that has been well arranged (Fig. 5.4) for discharge into the stream, and then into the impoundment. The Rational Method and Manning equation were used in the design of runoff and drains respectively. Raw water shall be taken from the impoundment and led by gravity into the low-lift raw water booster pump, which raises the water to the treatment plant. The treated water then gravitates to a clear water tank and is then transferred by high lift-pump to an overhead storage reservoir, from where the treated water is distributed by gravity to the distribution systems. (Figure 5.5)

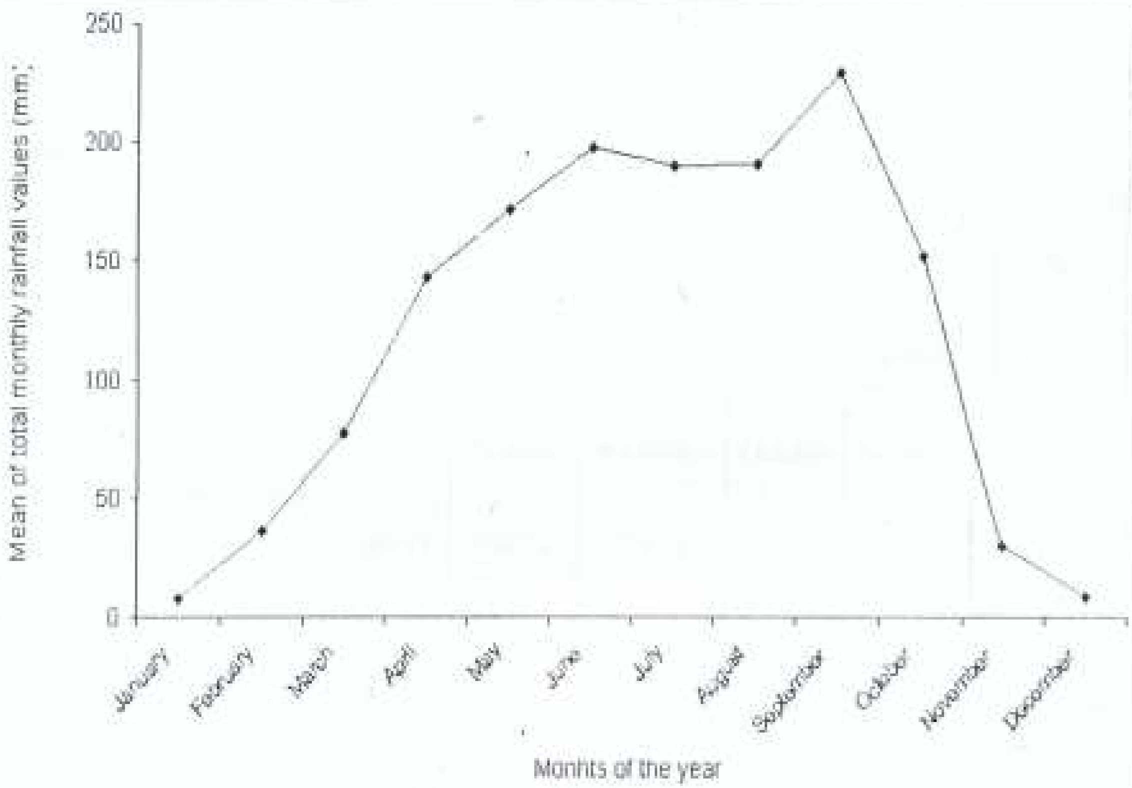


Figure 5.2: Mean of total monthly rainfall of Akure

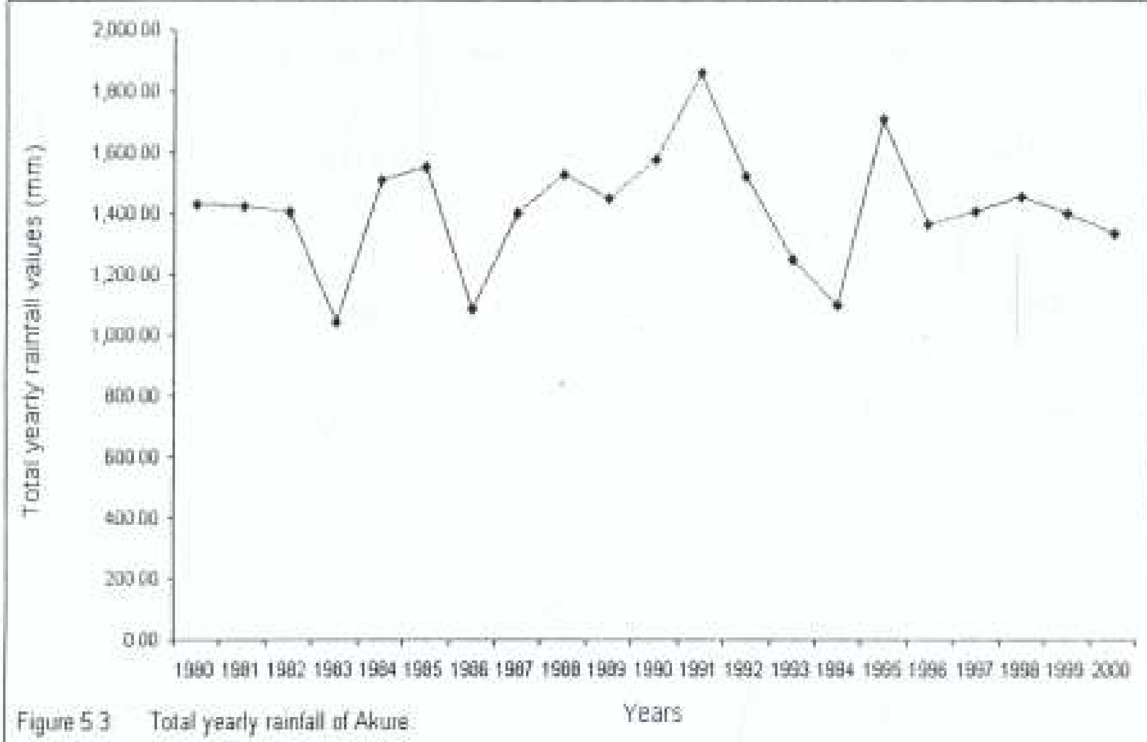


Figure 5.3 Total yearly rainfall of Akure

Table 5.1 Mean of total monthly rainfall value of Akure.

Month	January	February	March	April	May	June
Mean of total monthly rainfall (mm)	7.83	36.58	77.26	143.23	171.94	197.68
Month	July	August	September	October	November	December
Mean of total monthly rainfall (mm)	189.77	190.56	229.73	152.23	30.35	8.98

Table 5.2 Total yearly rainfall value of Akure.

Year	1980	1981	1982	1983	1984	1985	1986
Total yearly rainfall (mm)	1,429.80	1,424.20	1,405.30	1,044.50	1,507.40	1,554.20	1,085.80
Year	1987	1988	1989	1990	1991	1992	1993
Total yearly rainfall (mm)	1,401.4	1,525.20	1,449.90	1,577.90	1,862.40	1,518.90	1,249.20
Year	1994	1995	1996	1997	1998	1999	2000
Total yearly rainfall (mm)	1,095.50	1,707.40	1,366.20	1,407.60	1452.60	1398.20	1332.80

5.4 Siting of Proposed Treatment Plant

The proposed treatment and impoundment shall be sited within the estate (sheet 2) near the stream in an area already taken over by the flooding caused by the stream runoff flows. The area was chosen because the state government, through an agency charged with taking care of the estate, i.e the Ministry of Lands has marked the area as unsuitable for building. It is the authors believe that, once the water is taken care of the place will be useful for the proposed plant.

5.5 The Proposed Treatment Plant Components

The proposed treatment plant is designed for maximum flow rate of 1,750cubic meter per day based on the daily demand of the state's estimated population and a 16 hour daily working operation. The operation units shall consist of impoundment, water intake, aeration system, coagulation and flocculation tanks, sedimentation and clarification tanks, filtration tanks, disinfection, treated water storage tanks, and overhead storage tanks. The water treatment block and the hydraulic profile diagrams are shown in Figs. 5.4 and 5.6 respectively.

5.5.1 Intake Works

The water intake shall consist of a structure of dry chamber to be constructed along floodwall of the impoundment. The impoundment embankment on which the intake structure shall be constructed shall be against flooding, which will involve compacted stone filling, reinforced concrete retaining structure and hand rails. The structure shall contain low lift pump sets extending 10m into the impoundment with bell mouth end. The pump sets shall discharge raw water through a cast iron pipe into the treatment plant. Values used for the head losses coefficient in the system are taken from Table. 5.3.

Table 5.3 Some minor losses of head coefficient to illustrate head loss in water transport system*.

Coefficient		Coefficient	
Name	K	Name	K
Check valve	8	Exit loss	1.0
Glove valve	10	Entrance loss	
Gate valve	0.2	Projecting pipe	1.0
90° elbow	0.3	Rounded	0.25
45° elbow	0.2	0.5 contraction	0.33
22.5° elbow	0.15	2X expansion	0.56

- All valves wide open. Partial closure will be higher. Exact values are provided by manufacturers.

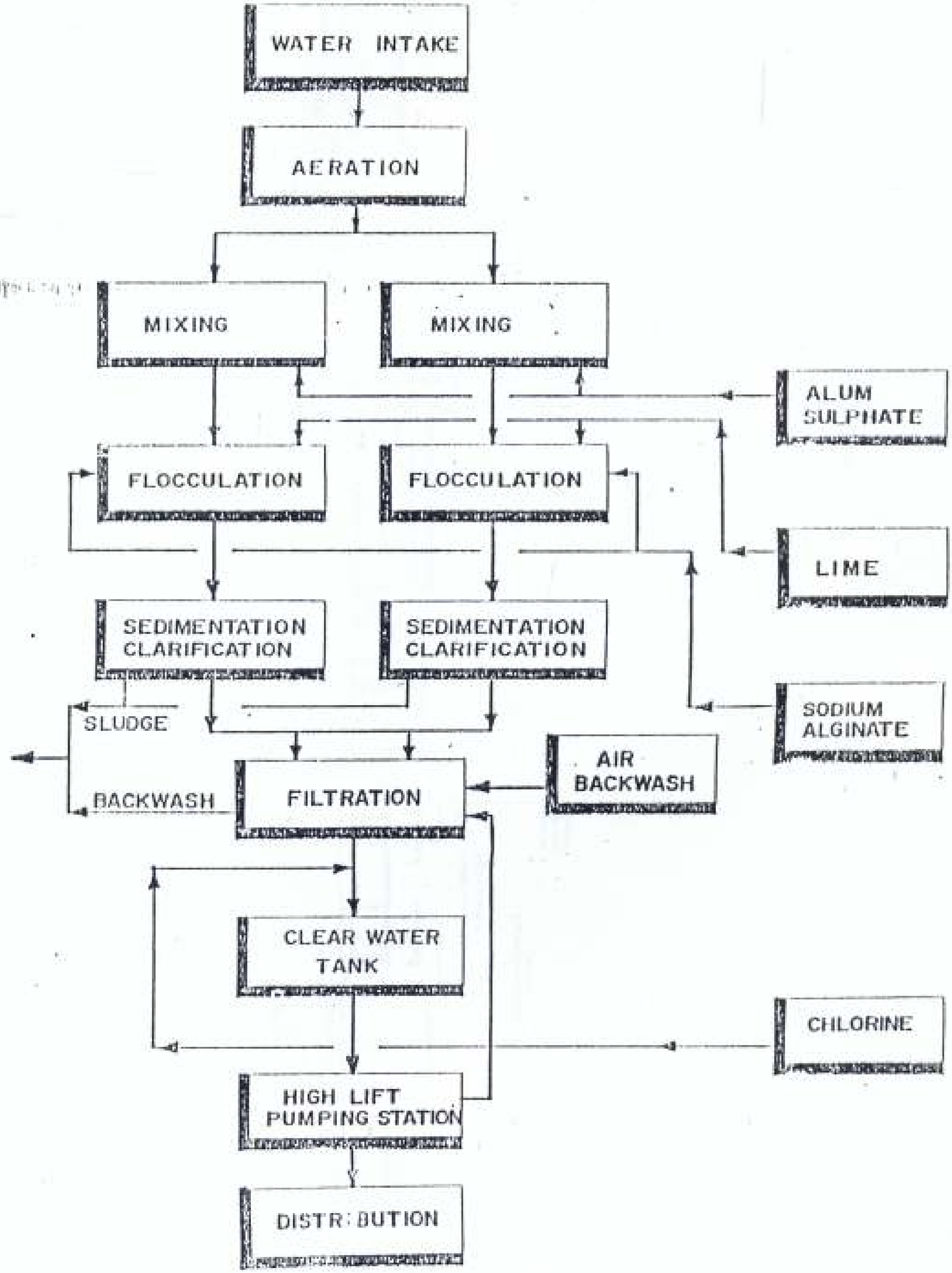
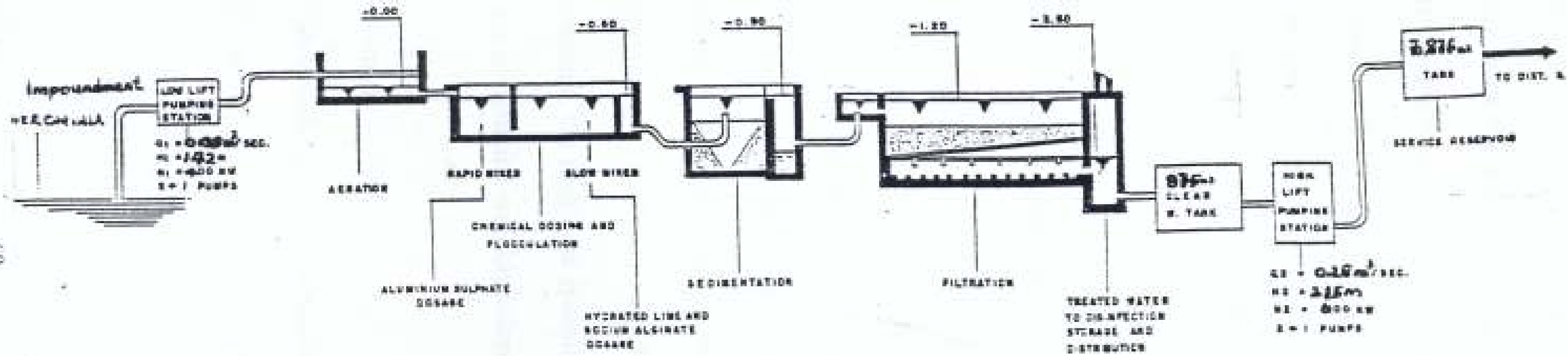


Figure 5.5 Schematic diagram of the proposed treatment plant's operation and process



Figures 5.6 The hydraulic profile diagrams of the treatment plant.

5.5.2 Aeration

Raw water from the intake structure shall be delivered through the ductile iron pipes to the treatment plant. Magnetic flow meter to monitor the amount of raw water .to be treated shall be installed before the aeration. The aeration could be completely bypassed and water could flow directly to the mixers. Contact time shall not be less than 15mins while the cascade aeration should be considered.

5.5.3 Coagulations and Flocculation

In order to guarantee the greatest flexibility in operations, the plant shall split into two separate but equivalent lines. This section shall be two successive stages, rapid mixing and slow mixing of good flocs formation. The aluminum sulphate shall be dosed to rapid mixing while hydrated lime and sodium alginate are dosed at the beginning of slow mixing. The tanks will be equipped with mechanical stirrers. To avoid the associated maintenance problems, the hydraulic system for rapid and slow mixing water with chemical for coagulation and formation of flocs should be applied

5.5.4 Clarification

The sedimentation basin construction (Fig. 5.7) which divides the "flowing deep" stream of water into many shallow streams flowing separating in parallel conduits has been proposed. The formed sludges are self-clearing because of the inclination of sedimentation conduits. The flocculated water is introduced from the lower side and it is collected as clear water on the upper side by a channel system while the sludge will flow inside the ducts in the opposite direction. The clarified water will be collected in channels carrying it to the filters.

5.5.5 Filtration

Filtration shall be done in rapid gravity, single media, contact rates, variable level filters. All filtering operations will be controlled and carried out from the operating gallery, see Fig. 5.9 for filters section. All control valves shall be operated by pneumatic powered actuators and shall be equipped with manual control for emergencies. Filter backwashing shall be by air blown by blowers and by water pumping by centrifugal horizontal pumps. A backwashing flow meter of the magnetic type shall be installed on the outlet side of the backwashing pumps to control the amount of backwashing water. After filtering and having been dosed with chlorine the water shall be conveyed to filters and for preparing the reagents as well as for distribution to the treatment plant network and by high lift pumps to the reservoir and distribution networks.

5.6 Chemical Dosage

Aluminum Sulphate, hydrates lime and sodium alginate are the main chemicals to be used in this proposed treatment plant. See Fig. 5.10 for the dosing flow diagram.

5.6.1 Aluminum Sulphate (Alum) - The alum solution shall be prepared in two separate stages, that is, dissolving the reagent in easily accessible tanks to produce 10-20 percent solution strength by means of recycling pumps and the same recycling pumps, suitably connected. The alum shall be dosed in proportion to the flow of water to be treated. The tanks shall be constructed and equipped with bottom outlet, overflow and level indicator alarm.

5.6.2 Hydrated Lime: the hydrated lime slurry shall be prepared in steel tanks equipped with stirrers and a bottom outlet, overflow and level indicator alarm and a dust exhaust fan. The dosage system shall consist of metering pumps operating on a signal from a pH meter.

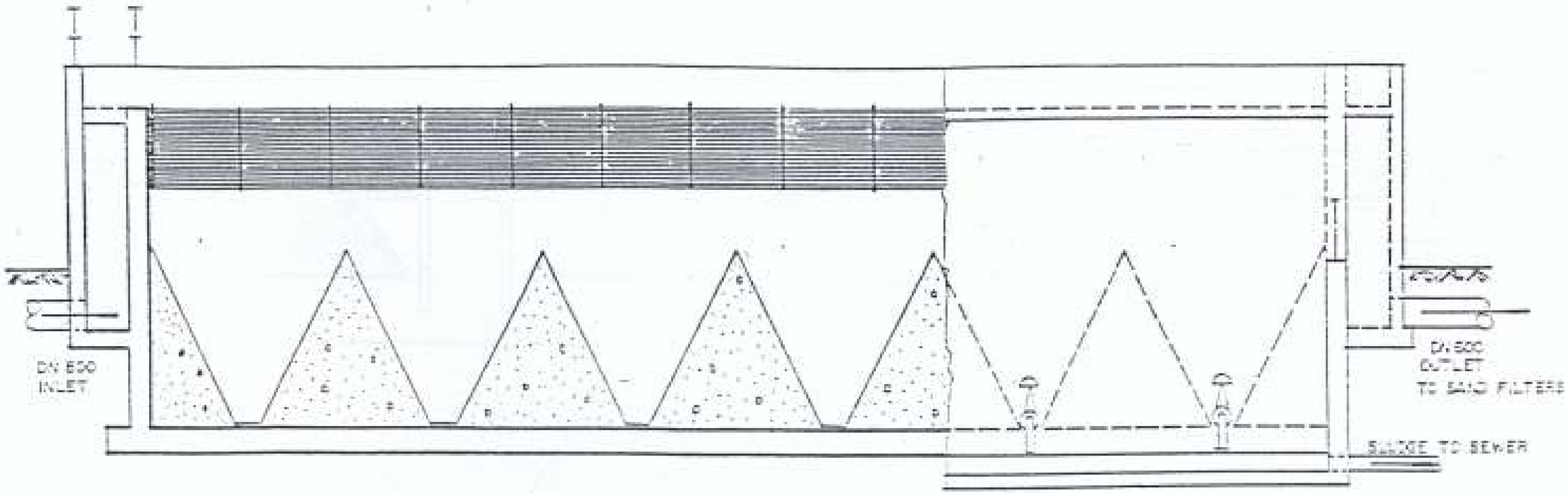


Figure 5.7 The cross section of sedimentation basin

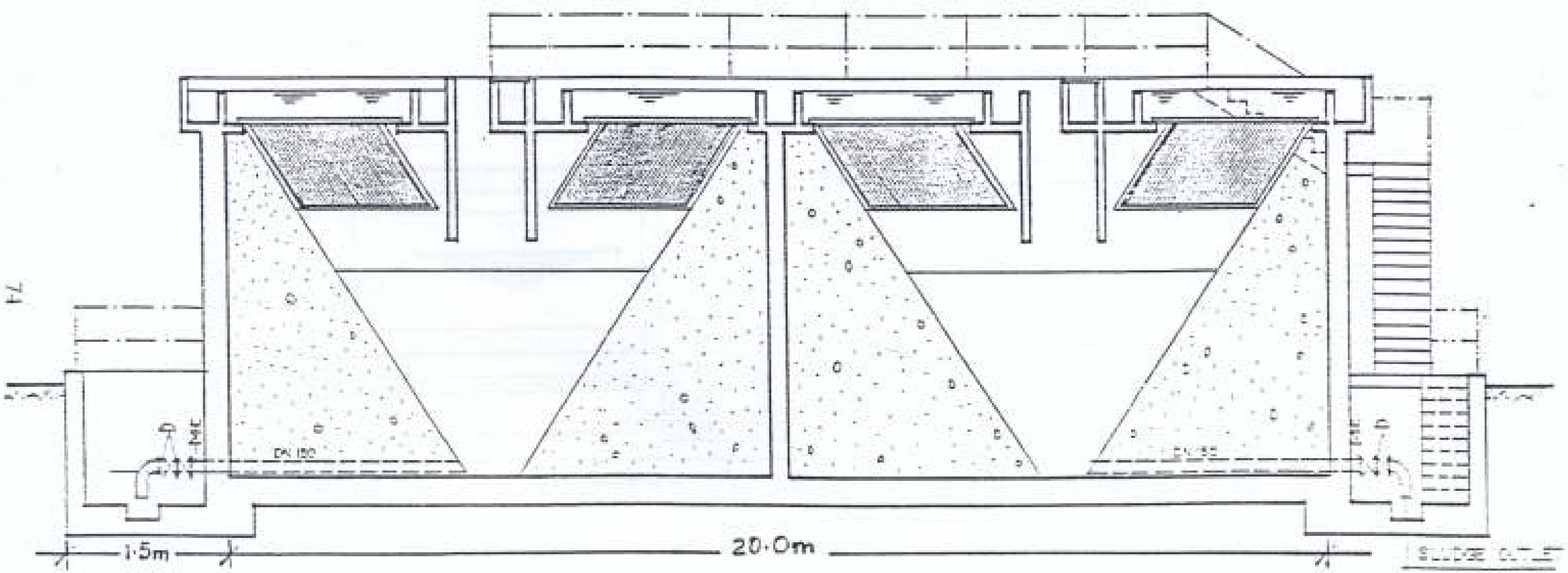


Figure 5.8 The cross section of sedimentation basin.

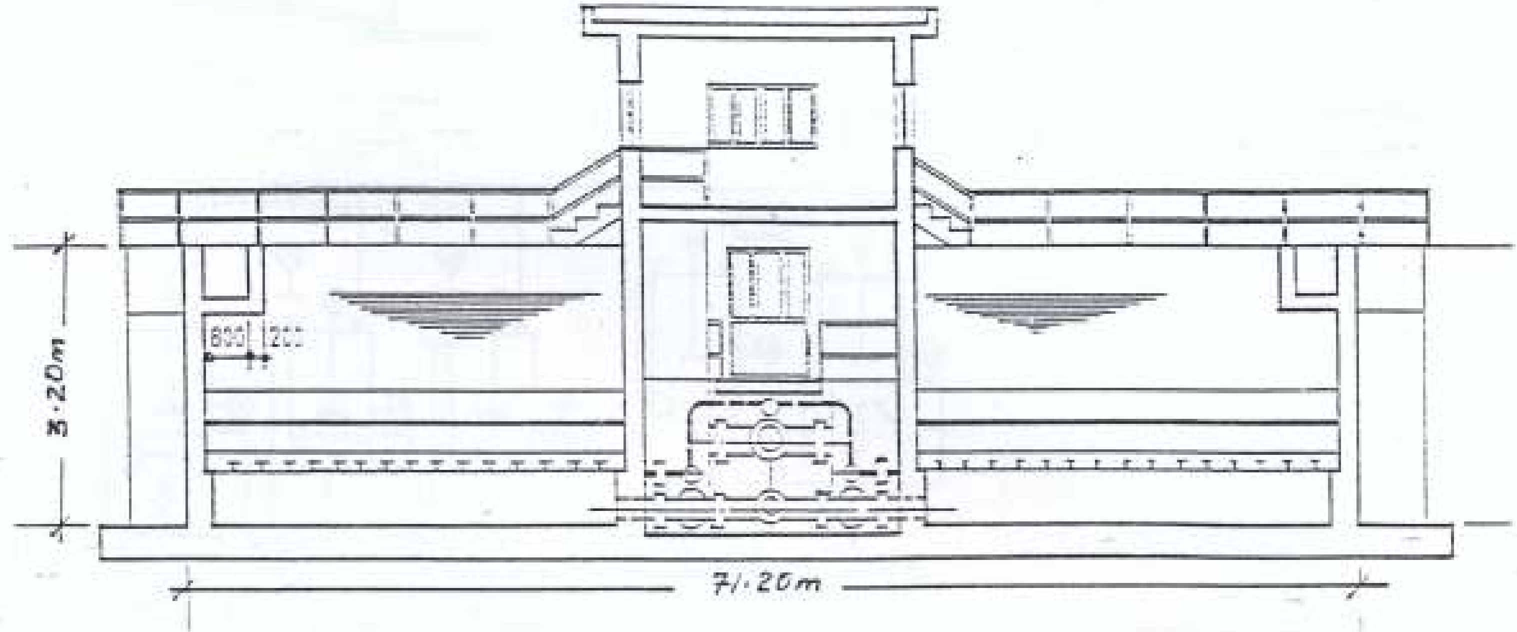


Figure 5.9 The cross section of sand filter

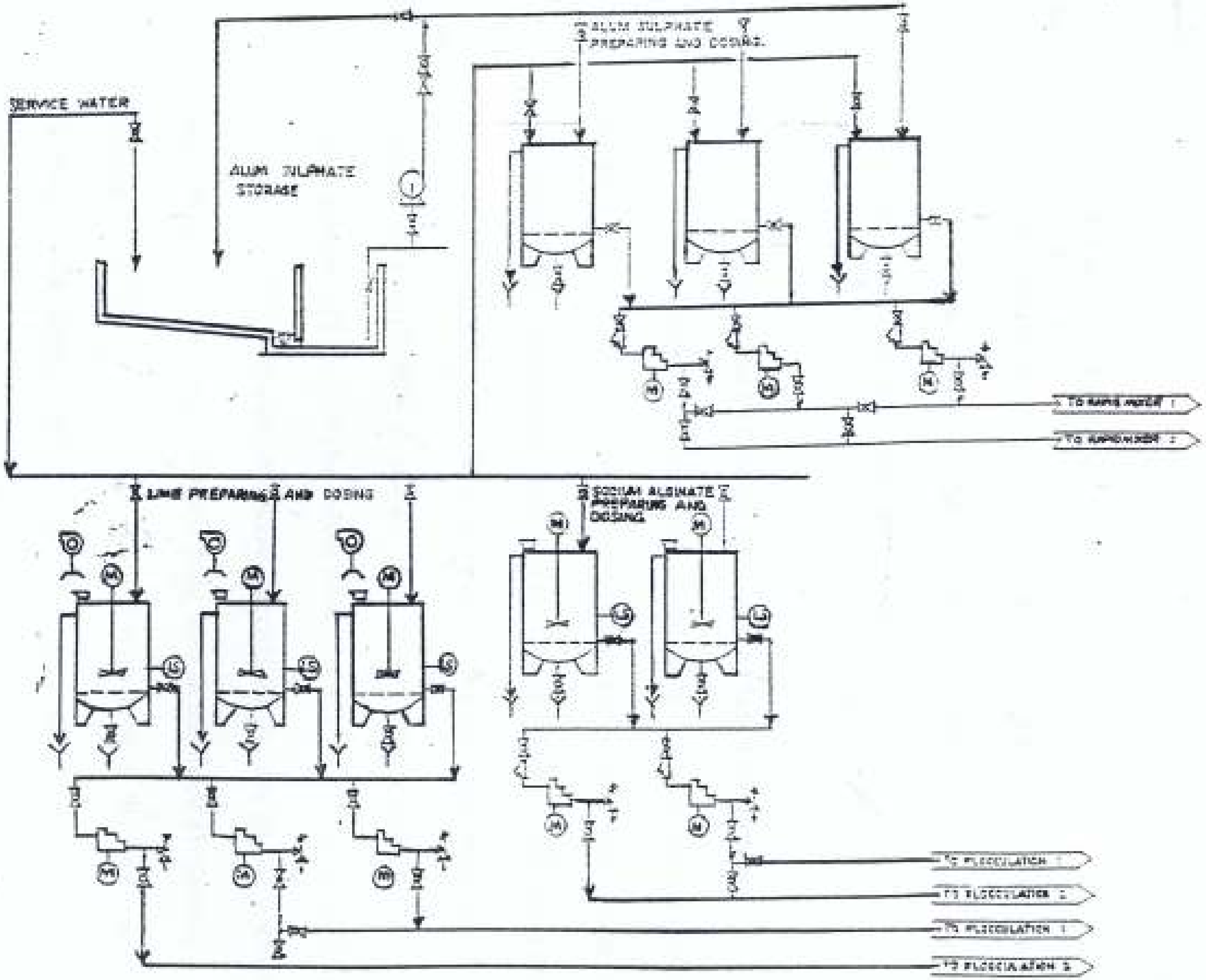


Figure 5.10 Chemical dosing flow diagram

5.6.3 Sodium Alginate: Sodium alginate dosing is foreseen to facilitate flocculation and hence separation of any suspended matter. Dosage shall be done in proportion to the flow of water to be treated by means of metering pumps which shall function continually.

5.6.4 Chlorination: The chlorination plant shall consist of pre-chlorination and post-chlorination in the pipe existing from the filtration section. The per-chlorination will be used when necessary to control algae growth, etc.

5.7 Design of the Proposed Treatment Plant Components

The design of the main component parts of the proposed treatment plant (Table 5.4) were carried out using the established methods to determine the components' sizes and quantities required. The components interconnecting the pipes shall be of cast iron pipes. The summary results are shown in Appendix A and B. Other aspects of the treatment plant are descriptively applied for the convenience of this work. Pipes of 300mm, 200mm, and 100mm diameter were selected for the design of the conduits for which the discharge of each pipe was determined (Equation 2.13). Using the results in Appendix C, low density area of the estate are to be serviced with 300mm diameter pipes, medium density area with 200mm diameter pipes, and 100mm diameter pipes for the high density area. The choice of the different diameters is to maintain fairly uniform pressure, since the low density areas is nearer the reservoir, and the same large pipes must convey water to the higher density areas. This is in conformity with the distances from the overhead storage tank which is to be located in the low density area.

Table 5.4 Summary results of designed treatment plant components

Component name	Size	Quantity	Capacity
Raw water impoundment	4.5m x 200m x 400m	1	360,000m^3
Mixing tank	6.0m x 5.0m x 3.5m	1	110m^3
Sedimentation tank	20.0m x 10.0m x 3.5m	2	700m^3
Filtration tanks	70.0m x 19.0m x 3.5m	2	2,660m^3
Treated water tanks	30.0m x 22.0m x 3.0m	3	7,875m^3
Low lift pumps	-	2	400KW
High lift pumps	-	2	600KW

5.8 Distribution Network

The open channel drainage system is adopted for transporting the stormwater or runoff to the stream and impoundment for pre-treatment storage while the treated water distributions will be done through the conduit piping. From the known discharge and slope for ache drain, the hydraulic section, that is, flow area, is calculated using the Manning's Equation.

The concentration times of catchment were calculated using Eq.2.9

For catchment A1:

Concentration time, Tc = $0.0663 \times 150^{0.77} / 0.0472^{0.385}$

$= 10.18$mins.

for the quantity of runoff on the catchment, equation 2.8 was used

that is $Qc = .000278 \times 16.500 \times 0.5 \times 154.91 \times 10^{-4}$

$= 0.355m^3/sec.$

this procedure was used for all other catchments and their results are shown in Appendix B.

To determine the sectional dimensions of the drains,

Let the drain width = b and the depth = 1.5b

Area of drain, $A = bm \times 1.5bm = 1.5b^2$

Where the drain is ¾ full

Wetted perimeter of drain, $P = (2 \times ¾ \text{ of } 1.5b) + b$

$= (2.25b) + b$

$= 3.25b$

$A/P = R = 1.5b^2 / 3.25b$

$= 0.46b$

$R^{2/3} = 0.46^{2/3} b^{2/3}$

$= 0.60b^{2/3}$

$AR^{2/3} = 1.5b^2 \times 0.60b^{2/3} = 0.9b^{8/3}$

$= 0.9b^{2.667}$

Substituting into equation 2.12

$Q = n^{-1}AR^{2/3} S^{1/2}$

$= n^{-1} \times 0.9 \times b^{2.667} \times S^{1/2}$

$b = 2.667\sqrt{} Qn / 0.9 \times S^{1/2}$

if $n = 0.011$ (table 2.7) then,

$$b = 2.667\sqrt{0.011Q / 0.9 \times S^{1/2}} \qquad \text{.......................5.1}$$

Eq. 5.1 was used to calculate the sectional dimensions of the drains and the results are shown in Appendix C. The Q value of the largest contributing catchment drains and the slopiness of the road were used for the calculations of the drains.

For the distribution mains, to determine the pipe dimension, Eq. 2.18 was used, as shown;

$$Q = 0.278CD^{2.63} S^{0.54}$$

$$AV = Q = 0.278CD^{2.63} S^{0.54}$$

$$V\pi D^2/4 = 0.278CD^{2.63} S^{0.54}$$

$$D^2 / D^{2.63} = 4 \times 0.278C\, S^{0.54} / V\pi$$

$$D^{2-2.63} = 1.112C\, S^{0.54} / V\pi$$

$$D^{0.63} = 1.112C\, S^{0.54} / V\pi$$

if C = 100 (table 2.6) and V = 1.0m/sec,

$$D^{0.63} = 1.112 \times 100\, S^{0.54} / 1 \times \pi$$

$$D^{0.63} = 111.2\, S^{0.54} / \pi$$

$$D^{0.63} = 35.4S^{0.54}$$

$$D = {}^{0.63}\sqrt{35.4S^{0.54}} \qquad \text{...............5.2}$$

Equation 5.2 was used to design for the whole conduit and the results are shown in Appendix D. Distribution pipelines are sized to deliver the peak daily demand. Dual-tap standpipes will be connected directly to a pipe laid according to the slopiness of the roads (Table 5.5). There will be inspection chambers at every change in direction of pipes, so that faults can easily be detected and corrected.

Table 5.5 Value of calculation of Spot Heighten on Estate Roads.

Roads Number	Distance of Roads	Slope Value	Roads Number	Distance of Roads	Slope Value
1.	250	0.076	39.	733	0.023
2.	381	0.052	40.	147	0.135
3.	154	0.071	41.	147	0.052
4.	785	0.036	42.	280	0.016
5.	60	0.012	43.	541	0.047
6.	96	0.047	44.	296	0.017
7.	273	0.034	45.	251	0.030
8.	482	0.049	46.	170	0.048
9.	184	0.083	47.	.469	0.034
10.	360	0.025	48.	63	0.012
11.	606	0.013	49.	370	0.080
12.	190	0.102	50.	277	0.074
13.	230	0.154	51.	140	0.090
14.	473	0.078	52.	102	0.024
15.	199	0.082	53.	100	0.068
16.	224	0.062	54.	316	0.046
17.	190	0.071	55.	653	0.013
18.	390	0.084	56.	100	0.074
19.	800	0.091	57.	110	0.083
20.	242	0.043	58.	580	0.077
21.	250	0.029	59.	184	0.067
22.	892	0.066	60.	100	0.088
23.	240	0.085	61.	272	0.009
24.	351	0.094	62.	923	0.109
25.	100	0.017	63.	336	0.028
26.	1.041	0.080	64.	111	0.093
27.	64	0.019	65.	77	0.090
28.	462	0.074	66.	98	0.012
29.	226	0.095	67.	73	0.018
30.	202	0.039	68.	233	0.015
31.	165	0.126	69.	80	0.020
32.	400	0.101	70.	498	0.014
33.	88	0.037	71.	90	0.013
34.	188	0.069	72.	90	0.013
35.	850	0.084	73.	64	0.115
36.	530	0.076	74.	95	0.014
37.	150	0.020	75.	49	0.028
38.	485	0.018			

5.9 Economic Evaluation of Existing Water System

The economic value of the existing systems of water supply was studied. There are 919 houses using hand-dug wells and 9 houses using boreholes. Their estimated cost is given in Table 5.6.

Table 5.6: Estimated values of present system of water supply in the estate.

Item	Boreholes (₦)	Hand-dug wells (₦)
Boring or excavation	400,000.00	40,000.00
Ringing	-	4,000.00
Water tank	30,000.00	30,000.00
Tank wall	45,000.00	45,000.00
Piping	30,000.00	30,000.00
Miscellaneous	10,000.00	3,000.00
Total per plot	515,000.00	152,000.00
Total for estate	4,635,000.00	139,688,000.00
General Total	144,323,000.00	
General Maintenance (10% of Insdtallation)	14,432,300.00	

5.9.1 Proposed System (Treatment Plant and Distribution)

Estimates of the proposed treatment plant and other alternative source of water supply system were prepared as shown in Table 5.7. There are three methods available for cost estimation, these are:

(a) concept cost estimating;

(b) detailed cost estimating – based on Bill of Quantity;

(c) comparison cost estimating – based on past experience.

The comparison cost estimating method was used for the costing of items in this project because construction and maintenance of these items are always the same; the only items that change are the sizes, numbers of these items and their prices. The estimated values of the proposed treatment plant and the public water system supply to the estate are given in Table 5.7. Forms of payment and charges on annual basis to the consumers can be estimated using the interest factors summarized in Table 2.8.

The variation in the cost of the common items is due to the difference in the quantity of the items used for the various alternatives. Where only two overhead storage tanks are provided for the proposed treatment plant, three each is provided for the other alternatives. The public water distribution is more expensive because the whole water distribution systems have to be upgraded from the dam to the consumer's point of service.

Table 5.7 Estimated values of all alternatives water supply sources to the estate.

Item	Proposed treatment plant (₦)	Public water distribution (₦)
Drainages	120,000,000.00	-
Raw Water Impoundment	60,000,000.00	-
Aeration Structure	20,000,000.00	-
Mixing tank	20,000,000.00	-
Sedimentation Tanks	40,000,000.00	-
Filtration Tanks	40,000,000.00	-
Treated Water Tanks	60,000,000.00	-
Lift pumps	5,000,000.00	20,000,000.00
Piping	90,000,000.00	120,000,000.00
Elevated storage tank/s	80,000,000.00	150,000,000.00
Supplementary Structures	100,000,000.00	205,000,000.00
Total	635,000,000.00	475,000,000.00
Maintenance and General Services	69,000,000.00	70,000,000.00

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusion

The Government Residential Estate in Ilesha Road is expected to develop rapidly as a result of its nearness to establishments of important institutions such as the Federal University of Technology, Akure, the Local Government Secretariat and major roads like the Ilesha and Ondo roads. Therefore, there is an urgent need to develop the much needed infrastructure, especially water supply. There is presently no public water supply in the project area. As a result most people in the area rely on water from well, streams and rainwater harvesting. All the wells are privately owned. Water from the stream is unsafe for human consumption. Thus, the provision of adequate and potable water based on this project will alleviate the problem.

Estimates of water requirement of the project area are base on the daily per capita demand of 1,750 l/day. Population forecast is based on an annual growth rate of 3.0 percent.

Geologically, the area is situated in the Precambrian basement complex as bedrocks are found on the surface or at shallow depths, the quantity of ground water is relatively small, and cannot be considered as a major source of water for the proposed scheme. Storm water combined with stream water is a more dependable source of the raw water for the proposed project.

The components of the proposed treatment plant are arranged in such a way that maximum use of space is ensured, with easy movement of men and materials. Initial pre-treatment, that is, settlement in the impoundment, consisting of chemical coagulation, flocculation and sedimentation has been considered for use. The proposed treatment plant is designed for maximum flow rate of 1,750 m^3/day based on 16h daily operation. The operation consist of water intake, aeration system, coagulation and flocculation, sedimentation and clarification, filtration, and disinfections

The water intake consists of a structure of dry chamber to be constructed along floodwall of the storage lake. The impoundment on which the intake structure is to be constructed to prevent flooding, consists of compacted stone filling, reinforced concrete retaining structure and hand rails. The structure will also have a low lift pump sets extending 10meters into the lake with bell mouth end. The pump sets shall discharge into a ductile iron pipe as close as possible to the treatment plant.

Raw water from the intake structure will be delivered through the ductile iron pipes to the treatment plant. Magnetic flow meter to monitor the amount of raw water to be treated shall be installed before the aeration. The aeration could be completely bypassed and water could flow directly to the mixers, contact time shall not be less than 15minutes while the cascade aeration should be considered.

In order to guarantee the greatest flexibility in operations, the plant will be split into two separate but equivalent lines. This section shall be two successive stages, rapid mixing and slow mixing of good flocs formation. The aluminum sulphate is feed into the rapid mixing while hydrated lime and sodium alginate are dosed at the beginning of slow mixing. The tanks will be equipped with mechanical stirrers. To avoid the associated maintenance problems, the hydraulic system for rapid and slow mixing water with chemical for coagulation and formation of flocs should be applied

Sedimentation will be adopted because it provides for sedimentation-basin construction which will divide the "flowing deep" stream of water into many shallow streams flowing separate in parallel conduits. The formed sludges are self-clearing because of the inclination of sedimentation conduits. The flocculated water is introduced from the lower side and it is collected as clear water on the upper side by a channel system while the sludge will flow inside the ducts in the opposite direction. The clarified water will be collected in channels carrying it to the filters.

Filtration will be done in rapid gravity, single media, contact rates, variable level filters. All filtering operations will be controlled and carried out from the operating gallery. All control valves shall be operated by pneumatic powered actuators and shall be equipped with manual control for emergencies. Filter backwashing shall be by air blown by air blowers and by water pumping by centrifugal horizontal pumps. A backwashing flow meter of the magnetic type shall be installed on the outlet side of the backwashing pumps to control the amount of backwashing water. After filtering and having been dosed with chlorine the water shall be conveyed to filters and for preparing the reagents as well as for distribution to the treatment plant network and by high lift pumps to the reservoir and distribution networks.

All chemical dosage of alum solution, hydrated lime slurry, sodium alginate, etc, shall be prepared in their respective ratios and added to the water at their respective required stages in the treatment line. The chlorination plant shall consist pre-chlorination and post-chlorination in the pipe existing from the filtration section.

Distribution pipelines are sized to deliver the peak daily demand. Dual-tap standpipes will be connected directly to a pipe on both sides of the roads to save consumers from having to cross the roads. The transmission main sizes were selected to minimize total costs, which include the pipe and power costs. The pipeline should be laid where they will be accessible, where excavation is simple, and where construction will not interfere with traffic. The looped networks distribution system will be adopted because of its greater reliability of service, elimination of dead ends, and improve flow condition during periods of high local demand.

The government can either take a loan to sponsor the project or handle it with budgeted money.

6.2: RECOMMENDATIONS:

The State Government should as a matter of urgency start the construction of water supply treatment plant and other social amenities in general to Ilesha Road Government Housing Estate, Akure and other areas of the state lacking such facilities. It is advisable to start the project on time since the value of money depreciates constantly in the Country.

Further detailed studies should be carried on all the aspects touched in this work before the realization of this project. Changes that might have taken place between the time of this study and its implementation time could result in higher prices than those mentioned in this study. In particular, a detailed pipe distribution analysis should be undertaken to confirm the sizes of suggested commercial pipes of 300mm, 200mm and 100mm diameter.

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Appendix A

Design sheet 1

REF	CALCULATION	RESULT
	POPULATION ESTIMATION	
	Assuming a population of 12, 12 and 8 people per plot for high, medium and low density plots respectively. Therefore population for the estate will be: High density area = 272x12 = 3,264 people Medium density area = 285x12 = 3,470 people Low density area = 371x 8 = 2,968 people Total population = 9,652 people Growth rate of 3% adopted for population increase in 70years $P_{30} = 9{,}652(1+0.03)^{70}$ Total population = 17,433people Total population for design = 17,500 people	Total population for design = 17,500 people
	CALCULATION OF REQUIRED WATER VOLUME	
	Water requirement of one person = 100litres/cap/day Therefore required volume of water = 17,500 x 100 = 1,750,000litres/day = 1,750cubic meter/ day	Required volume of water = 1,750 m^3/day
	DESIGN OF RAWWATER IMPOUNDMENT	
	Designing for a 6months of 31days capacity impoundment = 1,750 x 186 = 325,500m^3 cap adopt a volume of 350,000m^3 capacity Adopt a depth of 4.5m Area of impoundment = volume/length = 350,000/4.5 = 77,7778m^2 Adopt area = 200m x 400m = 80,000m^2 Volume of impoundment. = 4.5 x 200 x 400 = = 360,000m^3 Detention time, t_d = V/Q = 36,,000 / 1,750 x 3 = 68.57days	Volume of impoundment = 360,000m^3 Detention time, t_d = 69days

Design sheet 2

REF	CALCULATION	RESULT
	Sectional area of impoundment = 4.5 x 200 = 900m² Overflow rate, r = Volume/Area = 360,000 / (900 x 24x 60) = 0.28m³/m²/day	Overflow rate, r = 0.28m³/m²/sec
	DESIGN FOR MIXING TANK	
	Designing for a three days capacity tank = 1,750 x 3 = 5,250m³ capacity	Provide a mixing tank capacity of = 5,250m³
	Adopt a detention time, t_d = 30mins = 0.02day Volume of tank = 5.250 x 0.02 = 105m³ Adopt a tank depth of 3.5m Provide a mixing tanks size of = 105 m³ / 3.5m = 30m² Mixing tank size of = 6.0m x 5.0m x 3.5m = 105m³ Total volume of tank = 110m³ /day	Detention time, t_d = 30mins = 0.02day Provide a mixing tank capacity of 105m³ Provide a mixing tanks = 110m³
	DESIGN FOR SEDIMENTATION TANK	
	Adopt alum dosage of 40mg/l = 0.04mg/ m³ Required content = 0.04 x 5250 = 210mg/m³ Surface loading rate: r = Q/V = 5250/110 = 47.73mg/sec Provide loading rate = 48.0mg/sec To remove taste and odor add 6500kg/10⁶m³ of activated carbon Adopt a detention time, t_d = 6hours t_d = 24V/Q 6 = V24 /5250 With depth of 3.5m Area = Volume/depth = 700/3.5 = 200m² Provide a sedimentation tank size of = 20m x 10m x 3.5m = 700m³	Alum content = 210 mg/m³ Provide loading rate = 48.0mg/sec Provide a sedimentation tank size of = 700m³

Design sheet 3

REF	DESIGN FOR FILTRATION TANK	RESULT
	Adopt a filtration rate of 100 m^3 / m^2 .min Area of tank = flow rate / filtration rate = 5250 x 100 / 1,440 = 525,000/1,440 = $364.6m^2$ Adopt volume of tank = $370m^3$ Backwash rate at 90% per day = 0.9 x 370 / 24 =13.875 m^3 / day Backwashing rate in percentage = 13.875 / 100 = 0.14% Adopt a detention time, t_d = 12hours t_d = 24V/Q 12 = V24 /5250 Volume of tank = 12 x 5,250 / 24 = $2,625m^3$ Adopt 2-unit tanks of $2,625m^3$/2 = 1,312.5 m^3 Adopt a tank of depth of 3.5m Area = Volume/depth = 1,312.5/3.5 = $375m^2$ Provide filtration tanks size of = 19m x 70m x 3.5m = $1,330m^3$ Provide 2-unit filtration tanks size of = $1,330m^3$ x 2 = $2,660m^3$	Filtration rate of 100 m^3 / m^2 .min Adopt a detention time, t_d = 12hours Volume of tank = $2,625m^3$ Provide 2-unit filtration tanks size of = $2,660m^3$
	Filtration Aggregates	
	Adopt coarse-to-fine media to a cover a depth of 1m. Provide a 170mm of garnet layer with grain size of 0.5mm, 230mm of silica sand with grain size of 0.4mm, and 650mm of coal with grain size of 0.6m.	Adopt coarse-to-fine media

Design sheet 4

REF	CALCULATION	RESULT
	Chlorination Content	
	Adopt a 5kg/day chlorine and residual after 65mins contact of 0.25mg/l Dosage = 5 x 1000/5250 = 0.95mg/m³ Chlorine demand = 0.95 − 0.25 = 0.70mg/m³ Provide chlorine = 0.70mg/m³	Provide chlorine = 0.70mg/m³
	TREATED WATER STORAGE TANKS	
	Adopt a detention time, t_d = 36hours t_d = 24V/Q 36 = V24 /5250 Volume of tank = 36 x 5250/24 = 7,875m³ Adopt 3-unit tanks of = 7,875m³ / 3 = 2,625 m³ with depth of 3.0m Area = Volume/depth = 2,625/3.0 = 875m² Provide a mixing tank size of = 22m x 30m x 3.0m = 2,625m³	Detention time, t_d = 36hours Adopt 3-unit tanks of = 7,875m³ each.
	LOW LIFT PUMP	
	Period for pumping = 6 hours under normal condition Rate of water flow = Q/t = 7,920/6x60x60 = 0.28m³/sec Diameter of pipe = 300mm Using equation 2.8 for the suction head, Q = $0.278CD^{2.63}S^{0.54}$ = $0.278CD^{2.63}(h_f/L)^{0.54}$ $(h_f/L)^{0.54}$ = $0.278CD^{2.63}/Q$ where C = 100 (see table 2.6 using old cast iron) $(h_f/L)^{0.54}$ = $(0.278x100x0.3^{2.63})/0.28$ h_f/L = 0.54 x 4.1 = 14.20 Using equation 2.8 for the suction head, Q = $0.278CD^{2.63}S^{0.54}$	Rate of water flow = 0.28m³/sec

Design sheet 5

REF	CALCULATION	RESULT
	$= 0.278CD^{2.63}(h_f/L)^{0.54}$	
	$(h_f/L)^{0.54} = 0.278CD^{2.63}/Q$ where C = 100 (see table 2.6 using old cast iron)	
	$(h_f/L)^{0.54} = (0.278 \times 100 \times 0.3^{2.63})/0.28$ h_f/L	
	$= 0.54 \times 4.1$	
	$= 14.20$	
	Suction side of pump:	
	Let L = 10m	
	$h_f = 10 \times 14.20$	
	$= 142m$	
	Using projecting pipes, k = 1.0 (see table 5.8)	
	$h_f = 1.0 \times 0.28^2 / 2 \times 9.81$	
	$= 0.004m$	
	Using 3-number projecting pipes $= 0.004 \times 3$	
	$= 0.012m$	
	Using a standard 90° elbow pipe bend of k = 0.3 (see table 5.8)	
	$h_f = 0.3 \times 0.28^2 / 2 \times 9.81$	
	$= 0.0012m$	
	Total head loss on the suction side of pump	Total head loss on the
	$= 142 + 0.012 + 0.0012$	suction side of pump
	$= 142.013m$	= 142.013m
	Delivery side of pump:	
	Using equation 2.8 for the suction head, Q	
	$= 0.278CD^{2.63}S^{0.54}$	
	$= 0.278CD^{2.63}(h_f/L)^{0.54}$	
	$h_f = (10.7Q^{1.85}L)/(C^{1.85}D^{4.87})$	
	where C = 100 (see table 2.6 using old cast iron) and L = 15m	
	$= (10.7 \times 0.28^{1.85} \times 15)/(100^{1.85} \times 0.3^{4.87})$	
	$= 1.07m$	
	Using projecting pipes of k = 1.0 (table 5.8)	
	$h_f = 1.0 \times 0.28^2 / 2 \times 9.81$	
	$= 0.004m$	
	Using check valve of k = 8.0 (table 5.4)	
	$h_f = 8.0 \times 0.28^2 / 2 \times 9.81$	
	$= 0.032m$	
	Using gate valve of k = 0.2 (table 5.4)	
	$h_f = 0.2 \times 0.28^2 / 2 \times 9.81$	
	$= 0.0008m$	

REF	CALCULATION	RESULT
	Using check valve of k = 8.0 (table 5.4)	
	h_f = $8.0 \times 0.28^2 / 2 \times 9.81$	
	= 0.032m	
	Using gate valve of k = 0.2 (table 5.4)	
	h_f = $0.2 \times 0.28^2 / 2 \times 9.81$	Total head loss on the delivery side of pump
	= 0.0008m	= 1.11m
	90° Using a standard elbow pipe bend of k = 0.3	
	h_f = $0.3 \times 0.28^2 / 2 \times 9.81$	
	= 0.0012m	Total head loss on the system = 143.12m
	Total head loss on the delivery side of pump	
	= 1.07 + 0.004 + 0.032 + 0.008 + 0.0012	
	= 1.11m	
	Total head loss on the system, H = 142.01 + 1.11	Pump capacity = 400KW
	= 143.12m	
	Pump capacity:	
	H_P = QρgH = $0.47 \times 9.81 \times 100 \times 143.12$	
	= 393122W	
	= 393KW.	
	HIGH LIFT PUMP	
	Period for pumping = 6 hours under normal condition	
	Rate of water flow = Q/t = 7,920/6x60x60	Rate of water flow = $0.28m^3/sec$
	= $0.28m^3/sec$	
	Diameter of pipe = 300mm	
	Suction side of pump:	
	Using equation 2.8 for the suction head, Q	
	= $0.278CD^{2.63}S^{0.54}$	
	= $0.278CD^{2.63}(h_f/L)^{0.54}$	
	$(h_f/L)^{0.54}$ = $0.278CD^{2.63}/Q$	
	where C = 100 (see table 2.11 using old cast iron)	
	$(h_f/L)^{0.54}$ = $(0.278 \times 100 \times 0.3^{2.63})/0.28$	
	h_f/L = 0.54×4.19	
	= 14.20	
	Let L = 15m	
	h_f = 15×14.20	

REF	CALCULATION	RESULT
	= 213m = 0.004m Using a standard 90^0 elbow pipe bend of k = 0.3 (see table 5.8) $h_L = 0.3 \times 0.28^2 / 2 \times 9.81$ = 0.0012m Using gate valve of k = 0.2 (table 5.8) $h_L = 0.2 \times 65 \times 0.4^2 / 2 \times 9.81$ = 0.08m Total head loss on the suction side of pump = 213 + 0.004 + 0.008 + 0.0012 = 213.m	Total head loss on the suction side of pump = 213m
	Delivery side of pump Using equation 2.8 for the suction head. Q $= 0.278CD^{2.63}S^{0.54}$ $= 0.278CD^{2.63}(h_L/L)^{0.54}$ $h_L = (10.7Q^{1.85}L)/(C^{1.85}D^{4.87})$ where C = 100 (see table 2.6 using old cast iron) and L = 15m $= (10.7 \times 0.28^{1.85} \times 20)/(100^{1.85} \times 0.3^{4.87})$ = 1.43m Using projecting pipes of k = 1.0 (table 5.8) $h_L = 1.0 \times 0.28^2 / 2 \times 9.81$ = 0.004m Using check valve of k = 8.0 (table 5.4) $h_L = 8.0 \times 0.28^2 / 2 \times 9.81$ = 0.032m Using a standard 90^0 elbow pipe bend of k = 0.3 $h_L = 0.3 \times 0.28^2 / 2 \times 9.81$ = 0.0012m Total head loss on the delivery side of pump = 1.43 + 0.004 + 0.032 + 0.008 + 0.0012 = 1.47m	Total head loss on the delivery side of pump = 1.47m
	Total head loss on the system. H = 213 + 1.47 = 214.47m	Total head loss on the system = 214.47m
	Pump capacity: H_P = QegH = 0.24 x 9.81 x 1000 x 214.47 = 589,099W = 589KW	Pump capacity = 600KW

Sheet 1: Catchments characteristics and drains values of the estate.

Codes	Length m	Slope $X10^{-2}$	Concentration Time mins	Runoff Intensity mm/hour	Runoff Coefficient	Area m^2	Catchments Quantity m^3/sec	Number of Catchment Drains	Contributing Drains	Contributed Quantity m^3/sec	Flowing Quantity m^3/sec
A1	150	4.72	10.18	73.05	0.5	16500	0.168	1	-	-	0.168
A2	100	2.68	9.26	72.78	0.5	9900	0.100	2	-	-	0.100
A3	80	2.15	8.49	72.55	0.5	10240	0.103	3	-	-	0.103
A4	80	2.09	8.58	72.57	0.5	7200	0.073	4	-	-	0.073
A5	310	9.32	13.70	74.11	0.5	41880	0.431	5	4,12	0.205	0.636
A6	251	4.97	14.83	74.45	0.5	14339	0.148	6	-	-	0.148
A7	73	1.64	8.78	72.63	0.5	8030	0.081	7	1,8	0.336	0.417
A8	150	4.73	10.17	73.05	0.5	16500	0.168	8	-	-	0.168
A9	158	4.88	10.45	73.14	0.5	14694	0.149	9	-	-	0.149
A10	70	2.01	7.86	72.34	0.5	8050	0.081	10	2,3	0.203	0.284
A11	250	8.65	11.94	73.58	0.5	27292	0.279	11	-	-	0.279
A12	135	3.39	10.66	73.20	0.5	12960	0.132	12	-	-	0.132
A13	90	2.72	8.49	72.55	0.5	5850	0.059	13	-	-	0.059
A14	25	0.98	4.69	71.41	0.5	3700	0.037	14	15,38	0.862	0.899
A15	130	3.96	9.75	72.93	0.5	16770	0.170	15	-	-	0.170
A16	120	3.61	9.50	72.85	0.5	15600	0.158	16	-	-	0.158
A17	154	4.80	10.32	73.10	0.5	14694	0.149	17	-	-	0.149
A18	78	2.50	7.86	72.36	0.5	7722	0.078	18	9,10	0.433	0.511
A19	184	4.16	12.50	73.75	0.5	14904	0.153	19	-	-	0.153
A20	60	1.95	7.06	72.12	0.5	4800	0.048	20	-	-	0.048
A21	110	2.87	9.71	72.91	0.5	10100	0.102	21	16,17	0.307	0.409

Sheet 2: Catchment's characteristics and drains values of the estate.

Codes	Length	Slope $x10^{-2}$	Concentration Time	Runoff Intensity	Runoff Coefficient	Area	Catchments Quantity	Number of Catchment Drains	Contributing Drains	Contributed Quantity	Flowing Quantity
	m		$mins$	$mm/hour$		m^2	m^3/sec	m		m^3/sec	m^3/sec
A22	98	2.17	8.50	72.55	0.5	8.722	0.088	22	18.20	1.235	0.088
A23	40	1.87	5.25	71.58	0.5	3.520	0.035	23	21.22	2.294	0.035
A24	96	2.88	8.73	72.62	0.5	8.544	0.086	24	-	-	0.086
A25	80	2.34	8.22	72.47	0.5	6.640	0.067	25	-	-	0.067
A26	88	2.69	8.38	72.51	0.5	7.656	0.077	26	-	-	0.077
A27	190	4.35	12.60	73.78	0.5	15.960	0.164	27	25.26	0.144	0.308
A28	95	2.82	8.73	72.62	0.5	15.950	0.161	28	-	-	0.161
A29	92	2.75	8.59	72.58	0.5	8.280	0.084	29	5.19	0.789	0.873
A30	90	2.84	8.35	72.51	0.5	7.380	0.074	30	27.29	1.181	1.255
A31	98	2.80	8.96	72.69	0.5	10.878	0.110	31	6.13. 32	0.284	0.394
A32	64	1.73	7.77	72.33	0.5	7.650	0.077	32	-	-	0.077
A33	90	2.89	8.30	72.49	0.5	9.812	0.099	33	36	4.097	4.196
A34	180	4.94	11.51	73.45	0.5	18.300	0.187	34	23.24	1.177	1.364
A35	70	2.94	6.79	72.04	0.5	3.200	0.032	35	28.34	1.525	1.557
A36	110	3.92	8.61	72.58	0.5	8.910	0.090	36	35.69	4.007	4.097
A37	90	2.69	7.38	72.21	0.5	9.810	0.098	37	30.33	5.451	5.549
B1	100	3.70	8.18	72.54	0.5	8.900	0.090	38	39.40	0.602	0.692
B2	49	1.75	6.30	71.89	0.5	5.530	0.055	39	-	-	0.055
B3	150	3.72	11.15	73.35	0.5	21.000	0.214	40	41.42	0.333	0.547
B4	190	4.84	12.09	73.63	0.5	22.040	0.226	41	-	-	0.226
B5	106	2.73	9.62	72.89	0.5	10.600	0.107	42	-	-	0.107
B6	230	5.82	13.05	73.92	0.5	34.040	0.350	43	-	-	0.350
B7	200	4.85	12.57	73.77	0.5	20.200	0.207	44	-	-	0.207
B8	100	2.72	9.21	72.63	0.5	9.000	0.091	45	44	0.207	0.298

Sheet 3: Catchments characteristics and drains values of the estate.

Codes	Length	Slope $x10^{-2}$	Concentration Time	Runoff Intensity	Runoff Coefficient	Area	Catchments Quantity	Number of Catchments Drains	Contributing Drains	Contributed Quantity	Flowing Quantity
	m		$mins$	$mm/hour$		m^2	m^3/sec	m		m^3/sec	m^3/sec
B9	197	4.96	12.32	73.70	0.5	18,464	0.189	46	-	-	0.189
B10	48	2.73	4.54	71.36	0.5	4,320	0.043	47	43.46-	0.396	0.439
B11	36	2.93	4.07	71.22	0.5	3,240	.0032	48	47.49	0.573	.0573
B12	100	3.85	8.33	72.50	0.5	10,100	0.102	49	-	-	0.102
B13	100	3.80	8.36	72.51	0.5	9,200	0.093	50	-	-	0.093
B14	100	3.84	8.32	72.50	0.5	11,000	0.111	51	-	-	0.111
B15	99	3.85	8.28	74.48	0.5	14,256	0.148	52	45.53	0.517	0.665
B16	224	4.89	13.80	74.14	0.5	21,280	0.219	53	-		0.398
B17	90	2.84	8.53	72.51	0.5	8,550	0.086	54	-		0.196
B18	140	2.94	11.58	73.45	0.5	20,720	0.212	55	52	0.665	0.877
B19	50	2.90	5.27	71.58	0.5	4,850	0.048	56	55.65	1.223	1.271
B20	101	2.93	9.02	72.71	0.5	10,706	0.108	57	50.51	0.204	0.312
B21	113	2.98	9.77	72.93	0.5	6,667	0.068	58	-		0.148
B22	150	3.96	10.89	73.27	0.5	17,700	0.180	59	57.58	0.380	0.560
B23	100	2.99	8.88	72.66	0.5	9,200	0.093	60	59.68	1.623	1.716
B24	283	7.99	13.55	74.07	0.5	25,187	0.295	61	-		0.465
B25	250	7.96	12.53	73.70	0.5	23,500	0.241	62	-		0.463
B26	102	2.91	9.11	72.73	0.5	10,404	0.105	63	-		0.231
B27	140	2.90	11.64	73.49	0.5	13,720	0.140	64	62.63	0.346	0.486
B28	280	7.97	13.45	74.04	0.5	33,600	0.346	65	-		0.618
B29	70	2.04	7.82	72.35	0.5	2,590	0.026	66	-		0.059
B30	100	3.02	8.85	72.66	0.5	9,000	0.091	67	-		0.205
B31	240	6.00	13.33	74.00	0.5	22,080	0.227	68	61,64,67	0.836	1.063
B32	100	2.84	9.06	72.72	0.5	9,800	0.099	69	60,70	2.351	2.450

Sheet 4: Catchments characteristics and drains values of the estate.

Codes	Length	Slope $x10^{-2}$	Concentration Time	Runoff Intensity	Runoff Coefficient	Area	Catchments Quantity	Number of Catchments Drains	Contributing Drains	Contributed Quantity	Flowing Quantity
	m		mins	mm/hour		m^2	$m^3/sec.$	m		$m^3/sec.$	$m^3/sec.$
B33	420	13.88	14.86	74.46	0.5	58,800	0.609	70	66	0.026	0.635
B34	100	2.86	9.03	72.71	0.5	10,000	0.101	71	-	-	0.101
B35	128	3.70	9.83	72.95	0.5	16,256	0.165	72	-	-	0.165
B36	197	7.76	10.37	73.11	0.5	18,443	0.187	73	71	0.101	0.288
B37	187	7.71	9.98	72.99	0.5	16,388	0.166	74	-	-	0.166
C1	172	7.54	9.44	72.83	0.5	17,888	0.181	75	-	-	0.181
C2	190	8.01	9.96	72.99	0.5	20,140	0.204	76	75,79	0.253	0.457
C3	100	2.80	9.11	72.73	0.5	11,800	0.119	77	76,80	0.558	0.677
C4	71	2.71	7.08	72.12	0.5	6,887	0.690	78	-	-	0.690
C5	140	4.71	9.66	72.90	0.5	11,800	0.072	79	-	-	0.072
C6	165	5.75	10.15	73.05	0.5	16,510	0.101	80	-	-	0.101
C7	93	2.79	8.40	72.52	0.5	9,579	0.058	81	77,78	1.367	1.425
C8	86	2.73	8.19	72.46	0.5	7,654	0.046	82	-	-	0.046
C9	86	2.74	8.18	72.45	0.5	13,020	0.079	83	-	-	0.062
C10	64	2.73	6.52	71.96	0.5	7,654	0.046	84	-	-	0.041
C11	116	4.75	8.33	72.50	0.5	10,208	0.062	85	83,84	0.125	0.187
C12	110	4.59	8.10	72.43	0.5	6,710	0.041	86	86	1.425	1.466
C13	88	3.76	7.37	72.21	0.5	8,624	0.052	87	-	-	0.072
C14	173	5.69	10.57	73.17	0.5	23340	0.143	88	31,37	5.943	6.086
C15	100	2.69	9.25	72.78	0.5	11,800	0.072	89	82,88	6.132	6.204
C16	156	4.68	10.52	73.16	0.5	32,448	0.330	90	89, 96	6.860	7.193
C17	86	2.76	8.15	72.45	0.5	8,428	0.085	91	-	-	0.376
C18	150	4.68	10.21	73.06	0.5	31,050	0.315	92	85,91	0.272	0.587
C19	304	15.83	11.00	73.30	0.5	36,856	0.376	93	90,92	7.780	8.156

Sheet 5: Catchment's characteristics and drains values of the estate.

Codes	Length	Slope $\times10^{-2}$	Concentration Time	Runoff Intensity	Runoff Coefficient	Area	Catchments Quantity	Number of Catchments Drains	Contributing Drains	Contributed Quantity	Flowing Quantity
	m		*mins*	*mm/hour*		*m^2*	*m^3/sec*			*m^3/sec*	*m^3/sec*
C20	147	3.82	10.87	73.26	0.5	20,850	0.212	94	-	-	0.212
C21	147	3.80	10.69	73.21	0.5	20,580	0.209	95	-	-	0.209
C22	130	3.78	9.77	72.93	0.5	15,860	0.161	96	95, 97	0.498	0.659
C23	150	3.78	10.91	73.27	0.5	28,350	0.289	97	-	-	0.289
C24	155	3.97	10.98	73.29	0.5	24,800	0.253	98	72,73	0.453	0.706
C25	200	5.73	11.62	73.49	0.5	105,800	1.081	99	94,98	0.918	1.999
C26	186	5.76	10.97	73.29	0.5	18,224	0.186	100	99, 103	2.931	3.117
C27	200	5.69	11.65	73.50	0.5	15,600	0.159	101	-	-	0.159
C28	96	2.70	8.79	72.64	0.5	7,008	0.071	102	101,104	0.360	0.431
C29	72	2.75	6.99	72.10	0.5	9,218	0.092	103	102,108	0.840	0.932
C30	251	6.77	12.99	73.97	0.5	19,540	0.201	104	-	-	0.201
C31	70	2.75	6.97	72.09	0.5	7,400	0.044	105	-	-	0.044
C32	63	2.76	6.42	71.93	0.5	4,851	0.029	106	-	-	0.029
C33	100	3.74	8.15	72.45	0.5	4,851	0.029	107	105,106	0.073	0.102
C34	90	3.25	7.93	72.38	0.5	9,900	0.060	108	107,109	0.349	0.409
C35	307	8.76	13.92	74.18	0.5	39,970	0.247	109	-	-	0.247
D1	730	27.36	17.50	75.25	0.5	68,580	0.430	110	-	-	0.430
D2	78	2.56	7.78	72.33	0.3	4,056	0.024	111	110	0.430	0.454
D3	110	3.86	8.66	72.60	0.3	10,670	0.065	112	-	-	0.065
D4	150	4.58	10.30	73.09	0.3	23,250	0.142	113	-	-	0.142
D5	280	8.55	13.09	73.93	0.3	22,680	0.140	114	112,113	0.207	0.347
D6	57	1.54	7.44	72.23	0.3	5,187	0.031	115	114,119	1.436	1.467
D7	120	4.53	8.71	72.61	0.3	12,120	0.073	116	-	-	0.073
D8	190	6.46	10.82	73.25	0.3	16,810	0.103	117	-	-	0.103

Sheet 6: Catchments characteristics and drains values of the estate

Codes	Length	Slope $x10^{-2}$	Concentration time	Runoff Intensity	Runoff Coefficient	Area	Catchments Quantity	Number of Catchments Drains	Contributing Drains	Contributed Quantity	Flowing Quantity
	m		*mins*	*mm/hour*		m^2	$m^3/sec.$	*m*		$m^3/sec.$	$m^3/sec.$
D9	91	3.54	7.74	72.32	0.3	8.918	0.054	118	115.116	1.540	1.594
D10	150	4.58	10.30	73.09	0.3	22.500	0.137	119	132.135	0.952	1.089
D11	240	7.69	12.11	73.63	0.3	12.240	0.075	120	-	-	0.075
D12	111	4.75	8.05	72.42	0.3	7.548	0.046	121	-	-	0.046
D13	60	2.69	6.24	71.87	0.3	1.800	0.011	122	120.121	0.121	0.132
D14	77	2.76	7.49	72.25	0.3	6.776	0.041	123	-	-	0.041
D15	66	2.70	6.71	72.01	0.3	3.828	0.023	124	125.128	0.319	0.342
D16	90	3.44	7.76	72.33	0.3	7.110	0.043	125	123. 126	0.151	0.194
D17	180	5.75	10.85	73.26	0.3	18.000	0.110	126	-	-	0.110
D18	140	4.68	8.04	72.41	0.3	4.070	0.025	127	122.124	0.474	0.499
D19	94	2.73	8.77	72.63	0.3	5.640	0.034	128	129.130	0.091	0.125
D20	90	1.72	8.49	72.55	0.3	5.670	0.034	129	-	-	0.034
D21	100	3.15	8.72	72.62	0.3	9.400	0.057	130	-	-	0.057
D22	272	8.72	12.71	73.81	0.3	25.314	0.156	131	-	-	0.156
D23	90	2.67	8.55	72.57	0.3	3.150	0.019	132	127.131	0.655	0.674
D24	110	3.98	8.56	72.57	0.3	6.314	0.038	133	-	-	0.038
D25	80	2.66	7.82	72.35	0.3	9.240	0.056	134	132.134	0.712	0.768
D26	208	5.79	12.10	73.63	0.3	16.428	0.101	135	136.137	0.083	0.101
D27	100	3.83	8.07	72.42	0.3	6.800	0.041	136	-	-	0.041
D28	100	3.85	8.06	72.42	0.3	7.000	0.042	137	-	-	0.042
D29	273	8.73	12.74	73.82	0.3	4.815	0.030	138	139.140	0.150	0.180
D30	98	3.80	7.97	72.39	0.3	7.840	0.047	139	-	-	0.047
D31	190	5.76	11.31	73.39	0.3	16.810	0.103	140	-	-	0.103
D32	346	14.63	12.53	73.76	0.3	32.137	0.198	141	-	-	0.198

Sheet 7: Catchments characteristics and drains values of the estate.

Codes	Length	Slope $(x10^{-3})$	Concentration Time	Runoff Intensity	Runoff Coefficient	Area	Catchments Quantity	Number of Catchments Drains	Contributing Drains	Contributed Quantity	Flowing Quantity
	m		mins	mm/hour		(m^2)	m^3/sec			m^3/sec	m^3/sec
D33	72	2.92	6.96	72.09	0.3	750	0.005	142	-	-	0.005
D34	337	14.78	12.23	73.67	0.3	18,518	0.114	143	-	-	0.114
D35	73	2.79	7.16	72.15	0.3	749	0.005	144	142,143	0.119	0.124
D36	160	5.77	9.90	72.97	0.3	8,320	0.051	145	144	0.124	0.175
D37	79	2.75	7.65	72.30	0.3	12,640	0.076	146	-	-	0.076
D38	80	2.76	7.72	72.32	0.3	5,600	0.034	147	146, 148	0.122	0.156
D39	100	3.76	8.13	72.44	0.3	7,600	0.046	148	-	-	0.046
D40	102	3.91	8.13	72.44	0.3	10,404	0.063	149	-	-	0.063
D41	151	4.75	10.20	73.06	0.3	10,721	0.065	150	-	-	0.065
D42	118	3.74	9.25	72.78	0.3	14,160	0.086	151	147,149	0.219	0.305
D43	85	2.77	8.07	72.42	0.3	6,970	0.042	152	150, 153	0.086	0.128
D44	90	2.77	8.43	72.53	0.3	3,510	0.021	153	-	-	0.021
D45	140	4.14	10.15	75.05	0.3	20,720	0.175	154	-	-	0.175
D46	80	2.74	7.73	72.32	0.3	6,000	0.036	155	152,154	0.305	0.341
D47	78	2.63	7.70	72.31	0.3	6,970	0.042	156	145	0.175	0.217
D48	273	8.80	12.70	73.81	0.3	7,840	0.048	157	155,156	0.558	0.606
D49	71	2.52	7.29	72.19	0.3	4,815	0.029	158	-	-	0.029
D50	76	2.72	7.45	72.24	0.3	782	0.005	159	151,157	0.911	0.916
D51	133	4.13	9.77	72.93	0.3	7,581	0.046	160	161,164	0.518	0.654
D52	300	9.73	13.14	73.94	0.3	71,100	0.438	161	162,163	0.034	0.472
D53	80	2.24	8.36	72.51	0.3	4,720	0.029	162	-	-	0.029
D54	70	1.76	8.27	72.48	0.3	756	0.005	163	-	-	0.005
D55	100	3.09	8.77	72.63	0.3	7,600	0.046	164	-	-	0.046

APPENDIX C

Sheet 1: Characteristics of Drains for the Estate.

Drains Number	Length	Contributing Catchment Drains	Slope of drains	Flowing Quantity	Calculated Depth	Adopted Drains	Width of drains	Area of drains
	m			m^3/sec	mm	mm	m	m^2
1.	250	1.2	0.076	0.168	160	300	450	0.135
2.	381	7.8.9	0.052	0.417	241	300	450	0.135
3.	154	17	0.071	0.149	154	300	450	0.135
4.	785	14.15.16.21.23.24.35.36	0.036	4.097	608	750	1.1125	0.844
5.	60	20	0.012	0.048	141	300	450	0.135
6.	96	24	0.047	0.086	159	300	450	0.135
7.	273	12.26	0.034	0.132	169	300	450	0.135
8.	482	4.5.29	0.049	0.873	321	450	675	0.304
9.	184	19	0.083	0.153	152	300	450	0.135
10.	360	25.27.30	0.025	1.255	418	450	675	0.135
11.	606	11.38.40.42	0.013	0.692	378	450	675	0.304
12.	190	41	0.102	0.226	169	300	450	0.135
13.	230	43	0.184	0.350	184	300	450	0.135
14.	473	44.46.47.48	0.078	0.573	251	300	450	0.135
15.	199	45.52	0.082	0.665	264	300	450	0.135
16.	224	53	0.062	0.219	183	300	450	0.135
17.	190	49.54	0.071	0.102	134	300	450	0.135
18.	390	50.51.55.56	0.084	1.271	334	450	675	0.304
19.	800	65.69.70	0.091	2.450	421	450	675	0.304
20.	242	63.64	0.043	0.486	264	300	450	0.135
21.	250	62	0.029	0.241	219	300	450	0.135
22.	892	3.10.18.22.58.61.66.67	0.066	0.511	249	300	450	0.135
23.	240	68	0.085	1.063	312	450	675	0.304
24.	351	57.59.60	0.094	1.716	366	450	675	0.304
25.	100	71	0.017	0.101	175	300	450	0.135
26.	1.041	6.31.33.37.72.73.74	0.080	5.549	586	600	675	0.304
27.	64	84	0.019	0.046	127	300	450	0.135
28.	462	75.76.77	0.074	0.677	270	300	450	0.135
29.	226	79.82	0.095	0.072	111	300	450	0.135
30.	202	83.85	0.039	0.187	188	300	450	0.135
31.	165	80	0.126	0.101	120	300	450	0.135
32.	400	78.81.91.92	0.101	1.425	337	450	675	0.304
33.	88	87	0.037	0.052	118	300	450	0.135
34.	188	86.111	0.069	1.466	366	450	675	0.304
35.	850	110.116	0.084	0.430	223	300	450	0.135
36.	530	112.114.115.118	0.076	1.467	360	450	675	0.304
37.	150	113	0.020	0.142	192	300	450	0.135
38.	458	119.135.137	0.018	1.089	421	450	675	0.304

Sheet 2: Characteristics of Drains for the Estate.

Drains Numbers	Length	Contributing Catchment Drains	Slope of drains	Flowing Quantity	Calculated Drains Depth	Adopted Drains Depth	Drains Width	Drains Area
	M			m^3 sec.	mm	mm	m	m^2
39.	733	88,89,90,93	0.023	8.156	856	900	1.350	1.215
40.	147	94	0.135	0.212	156	300	450	0.135
41.	147	95	0.052	0.209	186	300	450	0.135
42.	280	96,97	0.016	0.659	356	450	675	0.304
43.	541	98,99,100	0.047	3.117	613	750	1.125	0.844
44.	296	101,102	0.017	0.431	301	450	675	0.304
45.	251	104	0.030	0.201	203	300	450	0.135
46.	170	105,107	0.048	0.102	144	300	450	0.135
47.	469	103,108,109	0.034	0.932	353	450	675	0.304
48.	63	106	0.012	0.046	117	300	450	0.135
49.	370	161,163	0.080	0.472	233	300	450	0.135
50.	277	146,147,151	0.074	0.305	201	300	450	0.135
51.	140	154	0.090	0.175	157	300	450	0.135
52.	102	149	0.121	0.063	141	300	450	0.135
53.	100	148	0.068	0.046	100	300	450	0.135
54.	316	150,152,155	0.046	0.341	229	300	450	0.135
55.	653	117,138,140	0.013	0.180	228	300	450	0.135
56.	100	136	0.074	0.041	94	300	450	0.135
57.	110	133	0.083	0.038	90	300	450	0.135
58.	580	120,122,127,132,134	0.077	0.768	347	450	675	0.304
59.	184	128,129	0.067	0.125	146	300	450	0.135
60.	100	130	0.088	0.057	103	300	450	0.135
61.	272	131	0.009	0.156	231	300	450	0.135
62.	923	141,143,145,162	0.109	0.198	159	300	450	0.135
63.	336	124,125,126	0.028	0.342	251	300	450	0.135
64.	111	121	0.093	0.046	95	300	450	0.135
65.	77	123	0.090	0.041	91	300	450	0.135
66.	98	139	0.012	0.047	140	300	450	0.135
67.	73	144	0.018	0.124	186	300	450	0.135
68.	233	160,164	0.015	0.564	341	300	450	0.135
69.	80	142	0.020	0.005	55	300	450	0.135
70.	498	156,157,157,159	0.014	0.916	414	450	675	0.304
71.	90	153	0.013	0.021	102	300	450	0.135
72.	90	13	0.013	0.059	150	300	450	0.135
73.	64	32	0.015	0.077	161	300	450	0.135
74.	95	28	0.014	0.161	216	300	450	0.135
75.	49	39	0.028	0.055	127	300	450	0.135

APPENDIX D

Sheet 1: Characteristics of Conduits for the Estate.

Drains Number	Length m	Slope	Calculated Diameter mm	Adopted Diameter mm	Drains Numbers	Length m	Slope	Calculated Diameter mm	Adopted Diameter mm
1.	250	0.076	72	100	39.	733	0.023	38	50
2.	381	0.052	58	100	40.	147	0.135	96	100
3.	154	0.071	69	100	41.	147	0.052	58	100
4.	785	0.036	48	50	42.	280	0.016	31	50
5.	60	0.012	26	50	43.	541	0.047	55	100
6.	96	0.047	55	100	44.	296	0.017	32	50
7.	273	0.034	46	50	45.	251	0.030	43	50
8.	482	0.049	56	100	46.	170	0.048	56	100
9.	184	0.083	75	100	47.	469	0.034	46	50
10.	360	0.025	39	50	48.	63	0.012	26	50
11.	606	0.013	28	50	49.	370	0.080	74	100
12.	190	0.102	84	100	50.	277	0.074	70	100
13.	230	0.184	15	50	51.	140	0.090	78	100
14.	473	0.078	73	100	52.	102	0.121	92	100
15.	199	0.082	75	100	53.	100	0.068	67	100
16.	224	0.062	64	100	54.	316	0.046	55	100
17.	190	0.071	69	100	55.	653	0.013	27	50
18.	390	0.084	75	100	56.	100	0.074	70	100
19.	800	0.091	79	100	57.	110	0.083	75	100
20.	242	0.043	53	100	58.	580	0.077	72	100
21.	250	0.029	43	50	59.	184	0.067	66	100
22.	892	0.066	66	100	60.	100	0.088	77	100
23.	240	0.085	76	100	61.	272	0.009	23	50
24.	351	0.094	80	100	62.	923	0.109	87	100
25.	100	0.017	32	50	63.	336	0.028	42	50
26.	1,041	0.080	74	100	64.	111	0.093	80	100
27.	64	0.019	34	50	65.	77	0.090	78	100
28.	462	0.074	70	100	66.	98	0.012	26	50
29.	226	0.095	81	100	67.	73	0.018	33	50
30.	202	0.039	50	300	68.	233	0.015	30	50
31.	165	0.126	94	300	69.	80	0.020	35	50
32.	400	0.101	83	450	70.	498	0.014	29	50
33.	88	0.037	48	300	71.	90	0.013	28	50
34.	188	0.069	68	450	72.	90	0.013	28	50
35.	850	0.084	75	300	73.	64	0.015	30	50
36.	530	0.076	72	450	74.	95	0.014	29	50
37.	150	0.020	35	300	75.	49	0.028	42	50
38.	458	0.018	33						