ASSESSMENT OF THE UTILISATION OF OSSIOMO RIVER FOR WATER SUPPLY TO ABUDU TOWN, EDO STATE, NIGERIA

BY

OHILEBO, AIRAODION GEORGE.
(CVE./OO/7340)

THESIS SUBMITED TO THE SCHOOL OF POST-GRADUATE STUDENTS IN PARTIAL FULFILLMENT OF THE AWARD OF MASTER OF ENGINEERING (M.ENG.) IN CIVIL ENGINEERING (WATER RESOURCES AND ENVIRONMENTAL ENGINEERING OPTION)

DEPARTMENT OF CIVIL ENGINEERING
SCHOOL OF ENGINEERING AND ENGINEERING TECHNOLOGY
FEDERAL UNIVERSITY OF TECHNOLOGY
AKURE – ONDO STATE
NIGERIA.

AUGUST 2005

CERTIFICATION

This is to certify that this research work was carried out by Ohilebo, Airaodion, George, of the Department of Civil Engineering, School of Engineering and Engineering Technology, Federal University of Technology, Akure, Ondo State in partial fulfillment of the requirement for the award of Master of Engineering (M.Eng.) in Civil Engineering (Water Resources and Environmental Engineering Option). It has not been submitted elsewhere for the award of any Degree or Diploma.

AM Oguntuase 19/10/05

Dr. A.M. Oguntuase

Supervisor

FED. UNIV. OF TECH, AKURE * LIBRARY *

DEDICATION

This project is dedicated to the glory of God almighty (*Omnipotent, Omnipresent and Omniscient*), whose guidance, inspiration and strength made it possible for me to complete successfully, this Master Degree in Civil Engineering (Water Resources and Environmental Engineering option).

ACKNOWLEDGMENT

"Be therefore, thankful for the least gift and you shall be ready to receive greater".

With the above divine admonition, I wish to use this medium to express my sincere thanks and profound gratitude to all those who were kind enough to make this project work a success, by their various contributions in one way or the other.

I will not fail to express my gratitude and acknowledge the inestimable contributions of the people whose names appear hereunder:

(i) My project supervisor Dr. A.M, Oguntuase, whose fatherly advice and encouragement helped to make this project a success. He guided me in all stages of the work, and further assisted me with materials that I most needed for the completion of the work.

(ii) Dr. J.O, Babatola, my head of department, for his immense co-operation and encouragement. His frank discussions helped me in the choice of this project topic.

(iii) Dr. A.O. Owolabi, whose good and amiable nature, encouraged me throughout the programme.

(iv) Other members of staff in Civil Engineering Dept. who assisted me in many ways to keep pace with my work throughout the programme.

Finally, I want to put on record, the enduring love and moral support from my family, who helped me immensely to complete successfully this piece of work, and the entire Masters Degree programme. To this end , I express my profound gratitude to my wife, Lawrentta, fondly called "Lori" by me; also to my children – Kings, Victory, Royal and Goodness, especially in the computer typesetting of this project. May God richly bless every one. Amen.

ABSTRACT

This project focuses on the utilisation of Ossiomo River for water supply to Abudu town in Orhionmwon Local Government Area of Edo State. To achieve this goal, hydrological appraisals were carried out to determine the quantity and quality of the water available, as well as the adequacy and dependability of the supply. The data collected and analysed include the hydrological data of the river, rainfall data of the study area, and other information such as the census data of Abudu town and maps. Following a reconnaissance survey of the study area, a transverse survey was conducted to determine spot elevations along the proposed pipeline routes and the service reservoir location.

Base on the data analysis, a share intake system, treatment plant components and distribution network, were designed. Cost estimates were also computed.

The study shows that Ossiomo River is a perennial river with the capacity to sustain water supply scheme for Abudu town and environs. The minimum annual discharge of the river is $10.64m^3/s$, which is considered as the safe yield for the design of the scheme. With a total water requirement of $0.138m^3/s$ to the town, the scheme users of the river can still have sufficient water. In terms of the financial implications of the project, a modest total cost of N240,000,000, has been proposed. This is quite feasible and within the executive scope of local and state governments, and with possible assistance from the federal government.

TABLE OF CONTENTS

Tables

Figures

CHAPTER ONE

INTRODUCTION

1.1 General.

The importance of adequate water supplies for the health and welfare of the inhabitants of human settlements is generally recognised. What is lacking is the recognition that water is one of the location factors in urban and rural planning, a particular relevant factor in the developing countries, where scarcity of capital does not permit transportation of water from distant sources.

The daily water demand of a human being varies between 1.5 and 20L depending on climate and physical activity. Daily per capita in-house water uses average about 60 to 80 litres in rural areas supplied by public wells and about 100 to 120 litres in residential districts of urban areas supplied by house connections. (United Nations 1986).

Several attempts have been made by the Federal , State and Local Governments in Nigeria to provide potable water supply in urban, semi-urban and rural towns. However, despite several interventions including the World Bank Financed National Water Rehabilitation Project in urban areas, the FGN/UNICEF Joint Programme on Rural Water Supply and Sanitation, Petroleum Trust Fund (PTF), Directorate of Food, Roads, and Rural Infrastructures (DFRRI), the National Bore Hole Programme and other External Support Agencies (ESAs), Non Govemmental Organisation (NGOs) and Private Interventions, national Coverage of Water Supply is still below 50% (Diyoke, 2001).

All over the world, a safe reliable water supply is vital to the production of food and fibres, to the health and well being of communities (who need water for consumption and sanitation), to the production of industrial goods and services, and to the production of energy in the form of hydroelectric power. In fact, water is the most important common element in environmental and development issues today.

1.2 Objective and Scope of the Project

The main objectives of this project are to:

(i) Evaluate the existing water supply scheme as well as appraisal of present and future waters demand for Abudu town.

(ii) Determine the water quality and quality, and the reliability of Ossiomo river as a source of water supply.

(iii) Design a suitable water scheme for Abudu town.

The scope of this work has been limited to the quantity and quality assessment of a river, and its use for water supply to a community. Comprehensive feasibility studies leading to full Engineering project work has not been attempted because of time and economic constraints.

1.3 Project Justification

The fact that the national coverage of water supply in Nigeria is below 50%, coupled with the policy thrust of the international communities to make potable water available in developing nations of the world, (Oyegun, 1986), the Abudu water supply scheme will obviously boost national coverage of water supply. Other grounds that have been considered are:

(i) The community has been without potable water since the only water borehole (sunk by the then Bendel State Government) collapsed about 15 years ago.

(ii) Abudu is not only a growing semi-urban town, but it is also the administrative Headquarters of Orhionmwon Local Government Area of Edo State.

(iii) The town also accommodates the permanent orientation camp for the National Youth Service Corps in Edo State.

(iv) As a developing semi-urban town, it houses a lot of public and private establishments among which are schools, banks and hospitals. A leprosarium hospital is also located in this ancient town.

(v) The level of social, economic and political activities are on the increase because of its nearness to the state capital, Benin City.

About 80% of all diseases and one third of all death in developing nation are caused by contaminated water. (Mara, 1974). Lack of potable water has created problem to the inhabitants of Abudu town. The water consumed by Abudu residents is contaminated. The roofs from which rainwater is collected are dirty, especially when collecting the first rains of the year. The roofs are dirty during the dry season when no rainfall is experienced, and are not washed before collecting the water.

The dug- wells (underground cement lined water storage reservoir) used to conserve harvested rainwater and also for the procurement of water from water tanker vehicles are not washed often. Some stay for more than three years without being washed, and the water stored in are used for domestic purposes. Living things such as lizards, rats, insects etc., fall into the reservoir and consequently contaminate thus making it unfit for consumption.

Occasionally there is no water in those reservoirs and water from tankers are stored for community use without treatment. The community undoubtedly is faced with acute water problem. The greatest problem is to get potable water because the few existing sources of water are raw and contaminated.

CHAPTER TWO

LITERATURE REVIEW

2.1 General

Whatever sources of water is utilised – lakes, rivers, springs or wells – the available supply depends on the nature and size of the catchments. For example, the water discharge by a river is that part of the rainfall, which, having been precipitated on the catchments, is not completely lost by evaporation or percolation into the ground, and it, includes that part of the water that has percolated into the ground, and is returned to the surface by springs. In assessing the value of a source of supply, therefore rainfall records should be compiled, and the area of the catchments should be determined as accurately as possible, so as to estimate the probable quantity of water available or where the flow can be gauged. The gauging should be taken over a period of several years.

In estimating the supply which, river can afford all times, the flow of the river in the driest years should be known. However, if this information is not available, the discharge during a normal year should be determined and reduced according to the relation between the rainfall of that year and that of the average of the three driest consecutive years on record. When investigating potential water supplies it is advisable to inspect rainfall records of as long as possible, up to 35 years. (Escritt, 1978)

2.2 Evaluation of Surface Water Resource

Whatever the method or approach taken to water resources development and management, it is necessary to carry out an inventory of available water resources – sources, existing uses and characteristics of the water, especially quantity, quality and its spatial and temporal distribution patterns. The hydrologic appraisal of water resources is a fundamental requirement for the planning, design, construction and operation of water resource projects. Such an appraisal involves the collection and analysis of hydrologic data and information as well as data on social-economic conditions that impinge on water utilisation. The objective of such an appraisal is to determine the source, extent, adequacy and dependability of supply and other characteristics of water, which will form the basis of policy regarding control and utilisation of water. (Ayoade, 1988.)

The evaluation of water resources in a given area is best done within the framework of the hydrologic or water balance equation usually expressed as follows:

$$P = Q + E \pm \Delta S \pm \Delta G \qquad (2.1)$$

where P = Precipitation,

Q = Runoff,

E = Evapotranspiration,

ΔS = Change in surface water storage,

ΔG = Change in ground water storage.

The above equation attempts to show the various ways by which the precipitation incident in a given area is disposed of. Precipitation represents the gross water resources available in a given area. These resources are, however, subject to natural losses through processes of evapotranspiration. It is the amount of rainfall less evaporation that really constitutes the effective water resources of a given area.

Water resources evaluation is done with the primary objective of meeting the demand for water resources and considers both the quantity and quality of available water resources and the possibility of their development within existing technological and socio-economic systems. Evaluation of water resources is therefore not complete without consideration of the demands for water, which are controlled by socio-economic factors. The major abstractive uses of water are for irrigation and water supply for domestic and industrial uses. The water requirements for these purposes now and in the future must be known in order to facilitate rational planning and optimum utilisation of the available water resources.

2.3 Development of Water Supply

The most important thing under water supply schemes is the selection of source of water, which should be reliable and have minimum number of impurities. After the selection of source of water, the next step is to construct intake works to collect and convey it to the treatment plant, where it is treated. The type of treatment processes directly depends on the impurities in water at the source and the quantity of water required by the consumers.

When the water is treated, it is stored in clear water reservoir, from where it is distributed to the consumers. The distribution system depends on the elevation of clear water reservoir and the topography of distribution area. Water directly flows under gravity to the low-level areas from the elevated tanks. Pumping is required for flows to the high level grounds. The flow diagrams of water supply systems and processes are shown in Fig 2.1

A water supply system capable of supplying a sufficient quantity of potable water is a necessity for an urban town. The elements (Fig 2.2 and Table 2.2) that make up a modern water supply system include:

(i) Source(s) of supply

(ii) Storage facilities

(iii) Transmission (to treatment) facilities

(iv) Treatment facilities

(v) Transmission (from treatment) and immediate storage facilities.

(vi) Distribution facilities.

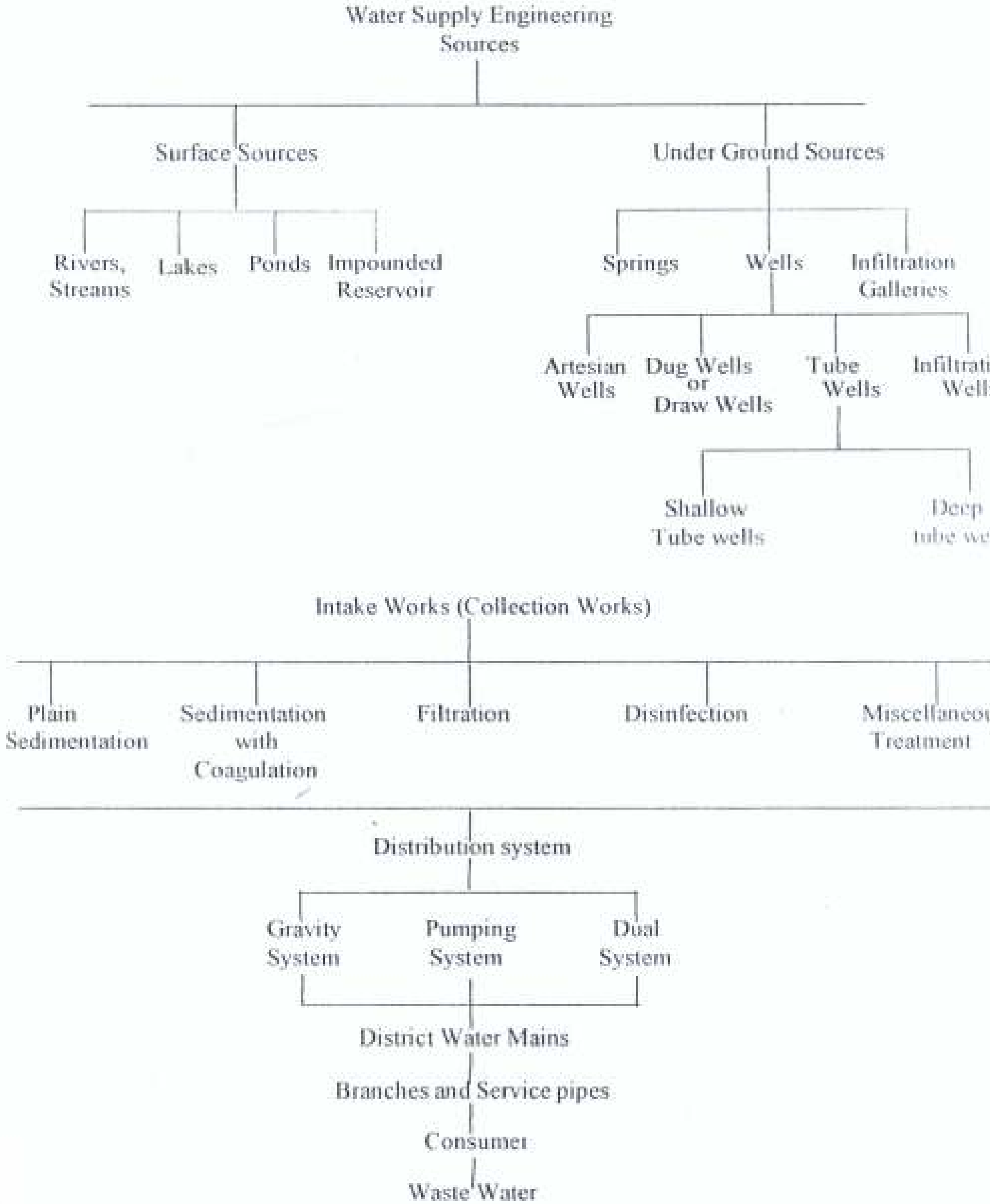


Fig 2.1 Flow Diagram of Water Supply Scheme

(Source: Birdie and Birdie, 1992)

In the development of public water supply, the quantity and quality of the water are of paramount importance. The relationship of these two factors to each of the functional elements is indicated in Table 2.1. As shown in Fig 2.2, not every functional element will be incorporated in each water supply system. For example, in systems where ground water is the source of water supply, storage and transmission facilities usually are not required. In some instances, treatment facilities may not be necessary.

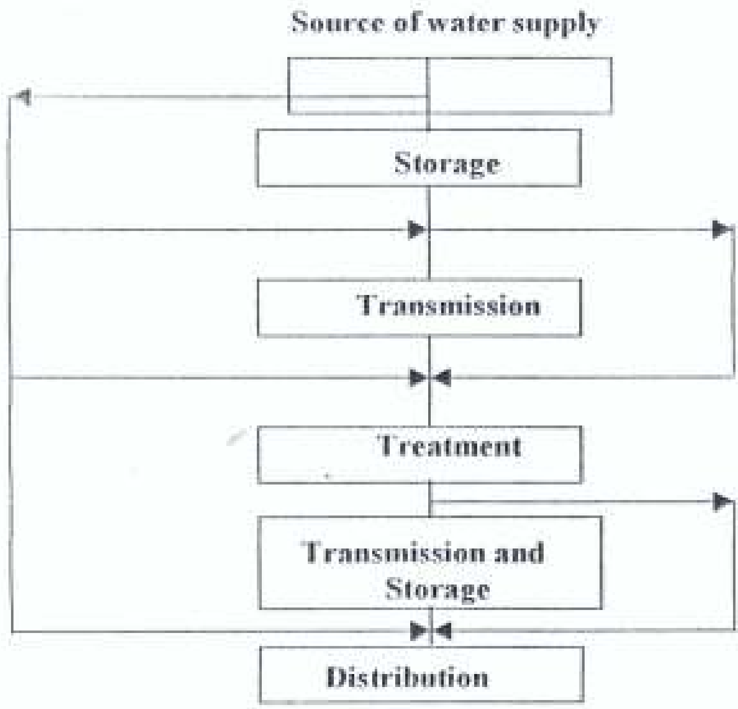


Fig 2-2 Interrelationship of the Functional Elements of a Municipal Water Supply System

(Source: Linsley and Franzini, 1979)

Table 2.1 The Functional Elements of Public Water Supply System

Functional Element	Principal Concerns in Facilities Design [Primary/Secondary]	Description
Source(s) of supply	Quantity/Quality	Surface water sources of supply such as rivers, lakes and reservoir or ground water source
Storage	Quantity/Quality	Facilities used for the storage of surface water usually located at or near the source of supply
Transmission	Quantity/Quality	Facilities used to transport water from storage to treatment facilities
Treatment	Quantity/Quality	Facilities used to improve or alter the quality of water.
Transmission and storage	Quality/Quantity	Facilities used to transport treated water to intermediate storage facilities and to one or more points for distribution.
Distribution	Quantity/Quality	Facilities used to distribute used to distribute water to the individual users connected to the system.

(Source: Linsley and Franzini,1979)

2.4 Water Uses

Water uses varies from one area to the other, depending on the climate, characteristics of the area concerned, population, industrialisation, and other factors. In a given area, water use also varies from season to season, day to day, and hour to hour. Thus, in planning of a water supply system the probable water use and its variations must be estimated as accurately as possible.

Domestic water demand includes the quantity of water required in the houses for drinking bathing, cooking, washing, etc. The quantity of water required for domestic use mainly depends on the habits, social status, climatic conditions and customs of the people. In most developing nations of the world, the domestic consumption of water under normal conditions is about 135 litres/day/capita. In developed countries this figure may be as high as 350 litres/day/capita. (Birdie and Birdie, 1992).

Commercial buildings and commercial centres include office buildings, warehouses, stores, hotels etc. The water requirements of commercial and public places may be up to 45 litres/day/capita. (Birdie and Birdie, 1992).

Quantity of water required for public utility purposes such as for washing and sprinkling of roads, cleaning of sewers, watering of public parks, gardens, public fountains etc. comes under public demand. To meet the water demand for public use, provision of 5% of the total consumption is made while designing the water works for the city.

All the water, which goes to the distribution pipes, does not reach the consumers. Some portion of this is wasted in the pipelines due to defective pipe-joints, cracked and broken pipes, etc. Generally an allowance of 15% of the total quantity of water is made to compensate for losses, thefts and wastage of water.

2.5 Requirements of Water Supply System

A suitable water supply system, according to Birdie and Birdie, (1992) should satisfy the following requirements:

(i) Quality of water supplied should be satisfactory (should be free from impurities and bacteria).

(ii) Water should be available in adequate quantity

(iii) Water should be readily available to the users

(iv) It should be cheap

(vi) It should be readily disposed off after it has served its purpose.

The management and design of water supply requires a knowledge of the quantities of water needed with reference to the desired population to be served. In order that the laid/planned system does not become inadequate due to population and consequent demand of water supply for various requirements, not only the present but also future requirements should be considered while planning water supply works.

The following factors are considered while planning water supply project.

(i) Design Period

The design period is the period for which the proposed system including its component structure and equipment would be adequate. The period should neither be too short or too long. Mostly water works are designed for periods of 22-30 years, which is a fairly good period (Henry and Heinke, 1989).

(ii) Population at the End of the Design Period

When the design period is fixed, the next step is to determine the population at the end of the design period, which will ensure that future water

requirement of the community is calculated and adequately provided for in the overall design works.

(iii) Per Capita Demand

For the purpose of estimation of total requirement of water, the demand is calculated on an average basic, which is expressed in litres/capita/day.

If Q is the total quantity of water required daily for the population of the town, P, the per capita demand will be:

Per Capita Demand = Q/P (litres/capita/day: lpcd) (2.2)

2.6 Yield Estimation for Surface-Water Supply

The quantity of water that may be drawn continuously from a stream or lake depends upon the area of the watershed, the topography, vegetation, rainfall, climate, and amount of storage. The maximum quantity of water that may be drawn continuously after deducting losses due to evaporation from the proposed reservoir surface, leakage through and under dams, and necessary withdrawals for reparian owners downstream is called *safe yield*. The estimated safe yield must exceed the estimated future demand if a proposed water supply is to be adequate. The safe yield of a source of supply can be estimated only from the records of the past. The most reliable information from which to predict yield is a hydrograph of the stream for a long period of years at the location of the proposed intake or dam. The hydrograph should be of sufficient duration to include a period of extreme drought.

To calculate yield, (Tort, *et al*, 1974) it is not generally possible to satisfactorily create a flow record at an intake site from rainfall data; there is no substitution for a careful examination of long reliable series of runoff measurements at the location concern. Self evidently the *gross yield* is the minimum flow on record, while the *net yield* is simply the amount remaining after deductions of any prescribed residual flow from the gross figure. Where it is felt preferable to estimate an intake yield, the following procedures are recommended:

(i) Locate a flow record of adequate length for the river concerned, or for one hydrologically similar.

(ii) Adjust this record back to natural conditions, if necessary.

(iii) Locate the lowest flow each year and rank the resulting series of extreme values from the smallest (rank 1) upwards.

(i) Plot these on an appropriate probability paper which will permit straight line

$$\frac{m}{(n+1)} \times 100\%$$

interpretation, plotting the prints at :-

(v) where n is the number of years of record and m is the rank number.

(vi) From the chosen flow probability line read off the flow for varying risks,

(vii) Transpose these to the intake site being studied, either by correlation or by the ratio of the estimated mean flow of the sites concerned.

(viii) The transformed figures from (vii) are then plotted on a gross yield/return period graph.

2.7 Intakes

The main function of the intake works is to collect the water from the surface source (within limitations of the water level) and then discharge water so collected, by means of pumps or directly to the treatment plant. Intakes are structures, which essentially consist of openings, grating, or strainer through which the raw water from river or reservoir enters, and is carried to a sump well by conduits. Water from the sump well is pumped through the rising mains to the treatment plant (Willcock, 1983).

A variety of intake systems have been successfully employed. Broad classifications of the principal types are intake towers, shore intakes, submerged intakes, siphon - well intakes. Many variations of these principal types have been used.

Intake towers are commonly employed in reservoirs and are usually located in the deepest part of a lake or reservoir. Towers have also been employed as part of a river intake system. The intake tower is located in the river channel, and water is conducted from it by gravity pipes or tunnel to the intake pumping station, located on the shore.

A shore intake system is frequently employed for large installation on rivers. It usually consists of a large concrete structure equipped with grated ports protected by coarse bar racks. Mechanical screens are usually placed in a well behind the ports. Such intake structure generally includes space for raw-water pumping equipment.

Submerged intakes may be constructed as "cribs" surrounding an upturned, bell-mouth inlet connected to an intake conduit. The intake conduit conveys water to the shore shaft, which serves as a pumping suction well.

Siphon-well intakes are employed in rivers and may consist of a shore structure, which receives water from the river via a siphon pipe. See fig 6.1 at the appendix. The siphon pipe inlet may be submerged crib equipped with a trash rack or simply a screen section attached to the open pipe. Siphon-well intakes have a record of satisfactory service and are generally less costly than other types of shore intakes. (American Society of Civil Engineers and American water works Association, 1990)

While selecting sites for intake works, the following points should be taken into consideration:

(i) Intake should be so located that the best available quality of water is taken. This makes purification economical.

(ii) Existence of heavy current at the intake site should be avoided. This type of current endangers the safety of the intake structure.

(iii) The required quantity of water should be available under the least discharge conditions.

(iv) The site should be easily accessible

(v) It is desirable to have an intake from which additional water may be available for future requirements.

(vi) The site should not be located in a navigational channel as this type of water is polluted and contaminated.(Birdie and Birdie,1992)

The following factors should be consider while designing an intake:

(i) The intake work should be capable of resisting external forces caused by heavy waves and currents, impact of floating and submerged bodies and ice pressure.

(ii) The intake work should have sufficient weight to prevent it floating by the up thrust of water and getting washed away of the current. This objective is achieved by providing massive mansory intake structure and providing stone pitching at the bottom.

(iii) If intake work is constructed in a navigational channel, it should be protected from the blows of moving ships and steamers by group of piles all around.

(iv) The foundation of intake should be taken sufficiently deep so that they may not be undermined and current does not overturn the structure.

(v) The inlets should have a screen to prevent large objects and fishes getting into the drawn water.

(vi) The inlet should be large enough to allow the designed quantity of water to be drawn.

(vii) The position of inlets should be such that they can admit water in all seasons from near the surface of water, where quality of water is comparatively better. Also the number of inlets should be more, so that if one is blocked, water can be drawn from others. To cater for variation in the water level, sets of inlets should be located at different levels.(Birdie and Birdie,1992)

Intakes in large rivers should be so located that the parts are submerged at all stages of the river to a sufficient depth to avoid troubles with possible ice cakes or floating debris and to preclude the entraining of air. The ports should also be several centimeters above the bottom of the stream so that sand and gravel being transported on the bottom will not be drawn into the intake.

In order to meet these requirements, it is often necessary to locate the intake in the deepest part of the stream and away from the shore, particularly if the river is subject to large fluctuation in stage. Under these conditions, the water must be conveyed from the

Intake to the shore through a pipe laid in the river bottom or through a funnel constructed under the bottom. Pipes laid in or on the river bottom should be securely anchored.

If the river stage does not fluctuate too much, it is sometimes possible to construct the intake on the shore of the river (shore intake). The necessity for a tunnel or subaqueos pipeline is thus avoided, and the design of the intake structure is simplified. (Wilcock, 1983).

2.8 Water Quality

The utility of water is limited by its quality, which may make it unsuitable for particular uses. Assessment of water quality is therefore an important aspect of water resources evaluation. As part of the water resources survey, available data on water quality should be collected and studied carefully. If no data on water quality is available, water samples should be collected and analysed.

In this modern rapidly changing world, the water engineer must therefore be concerned with water quality in addition to the traditional recognised evaluation of water.

The principal features of water quality in the streams, rivers and lakes with which the water Engineer is most concerned may be considered in three main groups – physical, chemical and biological.

Water-quality standards are published by various agencies and are influx. The U.S. Public Service (USPHS), The World Health Organisation (WHO) and the Environmental Protection Agency (EPA) are all organisations that have published drinking – water standards for both raw and finished potable water. In general, these standards are similar and have changed as knowledge of the health effects of various water-quality parameters has increased. (Wanielista, *et. al.*, 1984)

The International Standards were first published by the World Health Organisation (WHO) in 1958; a second edition was published in 1963. The current edition was issued in 1984. These standards are intended to supersede both the European standards for drinking water and the International Standard for drinking water. The WHO standard has prescribed the quality expected of potable water as in Table 2.2.

Records of people's desire to improve water quality date back to as early as 2000 BC. Since that time, advances in water treatment have continuously been made. Three basic objectives of water treatment according to Steel and McGhee,(1979) are:

(i) Production of water that is safe for human consumption

(ii) Proving aesthetically acceptable water for human consumption

(iii) Providing water for human consumption economically.

The production of biologically and chemically safe water is the primary goal in the design of water treatment plants. Anything less is unacceptable. A properly designed plant is not a guarantee of safety. However, skilful and alert plant operation and attention to the sanitary requirements of the source of supply and the distribution system are equally important. The second basic objective of water treatment is the production of water that is appealing to the consumer. Ideally, appealing water is one that is clear and colourless, pleasant to the taste, odourless, and cool. It is non-staining, neither corrosive nor scale forming, and reasonably soft.

Table 2.2 **Chemical Substances affecting Portability of Water**

(after, WHO 1984)

Substance	Maximum acceptable concentration	Maximum allowable concentration
Total solids	580 mg/l	500 mg/l
Colour	5 units	50 units
Turbidity	5 units	25 units
Taste	Unobjectionable	
Odour	Unobjectionable	
Iron (Fe)	0.3 mg/l	1.0 mg/l
Manganese (Mn)	0.1 mg/l	0.5 mg/l
Copper (Cu)	1.0 mg/l	1.5 mg/l
Zinc (Zn)	5.0 mg/l	15.0 mgl
Calcium (Ca)	75 mg/l	200 mg/l
Magnesium (Mg)	50 mg/l	150 mg/l
Sulphate (SO4)	200 mg/l	400 mg/l
Chloride (CL)	200 mg/l	600 mg/l
Fluoride (F)	0.5 mg/l	1.5 mg/l
Nitrate (NO_3)	50 mg/l	100 mg/l
Ca Co_3	-	120 mg/l
pH	7.0 – 8.5	6.5 – 9.2
Magnesium + Sodium Sulphate	500. mg/l	100 mg/
Phenolic substances	0.001 mg/l	0.002 mg/l
Carbon chloroform extract	0.2 mg/l	0.5 mg/l
Alky-benzy sulfonates	0.5 mg/l	1.0 mg/l
Lead	-	0.05 mg/l
Arsenic	-	0.05 mg/l
Selenium	-	0.01 mg/l
Chromium	-	0.05 mg/l
Cyanide	-	0.02 mg/l
Cadmium	-	0.01 mg/l
Barium	-	1.0 mg/l

(Source: Porta,1998)

The consumer is principally interested in the quality of water delivered to the tap, not the quality at the treatment plant.. Therefore, water utility operation should be such that quality is not impaired as water flows from the treatment plant through the distribution system to the consumer. Storage and distribution systems should be designed and operated to prevent biological growths, corrosion, and contamination by cross-connections. In design and operation of both treatment plant and distribution system, the control point for the determination of water quality should be the consumer's tap.

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. Alternative development should be based upon sound engineering principles and under consideration of flexibility for changing future conditions, emergency situations, operating personnel capabilities, and future expansion.

One of the great achievements of modern technology has been to drastically reduce the incidence of water-borne diseased such as cholera and typhoid fever. These diseases are not longer the great risks to public health that they once were. The key to this advance was the recognition that the contamination of public water supplies by human wastes were the main source of infection, and that it could be eliminated by more effective water treatment and better wastes disposal. Filtration of drinking water was used as early as 1802 in Paisley, Scotland, and water renders in London, England in 1828. (Wanielista, *et. al.*, 1984)

Today's water treatment plants are designed to provide water continuously that meets drinking water standard at the tap. There are four main consideration involved in accomplishing this: source selection, protection of water quality, treatment methods to be used, and prevention of re-contamination.

The actual selection of treatment processes depends on the type of water source and the desired water quality. Fig 2.3 shows a schematic outline of a typical surface water treatment plant.

Occasionally, raw water with low turbidity can be treated by plain sedimentation (no chemicals) to remove larger particles and then filtration to remove the few particles that failed to settle out. Usually, however, particles in the raw water are too small to be removed in a reasonably short time through sedimentation and simple filtration alone.

Following filtration and before it flows into the storage reservoir, the water is disinfected, usually with chlorine. Fluoride may also be added because of its ability to retard tooth decay. Treated water is then pumped by the water high-lift pumps into the distribution system to serve customers and to maintain water levels in storage reservoirs if required. The treatment units usually adopted in water treatment are given in Table 2.3, while the guidelines for the selection of pumps is in Table 2.4. The economic velocities in individual suction and delivery pipe work for water supply are as given in table 2.5.

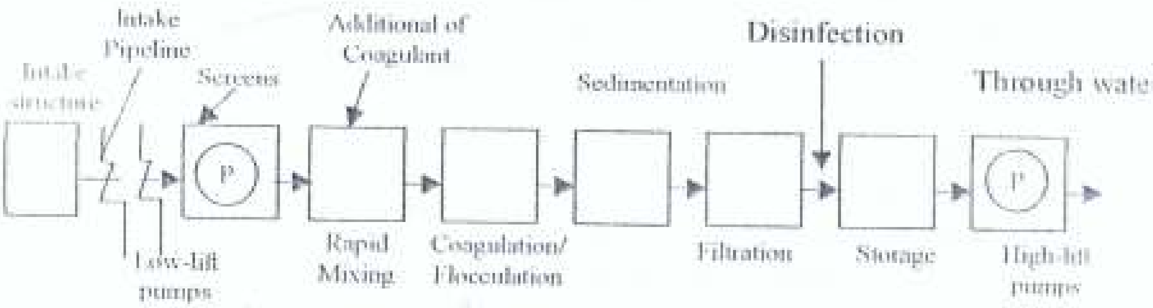


Fig. 2.3 Schematic of Water Treatment Plant using Surface Water Source

(Source: Henry and Heinke,1989)

Table 2.3 TREATMENT UNIT AND APPLICATION

Unit or Processes	What it is used for
1 Screen	Removal of floating matter
2 Plain	Removal of fine discrete particles, which are allowed to settle under gravity and then be removed.
3 Coagulation and Flocculation	Addition of some chemicals to coagulate the finer and lighter flocculent materials, which cannot settle under Process (2). By allowing them to form a larger mass (Flocs) they can now settle.
4 Secondary Sedimentation	This is the settling tank with short detention time where the flocculent materials that have come together in process (3) settle down and are then removed.
5 Filtration	This is employed to remove very fine particles and colloidal matters, which might have escaped from the sedimentation tank. Some microorganisms are also removed by filtration. The filter is constructed of porous materials such as zoned graded sand and gravel.
6. Disinfection	The effluent from the filter may still contain some bacteria. For this reason Disinfection is necessary. Disinfection may be achieved by chlorination, Ozonation or sonozone process.
7. Aeration	Water, which has been treated by sedimentation, Filtration and Disinfection will be safe, but not necessarily attractive. Aeration unite removes odour and colour (iron and maganese) by mixing and exposing water to air. Aeration can be created through hydraulic jump.
8. Chemical Precipitation	Removes such chemicals in water like iron, maganese and hardness.
9. Ion exchange (e.g. Zeolite bed).	By exchange of ion, this process also achieve the removal of hardness.

(Source: Agunwamba , 2002)

Table 2.4 Guideline for the Selection of Pumps

$Q\ m^3/s$	No of pumps working	No of pumps on stand-by	Total pumps required
0.032	1	1	2
0.032-0.095	2	1	3
0.095-0.189	3	1	4
0.189 and above	4	2	6

(Source: Wogu, 1997)

Table 2.5. Economic Diameter of Pumping Station Pipe-Work and Valves

Discharge of pump (m^3/min)	Economic diameter of suction and delivery pipe-work and valves connected to that pump (mm)	Economic velocity (m/min)S
0.4 m^3/min	100 mm	49.4 m/min
0.7	125	53.1
1.1	150	58.6
1.5	175	62.2
2.2	200	67.7
3.0	225	73.2
3.9	250	76.9
6.0	300	84.2
10.5	375	91.5
16.6	450	102.5
24.4	525	109.8
34.0	600	117.1
45.7	675	124.4
59.5	750	133.6
75.5	825	139.1
93.9	900	144.6

2.9 Hydrologic Measurement

Hydrologic measurements are made to obtain data on hydrologic processes. These data are used to better understand these processes and as a direct input into hydrologic simulation models for design, analysis, and decision making.

Hydrologic processes vary in space and time, and are random, or probabilistic, in character. Precipitation is the driving force of the land phase of the hydrologic cycle, and the random nature of precipitation means that prediction of the resulting hydrologic processes (e.g., surface flow, evaporation and stream flow) at same future time is always subject to a degree of uncertainty that is large in comparison to prediction of the future behaviour of soils or building structures. These uncertainties create a requirement for hydrologic measurement to provide observed data at or near the location of interest so that conclusion can be drawn from on-site observation.

Measurement of channel flow is of particular relevance to the civil engineer because stream flow comprises a major component of output from the drainage basin system. In principle, the measurements are easier to make than those of most other aspects of drainage basin dynamics because the flow is concentrated within the channel and it is possible to isolate the total volume of water discharging from a catchment of known area.

In detail, however, the precise methods of measurement are complex and the associated science of hydrometry (Gregory and Walling, 1979) has developed into a specialised discipline. British standard practices have been developed as detailed specifications for several types of measurement and in several other countries stream ganging manual have appeared. The major stream flow parameters that are measured are those stage or water level, velocity and discharge, and their variation through time.

Depth is measured by several methods. They include graduations on the wading rod, tags on the cable which supports the meter- and-weight assembly, or a mechanical indicator attached to the reel on which the supporting cable is wound. Water surface elevation measurements include both peak levels (flood crest elevations) and stage as a function of time. These measurements can be made manually or automatically.

Continuous records of water level are maintained for the calculation of stream-flow rates. The level of water in a stream at any time is referred to as the gauge height.

Among the different methods that have been developed for the measurement of stream velocities, are mechanical devices, which are rotated or deflected by the current, instruments for the conversion of velocity head into potential head, chemical methods, and others. (Chow, 1964)

Because of the widely varying conditions of stream-flow, ranging from very low to very high and turbulent velocities, extremely rough cross sections, and the like, no single method of velocity measurement is universally applicable. The rotating meter covers the greater range of conditions, but is not suitable for velocities below about 30mm/sec, nor under conditions of extreme turbulence. The methods for velocity measurements includes; Rotating meter (Current Meter), Dynamometer, Float, Chemical and Electrical Methods. (Chow, 1964)

Discharge measurements in natural channels are usually determined by using the following methods :- (i) Current-meter measurements. (ii) Area – velocity method. (iii) Slope – Area Method. (iv) Weir measurements. (Handbook of applied Hydraulics 1979)

2.10. Distribution of Water

After complete production, the water is stored in clear water storage tanks from where it is distributed to the consumers by means of distribution system.

The distribution system consist of pipes of various sizes, valves, pumps, metres, distribution reservoirs, hydrants, stand post etc. A good distribution system according to Singh, 1986, should satisfy the following conditions :

(i) Purity of water should not be affected during conveyance of water to consumers.

(ii) The pressure of water at each point should be ample.

(iii) Required amount of water at each point should reach each point.

(iv) The most important requirement is that in the event of breakdown or repair of one line, water should reach that locality from other lines.

For efficient distribution it is required that water should reach every consumer with required rate of flow. Enough pressure is therefore needed in the pipeline to force the water to reach every point in the system. Depending upon the method of distribution, the distribution system is classified as follows:

(i) gravity system,

(ii) pumping system,

(iii) dual system or combined gravity and pumping system.

However, for the purpose of this study, the applicability of the dual system is discussed.

The dual system is also known as combined gravity and pumping system. The pump is connected to the marine as well as to an elevated reservoir. In the beginning when demand is small, the water is stored in the elevated reservoir, but when demand increases the flow in the distribution system comes from both the pumping station as well as elevated *reservoir*.

In this system, water comes from two sources, one from reservoir and the other, from pumping station, hence it is called a dual system. This system is more reliable and economical, because it requires uniform rate of pumping but meets the low as well as maximum demand. The water stored in elevated reservoir meets the requirements of demand during break down of pump and fire fighting (Davis and Sorensen, 1979).

There are four main systems of water distribution: viz

(i) Dead End or Tree-System, (ii) The Grid Iron System, (iii) Circular or Ring System, (iv) Radial System.

Water pressure in the distribution network range from 130-260 Kpa (20-40psi) in the residential areas with buildings not over four stories in height to 400-500 Kpa (60-75psi) in areas with taller commercial or residential buildings. (Henry and Heinke 1989)

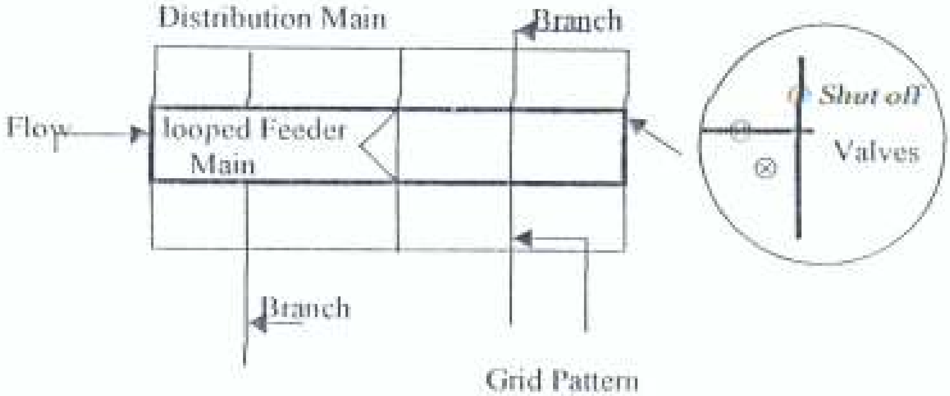


Fig.2.6. Distribution System Configuration

(Source: Henry and Heinke, 1989)

Storage is necessary in any municipal water demand, to provide fire protection, and for emergency needs. Residential water demand varies according to reasonable predictable patterns over the day. High lift pumps at the treatment plants are not normally designed to meet these changes in demand. Instead, the usual practice is to pump water into the distribution system at a fixed rate for a given period and allow a reservoir to supply extra water if demand exceeds this rate or receive water if demand is less than the pumping rate.

Operating storage is used to meet variable water demands while maintaining adequate pressure on the system. Where information on water demand is available, storage volume can be calculated or found graphically (from a mass diagram, for requires that the flow in each pipe be assumed so that the principle of continuity is satisfied at each junction.

A correction to the assumed flow is computed successively for each pipe loop in the net work until the correction is reduced to an acceptable magnitude. If Qa is the assumed flow and Q the flow in a pipe, the correction Δ is $Q-Q_a$,and

$$Q = Qa + \Delta \qquad (2\text{-}11)$$

Expressing head loss by

$$h_L = k\,Q^x \qquad (2\text{-}12)$$

the condition that the head loss around any closed loop be zero, gives

$$\Sigma\, k\,(Q_a + \Delta)^x = 0 \qquad (2\text{-}13)$$

Expanding this summation,

$$\Sigma k Q^x_a + \Sigma x k \Delta Q_a^{x-1} + (x-1)\backslash 2\Sigma x k \Delta^2 Q_a^{x-2} + \text{-----} = 0 \qquad (2\text{-}14)$$

If Δ is small, the third and all succeeding terms of the expansion may be neglected. Hence,

$$\Sigma\, k\, Q_a^x + \Delta \Sigma x k Q_a^{x-1} = 0 \qquad (2\text{-}15)$$

where Δ has been removed from the summation since it is the same for all pipes of the loop. Solving for Δ gives:

$$\Delta = \frac{-\Sigma k Q_a^x}{\Sigma(x k Q_a^{x-1})} \qquad (2\text{-}16)$$

In applying Eq (2-16). The direction of flow as well as the rate of flow in the circuit must be assumed. The numerator of Eq. (2-16) is then the algebraic sum of the head loss in the circuit, with due regard for sign. The correction Δ must be applied in the same sense to each pipe in the loop. If the counter clockwise direction is assumed positive, Δ is added algebraically to flows assumed in the counter clockwise direction and subtracted from flows in the clockwise direction. In Eq (2-12), Δ was given the same sign in all pipes. Because of this, the denominator (Eq.2-16) is taken as the absolute sum without regard to the signs of the individual items in the summation.

Since the hardy cross method assumes a constant value of x, usually given as 1.85 or 2. Minor losses are generally neglected, but they can be introduced by substituting an equivalent length of pipe. (Linsely, and Franzini,1979)

The system must be divided into two or more loops, such that each pipe in the network is included in at least one loop. Values of Δ are computed for each loop, and the assumed flows are made until the desired accuracy is attained.

In the design of a pipe network care should be taken to ensure that proper flow rates are delivered where needed at adequate pressure. For most distribution system a static pressure of 400 to 500KN/m^2 is recommended at all points of delivery. Nowhere in the system should pressure be permitted to drop to the point of where they approach cavitation.

There are three requirements for a pipeline, namely:

(i) It must convey the quantity of water required;

(ii) It must resist all external and internal forces coming upon it;

(iii) It must be durable and have a long life.

In order to deal with this subject in an orderly fashion it is necessary to classify pipelines into three categories of usage, which may be defined as follows:

'Mains'-pipelines for the conveyance of water over long or short distance and for the distribution of waters through towns, and in general, any large-sized pipeline.

'Services pipe'-the individual supply line to a house farm, or block of flats, laid underground from a main to the building.

'Plumbing pipes'- pipework within a building for the distribution of water to the various appliances.(Twort,*et. al.*,1974)

Most water engineers will have to choose the size of main to be adopted in the circumstances, some commonly held principles may be quoted (as a rough guide) as given in Table 2.6

Table 2.6 Size of Distribution Mains

Diameter of main	Uses in a distribution system
75mm	Suitable only for supply to self-contained housing site of small or moderate size from which no further mains extensions are possible, e.g. very suitable for culs-de-sac, not suitable for throughways'.
100mm	A favourite distribution pipe size, which will normally permit of some moderate further extension under good pressure. Where plenty of cross connections at road junctions are made. Very suitable for completely built up housing areas, also suitable for moderate lengths of rural work where no large factories or housing estates are expected to grow up, provided that a ring main can be laid instead of a spur extension. Not suitable at all for bulk conveyance of water to an area were there is any likelihood of growth.
150mm	Suitable for laying along a road along which housing estate are developing, provided some limit of their development can be seen. Suitable for quite long lengths of rural areas particularly if connected ring main to the source or from a trunk main. Suitable for bulk conveyance of water to an isolated area provided it is of moderate size and unlikely to develop. Suitable for reinforcing supplies to an existing area of over-draws in a built-up area. Suitable for normal industrial demands. Not suitable for large industrial demands, fast or large developing areas, long distance conveyance of water.
225mm (Also 200 and 250mm)	Normally classed as a trunk main. Its particular use in the distribution system is that of a 'reinforcement' main, a kind of leading artery in the distribution system taking a good flow and pressure right into the heart of distribution areas, or leading through such areas as a ring main feeder. Suitable for long lengths of rural work. Suitable for bulk conveyance of water to separate areas. Should normally be suitable for industrial estate and factory areas, except where individual very large consumers exist.
300mm and over	These are regarded as trunk mains.

(Source: Twort ,*et al.* 1974)

CHAPTER THREE

THE STUDY AREA

3.1 General

Abudu town is the administrative headquarters of Orhionmwon Local Government Area of Edo State. The Local Government Area (LGA) was created in 1976 and even after the present Uhumwode LGA, was created out of the old Orhionmwonn LGA in 1991, Abudu town still remains the administrative headquarter of the new Orhionmwon local government area. Other towns are, Urhionigbe, Ugo Niyekeorhionmwon, Igbanke, Evboesi, Evbuehighae, Orogho, Umoghunmun –N'okua, Evbobemwen, Ugboko –N' Umagbae, Ugboko – N' Iro, Oza – Aibiokunla, Sokponba, Evbuobanosa, Ebuokabua and Ogan.

Orhionmwon LGA is situated in the dense forest of the Ancient Benin kingdom in Edo State. The LGA has boundaries with Igueben LGA, Uhunmwode LGA, and Ikpoba – Okha LGA, all in Edo State. On another side, it has boundaries with Ika South LGA, Ethiope East and West LGAs, and Ukuani LGA, all in Delta State. (See fig. 3.1) Orhionmwon LGA has twelve political wards namely, Aibiokular I and II (Abudu town), Iyoba, Ukpato, Igbanke East and West, Urhonigbe North and South, Ugu, Evboesi, Ugboko and Ugbeka.

Culturally, the people of Abudu and the entire local government area, are Edo speaking. Cultural values have portrayed their inherited traditional respect for monarchical institution. Their main staple foods are Cassava, Yam and Plantain. The common tree crops in the area are Rubber and Palm trees.

3.2 Location

Abudu town is located between latitude 6^0 41'E -6^0 49'E and longitude 4^0 32'E-4^0 36'E in the eastern part of Edo. Since Abudu town is located along the Benin – Asaba Express highway and it is 45km. from Benin City – the capital city of Edo State.-(Fig 3.2)

3.3 Level of Infra-Structural Development

The town is connected to the National Electricity Grid. It has a medium level public and private establishments: schools, banks, hospitals and good earth roads. All these have impacted positively on the socio – physical landscape of Abudu town. With the birth of the Edo State National Youth Service Corps (NYSC) permanent orientation camp in Abudu, the town has witnessed noticeable increase in the tempo of urbanisation and agglomeration processes.

Natural sources of water in the area include surface water, ground water and rain water. The area has no surface water supply scheme, and the only groundwater scheme (borehole), developed by the Edo State Government in 1985 has since collapsed and out of use for the past 15 years. The teaming population depends largely on rainwater harvesting from rooftops and untreated river water from Ossiomo river. However, some few wealthy individuals obtain their water supply from private boreholes, which they also commercialise. Discussions with the owners indicate the depths of boreholes vary from 30m to 60m.

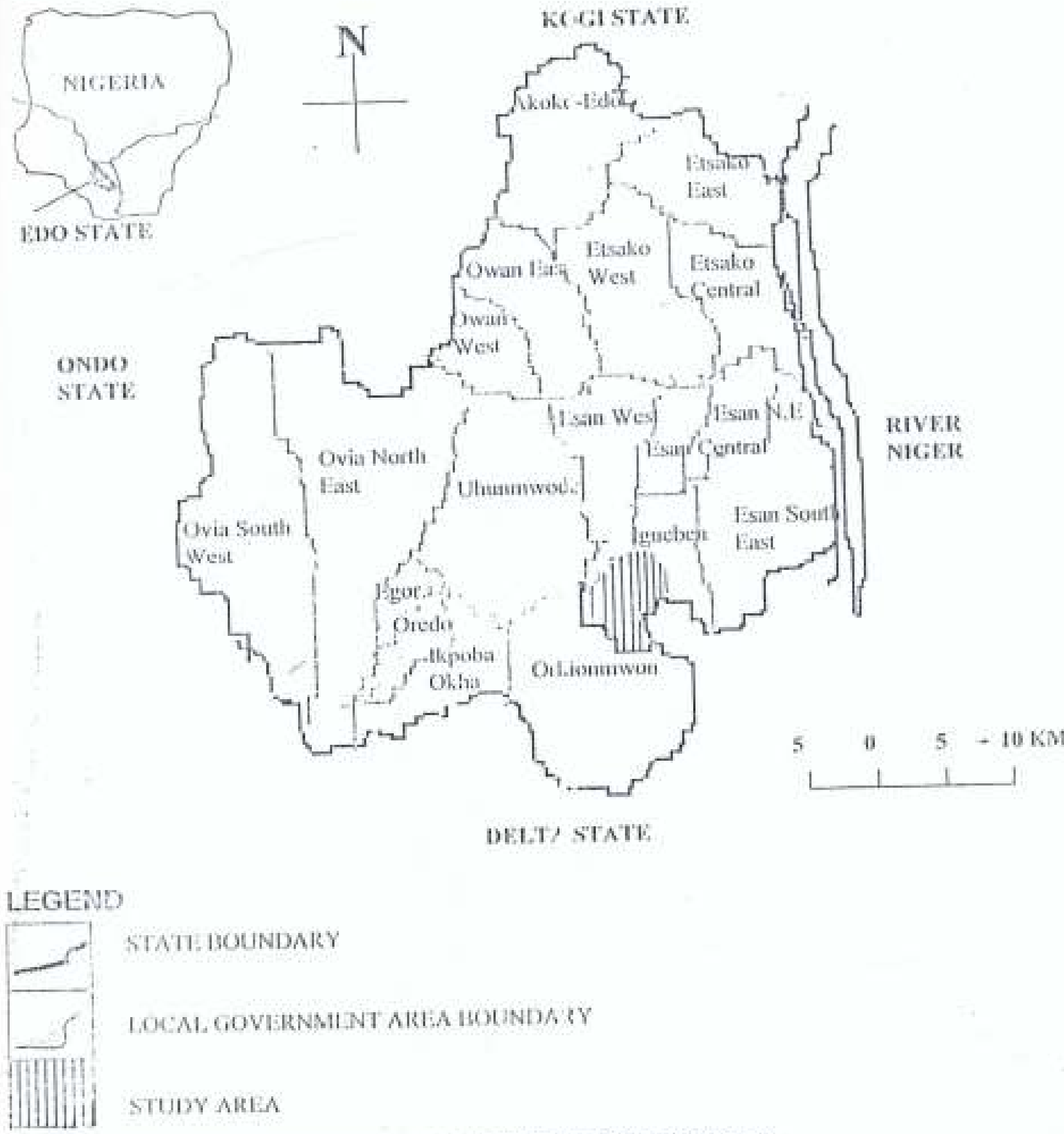


FIG. 3.1 EDO STATE SHOWING ABUDU THE STUDY AREA.

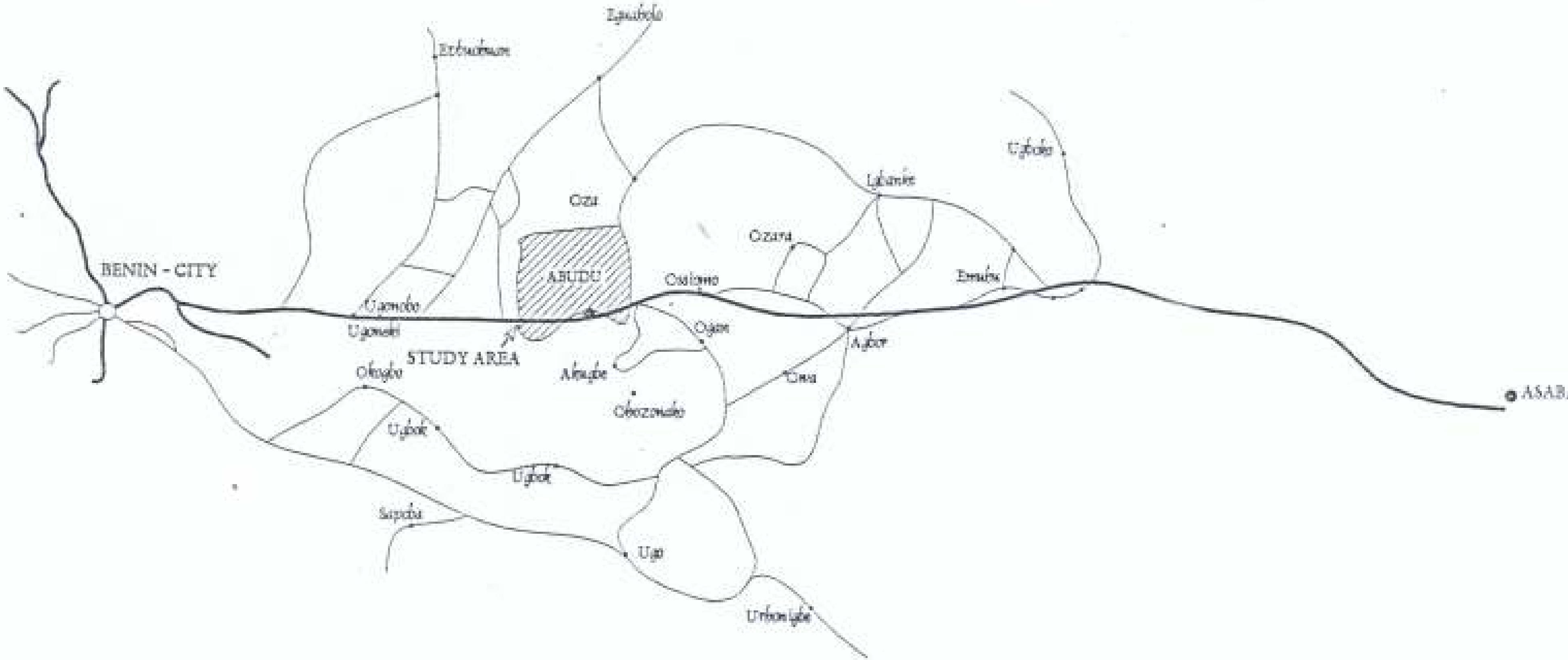


FIG .3.2. LOCATION OF ABUDU ALONG BENIN - ASABA TRUNK ROAD.

3.4 The Physical Landscape

(i) Relief

The entire Abudu area is on a relatively steep terrain that marks the sideof a plateau. The East- facing slope that leads down Ossiomo valley is relatively steep with average slope of 4".48'. The lignite group of rocks, consisting of clays underlies the entire area, fine- grained sands, lignite and carbonaceous shaley clays. (Idahosa, 1986).

(ii) Drainage:

Ossiomo River drains the entire Orhionmwon LGA, including Abudu town. The river rises from its source at Iruekpen village in Esan West LGA of Edo State and maks its meandering runs through Abudu and Ologbo towns before discharging into Boom River (Fig.3.3). Ossiomo River is characteristically deeply incised, with rocky bed and steep banks, dense forest and hill ranges hug the river banks at some sections of its run as it passes through Abudu hill ranges. The watershed of Ossiomo river basin at Abudu is shown in Fig.3.4.

It is important to note that at its source, the people call it Ibiekuma River, while at Abudu and in the entire Local Government Area of Orhionmwon, it is called Orhionmwon River. The Local Government Area was named after the river because it cuts across the entire Local Government Area. Officially and on the topographic map of Nigeria, the river was named after the village within the Abudu area called ***Ossiomo***, where the first leprosarium hospital in the then Western Region of Nigeria is located.

FIG 3-3 LOCATION OF OSSIOMO RIVER WITHIN THE STUDY AREA - ABUDU, EDO STATE

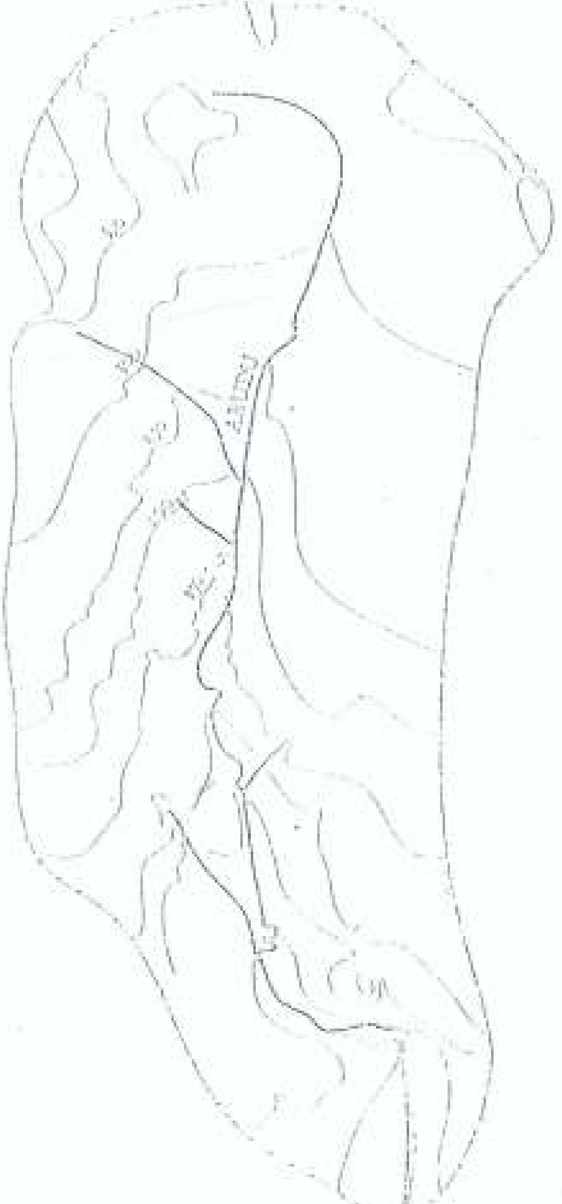


FIG 3-4 OSSIOMO RIVER CATCHMENT AREA

3.5 Climate

The climate of Abudu town is of the sub-equatorial type. It is typical of the climate of the southern Nigeria. The Abudu area experiences the humid tropical climate, which is characterized with wet and dry seasons. The dry seasons which lasts between the months of November and March usually coincides with the period of high sunshine, while the wet season which last between the months of April and October coincides with the period of low sun. This variation between the seasons is caused by the latitudinal migration of the Inter-Tropical Convergence Zone (I.T.C.Z.) (Idahosa, 1980)

The rain usually falls from convectional storms hence is unevenly spread throughout the days of the wet seasons. The mean annual rainfall is estimated at 2200 mm. The months of July and September have highest rainfalls, while the months of December and January have the lowest rainfalls in the year. Fig. 3.5 shows the Annual rainfall from 1978 – 2002 , while Fig. 3.6 shows the mean monthly rainfall. The tabulated monthly rainfall from 1978 – 2002, is shown in Appendix A.

The mean monthly temperature is about $27^{o}C$, while the extreme maximum rarely exceed $40^{o}C$. The months of January to March experience high temperature, while the lowest temperature is recorded in June and July.

The relative humidity ranges from 40% in the morning to about 70% in the afternoons. Over most of the areas the altitude modifies temperature thereby eliminating extremes. In general, the relatively flat top of the plateau are relatively cooler, than other parts of the Delta

The vegetation is characterised by trees and grasses. The trees are usually between 6-12m in height, strongly rooted and with flattened crowns. The grasses are often long reaching up to 3.5m. The climate and soil, favours the growth of dense savannah woodland. Apart from the tropical hard wood timbers, such as Iroko, Obeche and Agba, industrial and food crops including the Oil Palm, Rubber, Cocoa, Plantain and many local important fruits thrive well in the forest.

The traditional method of farming, which is rotational bush fallow cultivation, has left its mark on the vegetation. The over cultivated abounded fields, especially when on poor soils, get colonised by grasses and subsequently a new type of vegetation known as derived Savannah now replaces the original richer vegetation.

The soil types are ferrisoils on loose sandy sediments. The area is composed of the basement complex of the Precambrian era. The Ossiomo River overlies detritus of Bendel Ameki shales and sandstones with a unique toposequence (Aiboni, 1988).

Soils are less leached and consequently retain the advantage of a good rooting depth. The top soil, when freshly cleared of forest, contains about 5% organic matter and can thus sustain good food crops for several years.

3.6 Population

The 1991 census figure put the population as 11,271, made up of 5,482 males and 5,789 females. However, assuming the growth rate of 3% in the light of its fast growth by reasons of urbanization, the population as at year 2003 is estimated to be about 20,000 people.

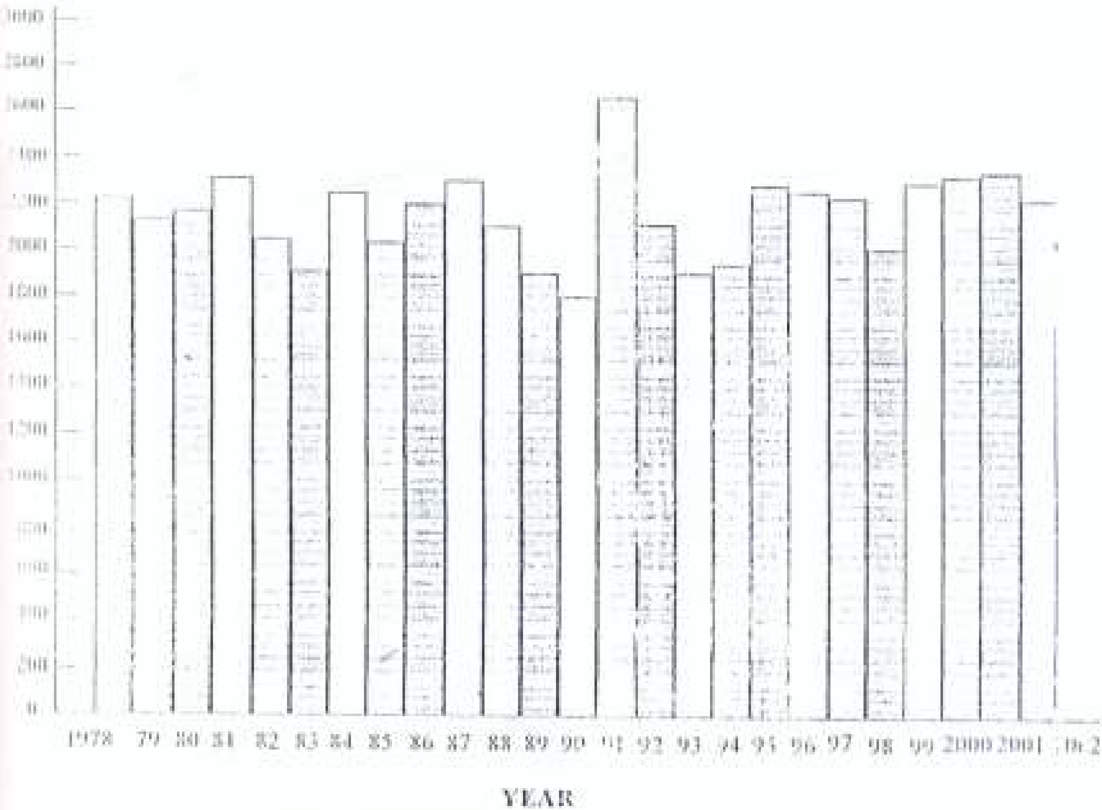


Fig. 3.5 ANNUAL RAINFALL FOR THE YEAR 1978-2002

Source: Department of Meteorological Services: Benin city, 2001

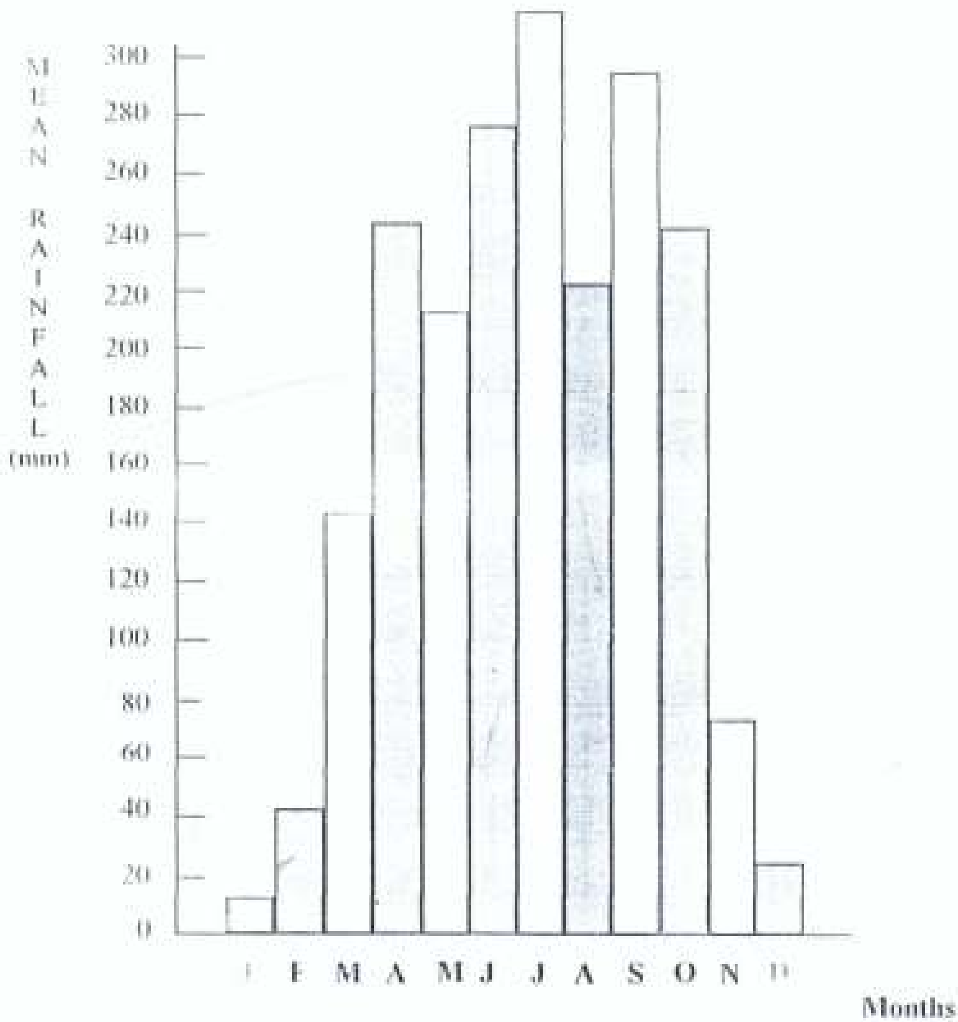


Fig. 3.6 Mean Monthly Rain Fall

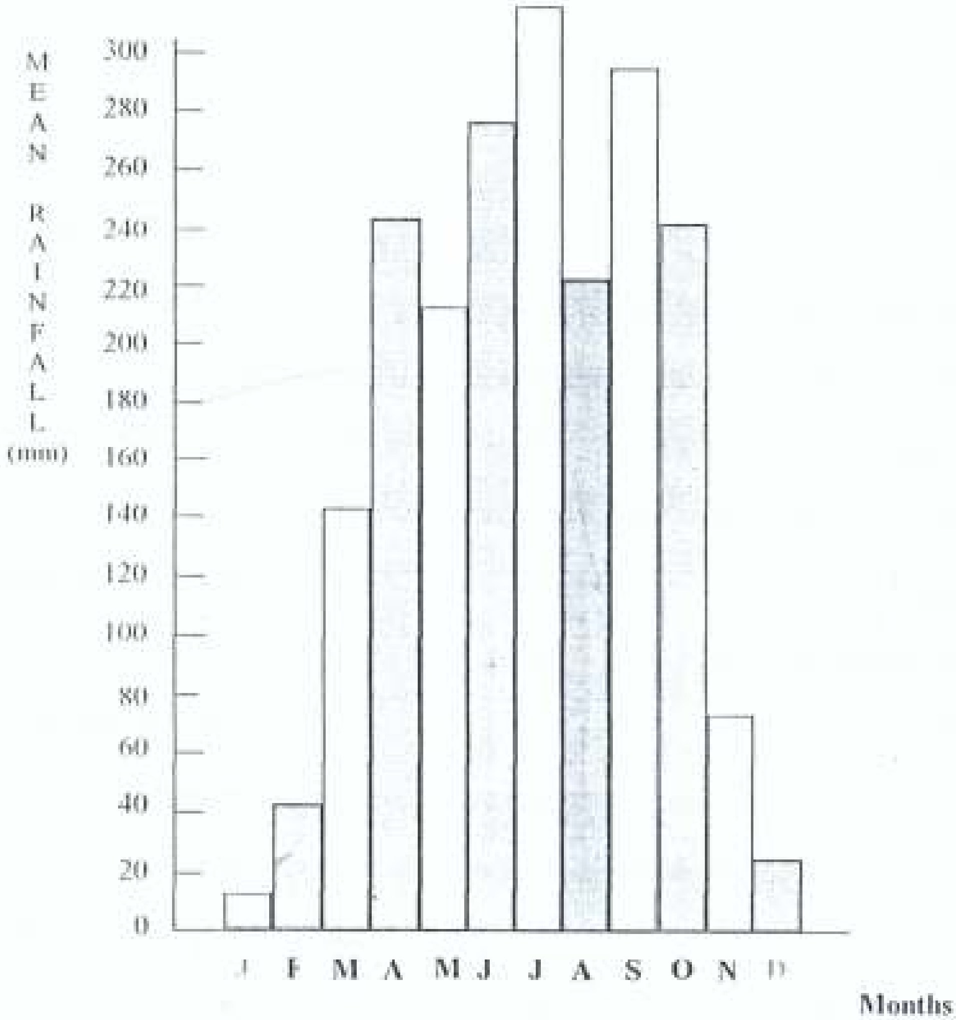


Fig. 3.6 Mean Monthly Rain Fall

CHAPTER FOUR

RESEARCH METHODOLOGY

4.1 General

This work was undertaken in three phases. The preliminary phase focused on the collection of data from relevant public and private agencies, responsible for publishing or keeping relevant hydrological data. The other phase involved reconnaissance survey of the study area with a view to having first hand knowledge of its water situation, in addition to studying the physiography and hydrology of Ossiomo river basin. This led to the field study in which the collection of field data and quantitative measurements of relevant hydrologic parameters such as, stream flow, and water quality were conducted. The third and final phase included data analysis, design works and appropriate recommendations to solve the perennial problem of potable water supply to Abudu community and environs.

4.2 Hydrological Data and other Information

The following data were obtained in the course of this project.

(i) Topographical Map

The 1:50,000 topographical maps were obtained from Federal Survey, Lagos. They showed the course of River Ossiomo (the main water source in the project area - Abudu) from its source at Ibiekuma village in Iruekpen – (Ekpoma Esan West LGA of Edo State), through the study area, to its final discharge into Benin River. The map was useful in the study of the physiographic characteristics of the River basin.

(ii) Rainfall Data

The source of the rainfall data is the Department of Meteorological Services in Benin City. (1978-2000). This was used to estimate the average rainfall and range, and to find the rainfall during the three driest years, usually associated with shortage. The rainfall data are from these recorded with a radar equipment installed at Benin City Airport.

(iii) Published Hydrological Information

Benin-Owena River Basin Authority – Federal Ministry of Water Resources Yearbooks, vols. (I, II & III) were obtained for reference purposes. All data in respect of hydrologic measurements - stream flow data, for River Ossiomo at Abudu gauging station were examined, and relevant data extracted. The yearbooks covered the water stage/discharge measurements carried out between 1989 and 1994

(iv) Population Census Figure

The latest (1991) Population figure for Abudu was obtained from the National Population Commission Office in Benin City, from which the population, were projected. From this, the anticipated population for a design period of 25 years, were estimated for the purposes of working out total water requirements, and other design parameters.

4.3 Other Information/Data

The state of existing water supply scheme was obtained from Edo State Urban Water Board, Benin City. This information revealed the existence of a water supply scheme in Abudu, which has since collapsed due to age and lack of maintenance. The scheme consists of a borehole water supply facility, which collapsed in 1989, and an overhead, $200m^3$ capacity Braithwaite tank that served as service reservoir.

4.4 Field Observation/Measurements

The objectives are to determine the extent, and character of water on which an evaluation of control and utilisation are to be based. In pursuance of this, and after identifying Ossiomo river as a potential and dependable source of water supply for Abudu community, the following quantitative measurements were carried out:

(i) Stage (ii) Discharge (iii) Water Quality.

It should be noted that stage and discharge measurements were carried out for year 2003. The measurements were used in conjunction with previous measurements (1989 to 1994) on Ossiomo river, as recorded by Benin Owena River Basin Authority.

4.5 Measurement of Stream Flow (Year 2003)

As there was no official activity in respect of the streamflow measurement of Ossiomo River for the year 2003 by the Benin Owena River Basin Authority, the author, with the assistance of Benin Owena River Basin Authority, conducted the measurements.

Measurements were taken to cover both the dry and rainy seasons. The dry season measurements were carried out in the months of January, February and March, while that for rainy season, were taken in May, June and July. The measurements were carried out with Current Meter, using the Bridge Boom method with 25kg-iron weight, (as sounding weight) and hand – operated reels and boom.

4.6 Water Quality Measurements

Ossiomo River has no previous records regarding its water quality. The water sampling was planned to cover both the dry and rainy seasons. Representative water samples were taken in the months of January-March (Dry Season) and May to July (Rainy Season). The water sample analysis was carried out at the Benin-Owena River Basin/University of Benin, Joint Analytical Research Laboratory in Benin City.

4.7 Data Analysis

Analysis of the data collected (both from agencies and from field) consist of the followings:

(i) Population projections for water requirements

(ii) Hydrological evaluation of the Ossiomo water shed

(iii) Rainfall data, to obtain monthly and annual means, and estimates for the dry years.

(iv) Stream-flow analysis to obtain maximum and minimum discharges

(v) Water quality analysis for treatment plant design parameters

(vii) Topographic analysis for distribution network

4.8 Design Work

Items designed for were:

(i) Intake systems

(ii) Treatment works

(iii) Pipeline distribution network

CHAPTER FIVE

RESULTS AND DISCUSSION

5.1 Results

(i) Ossiomo River - Stream Flow Records

Extracts from published streamflow records (Table 5.1) for Abudu Ossiomo River, by Benin Owena River Basin Authority Federal Ministry of Water Resources, showed that the minimum annual water discharge of the river between 1989 and 1994 was $10.64m^3/s$, while the maximum discharge was $138m^3/s$. The minimum discharge was recorded in 1990, while the maximum discharge was in 1991.

The corresponding minimum and maximum water stages were also shown to be 1.12m and 3.08m respectively. (Table 5.2)

Table 5.1 Ossiomo River-Abudu Station Water Discharge

Year	1989	1990	1991	1992	1993	1994
Maximum	$95m^3/s$	$41.8\ m^3/s$	$138\ m^3/s$	$125\ m^3/s$	$80\ m^3/s$	$77.7\ m^3/s$
Mean	$22.18\ m^3/s$	$19.29\ m^3/s$	$28.14\ m^3/s$	$36.6\ m^3/s$	$32.74\ m^3/s$	$37.14\ m^3/s$
Minimum	$14.28\ m^3/s$	$10.64\ m^3/s$	$11.08\ m^3/s$	$11.3\ m^3/s$	$15.12\ m^3/s$	$14.2\ m^3/s$

(Source: Benin Owena Basin Dev. Authority, Hydrological Year Book, Vol. I-III.)

Table 5.2 Ossiomo River- Abudu Station Water Stages

Year	1989	1990	1991	1992	1993	1994
Maximum	265cm	192 cm	308 cm	295 cm	250 cm	247 cm
Minimum	126 cm	112 cm	114 cm	115 cm	124 cm	126 cm

(Source: Benin Owena Basin Dev. Authority, Hydrological Year Book, Vol. I-III)

Extracts from discharge measurement carried out by the author for the dry and rainy seasons are as shown in Table 5.3, while the discharge field measurement notes are in appendix B (1-6)

Table 5.3 Ossiomo River- Abudu Station Water Discharge (year 2003)

Dry Season			**Rainy Season**		
January	**Feb.**	**March**	**May**	**June**	**July**
M^3/s	**m^3/s**	**m^3/s**	**m^3/s**	**m^3/s**	**m^3/s**
15.2	14.3	12.8	19.5	26.2	28.6

(ii) Water Quality

The laboratory results of the representative water sample (dry and wet seasons) from Ossiomo River, are as shown in Table 5.4.

(iii) Population Projection, and Water Requirements

(a) Design Population for Abudu Town

Design period =25yrs

Growth rate =3%per annum (National Population Com. Benin)

Population Figure =11271 (1991 Census)

Estimated population

(year -2003) $=11271\ (1.03)^{12} = 16070$

Estimated population for year 2028

(25years period) $=16\,070(1.03)^{25} = 33650$

Hence Design population is approximately **34000** people

(b) Total Water Requirement

Unit Water Consumption = 120 Litres/ Capita/ Day (Edo water board)

Total Domestic Demand = $120 \times 34000 = 4080 m^3/day$

Allowing for non-domestic consumption, 30% = 1228 m^3/day

Total dom. + Non Dom = 5308(m^3/day)

Add waste/leakages 25% = 1326

Total Avg. Daily Demand = 6334 (m^3/day)

∴Average daily demand = 6334 (m^3/day)

Using a peak factor of 120%

∴ Peak day demand = **7961(m^3/day)**

Table 5.4 Water Analysis Results (Ossiomo River Abudu, Edo State).

...history	Dry season			Rainy season			WHO Standard	Remarks	
...mpled	Water 24/01/03	Water 24/02/03	Water 24/03/03	Water 22/05/03	Water 22/06/03	Water 22/07/03	W.H.O	OK	Not OK
...n	Abudu	Abudu	Abudu	Abudu	Abudu	Abudu	Standard		
...ng point	Ossiomo River	Ossiomo River	Ossiomo River	Ossiomo River	Ossiomo River	Ossiomo River			
...rature	27^0C	27^0C	27^0C	26^0C	25.8^0C	25.3^0C			
Ph	6.2	6.86	6.81	7.2	6.60	6.78	6.5/ 8.5		
...urbidity (NTU)	5.0	5.80	4.06	5.02	3.54	3.41	<1		
...tal dissolved ...olids	27.2	28.0	15.60	45.0	45.0	43.0	1000		
...tal suspended ...olids	3.80	4.0	3.95	5.24	5.8	4.06	-		
...mmonia – N	1.02	0.8	1.63	2.0	0.85	0.93	0.05		
...tal Alkalinity	52	60	65.00	55.0	58.0	60.20	-		
...linity(chloride)	12.01	12.0	15.80	19.31	19.30	19.04	250		
...NIONS									
Nitrate	0.43	0.45	0.74	0.14	0.18	0.10	10		
Sulphate	1.70	1.92	2.32	1.73	1.95	1.85	400		
Phosphate	0.45	0.55	0.63	0.98	0.98	1.01	-		
...ETALS									
Sodium	18.03	18.8	19.10	20.40	20.35	20.40	200		
Potassium	7.21	6.89	7.35	9.05	8.15	8.08	-		
Calcium	14.0	12.81	16.03	12.41	14.21	14.51	75		
Magnesium	3.92	4.92	5.97	5.32	5.41	6.00	50		
...tal Hardness	42	42	45.00	40.00	43.00	42.05	500		
...ACE ELEMENTS									
...tal Iron	0.28	0.28	0.30	0.31	0.28	0.31	0.3		
...ad	0.15	0.18	0.13	0.18	0.19	0.17	0.05		
...hromium	0.048	0.06	0.04	0.01	0.10	0.92	0.05		
...opper	0.19	0.12	0.24	0.44	0.48	0.51	1.0		
...anganese	0.03	0.03	[illegible]	0.08	0.11	0.13	0.1		
...nc	1.21	1.10	1.32	0.79	1.90	2.01	5.0		
...dmium	ND	0.001	ND	ND	ND	ND	0.005		

Notes: All measurements in mg/l, except where indicated.

N.D = Not detected

W.H.O = World Health Organization

5.2 Discussion

(i) Rainfall

From Rainfall records, the three driest years on record over a period of twenty-five years were 1983, 1990,and 1993, with annual rainfalls of 1880mm, 1753mm and 1861mm respectively. Furthermore, stream-flow measurements in Tables 5.1 – 5.3, showed that the minimum annual stream discharge of Ossiomo river recorded in 1990 and 1993 were $10.64m^3/s$ and $15.12m^3/s$ respectively.

A comparison of the annual rainfall of the three driest years on record over a period of twenty-five years, showed that the minimum annual stream discharge of Ossiomo river recorded in 1990 and 1993 can be associated with the annual rainfalls of 1753mm and 1861mm for the same period. Therefore a minimum river discharge of $10.64m^3/s$ can be safely adopted as the minimum discharge of Ossiomo river over a twenty-five year period, as the annual rainfall within the twenty-five years period, has been in the range of 1700mm to 2600mm.

(ii) Ossiomo River Hydrographs

Ossiomo river at Abudu region is located on latitude 6^0,N18^0 Longitude 6^0 E 02^0 and has a drainage area of $1230.65km^2$ (BORDA, 1997). The river is about 2.47km within the Abudu area, with an average width of approximately 36.4m.

The discharge hydrograph shown in Fig 5.1 shows considerable seasonality in its stream flow characteristics. The hydrograph shows a peak flow in the month of September during and following heavy rainfall that reflects the tropical rainy season of the region. A closer scrutiny of the hydrograph, reveals a

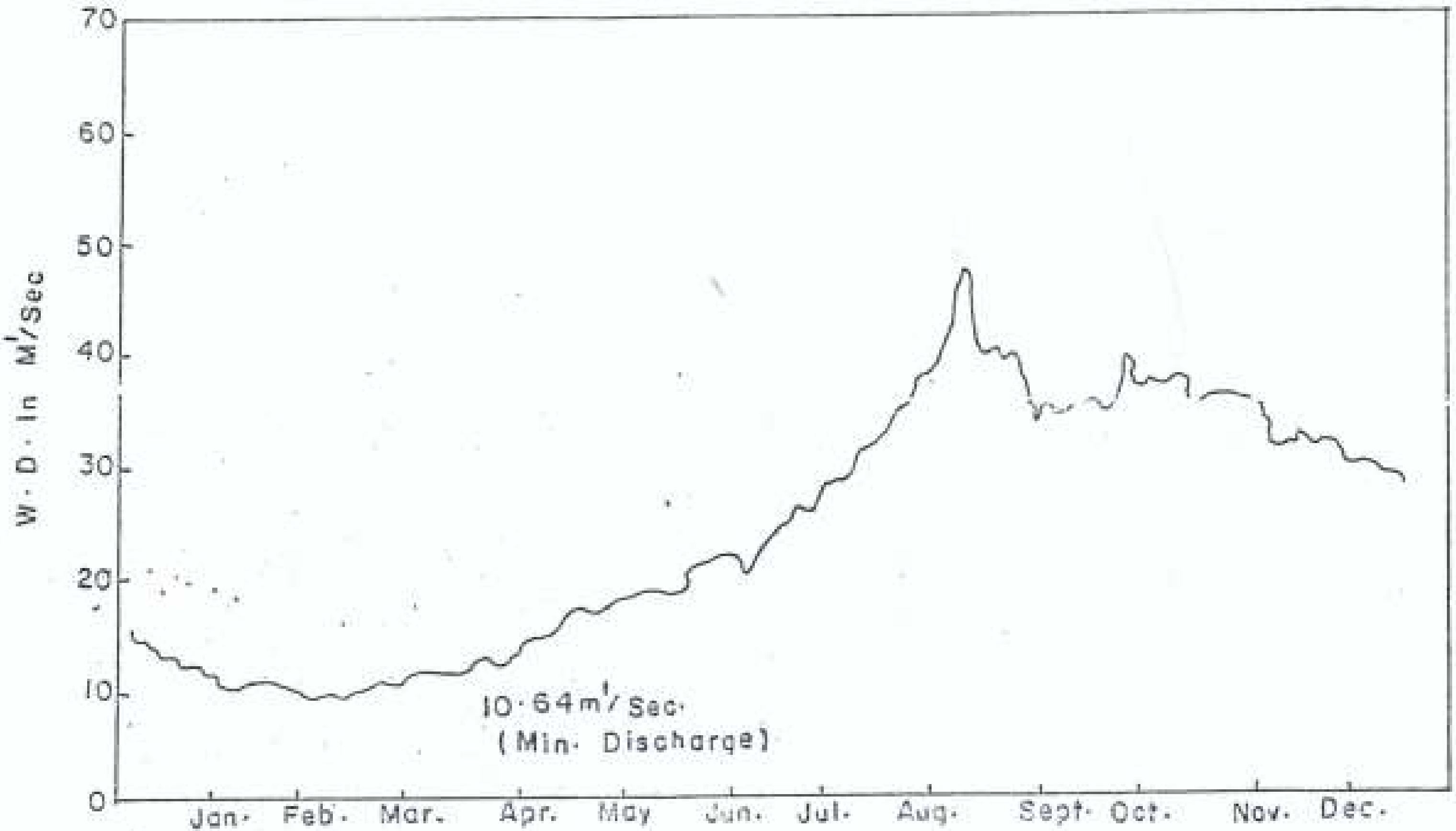


FIG · 5 · 1 : WATER DISCHARGE HYDROGRAPH (1990)·
OSSIOMO RIVER AT ABUDU

long period of high flow that begins in May and does not end until December. In fact from available records, (1989 to date) it is safe to assume that about 80 to 90% of the annual flow occur during the four months of the heavy rainfalls in the rainy seasons.

Also from the water level hydrogaph shown in Fig 5.2, the increase in water level starts substantially in April and rises to a peak in September. Finally, from the above hydrograph analysis, the hydrograph has a perennial and continuos flow regime, in addition to a dependable discharge that can be possibly harnessed in future, and treated for public use and consumption.

(iii) Water Quantity

In estimating the supply a river can afford at all times, the flow of the river in the driest years, or during periods of low stream flow is an important factor. From records in Tables 5.1- 5.3, the minimum annual stream discharge of Ossiomo River, which is *10.64m³/s*, has been adopted in this paper as the *Design Safe Yield* of Ossiomo River.

This quantity is much more than the projected total water requirement of $0.138m^3/s$ calculated for Abudu Community (within the designed period of 25yrs). Therefore, Ossiomo River can be considered as a dependable source of water supply to Abudu town; as the water requirement of $0.138m^3/s$ is only 1.3% of the minimum flow-rate of the River. In the periods of low flows, downstream users of the Ossiomo river will still have sufficient water for their various uses.

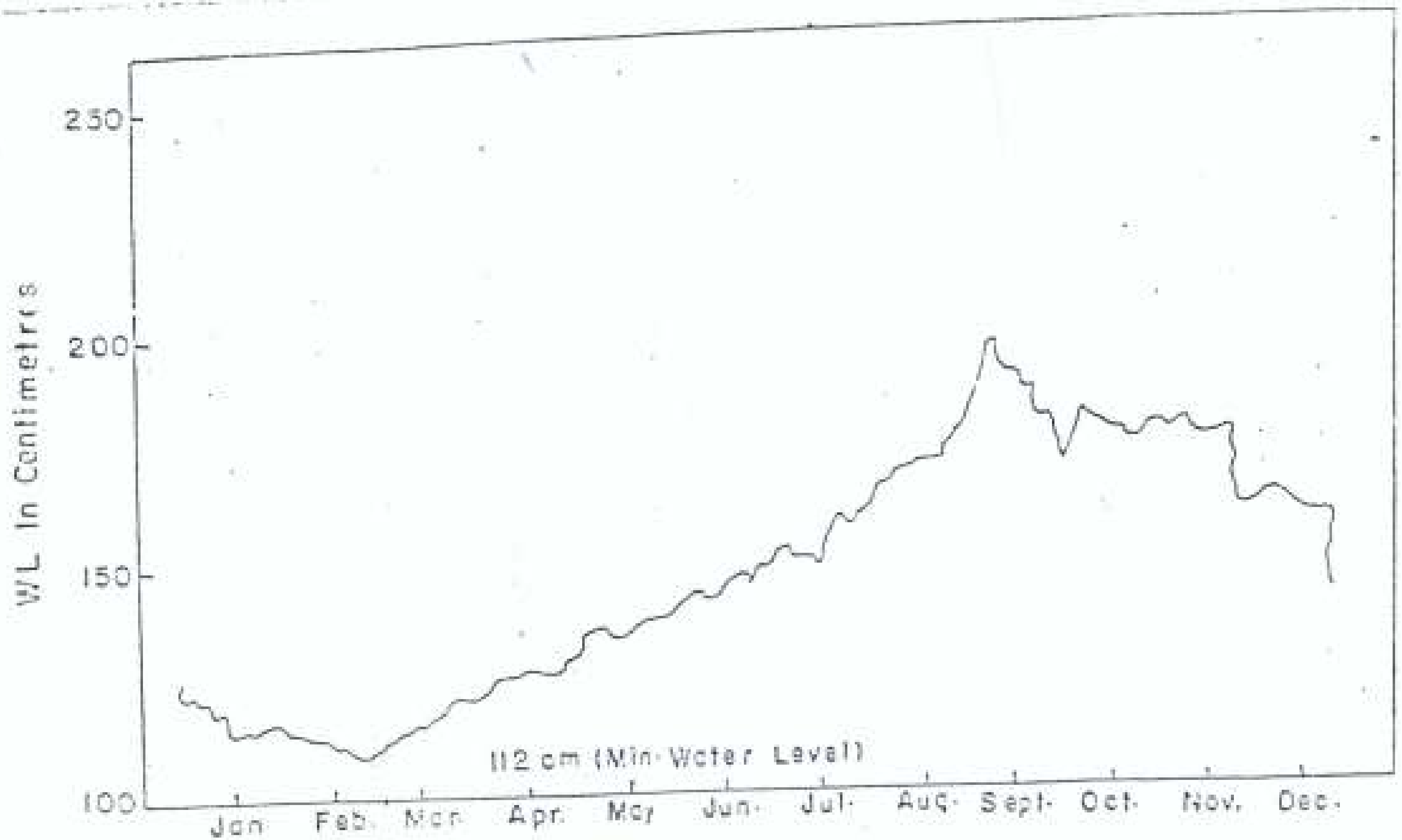


FIG. 5. 2: WATER LEVEL HYDROGRAPH (1990)
OSSIOMO RIVER AT ADUDU

(iv) Water Quality

In the evaluations of water quality, the impurities in the water are usually analysed and the level of impurities depends on the source of the water. The laboratory results (Table 5.4) of the water sample of Ossiomo river, have been cross checked with World Health Organisation (WHO) Standards, for potable water, and it is evident that some of the parameters fall short of the acceptable standards as it affects potable water.

Consequently, the water has to be treated before it will be suitable for use and consumption. Screening, Coagulation/ Flocculation, Sedimentation, Filtration and Disinfection, are the main unit operations that will be involved in the treatment processes.

(v) Design works (See Appendix C1 to C17)

Design of the main component of the water supply system has been carried out based on the relevant data arising from laboratory and field results as it affects water quality and quantity, population and water use.

The water supply facilities that have been designed for, consists of Collection, Treatment Works, Transmission, Pumping, Storage and Distribution System. The size of the proposed water supply project has been based on an average daily water per capita consumption rate.

The design parameters include:

(a) Design Period = 25yrs.

(b) Design Population = 34000

(c) Total Water Requirement = $7961m^3$/day

(d) Design Flow Rate = 0.138 m^3/s

(vi) Main Elements of Abudu Water Supply Project

The main elements of the Abudu water supply works, comprises the following:

(a) Intake Structure System

-The siphon well intake system - to be located at the shore of Ossiomo River.

-Detention time (Water Well) = 20min

(b) Pump Station

-Raw water pumping station located at the shore of Ossiomo River.

-Three pumps working, and one on standby.

-Raw water transmission line, 300mm-dia. ductile iron pipe to treatment works.

(c) Treatment Plant

-Treatment works with chemical dosing facilities. The main components are:

(i) Coagulation/Flocculation Tank

-Detention time of 20mins.

(ii) Sedimentation Tank

-Detention time for sedimentation tank is 2hrs.

(iii) Rapid Sand Filter

-Flow rate of $4m^3/m^2$-hr

(iv) Clear Well of $2390m^3$ capacity

The flow diagram for the treatment plant is as shown in Fig.5.3.

(d) Distribution

The conditions at the distribution ends are:

(i) The Service Reservoir is 15m high from the ground level and the depth of water column is 5m, hence the static pressure head at the starting point A is given by (15+5)m = 20m.

(ii) The maximum ground elevation = 310.34 m o.d

The minimum ground elevation = 295.35 m o.d

(iii) Pressure Head elevation at A = (308 + 20) = 328 m o.d

The distribution system adopted is the *Dual System or Combined Gravity and Pumping System,* while Ductile Iron Pipes have been used as Pipeline Mains and Sub- Mains in the distribution network.

The simplified diagrams of the Siphon Well Intake System are shown in Figures 5.4 and 5.5, while Figure 5.6 shows the simplified pipeline network, indicating final designed values of flows and direction. The Transverse Survey of the pipeline route, along the road is as contained in Appendix F (DWG. 01 – 07)

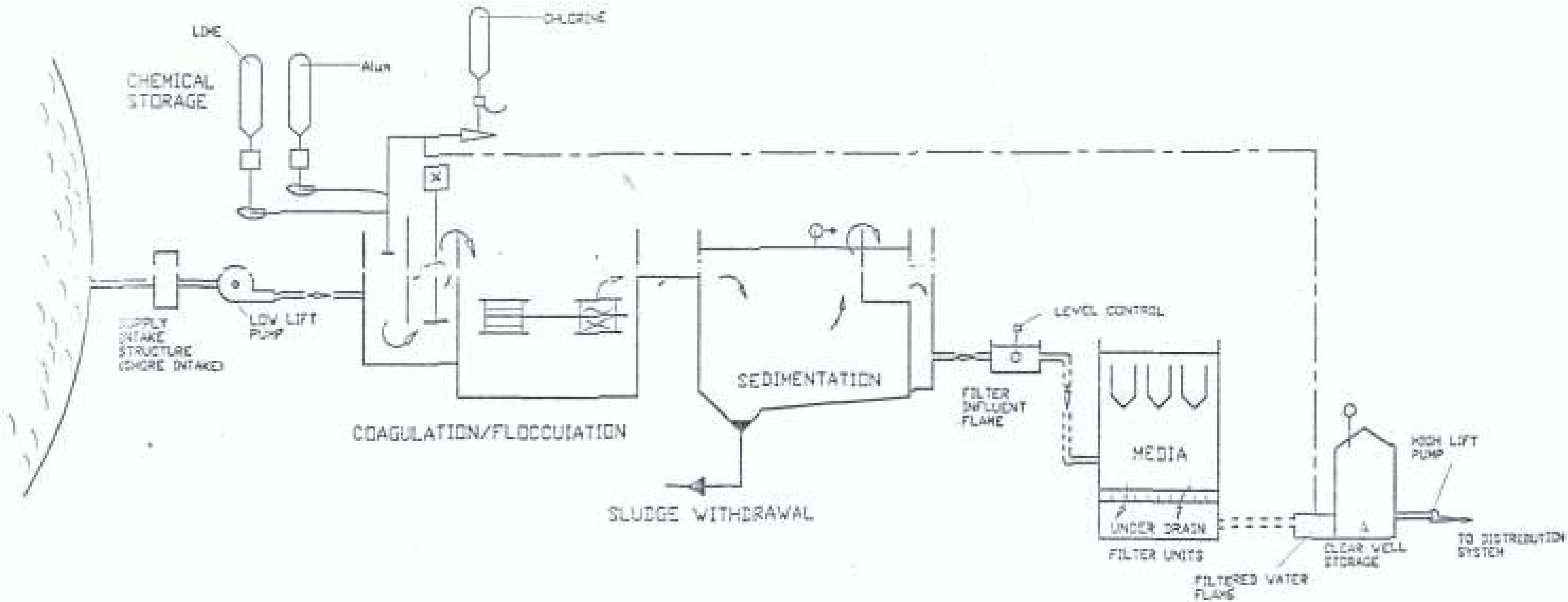


FIG 5.3 FLOW DIAGRAM OF WATER TREATMENT PLANT

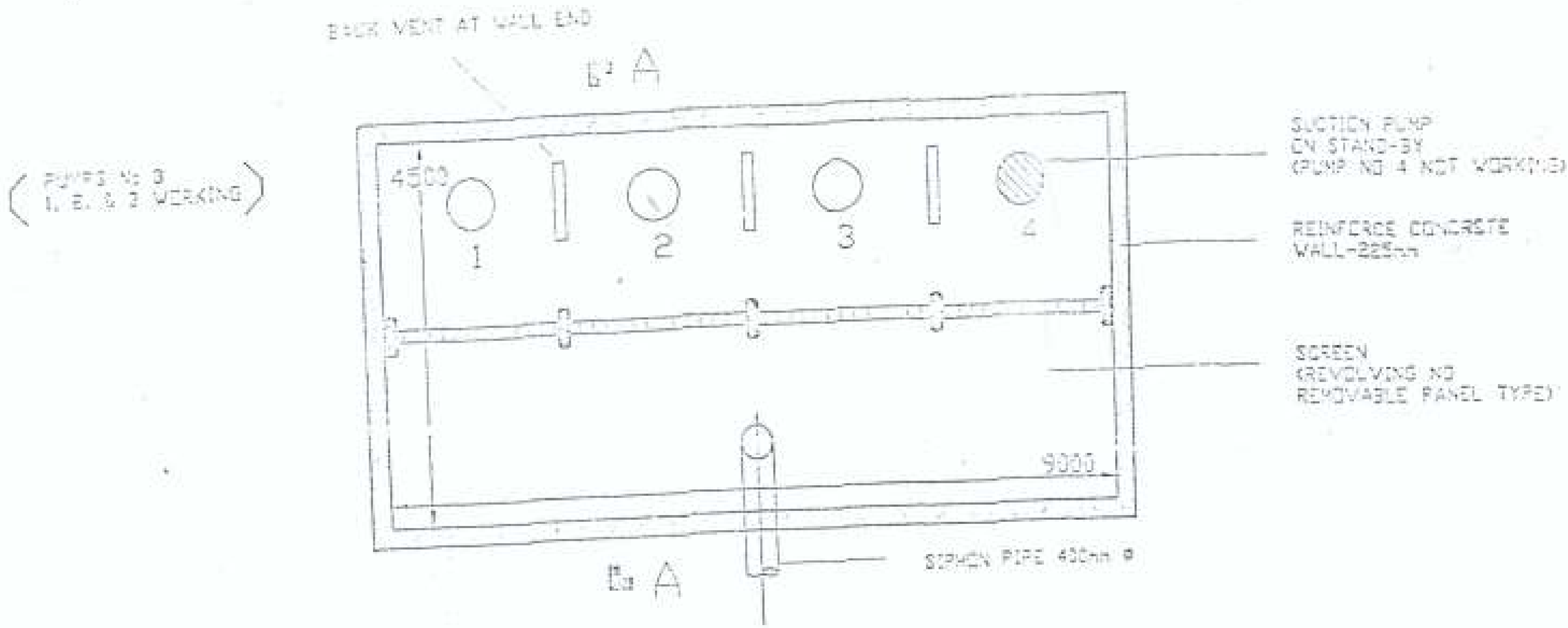


FIG 5·4 LAYOUT PLAN (SUMP WELL)

SIPHON WELL INTAKE SYSTEM (SHORE INTAKE)

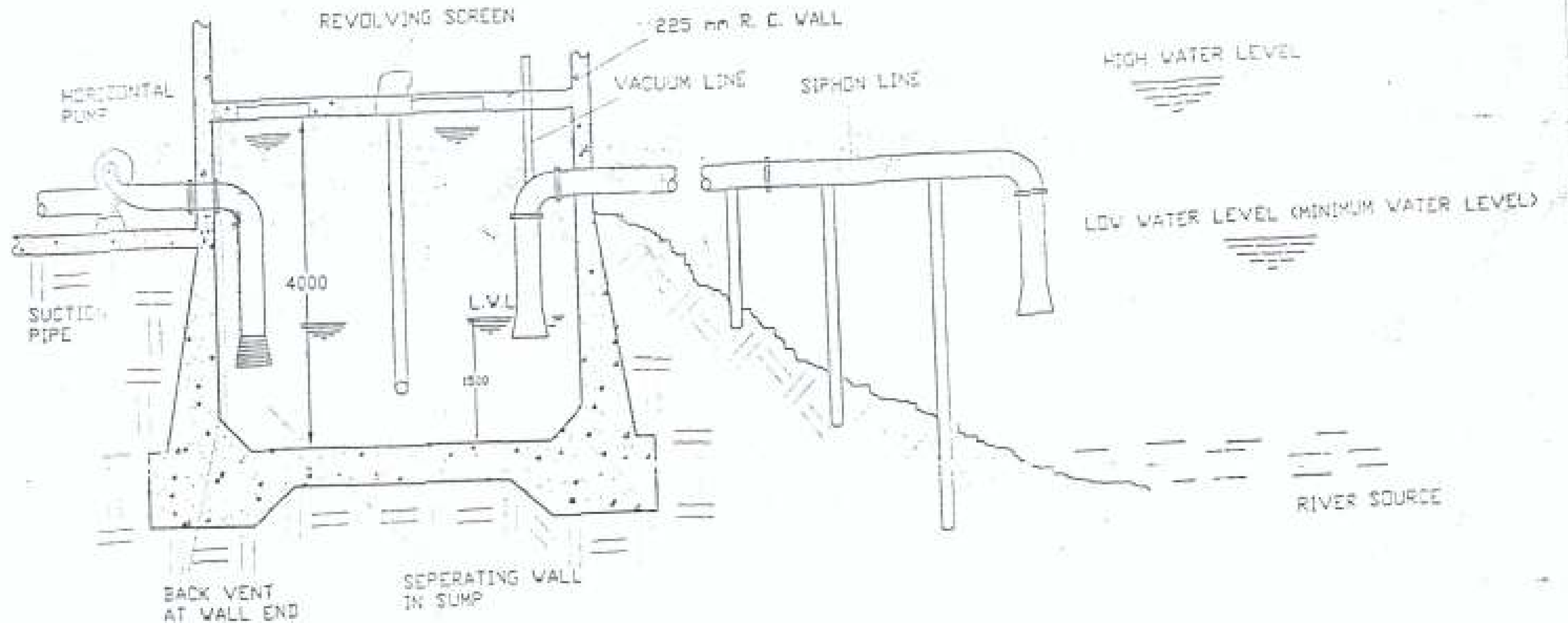


FIG 5.5 CROSS SECTION A-A

SIPHON WELL INTAKE SYSTEM (SHORE INTAKE)

Elevation of Pipe Nodes: –

Node	A	B	C	D	E	F	G	H	J	K	L	M
Elevation (m.o.d.)	312 .38	310 .34	302 .48	305 .12	302 .18	298 .45	295 .35	298 .13	299 .81	301 21	309 .12	296.3 7

(Source: Transverse survey by the author.)

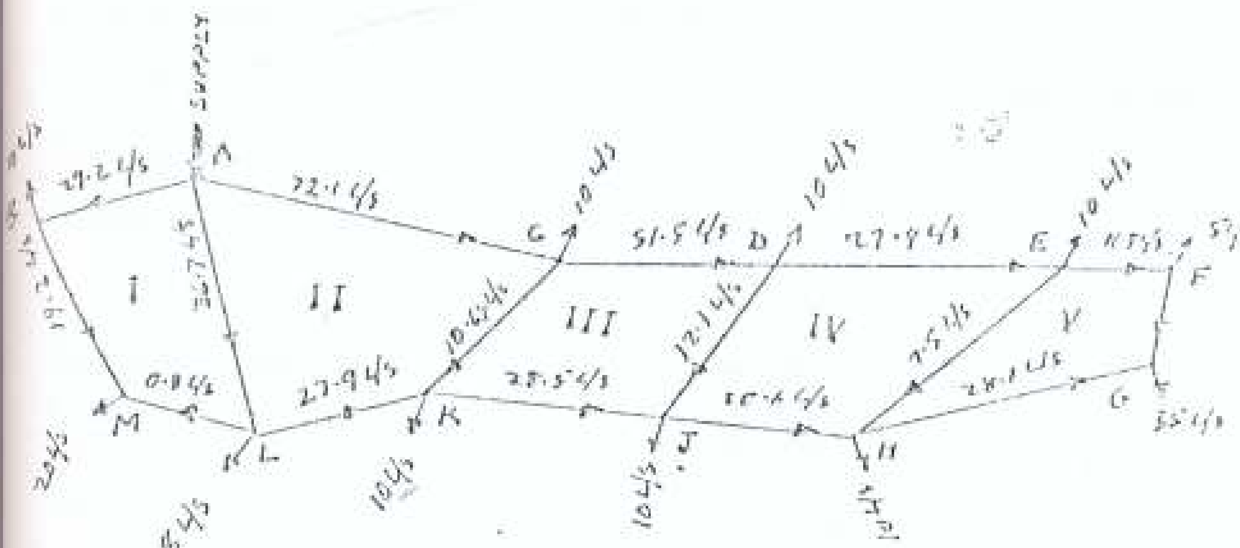


Fig.5.6 Simplified Net Work Diagram –Indicating Final Values

of Flows and Directions.

5.3 Financial Estimate

The financial estimate is made up of the sum of the estimated total cost and the fees for the consultants and the reimbursable bill for consultants. The cost of land acquisition has been excluded, because it is expected that land use decree of 1978 will be invoked by Government, as the provision of a water supply scheme, is usually seen as a social responsibility of any Government. The main source of power for the project is from the public power supply (NEPA). The generators provided, acts as stand-by, incase of power outages, low voltage supply and under frequency periods.

The total cost of the project is ₦ 239,802,500 as detailed in table 3.5, and summarised as follows:

Estimated Total Cost Project	=	₦ 225,000,000
Prime Consultant (Water Engineers)	=	₦ 5,637,500
Structural Engineers	=	₦ 3,265,000
Mechanical Engineers	=	₦ 2,725,000
Electrical Engineers	=	₦ 2,725,000

Reimbursable Expenses:

Maximum=1% of ETC

Take 0.2% of ETC for all Consultants

∴ Reimbursable Expenses	=	₦ 450,000

For a relatively small town of this size, water supply scheme can be procured for ₦ 240,000,000. This is quite feasible and within the executive scope of Local and State Governments, and with possible assistance of the Federal Govemment.

CHAPTER SIX

CONCLUSION AND RECOMMENDATIONS

6.1 Conclusion

This study showed that Ossiomo river discharge measurements during periods of low flows, vary from $10.64m^3/s$ to $12.8m^3/s$ over the six consecutive years, for which data is available.

The minimum flow rate of $10.64m^3/s$ recorded in 1990, (which for design purposes can be taken as the minimum yield, during periods of drought or low flows) can sustain the calculated design flow rate of $0.138\ m^3/s$ for the Abudu water supply scheme. In fact, the surface source can supply the Abudu town and environs if properly harnessed, as the water requirement for Abudu represents only 1.3% of the streamflow.

A comparative analysis of the water quality results and WHO's standard values, showed that some of the water quality parameters fall short of the acceptable international standard for potable water supply. In view of this shortfall in standard, there is a need for the treatment plant with the above calculated design parameters to meet with potable water supply standard.

Ossiomo river is a major water resource of the Local Government, and the State, which can be harnessed for domestic, industrial and agricultural purposes.

6.2 Recommendations

The Abudu water supply projects is to be designed to provide a safe reliable water supply to the urban community. To achieve this primary objective, and taking into cognizance, the data and experience gathered in this study, the following technical recommendations are to be adopted:

(i) Intake structure: An offshore intake structure (Siphon-Well Intake Type) is to be adopted, since the river stage does not fluctuate too much. This will remove the

(ii) necessity for a tunnel or subaqueous pipeline, thus simplifying the design or the intake structures.

(iii) Water treatment Components: Water treatment method to be adopted will involve the following main unite operations, (a) Screening, (b) Coagulation / Flocculation (c) Sedimentation (d) Filtration and (e) Disinfection. Accordingly, each component has to be designed for, so as to achieve standard operations in the treatment plant.

(iii) Other non-technical recommendations to ensure sustainability and efficient functioning of the project, are follows: -

(a) Institutionalizing preventive maintenance for water supply at all levels and making budgetary provisions for operations and maintenance (O/M)

(b) Proper funding of water supply projects for Abudu and the consideration of water as an economic commodity, rather than a social good, so that proper economic benefit can be derive for water supply.

(c) Future extension of the Abudu water project to neighbouring communities as Ossiomo River has the potentials to serve other town and villages in the Local Government Area (Orhionmwon).

REFERENCES

Agunwamba, C.J., (2000), Water Engineering Systems, Immaculate Publications Ltd., Enugu, Nigeria.

Aiboni, V, (1988), "Soil of a toposequence in Abudu, Bendel State", Paper presented at the 16th Annul Conference of Soil Society of Nigeria, Minna, pp 55-62.

American Society of Civil Engineers and American Water Works Association, (1990), Water Treatment Plant Design, 2nd Ed., Mc Graw-Hill Bk. Co. New York.

Ayoade J.O., 1988), Tropical Hydrology and Water Resources, Macmillan Publishers, London.

Benin-Owena River Basin Authority, (1977); Hydrological Year, Book, BORDA, Federal Ministry of Water Resources, Vols.I-III.

Birdie G.S and Birdie, J.S., (1992), Water Supply and Sanitary Engineering, Dhanpat Rai and Sons, Nai, Saraki, Delhi, India.

Chow V.T, Maidment D.R, May L.W, (1988); Applied Hydrology, Mc Graw-Hill Bk Company, New York.

Cornish, R.J., (1980); "The Analysis of Flow in Networks of Pipes", Journal of the Institution of Civil Engineers, London Vol. 13, pp 147-162.

Diyoke, I. O, (2001), " Sustainable National Water Supply Development", Paper presented at the Annul Conference, on Sustainable National Infrastructural Development, of the Nigeria Society of Engineers, Portharcourt, pp173-174.

Escritt, L.B., (1978); Water Supply and Building Sanitation: Public Health Engineering Practice, 4th Edition vol., McDonalds and Evans, London.

Feachem. R, Mc Garry M, Mara D. 1982; Water, Wastes and Health in Hot Climate, John Wiley and Sons Chichester, England.

Federal Ministry of Water Resources, (2000), "Implementation Strategy for Improved National Water Supply and Sanitation", Report of the Department of Water Supply and Quality Control, Federal Ministry of Water Resources, Abuja, Nigeria.

Gregory, J.K and Walling, E.D, (1979), Drainage Basin, Form and Process, Edward Arnold Ltd., London.

Hamidu, J.A (1990),"National Growth, Water Demand and Supply Strategies in the 1990s", Water Resources Journal, Vol.1No.1, pp 4 -11.

Henry. G.J and Heinke W.G, (1989), Environmental Science and Engineering, Prentice Hall, Englewood Cliffs, N.J, U.S.A.

Idahosa R.K, (1986); Geographical Regions of Benin, Heinemann Educational Books Ltd., Ibadan, Nigeria.

Karasik J.I, Krutzsch C.W, Warren H.F, Joseph P. M, (1976), Pump Handbook, McGraw- Hill Bk. Co.

Lencastre A, (1987), Handbook of Hydraulic Engineering, English Edition, Ellis Horwood Ltd., England.

Linsley, R. K and Franzini. J. B., (1979), Water Resources Engineering, 3rd Ed.,Mc-Graw-Hill Inc., New York.

Linsley R.K, Kohler M. A, Poulhus J.L, (1988), Hydrology for Engineers, McGraw -Hill, inc.USA.

Mara, D.D, (1974); Bacteriology for Sanitary Engineers, Churchill Livingstone, Edinburgh, England.

Marion C.B, (Streamflow measurement) in V.T Chow, (Ed.), (1964); Handbook of Applied Hydrology, Mc Graw-Hill Bk. Company, New York.

McBirney Marsh, (1987); "The UMFM Flowmeter", ProductBrochure, McBirney Marsh Inc. Gaithersburg, Md., USA.

Oyegun R.O., (1986); "Financial Viability and Economic Efficiency of Water Supply in Kwara State -The Case Study of Ilorin", AQUA, No 4, pp 207-213.

Porta, F, (1998); " Management Systems of Drinking Water Production and distribution Services in EU member states" AQUA, Vol. 47, No 4, pp 174 – 182.

Russell W.R, (Natural Channels) in Davis, C.V, and Sorensen, E.K (Ed.), (1979); Hand book of Applied Hydraulics, Mc Graw- Hill Bk.Co. New York.

Shaw E.M., (1988); Hydrology in Practice, Chapman and Hall, London.

Seelye E, (1960); Data Book for Civil Engineers Design, John Wiley and Sons Inc New York.

Singh, Gurcharam (1986), Standard Handbook of Civil Engineering, 5th Edition, Nem Chand Jain (Prop). Nai Sarak Delhi. India.

Smith, K (1972); Water in Britain, Macmillan, London.

Steel E.W, and Mc Ghee T.J, (1979), Water Supply and Sewage, 5th Edition, Mc Graw-Hill Inc. New York, USA.

Twort A.C, Hoathel R.C and Law, F.M., (1974); Water Supply, 2nd Edition.Edward Arnold, London.

United Nations (1986), " The Demand for Water," Natural Resources / Water Series,

Dept of Economic and Social Affair, UN, New York. No3, p p56-76.

Willcock B.D., (1983); Water Engineering, Evans Brothers, Ltd., London.

Wogu, N.J., (1977), Basic Pipeline Hydraulics, African Educational Services Ltd.,

Owerri, Imo State, Nigeria.

APPENDIX A

Monthly Total Rainfall from 1978 – 2002

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEPT	OCT	NOV	DEC	TOTAL
2002	0.0	115.3	163.5	278.1	199.4	344.9	411.3	200.1	294.8	112.2	97.2	TR	2216.8
2001	18.8	10.1	119.3	394.3	398.1	364.3	216.0	137.1	357.1	183.0	82.6	3.9	2281.8
2000	4.0	73.0	60.8	170.0	119.8	413.7	294.7	237.9	345.0	351.2	126.4	48.7	2245.2
1999	86.3	64.1	98.2	119.6	161.7	99.5	412.3	232.0	396.0	472.5	97.8	9.4	2250
1998	44.1	1.8	104.6	164.8	214.2	214.4	506.1	95.6	387.9	244.0	58.8	42.0	2013.4
1997	75.3	TR	104.0	230.9	305.3	208.4	285.0	160.2	222.9	338.4	176.6	98.9	2205.6
1996	70.	92.6	188.2	298.3	222.2	291.1	182.3	253.1	376.0	292.9	25.2	1.1	2270
1995	TR	50.6	165.4	215.2	226.9	246.4	283.8	280.8	383.3	240.2	124.7	9.4	[illegible]
1994	27.5	14.6	111.4	149.8	322.9	351.3	41.3	461.4	391.8	204.5	47.0	0.0	2173.5
1993	TR	9.6	135.0	95.4	198.2	288.8	191.4	354.1	257.6	174.2	108.1	48.6	1961
1992	TR	0.2	41.4	222.7	240.0	335.8	515.9	76.4	256.3	292.2	35.6	TR	2010.5
1991	TR	58.1	121.5	386.3	196.7	207.2	656.2	382.6	268.0	267.9	39.2	11.9	2397.2
1990	17.6	18.8	24.5	256.8	180.8	204.1	235.8	216.5	196.9	198.9	33.7	168.6	1753
1989	0.0	25.8	66.1	386.3	150.1	112.0	297.2	90.7	157.3	380.8	14.2	TR	[illegible]
1988	17.5	18.5	112.0	271.0	190.0	314.5	410.4	341.1	210.4	150.0	59.2	0.0	2103.6
1987	0.0	58.8	98.5	310.0	145.0	244.1	347.2	420.5	341.8	189.4	121.0	18.2	2294.4
1986	6.5	89.4	104.2	157.5	341.0	310.0	280.8	380.4	276.2	234.0	160.4	20.1	2360.4
1985	41.1	TR	165.4	230.0	180.5	276.1	310.0	49.2	410.1	344	10.1	42.1	2058
1984	TR	92.5	200.1	315.0	212.0	302.1	380.3	92.8	238.1	280.5	96.7	20.1	2230.8
1983	TR	60.0	220.3	189.5	169.5	255.0	237.6	120.0	389	202.0	35.0	2.1	1880
1982	7.0	39.2	215.4	229.1	210.3	258.0	333.1	240.5	280.2	174.4	72.0	TR	2050.2
1981	6.3	42.0	146.5	310.0	180.5	280.1	412.1	273.8	221.0	262.1	140.0	14.8	2200.2
1980	33.0	TR	198.3	291.0	111.0	190.0	582.0	98.4	230.0	310.3	97.0	0.0	[illegible]
1979	20.5	26.7	212.3	171.0	241.5	310.0	621.4	76.5	310.0	86.9	28.0	TR	[illegible]
1978	9.0	70.5	320.0	240.5	310.1	248.5	512.4	245.4	165.5	TR	60.0	2.1	[illegible]
Mean	17.3	41.3	139.96	240.9	217.1	276	358.3	220.7	294.5	239.5	78.22	22.5	

APPENDIX B1

DISCHARGE MESUREMENT NOTES- (YEAR 2003)

RIVER: Ossiomo — **PERIOD:** Dry Season

DATE: 24/01/03 — **LOCATION:** Abudu

Distance from initial point (m)	Depth (m)	No. of observation (m)	No. of Rev.	Time (Sec.)	Rev./Sec.	Velocity (m/s)		Area (m2)	Discharge (m3)
						At point	Mean in vertical		
2	0.68	0.6	50	10	5.0	0.57	0.57	0.6	0.342
4	0.83	0.6	22	10	2.2	0.25	0.25	4.25	1.063
OBSTRUCTION BY BRIDGE PIER									
12	0.95	0.6	26	10	2.6	0.296	0.296	4.75	1.406
14	1.89	0.2	70	10	7.0	0.781			
		0.8	40	10	4.0	0.46	0.621	3.78	2.347
16	2.84	0.2	80	10	8.0	0.91			
		0.8	36	10	3.6	0.41	0.66	5.68	3.7488
18	3.89	0.2	90	10	9.0	1.03			
		0.8	20	10	2.0	0.23	0.63	7.78	4.9014
20	6.83	0.2	66	10	6.6	0.752			
		0.8	10	10	1.0	0.114	0.433	13.66	5.915
22	6.2	0.2	70	10	7.0	0.798			
		0.8	33	10	3.3	0.376	0.587	12.1	7.28
24	2.63	0.2	20	10	2.0	0.228			
		0.8	15	10	1.5	0.17	0.199	5.26	0.15
26	1.68	0.2	10	10	1.0	0.114			
		0.8	5	10	0.5	0.057	0.855	5.04	0.431
30m									

Time Start: ...9.30am... **Finish:** ...10.30am

Method:......Bridge... Boom...

Weight used:...25kg.......................................

Discharge:... 15.2 m^3/s.......................................

APPENDIX B2

DISCHARGE MESUREMENT NOTES- (YEAR 2003)

RIVER: Ossiomo **PERIOD:** Dry Season

DATE: 21 02 03 **LOCATION:** Abudu

Distance from initial point (m)	Depth (m)	No. of observation (m)	No. of Rev.	Time (Sec.)	Rev./Sec.	Velocity (m/s) At point	Velocity (m/s) Mean in vertical	Area (m^2)	Disch (m³
0	0	0	0	0	0	0	0		
2	0.82	0.6	63	10	6.3	0.714	0.714	0.82	0.58
4	1.01	0.6	28	10	2.8	0.332	0.332	5.05	1.67
OBSTRUCTION BY BRIDGE PIER									
12	0.95	0.6	33	10	3.3	0.388	0.388	4.75	1.84
14	1.8	0.2	91	10	9.1	1.014			
		0.8	51	10	5.1	0.585	0.799	3.6	2.87
16	2.7	0.2	108	10	10.8	1.196			
		0.8	45	10	4.5	0.520	0.88	5.4	4.6
18	3.7	0.2	112	10	11.2	1.239			
		0.8	19	10	1.9	0.233	0.736	7.4	5.4
20	6.5	0.2	83	10	8.3	0.928			
		0.8	11	10	1.1	0.145	0.537	13.0	6.9
22	5.9	0.2	91	10	9.1	11.014			
		0.8	42	10	4.2	0.487	0.751	11.8	8.8
24	2.5	0.2	25	10	2.5	0.299			
		0.8	03	10	0.3	0.064	0.182	5.0	0.9
26	1.6	0.2	08	10	0.8	0.115			
		0.8	05	10	0.5	0.084	0.099	4.8	0.4
30m	-								

Time Start: ...10.05am... **Finish:** ...11.04am

Method: ......Bridge... Boom...

Weight Used: ...25kg..................

Discharge: ...11.3 m^3/s

APPENDIX B3

DISCHARGE MESUREMENT NOTES- (YEAR 2003)

RIVER: Ososomo PERIOD: Dry Season

DATE: 24/03/03 LOCATION: Abudu

Distance from initial point (m)	Depth (m)	No. of observation (m)	No. of Rev.	Time (Sec.)	Rev./Sec.	Velocity (m/s)		Area (m^2)	Discharge (m^3)
						At point	Mean in vertical		
0	0	0							
2	0.95	0.6	40	10	4.0	0.456	0.456	0.95	0.433
4	1.52	0.6	78	10	7.8	0.889	0.889	7.5	6.668
OBSTRUCTION BY BRIDGE PIER									
12	1.9	0.6	45	10	4.5	0.513	0.513	9.5	4.871
14	2.0	0.2	76	10	7.6	0.114			
		0.8	15	10	1.5	1.30	0.707	4.0	2.828
16	1.0	0.2	102	10	10.2	1.163			
		0.8	30	10	3	0.342	0.7525	6.0	6.02
18	4.1	0.2	98	10	9.8	0.114			
		0.8	14	10	1.4	1.564	0.839	8.0	6.712
20	7.3	0.2	68	10	6.8	0.775			
		0.8	14	10	1.4	0.160	0.4675	14.6	6.826
22	6.5	0.2	91	10	9.1	1.04			
		0.8	30	10	3.0	0.342	0.691	13.0	8.983
24	2.7	0.2	85	10	8.5	0.97			
		0.8	11	10	1.1	0.125	0.548	5.4	2.959
26	1.8	0.2	[illegible]	10	9.5	1.08			
		0.8	20	10	2.0	0.23	0.655	5.1	[illegible]
30m									

Time Start: ...9.30am... **Finish:** ...10.45am

Method:......Bridge...Boom...

Weight Used: ...25kg..............................

Discharge: 12.8 m^3/s..............................

APPENDIX B4

DISCHARGE MEASUREMENT NOTES (YEAR 2003)

RIVER: Ossiomo **PERIOD:** Wet Season

DATE: 22/05/03 **LOCATION:** Abudu

Distance from initial point (m)	Depth (m)	No. of observation (m)	No. of Rev.	Time (Sec.)	Rev./Sec.	Velocity (m/s)		Area (m^2)	Discharge (m^3)
						At point	Mean in vertical		
2	1.24	0.6	60	10	6.0	0.684	0.684	1.24	0.848
4	1.95	0.6	25	10	2.5	0.285	0.342	9.75	2.78
OBSTRUCTION BY BRIDGE PIER									
12	2.0	0.6	93	10	9.3	1.060	1.026	10.0	10.60
14	3.4	0.2	63	10	6.3				
		0.8	30	10	3.0	0.530	0.57	6.8	3.604
16	5.1	0.2	70	10	7.0				
		0.8	30	10	3.0	0.559	0.627	10.2	5.7018
18	7.0	0.2	80	10	8.0				
		0.8	30	10	3.0	0.679	0.587	11.0	8.778
20	9.4	0.2	58	10	5.8				
		0.8	95	10	9.5	0.872	0.741	18.8	16.394
22	8.2	0.2	75	10	7.5				
		0.8	28	10	7.8	0.587	0.587	16.4	9.627
24	5.4	0.2	60	10	6.0				
		0.8	25	10	7.5	0.485	0.485	10.8	5.238
26	3.1	0.2	90	10	9.0				
		0.8	20	10	2.0	0.627	0.627	9.3	5.8311
30m									

Time Start: 11 am **Finish:** 12 noon

Method: Bridge Boom

Weight Used: 25kg

Discharge: 19.5 m^3/s

APPENDIX B5

DISCHARGE MEASUREMENT NOTES-(YEAR 2003)

RIVER: Owaoma **PERIOD:** Wet Season

DATE: 22/06/03 **LOCATION:** Abuda

Distance from initial point (m)	Depth (m)	No. of observation (m)	No. of Rev.	Time (Sec.)	Rev./Sec.	Velocity (m/s)		Area $(m)^2$	Discharge $(m)^3$
						At point	Mean in vertical		
	0								
2	1.32	0.6	60	10	6.0	0.684	0.684	1.32	1.149
4	1.98	0.6	30	10	3.0	0.324	0.342	2.3	
OBSTRUCTION BY BRIDGE PIER									
12	2.20	0.6	90	10	9.0	1.026	1.026	11.0	
14	3.5	0.2	60	10	6.0	0.684			9.24
		0.8	40	10	4.0	0.456	0.57	7.0	
16	5.8	0.2	80	10	8.0	0.912			6.510
		0.8	30	10	3.0	0.342	0.627	11.6	
18	7.8	0.2	75	10	7.5	0.855			8.617
		0.8	28	10	2.8	0.319	0.587	15.6	
20	9.55	0.2	90	10	9.0	1.026			10.48
		0.8	40	10	4.0	0.456	0.741	19.10	
22	8.9	0.2	60	10	6.0	0.684			13.67
		0.8	25	10	2.5	0.785	0.485	17.8	
24	5.82	0.2	80	10	8.0	0.912			10.15
		0.8	30	10	3.0	0.324	0.627	11.64	
26	3.84	0.2	95	10	9.5	1.083			5.196
		0.8	25	10	2.5	0.285	0.684	11.52	
30									

Time Start: ...11.00am... **Finish:**...12.00 noon.

Method:......Bridge...Boom...

Weight Used:...25kg.................................

Discharge:...26.2 m^3/s.....................................

APPENDIX B6

DISCHARGE MESUREMENT NOTES- (YEAR 2003)

RIVER: Ossiomo **PERIOD:** Wet Season

DATE: 22/07/03 **LOCATION:** Abudu

[illegible]nce from point (m)	Depth (m)	No. of observation (m)	No. of Rev.	Time (Sec.)	Rev./Sec.	Velocity (m/s)		Area $(m)^2$	Discharge $(m)^3$
						At point	Mean in vertical		
2	1.68	0.6	60	10	6.0	0.684	0.684	1.68	1.149
4	2.3	0.2	80	10	8.0	0.91			
		0.8	20	10	2.0	0.22	0.565	11.5	6.498
OBSTRUCTION BY BRIDGE PIER									
12	3.5	0.2	60	10	6.0	0.68			
		0.8	33	10	3.3	0.476	0.528	17.5	9.24
14	5.2	0.2	80	10	8.0	0.912			
		0.8	36	10	3.0	0.34	0.626	10.4	6.510
16	7.23	0.2	65	10	6.5	0.74			
		0.8	40	10	4.0	0.456	0.998	11.46	[illegible]
18	8.0	0.2	70	10	7.0	0.798			
		0.8	45	10	4.5	0.513	0.655	16.0	10.48
20	9.6	0.2	80	10	8.0	0.912			
		0.8	45	10	4.5	0.513	0.712	19.2	13.67
22	8.9	0.2	70	10	7.0	0.789			
		0.8	50	10	5.0	0.57	0.684	17.8	10.15
24	6.0	0.2	56	10	5.6	0.638			
		0.8	20	10	2.0	0.228	0.433	12.0	5.196
26	10	0.2	80	10	8.0	0.912			
		0.8	53	10	5.3	0.604	0.76	12.0	9.12

Time Start : 10.00am **Finish :** 11.00am

Method : Bridge Boom

Weight used : 25kg

Discharge : 28.6 m^3/s

APPENDIX C1

DESIGN WORKS -MAIN ELEMENTS OF WATER SUPPLY

(1) Design Parameters

(i) Design Population for Abudu Town

Design period =25yrs

Growth rate =3%per annum (National Population Com. Benin)

Population Figure =11271 (1991 Census)

Estimated population

(Current year -2003) $=11271\ (1+3/100)^{12}=16070$

Estimated population for year 2028

(25years period) $=16\ 070(1+3/100)^{12}=33650$

Hence Design population is approximately **34000** people

(ii) Total Water Requirement

Unit Water Consumption = 120 Litres/ Capita/ Day (Edo water board)

Total Domestic Demand = 120 x 34000 = $4080m^3$/day

Allowing for non-domestic consumption. 30% = 1228 m^3/day

Total dom. + Non Dom. = 5308(m^3/day)

Add waste/leakages 25% = 1326

Total Avg. Daily Demand = 6334 (m^3/day)

∴ Average daily demand = 6334 (m^3/day)

Using a peaking factor of 120%

∴ Peak day demand = 1.20 (6634) = **7961(m^3/day)**

APPENDIX C2

Using 1 day of 16hrs pumping (6 am to 10 pm)

The peak day flow rate = 7961 / 16

∴ Design flow rate = **498m^3/hr or 0.138m^3/s**

(2) Intake Structure (Water Well)

Design Criteria :

Flow rate (Q) = 498m^3/hr

Detention time = 20min.

Dept of water = 4m

Shape = Rectangular (2:1)

Calculations:

Vol. of water = $\frac{20}{60} \times \frac{198}{1} = 166m^2$

w $= \frac{\sqrt{166}}{8}$ = 4.5m

∴ L = 9m.

Use well size of 4.5 x 9m x 5m (Fig 6.1, 6.2)

(3) Main Components of Water Treatment Plant.

(i) Coagulation /Flocculation Tank

Design Criteria:

Detention time = 20min.

Flow rate = 8.3m^3/min.

Dept of water = 3min

Shape = Rectangular (2:1)

Vol. of water = $20 \times 8.3 = 166m^3$

Use two tanks

Hence capacity of water in each tank = $83m^3$

$W = \sqrt{\frac{83}{6}} = 3.7m$

∴ L = 7.4m.

Use tank size of 4m x 8m x 3.5m

(ii) Sedimentation Tank

Design Criteria:

Detention = 2hrs

Flow rate = $498m^3/hr$

Dept of water = Rectangular (2:1)

Calculations;

Vol. of water = $2 \times 4 = 996m^3$

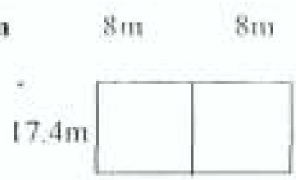
Use two tanks.

Hence capacity of each $498m^3$

Let one side be same with coagulation/ flocculation tank =8m (width)

∴ Length = $\frac{498}{8 \times 3.6} = 17.4m$

Use tank size of 8m x 17.4m x 4m



APPENDIX C4

(iii) **Rapid Sand Filter**

Design Criteria:

Flow = $498m^3/hr$

Filtration rate = $4m^3/hr/m^2$

Calculations:

Area of bed = $498/4 = 125m^2$

Use two filter beds.

$\therefore$ Each = $125/2 = 62.5m^2 = 63m^2$

Let L = 8m

W x 8 = $63m^2$

$\therefore$ W = 7.9m = 8m

Use 8m x 8m Filter Bed

Filter Media:

500mm layer of anthracite medium

250mm layer of graded gravel.

400mm layer of graded gravel

APPENDIX C5

(i) Clear Well

Design Criteria:

Provide 30% of daily output (Linsley and Franzini, 1979)

Flow rate	=	$498m^3/hr.$
Pumping for 16hrs. daily output	=	$7968m^3$
Shape of well	=	Rectangular (1.2 : 1.5)

Calculations:

Storage Capacity	=	$(30/100)(7968) = 2390m^3$
Let dept	=	4m
Using two tanks,		
Vol. for each tank	=	$1195m^3$
$\therefore 125w^2 4$	=	$1195m^3$
w^2	=	1195/5
hence W	=	15.5m.
and L	=	19.4m.

Use tank size (two numbers) of – 16m x 20m x 4m.

APPENDIX C6

(4) Pipe Line- Net Work Design

Design Data for Network Analysis

(i) Proportionality factor (k) $= 0.81fl/gD^5$

(ii) Frictional coefficient (f) $= 0.01$

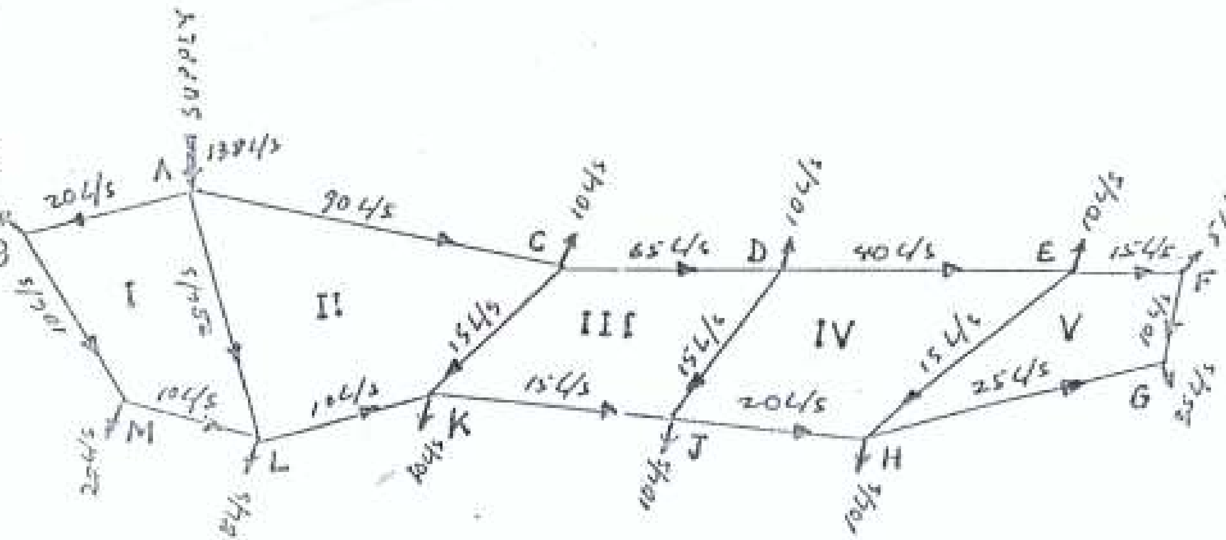
(iii) Head Loss $= kQ^{1.85}$

(iv) Correction factor for each Loop Δ (l/s) $= \dfrac{-\sum k\,Q^x}{\sum(x\,k\,Q^{x-1})}$

APPENDIX C7

Simplified Net Work Diagram –Showing Initial allocation of Flows

and Loop Identifiers. -



APPENDIX C8

Initial Allocation of flows and Loop Identifiers

Pipe	Pipe size (mm)	Pipe length (m)	Discharge Q (l/s)	Loop
A> B	300	500	20	(I)
BM	300	600	10	
ML	300	350	10	
LA	200	800	28	
AL	200	800	28	(II)
LK	300	500	10	
KC	200	550	15	
CA	300	200	70	
CK	200	550	15	(III)
KJ	300	580	15	
JD	200	600	15	
DC	300	1000	65	
DJ	200	600	15	
JH	700	750	20	(IV)
HE	200	450	15	
ED	300	800	40	
EH	700	450	15	
HG	300	500	25	(V)
GF	200	420	10	
FE	300	450	15	

APPENDIX C9

Flow Corrections:

First Trial Corrections

1st Correction = Loop (I)

Pipe	Pipe Size (mm)	Pipe Length (m)	Discharge Q l/s	K
AB	300	500	29	170
BM	300	600	19	1548
ML	300	350	-1	119
LA	200	800	-19	2064

Δ1 = 9 l/s

1st Correction = Loop (II)

Pipe	Pipe Size (mm)	Pipe Length (m)	Discharge Q (l/s)	K
AL	200	800	27.5	2064
LK	300	500	18.5	170
KC	200	550	- 6.5	1419
CA	300	1200	- 61.5	408

Δ= 8.5 l/s

1st Correction Loop (III)

Pipe	Pipe Size (mm)	Pipe Length (m)	Discharge Q (l/s)	K
CK	200	550	12	1419
KJ	300	580	20.4	1917
JD	200	600	- 9.6	1548
DC	300	1000	- 59.6	340

Δ 1 = 5.4 l/s

1st Correction Loop (IV)

Pipe	Pipe Size (mm)	Pipe Length (m)	Discharge Q (l/s)	K
DJ	200	600	11.8	1548
JH	300	750	22.2	255
HE	200	450	- 12.8	1161
ED	300	800	- 37.8	272

Δ= 2.2 l/s

APPENDIX C10

1st Correction Loop (V)

Pipe	Pipe Size (mm)	Pipe I Length (m)	Discharge Q (l/s)	K
FH	200	450	8.8	1161
HG	300	800	21	272
GF	200	420	-14	1083
FE	300	450	-19	153

Δ_1 = -4 l/s

Second Trial Corrections

2ND Correction Loop (I)

Pipe	Pipe Size (mm)	Pipe Length (m)	Discharge Q l/s	K
AB	300	500	33.7	170
BM	700	600	23.7	1548
ML	300	350	-3.7	119
LA	200	800	-22.8	2064

.7 l/s

2ND Correction Loop (II)

Pipe	Pipe Size (mm)	Pipe Length (m)	Discharge Q l/s	K
AL	200	800	28.4	2064
LK	300	500	24.1	170
KC	200	550	-41.9	1119
CA	300	1200	-75.9	108

Δ_2 = 5.6 l/s

APPENDIX C11

Correction Loop (111)

Pipe	Pipe Size (mm)	Pipe Length (m)	Discharge Q l/s	K
CK	200	550	14.2	1419
KJ	300	580	22.6	1917
JD	200	600	- 9.6	1548
DC	30	1000	- 57.4	340

Δ = 2.2 l/s

2nd Correction Loop (IV)

Pipe	Pipe Size (mm)	Pipe Length (m)	Discharge Q (l/s)	K
DJ	200	600	11.1	1548
JH	300	750	23.7	255
HE	200	450	- 7.3	1161
ED	300	800	- 36.3	272

Δ=1.5 l/s

2ND Correction Loop (V)

Pipe	Pipe Size (mm)	Pipe Length (m)	Discharge Q l/s	K
EH	200	450	7.5	1161
HG	300	800	22.4	272
GF	200	420	-12.6	1083
FE	300	450	-17.6	153

Δ = +0.8 l/s

APPENDIX C12

Third Trial Corrections

3rd Correction Loop (I)

Pipe	Pipe Size (mm)	Pipe Length (m)	Discharge Q l/s	K
AB	300	500	29.2	170
BM	300	600	26.7	1548
ML	300	350	11.7	119
LA	700	800	32.9	2064

$\Delta_3 = 3\ l/s$

3rd Correction Loop (II)

Pipe	Pipe Size (mm)	Pipe Length (m)	Discharge Q l/s	K
AL	700	800	36.7	2064
LK	300	500	27.9	170
KC	700	550	10.4	1419
CA	300	1200	72.1	498

Δ 3.8 l/s

APPENDIX C13

3rd Correction Loop (111)

Pipe	Pipe Size (mm)	Pipe Length (m)	Discharge Q (l/s)	K
CK	200	550	10.6	1419
KJ	300	580	22.8	1917
JD	700	600	- 10.9	1548
CD	300	1070	- 51.5	340

$\Delta_3 = 0.23\ l/s$

3rd Correction Loop IV)

Pipe	Pipe Size (mm)	Pipe Length (m)	Discharge Q (l/s)	K
DJ	200	600	12.1	1508
JH	300	750	30.6	253
HE	200	450	- 6.9	1161
ED	300	800	- 29.4	272

$\Delta = 1.2\ l/s$

3rd Correction Loop (V)

Pipe	Pipe Size (mm)	Pipe Length (m)	Discharge Q (l/s)	K
EH	200	450	7.5	1161
HG	300	800	28.1	272
GF	200	420	6.9	1083
FE	300	450	11.9	153

$\Delta_3 = 0.6\ l/s$

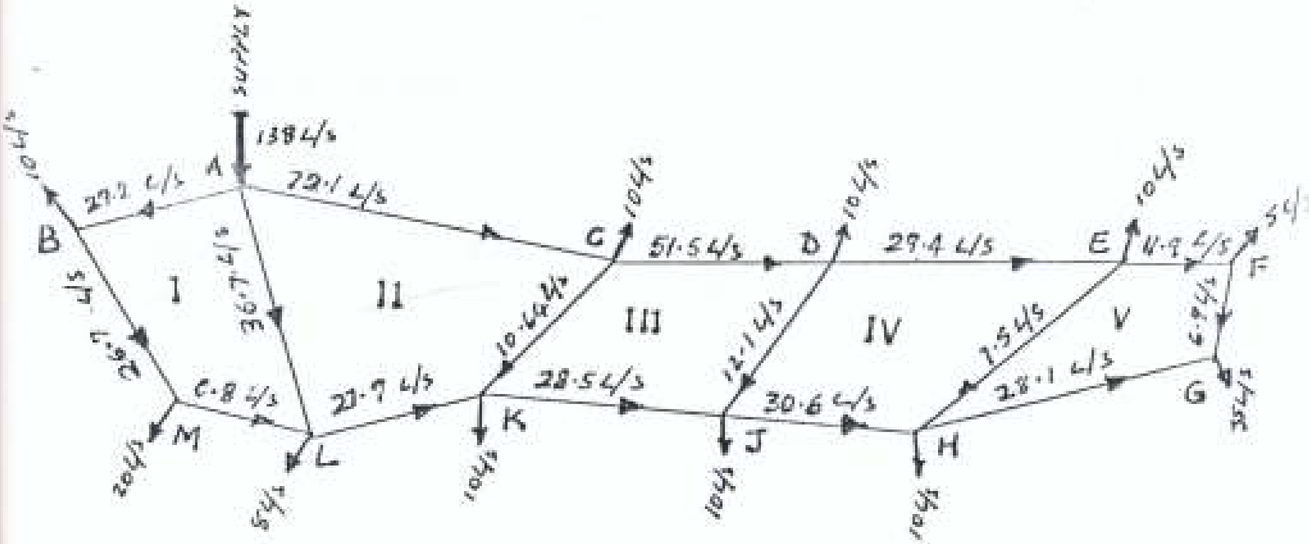
APPENDIX C14

Final Values of Flow: -

Final Values (Flows in direction of pipe Identifiers; e.g., A→ B)

Pipe	Q (l/s)	h_L (m)
AB	29.2	1.1
BM	26.7	2.2
AL	36.7	2.9
LM	0.8	2.4
AC	72.1	3.0
CK	10.6	3.0
LK	27.9	2.0
CD	51.5	5.0
DJ	12.1	5.5
KJ	28.5	7.5
DE	29.4	3.0
EH	7.5	7.2
JH	30.6	4.2
EF	11.9	3.5
FG	6.9	7.5
HG	28.1	3.8

APPENDIX C15



Simplified Network Diagram - Indicating Final Values of Flows and Directions

APPENDIX C16

Calculations of Total Heads and Pressure Heads at Nodes

The Conditions at the Distribution ends are:

(i) The Service Reservoir is 15m high from the ground level and the depth of water column is 5m hence the static pressure head at the starting point A is given by (15+5)m = 20m.

(ii) The maximum ground elevation = 308m o.d

The minimum ground elevation = 295m o.d

(iii) Pressure Head elevation at A = (308 + 20) = 328m o.d

Elevation of Pipe Nodes: –

Node	A	B	C	D	E	F	G	H	I	J	K	L
Elevation (m.o.d.)	308	308	307	305	304	301	295	297	300	305	306	305.6

(Source: Spot Levels carried out by the Author)

Pressure Heads at Nodes:-

(i) Node A

Elev. at A = 308m o.d

Pressure Head = 20.5m

Total Head = 328m o.d.

(ii) Node B

Elev. at B =308m o.d. h_L = 1.1m

Pressure Head = (328 – 308 -1.1) = 18.9m

Total Head = 326.9m o.d

APPENDIX C17

(iii) **Node L**

Elev.at L = 306 m o.d. h_L = 2.9m

Pressure Head = (326.9- 306 -2.9) = 18m

Total Head = 324 m o.d.

(iv) **Node M**

Elev.at M = 305.6m o.d. h_L =2.4m

Pressure Head = (324 – 305.6 – 2.4) = 16m

Total Head = 321.6m o. d

(v) **Node C**

Elev.at C = 307m o.d. h_L= 3.0m

Pressure Head = (328 – 307 – 3) = 18m

Total Head = 325m o.d.

(vi) **Node K**

Elev.at K = 305m o.d. h_L = 3.0m

Pressure Head = (325 – 305 – 3) = 17m.

Total Head = 322m o.d.

(vii) **Node D**

Elev.at D = 305m o.d. h_L = 5.0m

Pressure Head = (325 – 305 –5) = 15m

Total Head = 320m o.d.

APPENDIX C18

(viii) Node J

Elev.at J = 300m o.d. h_L = 5.5m

Pressure Head = (320 – 300 – 5.5) = 14.5m

Total Head = 314.5m o.d.

(ix) Node E

Elev.at E = 304m o.d. h_L = 3.0

Pressure Head = (320 – 304 – 3) = 13m

Total Head = 317m o.d.

(x) Node H

Elev.at H = 297m o.d. h_L = 4.2m

Pressure Head = (314 – 297 – 4.2) = 12.8m

Total Head = 309.8m o.d.

(xi) Node F

Elev.at F =301m o.d. , h_L = 3.5m

Pressure Head = (317 – 301 - 3.5) = 12.5m

Total Head = 313.5m o.d.

(xii) Node G

Elev.at G = 295m o. d. h_L = 3.8m

Pressure Head = (309.8 – 295 – 3.8) = 11m

Total Head = 306m o.d.

APPENDIX C19

Pipe Line Design Summary Table

(Flow in direction of Pipe Identifier; e g. A → B)

Final Values

Pipe Line (m)	Pipe Size (mm)	Pipe Length (m)	Flow Q(l/s)	Head Loss (m)	Node	Elevation (m o.d)	Pressure Head (m)
A → B	300	500	36.7	1.1	A	312.38	20
B M	300	600	26.7	2.2	B	310.34	18.9
A L	200	800	29.2	2.9	C	302.48	18.0
L M	300	350	0.7	2.4	D	305.12	15.0
A C	300	1200	72.1	3.0	E	302.48	13.0
C K	200	550	10.6	3.0	F	298.45	12.8
L K	300	500	[illegible]	2.5	G	295.35	11.0
C D	300	1000	57.2	5.0	H	298.13	12.8
D J	200	600	12.1	5.5	J	299.81	14.5
K J	300	580	22.8	7.5	K	301.21	17.0
D E	300	800	35.1	3.0	L	309.12	18.0
E H	200	450	7.5	7.2	M	296.37	16.0
J H	300	750	24.9	4.2			
E F	300	450	17.6	3.5			
F G	200	420	12.6	7.5			
H G	300	800	22.4	3.8			

APPENDIX C20

Remark On Design of Pipeline System:-

Design is satisfactory since:-

(i) Adequate pressure prevailed at all nodes of operation-***minimum pressure***, not less than 1 bar (10.2m) in the Pipe Line Net-work.

(ii) There is no indication of negative pressure at any node in the Network.

APPENDIX D1

Consultancy Fees: The New Professional Scale of Fees for Consultants in the Construction Industry

Prime Consultant

Cost of project.	Fees Payable as a Percentage of Projects.
Up to 5million	4.75%
Next 10 million or part thereof	4.55%
Next 15 million or part thereof	4.25%
Next 45 million or part thereof	4.0%
Next 75 million or part thereof	3.5%
Next 150 million or part thereof	3%
Next 200 million or part thereof	2.5%
Balance over 500 million	1.75%

Stage payment for all categories of professional are:

Stage 1 = 25% of Estimated total cost (ETC)

Stage 2 = 50% Estimated total cost (ETC)

Stage 3 = 25% of Total construction sum (TCS)

(Source: Federal Ministry of Works and Housing, 1996)

APPENDIX D2

Consultancy Fees: The New Professional Scale of Fees for Consultants in the Construction Industry

Structural Engineers

Cost of project	Fees Payable as a Percentage of Projects.
Up to 5 million	3.0%
Next 10 million	2.50%
Next 15 million	2.25%
Next 45 million	2.0%
Next 75 million or part thereof	1.75%
Next 150 million or part thereof	1.50%
Next 200 million or part thereof	1.25%
Balance over 500 million	1.0%

Stage payment for all categories of professional are:

Stage 1 = 25% of Estimated total cost (ETC)

Stage 2 = 50% Estimated total cost (ETC)

Stage 3 = 25% of Total construction sum (TCS)

(Source: Federal Ministry of Works and Housing, 1996)

APPENDIX D3

Consultancy Fees: The New Professional Scale of Fees for Consultants in the Construction Industry

Mechanical/Electrical Engineers

Cost of Project	Fees Payable as a Percentage of Projects.
Up to 5 million	1.95%
Next 10 million or part thereof	1.75%
Next 15 million or part thereof	1.55%
Next 45 million or part thereof	1.35%
Next 75 million or part thereof	1.15%
Next 150 million or part thereof	1.00%
Next 200 million or part thereof	0.85%
Balance over 500 million	0.65%

Stage payment for all categories of professional are:

Stage 1 = 25% of Estimated total cost (ETC)

Stage 2 = 50% Estimated total cost (ETC)

Stage 3 = 25% of Total construction sum (TCS)

(Source: Federal Ministry of Works and Housing, 1996)

APPENDIX E 1

Bill of Quantities (Abudu Water Scheme)

Estimated Total Cost (ETC)

S/No	Description of Item	Unit	Rate	Amount ₦ : K (million)
A	Mobilisation and demobilisation of men, materials and equipment.	L/S		3m
B	Intake works, consisting of Civil works-bulldozing of Access Roads, Construction of Intake well, Pump house, etc.	L/S		35m
C	Water treatment plant, consisting of Coagulation / Flocculation units, Sedimentation units, Filter units and Clear well storage units.	L/S		55m
D	Service Reservoir (Braithwaite) Elevated Storage 300m³.	L/S		3m
E	Supply and laying of Pipeline Network (Trunk and Sub-mains).	L/S		80m
F	Distribution System.	L/S		18m
G	Supply and Installation works – Low lift Pumps, High lift Pumps, Generating plants and Control panels.(Electro-Mechanical Components) and Electrification works.	L/ S		5m
H	Chemical dozers and all equipment connected with water treatment.	L/S		8m
I	Chemical / Pumping cost for 1 year.	L/S		6m
J	Construction of roads and Drainage works from Intake to Treatment Plant Premises.	L/S		10m
K	Contingency			2m
	Estimated total cost (ETC)			N 225m

APPENDIX E 2

CALCULATED CONSULTANTS FEES

Prime Consultants (Water Engineers)

4.75% of 5million	**₦ 237,500**
4.5% of 10million	**₦ 450,000**
4.25% of 15million	**₦ 637,500**
4% of 45million	**₦ 1800,000**
3.5% of 75million	**₦ 2,62500**
3% of 75million	**₦ 2250000**

Structural Engineers

3% of 5m	₦ 150,000
2.5% of 10m	₦ 250,000
2.25% of 15m	₦ 337,500
2% of 45m	₦ 90,000
1.75% of 75m	₦ 1,312,500
1.5% of 75m	₦ 1125000

Mechanical / Electrical Engineers

1.95% of 5m	₦ 97,500
1.75% of 10m	₦ 175,000
1.55% of 15m	₦ 232,500
1.35% of 45m	₦ 607,500
1.15% of 75m	₦ 862,500
1% of 75m	₦ 750000

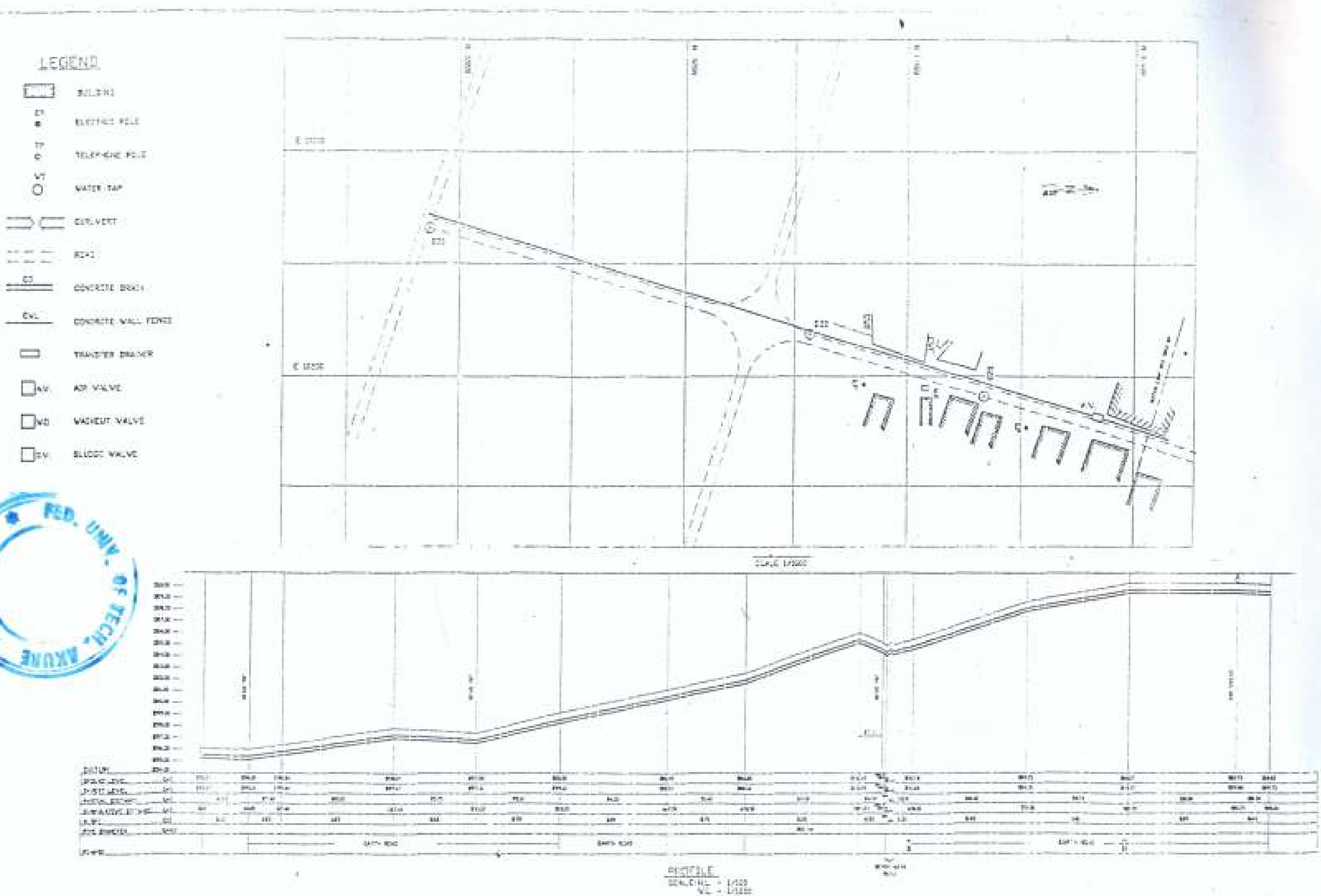
Reinbursable Expenses:

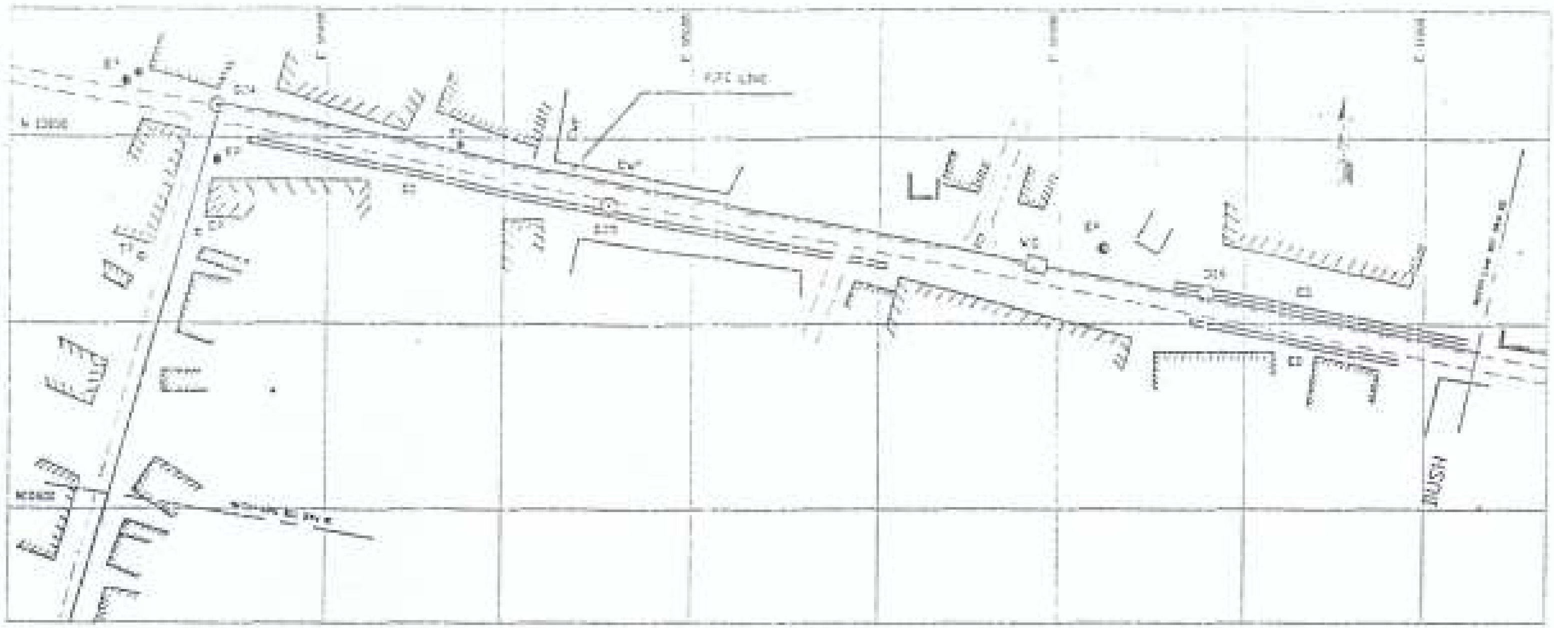
Maximum=1% of ETC

Take 0.2% of ETC for all Consultants = 0.2 of 225m	₦ 450000
Total cost of project	**₦ 240,000,000m**

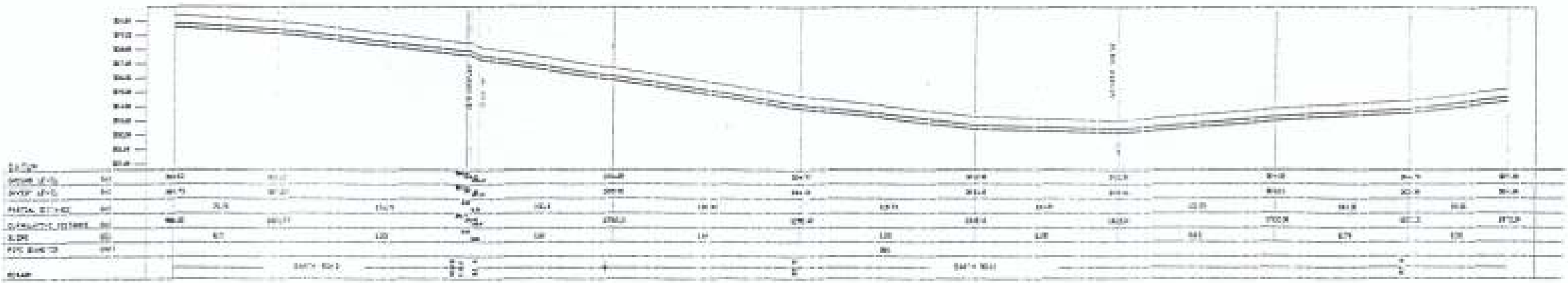
APPENDIX F (DRWG 1-7)

TRANSVERSE SURVEY DRAWINGS OF THE PIPE LINE ROUTES

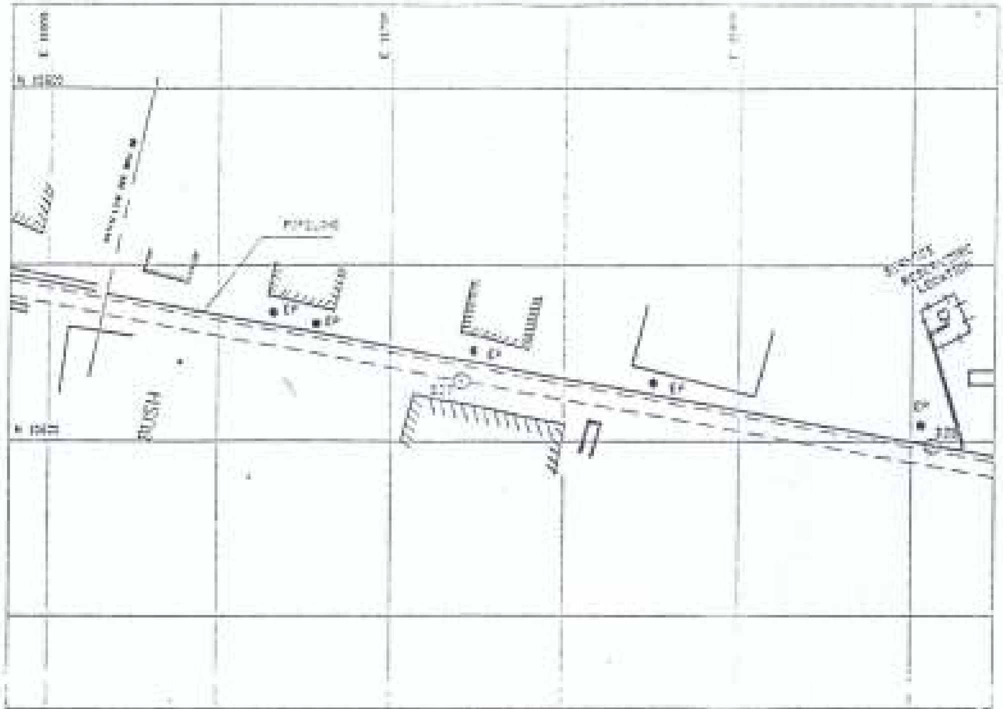

LEGEND
BUILDING
ELECTRIC POLE
TELEPHONE POLE
WATER TAP
CULVERT
ROAD
CONCRETE DRAIN
CONCRETE WALL FENCE
AIR VALVE
WASHOUT VALVE
BLOCK VALVE
SCALE 1/5000
DATUM
GROUND LEVEL
INVERT LEVEL
EARTH ROAD
PROFILE



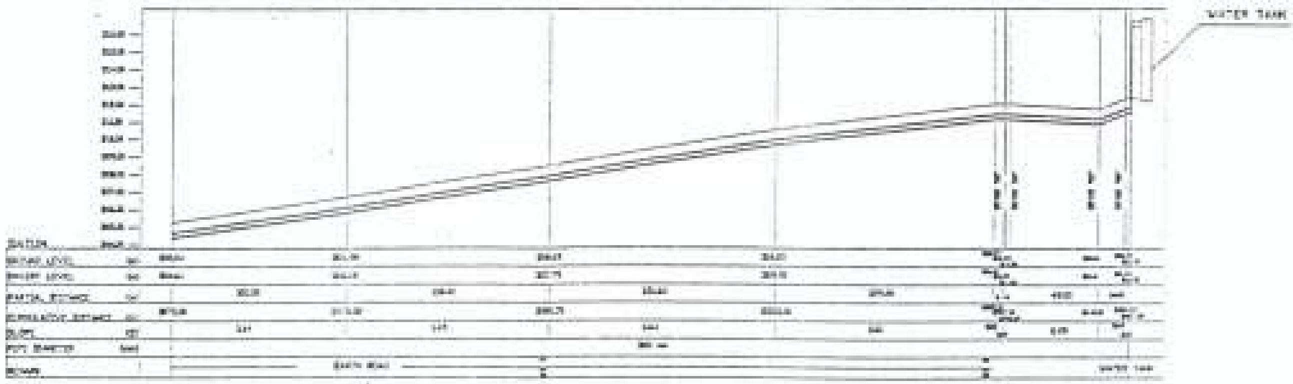
SCALE 1/1000



PROFILE



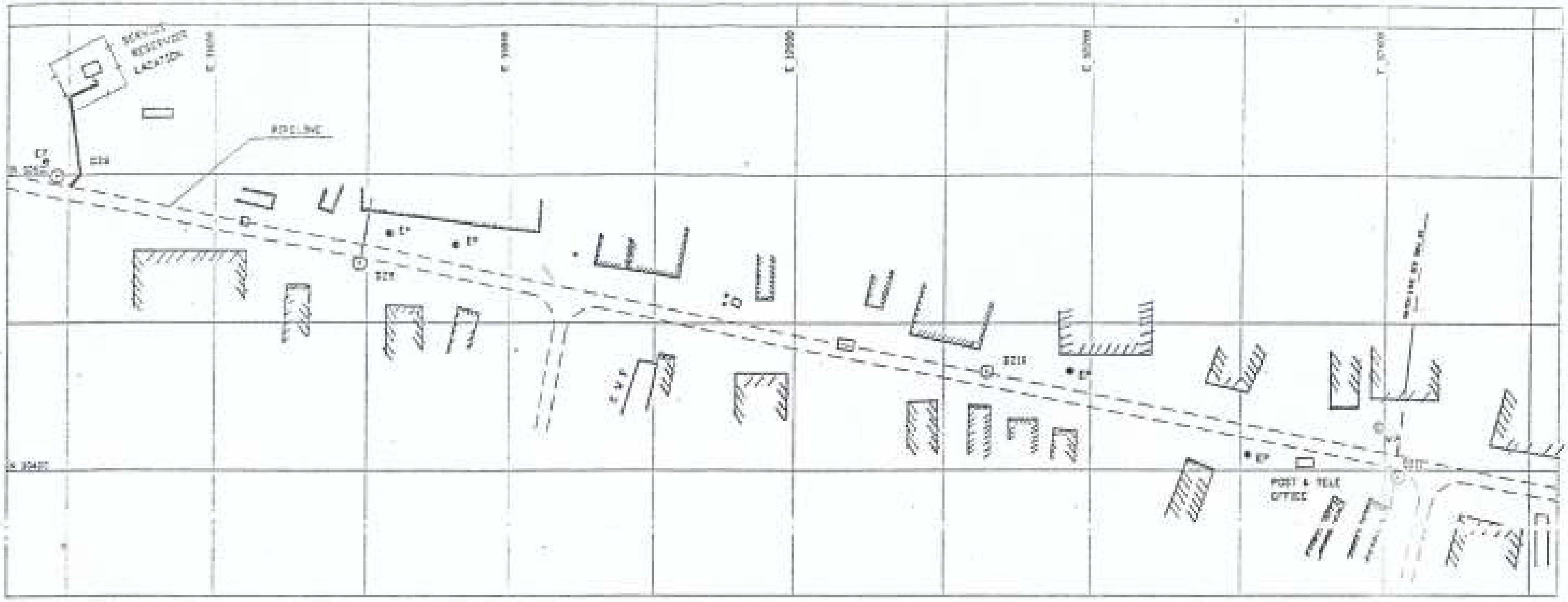
LAYOUT PLAN



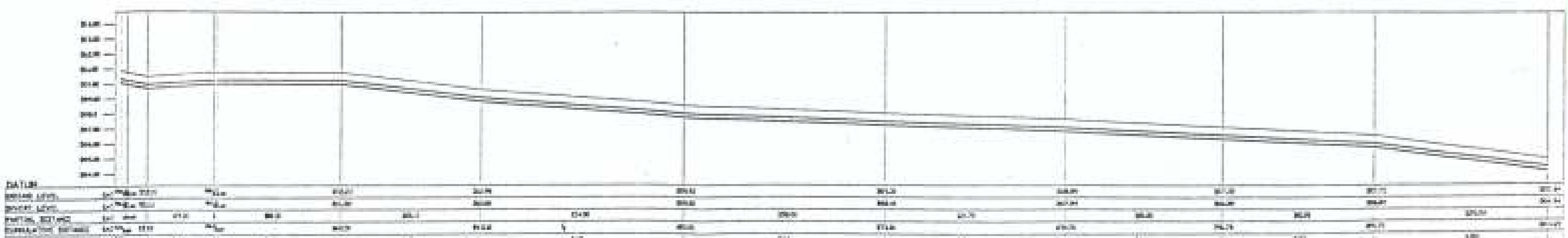
PROFILE

NOTE:

DWG 03



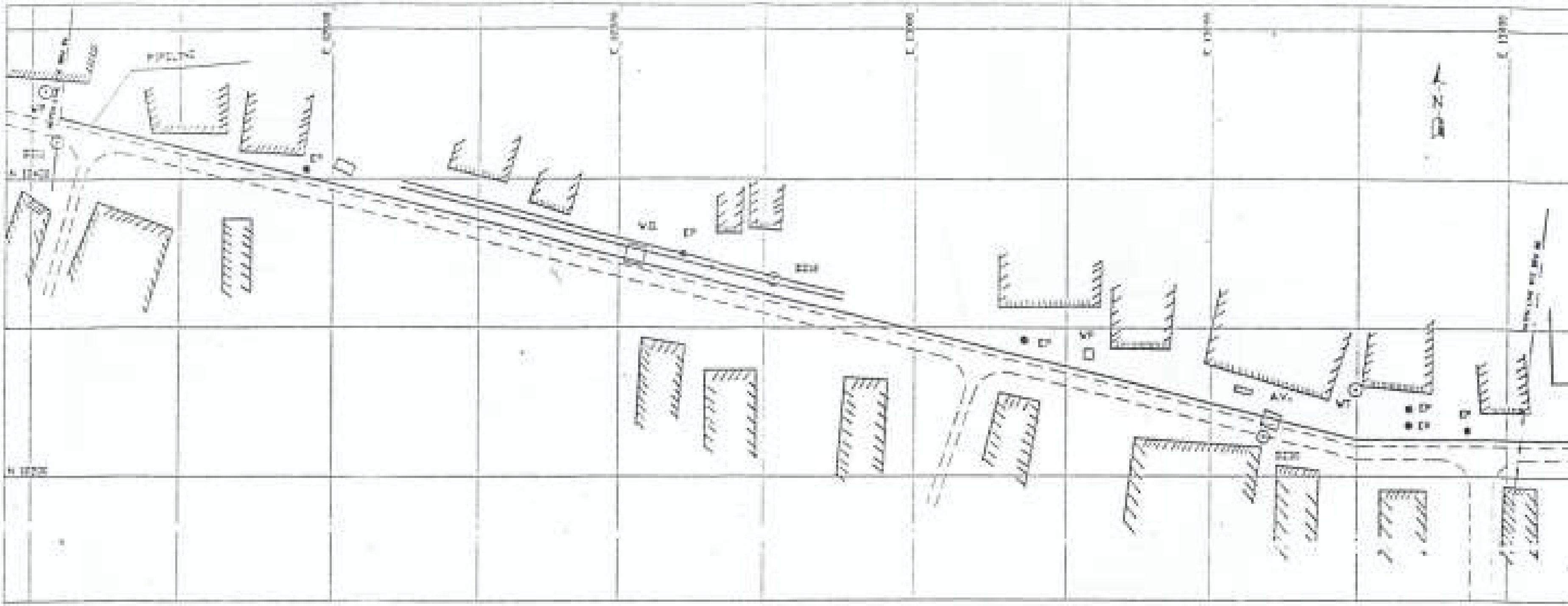
LAYOUT PLAN
SCALE 1/2000



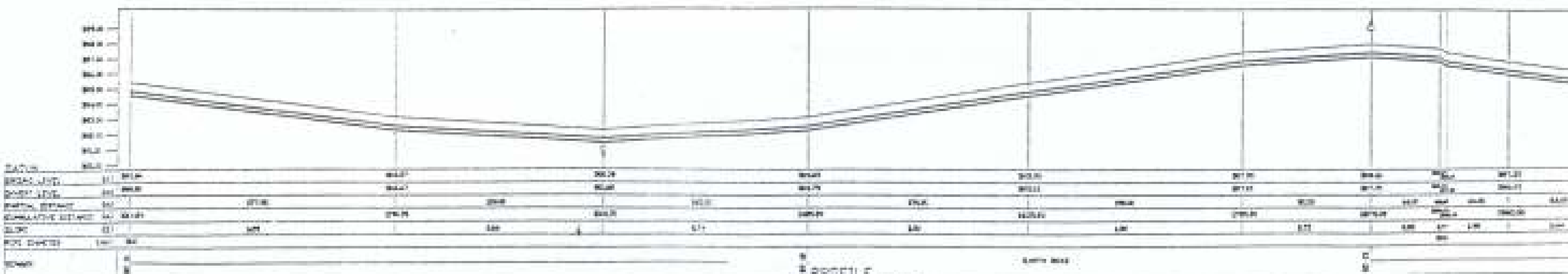
PROFILE

NOTE

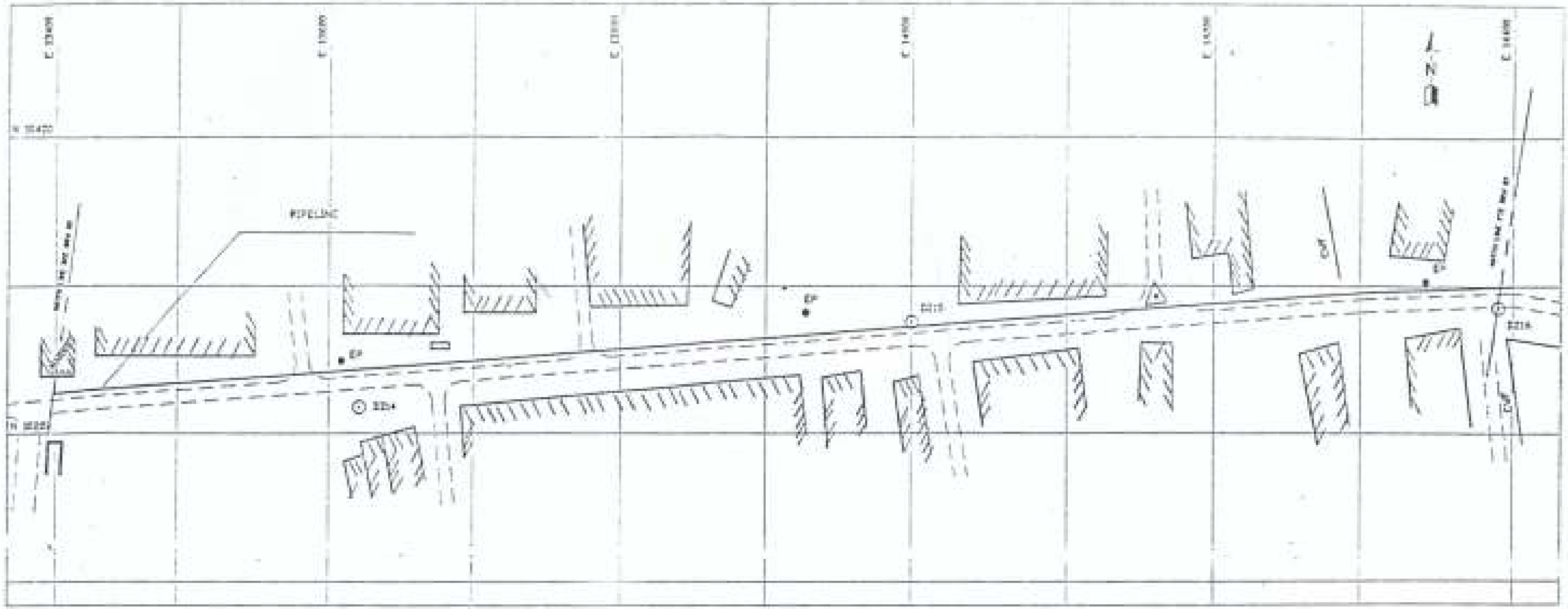
DWG C4



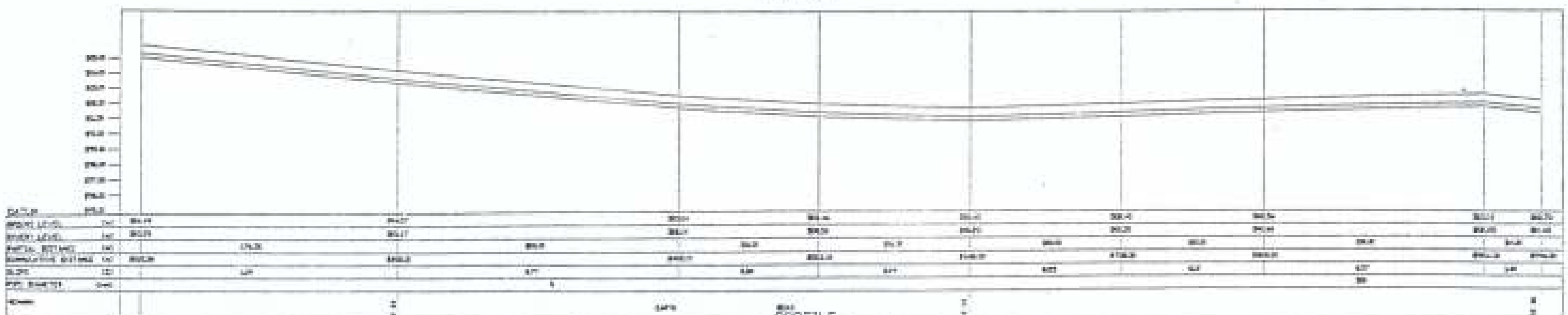
LAYOUT PLAN
SCALE 1/2000



PROFILE
SCALE-H.L. = 1/200
V.L. = 1/2000



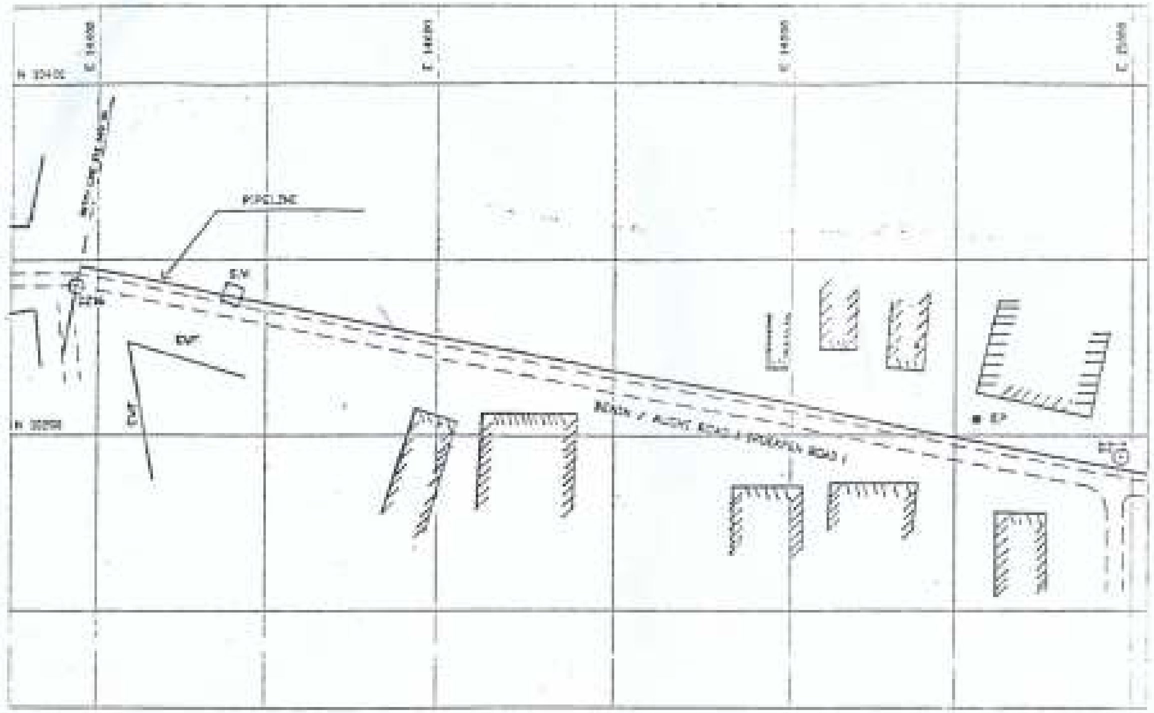
LAYOUT PLAN
SCALE 1/2000



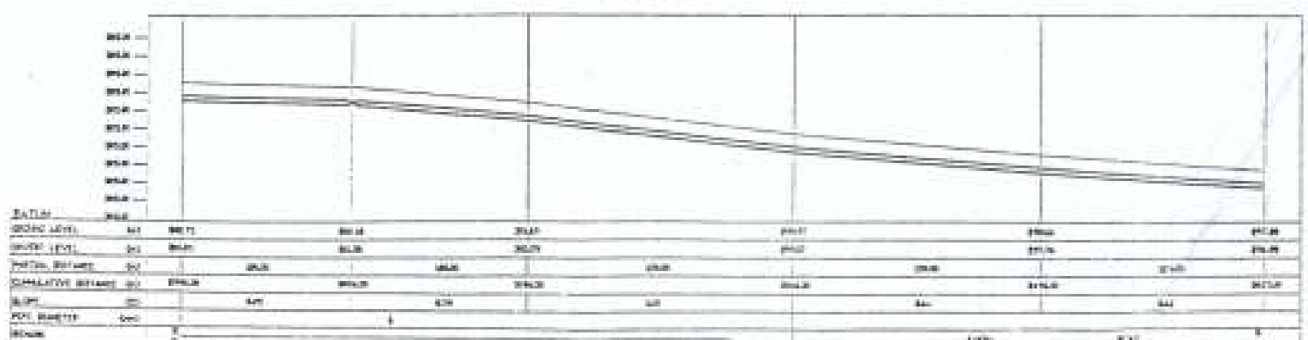
PROFILE
SCALE:H.L. = 1/100
V.L. = 1/2000

NOTE:
1. ALL DIMENSION ARE IN METERS
UNLESS OTHERWISE INDICATED

DWG 06



LAYOUT PLAN



PROFILE

NOTE:
1. ALL DIMENSION ARE IN METERS