OPTIMAL ALLOCATION AND DISTRIBUTION OF WATER SUPPLY: Case Study of Ekiti and Ondo States

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A THESIS IN THE DEPARTMENT OF CIVIL ENGINEERING , SUBMITTED TO THE SCHOOL OF POSTGRADUATE STUDIES FOR THE AWARD OF THE DEGREE OF DOCTOR OF PHILOSOPHY IN CIVIL ENGINEERING (WATER RESOURCES OPTION)

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DECLARATION

I, Josiah Oladele Babatola, declare that this thesis was Prepared entirely by me under the supervision of Dr. A. M. Oguntuase (Major Supervisor) and Dr. A. A. Olufayo (Co-Supervisor) both of Federal University of Technology, Akure. The thesis has not been presented, either wholly or partly, for any degree elsewhere before.

All sources of information contacted have been duly acknowledged.

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CERTIFICATION

This thesis entitled 'Optimal Allocation and Distribution of Water Supply: A Case Study of Ekiti and Ondo States by Josiah Oladele Babatola meets the regulations governing the award of the Degree of Doctor of Philosophy (Ph.D) in Civil Engineering of the Federal University of Technology, Akure, and is approved for its contribution to knowledge and literary presentation.

....	....
Dr. A. M. Oguntuase 13/5/04	Dr. A. A. Olufayo 13/5/04
Major Supervisor	Co- Supervisor

DEDICATION

This work is dedicated to the Almighty God and to my lovely wife – Olayinka and children – Olanrewaju, Oluwatosin and Monisola.

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ABSTRACT

Many of the Nigerian urban centres are facing erratic water supply due to reasons ranging from socio-economic to technical problems. Urban centres in Ondo and Ekiti States are also faced with these serious water supply problems.

This research work carried out socio-economic appraisal the water supply situation of the study area and technical appraisal of the existing water schemes. Results of the appraisal were used to develop two models viz:

Allocation of water to demand centres in the most economical way and transmission of water from one headwork to another in the most optimal way to ensure efficient use of the existing water schemes in the study area.

To find the best radius or diameter, the construction cost plus the fluid friction loss were expressed in terms of the pipeline size and this function minimized by the procedures of differential calculus.

To express the total cost as a function which can be differentiated, the overall size was characterized by one dimension such as radius. This dimension was treated as a variable for computation but as it varied, all other dimensions of the aqueduct cross-section varied accordingly.

In this work, water was allocated optimally to consumers where demand is greater than the available supply using the equations that were developed. The rate at which water is supplied to the consumers which is the principal determinant of the cost of the system and the least cost of supply was achieved through optimization techniques. When tested with the local cost data, these equations enable prediction of system costs for variety of design conditions and construction materials. Thus the equations provide the basis for decisions on the basic design variables. The models were tested with data from some water supply schemes in Ondo State. Some specific facts which result from our analysis are presented. The results of this study will be useful to designers, researchers, policy makers and nongovernmental organizations (NGOs) that are involved in water supply activities.

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a	=	trench width/pipe radius
a_e	=	area of excavation in canal cross section
a_l	=	area of lining in canal cross section
b	=	trench depth/pipe radius
C	=	cost to construct one unit length of tunnel, pipeline, or canal;
C_e	=	cost to excavate one unit length of canal;
C_l	=	cost of lining in one unit length of canal
C_p	=	cost of pipe in one unit length of pipeline
C_t	=	cost to excavate and backfill one unit length of pipe trench
D_p,	=	design factors pipelines (see Eq. 6.81)
e_p,	=	excavation costs per unit volume for pipelines
E	=	Value of energy lost fluid friction in one unit length of pipe Line per year;
f_p	=	fluid friction factors in Chezy formula for, pipelines, and
g	=	acceleration of earth gravity force;
H	=	hydrographic-shape factor for pressure pipelines
Hg	=	hydrographic-shape factor for gravity flow in pipelines
I	=	annual interest on construction cost plus annual payment needed to retire the debt during useful life of the structure;
k_p	=	thickness of pipe/radius
l_p	=	costs per unit volume of pipe lining
M	=	total cost of one unit length of pipeline per year;

n_p = number of kilogram-meters in 1 kw-hr.

n_t = number of seconds in 1 year;

p = value of unit of energy in flowing liquid, in Naira per kilowatt-hour;

Q = rate of fluid flow in pipeline

Q_a = average rate of fluid flow in pipeline

Q_m = maximum rate of flow in pipeline

q = rate of flow per year, in hectare meters per year;

R = Reynolds number for pipes;

R = hydraulic radius

r = radius of pipelines

s = energy gradient of pipelines

s_g = energy gradient of gravity flow

s_m = energy gradient when flow is maximum;

t = time, in years;

V_e = average volume of excavation in one unit length of trench

v = velocity of flow (average over the cross section);

v_m = maximum velocity of flow (average over cross section)

w = specific weight of liquid being conveyed; and

v = kinematic viscosity of liquid being conveyed

CHAPTER ONE

INTRODUCTION

1.1 General Introduction

Water is a wonder material. It is life and prosperity itself. Life cannot exist without water. With it, life prospers and happiness is prolonged. Remains of old civilization have shown their utter dependence on resources of water. "Water is one of the most valuable natural resources vital to the existence of human life" (Sonuga, 1984). Water is needed for food production, transportation, power and other amenities necessary for human existence. "Water is the symbol of advanced human civilization" (Sen, 1981). It is the only liquid available on the earth's crust under natural conditions and it is simultaneously present in all its three phases on the surface of the earth.

1.2 Problems of Urban Water Supply

The problem of water supply is global but it is much more pronounced in the developing countries. In developing countries, cities often suffer from serious water supply shortages due primarily to uncontrolled and unanticipated growth in urban population, inadequacy in the design of the urban network water supply systems apart from management problems of operation and maintenance and natural disaster.

The design of water supply systems requires an estimate of future demands and the optimal way to meeting these demands. The choice of the design capacity is a function of factors such as population growth and demand, source of funding, the character of the water supply, the qualities and quantities of water (both raw and final product). The general urban water demands consist of residential, industrial, commercial and fire fighting demands and losses incurred in the process of transmission.

Rapid growth in urban population is common to developing countries. The growing demand for water in addition to the limiting nature of the resource is the reason for exploring new trends in designing and managing water supply systems.

In industrialized countries, supplies of potable water are mandated by statutory regulations and therefore considered essential commodities. Hence, the designers of the water supply systems have considerable latitude in deciding how much capacity to provide, guided by estimation of population growth, demand, and financing considerations. Unfortunately developing countries are faced with a variety of problems and so cannot compete in any way with developed nations in this respect. Rather they are struggling to meet the World Health Organization (WHO) minimum standard both in quality and quantity of water due to factors such as high birth rates, excessive migration of people from rural area to urban cities, and high population growth rate of cities. During the past three decades, the populations of many cities in Asia, Africa and Latin America have doubled. This high population growth rate has been accompanied by substantial expansion of city boundaries and even higher levels of industrial and commercial activities (Wright, et al 1986; Clymo, 1989; Yearsley, 1989).

The massive migration of rural populace to urban cities is precipitated by factors ranging from technological advances in agriculture that make manual farm labour unpopular, urban-oriented educational policies to the rural population's perception of opportunities in the cities. Generally speaking, in cities of developing countries, the population projections of cities in developing countries are grossly underestimated.

Clymo (1989) observed that, although developing world cities vary greatly, they have one characteristic in common; there are always large numbers of people living in high-density areas. The huge changes witnessed in the demographic profiles, industrial and commercial activities have placed new and heavy demands on water supply that many cities in developing countries have been unable to meet. Solving water problems has thus become one of the chief tasks confronting governments in the developing countries primarily because, of the economic importance of their burgeoning cities.

Developing countries often turn to international organizations such as the World Bank, United States Agency For International Development (USAID), United Nations Development Programme (UNDP) for funds in form of loan and grants to finance their water supply schemes. Such a loan was obtained by the Kaduna State Water Board Nigeria for the expansion of Kaduna urban water supply system. A similar loan was obtained by Borno State Water Board, Nigeria. These loans are supposed to be repaid from user charges. Because of the importance of water to life, it has been the practice of most national governments to subsidize water supplies. As a result of this practice, loans are often not paid back when due, making it increasingly difficult to obtain additional loans even when critically required.

Solving water supply problems means addressing one society's and the individual's most basic needs. As such the quality and quantity of water supplied to a population play significant roles in its health and development. In recognition of this fact, the United Nation's International Drinking Water and Sanitation Decade (IDWSSD), 1980 to 1990 was adopted. Despite the IDWSSD declared goal, which is to provide two of the essential fundamental human needs, safe water and sanitation to all the people of the world, a lot still need to be done in the developing countries. Now that the financial volume required to rescue the situation is staggering, much needs to be done at the designing management stages towards minimization of total cost of efficient water supply.

1.3 Planning a Municipal Water System

In general, the following must be done in planning a municipal water system:

i Estimate the future population of the community, and study local conditions, to determine the quantity of water which must be provided;.

ii Locate a reliable source of water of adequate quality;.

iii Make provision for the necessary storage of water, and design the works required to deliver the water from the source to the community;.

iv Determine the physical, chemical and biological characteristics of the water.

v Design any necessary water treatment facilities;.

vi Design the distribution system, including distribution reservoirs, pumping stations, elevated storage, layout and size of mains and location of fire hydrants. The cost of pumping station (boosters) and power operation and maintenance are included in the cost of transmission.

vii Provide for the establishment of an organization, which will maintain and operate the supply, distribution, and treatment facilities. A transmission route may snake around the contours of the terrain thereby considerably increasing the overall length of the facility.

1.4 Components of Water Supply Systems

Components of water supply systems normally include the following:

i. Source of water;

ii. Collection of water at the source;

iii. Transportation from the source to a water treatment plant;

iv. Treatment plant – purification or treatment as may be required; and

v. Transmission and distribution networks for treated water to the consumers.

Figure 1.1 shows the interrelationship between the different components of urban water supply system. In considering the system, a least cost supply system that accommodates all these components can be designed as a solution to problem of water in cities of developing nations. Since distribution-networks consume a considerable part of the total expenditure incurred on water systems, an attempt is made in this study to use systems analysis from the multi-objective point of view to evaluate the existing water distribution networks.

1.5 Brief Historical Background and Current Realities of Water Supply in Nigeria

According to Allavi (1984), existing water undertakings in Nigeria, especially in urban cities, were initiated and constructed during British rule, about 100 years ago. Some of these undertakings are as shown in Table 1.1 below.

With the advent of regional system of government in 1954, water supply development and control responsibility moved to the respective governments whose capitals were based in Enugu, Kaduna and Ibadan. The development and management of waterworks were under the joint responsibility of the Ministry of Works and Transport, then known as the Water Authority and the Ministry of Local Government, which was known as the Prescribed Authority.

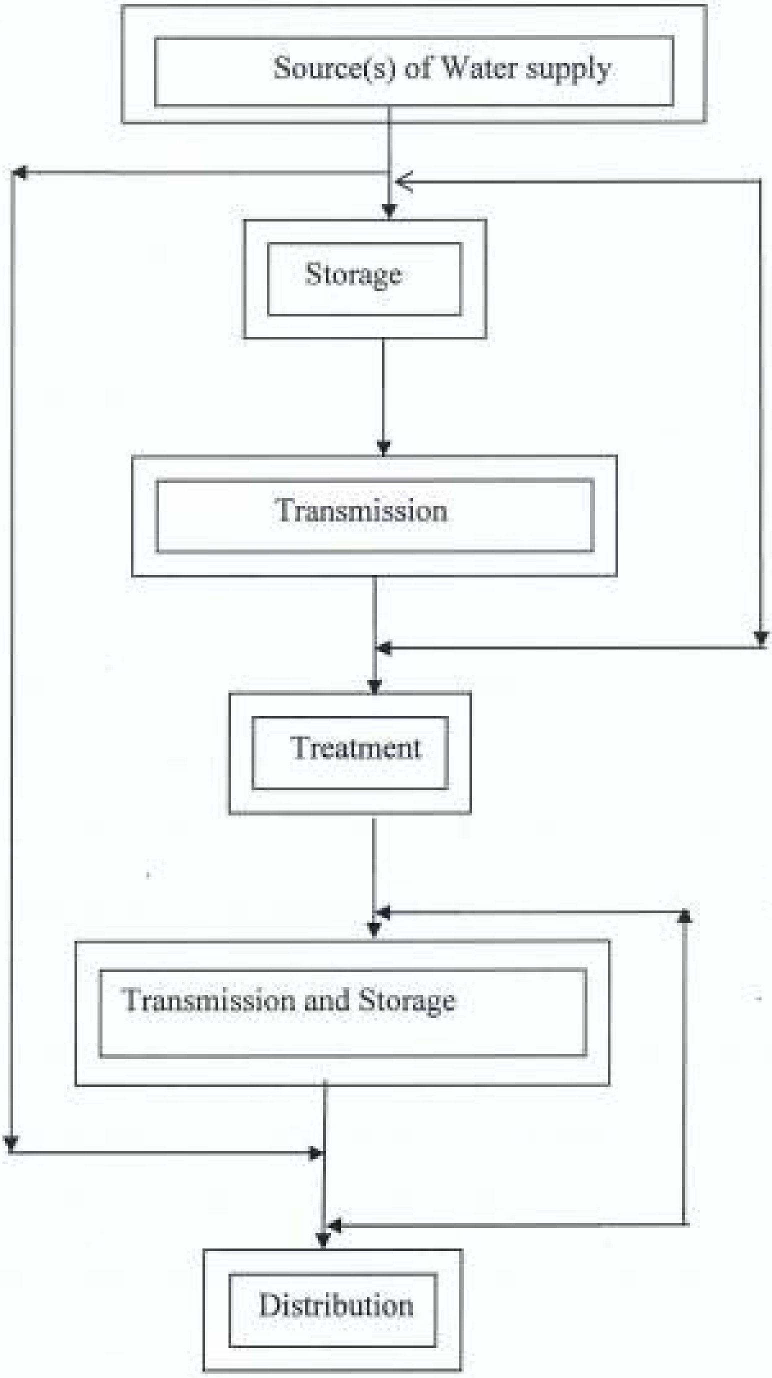


Fig 1.1: Interrelationship of the Functional Elements of Municipal Water Supply Systems.

Table 1.1: Early Water Schemes and Year of Commission

Town	Year Commissioned
Abeokuta	1911
Ijebu – Ode	1927
Oyo	1928
Ile – Ife	1936
Iseyin	1939
Ilaro	1940
Ibadan	1943
Otta	1947

(Source : Allavi, 1984)

There is no doubt that, the golden period of meaningful water projects investments especially on urban water supply facilities in Nigeria, remains that of the second and third National Development Plans. Nevertheless, the quest for the development of water supply has continued unto this day, albeit, with a subdued ardour and diminishing resources. But before the current realities in our urban public water supply can be adequately painted, two important issues, which will throw more light on the present plight of water supply, must be examined. They are the demographic antecedents, profiles and trends of the country and the introduction of Structural Adjustment Programme (SAP) into the Nigerian economy.

1.6 Objectives of the Research

The general objective of this research is the development of functional strategies to adequately meet water requirement of the urban communities in Ondo and Ekiti States.

The specific objectives of the research are to:

1. Evaluate the present status of water supply in the urban centers of Ondo and Ekiti States;
2. Assess the level of services of the existing water schemes in both States;
2. Identify the constraints in the realization of sustainable water supply;
3. Carry out some qualitative and quantitative assessment of some wells and boreholes in at least one of the state capitals;
4. Develop mathematical models that can be used for optimal allocation and distribution of the available potable water in the communities.

1.7 Justification of Research

The continuous deterioration of water supply situation in many Nigerian large and medium size cities exerts a major strain on the well- being of the dwellers. The welfare and even the productivity of the dwellers are greatly impaired by this poor state of water supply. There is a need to expand the scope of investigation on the water supply situation in the cities of Nigeria to be able to scientifically establish the immediate and remote causes of perennial water problems. There is also the need to identify the functional approach for adequately meeting the water requirement of the urban dwellers through the initiatives of these two parties using effective engineering and management tools.

When the rate of population growth and urbanization is extraordinarily high, as the case is in the present situation, the size of water supply facilities required would have to be large so as to be able to meet the challenges posed by the imbalance. In this study area, the research is needed to find the appropriate solution that will yield optimal allocation and distribution of the available water to the communities.

CHAPTER TWO

LITERATURE REVIEW

2.0 Definitions

Definitions of water supply include "an undertaking by a public, or sometimes a private, authority of conserving, pumping and piping potable water to consumers" (Chopra, 1989) and "the impounding, treatment, pumping and piping of drinking water for customers by a public authority" (Scott, 1981).

The total water supply to a city is usually distributed among the following four major classes of consumers; domestic, industrial, commercial and public, Domestic water use consists of water furnished to houses for drinking, bathing, washing, sanitary, culinary and lawn sprinkling purposes. Domestic use accounts for between 30 and 60% (225 to 270 litre per capita per day) of total water consumption in an average city.

2.1 Sources of Water Supply

The common sources of water in the study area are: Rainwater, Surface Water and Groundwater. Any of these sources, which are available in the study area, especially the last two, can be developed to serve as a source of water supply for consumption. However, the conventional headworks for surface water are the sources of water supply in the case of this study area.

Rain harvesting and storing water for future use is not a common practice. Rain water is normally collected for direct use from collected from roofs, and stored in cisterns, for small individual supplies. Rain water collected from prepared watersheds, or catches, can be stored in reservoirs, for large communal supplies.

Surface water is a common source of water in the study area. Water in this case is usually obtained from:

(a) streams, natural ponds and lakes of sufficient size, by continuous draft;

(b) streams with adequate flood flows, by intermittent, seasonal, or selective draft of clean flood waters and their storage in reservoirs adjacent tot he streams or otherwise readily reached from them;

(c) streams with low dry-weather flows in reservoirs impounded by dams thrown across stream valleys.

Groundwater is heavily used in this area of study. This is usually obtained from:

(a) natural springs

(b) infiltration galleries, basin or cribs

(c) wells or galleries with flows maintained by returning to the ground water previously with drawn from the same aquifer for cooling or similar purposes.

(d) wells, galleries and possibly springs with flows augmented from some other sources which are

  (i) spread on the surface of the gathering ground;

  (ii) carried into charging basins or ditches;

  (iii) led into diffusion galleries or wells.

2.2 Domestic Water Demand

The "demand" for improved water and sanitation services is not a simple concept and the factors or determinants which influence this demand are relatively complex as research from the World Bank Water Research Team (1993) demonstrated. It certainly cannot be directly correlated to household income as has commonly been done in the past. The three main factors which affect demand are (Garn, 1998).

Household income, gender, education, occupation and assets, as well as other local demographic characteristics

Available, reliability cost and convenience of existing services relative to proposed options; and

Household attitudes towards government sector policy and the service provider.

Table 2.1: **Some of the demand assessment tools currently in use**

Users: Engineers	Social Scientists	Economists
* Household/Revealed Preference Surveys	* Participatory Rapid Appraisal (PRA)	• Contingent valuation Methodology (CVM)
* Assumption based on aggregated estimates based on supply 'norm'	• Relative demand-based on community meetings	• Household/Revealed Preference Surveys

However, water supply schemes are planned to provide adequate water to the community to improve their health and thereby their economic welfare. It has been established that any consumption beyond 60 litres per capita per day does not contribute much to health.

2.2.1 Factors Affecting Domestic Water Demand

Several factors determine the household domestic use of water. Demand rates vary with time of day, month and year. The factors affecting domestic water demand are:

Climatic conditions: Climate affects the consumption of water. The determinants of demand are temperature and rainfall. Frequent baths and drinking are imminent in the hot season while this is reduced in the rainy season. .High temperatures also lead to high water use for air conditioning and where summers are hot and dry, much water is used for watering lawns.

House sizes and age of property: The bigger the house, the more likely the number of sanitary fixtures such as showers and toilets. This is a good measure of the volume of water consumed in the household. The age of property has a relationship with the type and number of sanitary fixtures in it.

At Fylde, U.K, it was observed that occupants of modern buildings use about 26% more water than their counterparts in older ones, (Twort et al, 1987). Tables 2.2 and 2.3 give some insight to these facts.

Table 2.2: Net Domestic Demand in Hot Climates

Usage	Mean Measured Consumption in 63 Households in a Hot Climate, Lcd (%)		Suggested Mean Design Values for Liberal Supply Lcd (%)	
(a) Sanitation use : Asian toilet plus anal cleaning	28	16	25	17
(b_1) Drinking, food preparation and cooking	14	8	12	8
(b_2) Dish – washing cleaning house e.t.c	13	8	12	8
(c) Ablution by showers and under running tap	85	48	60	40
(d) Clothes washing	35	20	30	20
(e) Miscellaneous: Yard washing, car cleaning, plant watering	Included above		11	7
Total	175	100	150	100

(**Source:** Twort et al, 1987)

Table 2.3: Range of Total Water Demand

Location	Typical Total Demand per Capital Reported
(a) Small towns with little industrial demand : Areas with low standards of housing settlement areas	90 – 150
(b) Low standard housing and a high proportion of stand pipe supplies	70 – 90
(c) Standpipes supplies: Rural areas with mainly standpipe supplies.	25 - 60

(**Source**: Twort et al, 1987)

Class of the consumers: The income of a consumer is the determining factor of the class to which the consumer belongs and it is an important characteristic of the population which affects the domestic use of water. The class is determined by the economic status and standard of living of the consumers and it differs greatly in various households of a city. The per capita use of water is much lower in the households of the lower class as compared to those of the upper class.

Size of household: Brandon (1984) stated that a good determinant of household water demand is household size. The volume of water used by people who carry it home seems to be associated with the number of people in the household in a variety of culture. Also the larger a household, the smaller the per capita use of water daily.

Effect of Sewerage: The effect of installation for sewerage is to increase the rate of water use because of increase in plumbing facilities. It has been noted that in sewered areas, the domestic water consumption is more than in unsewered places. Some data indicate an increase of 50%. A mean annual domestic water use was 9.4% to 29.3% higher in sewered areas than in areas where septic tanks are used in spite of better values of those houses (Twort et. al, 1987).

Losses: This is one of the most important factors that affect water consumption. Losses can be from leakages in reservoirs, trunk mains, distribution systems and plumbing systems of consumers services and metering and billing losses. Usually, metering and billing losses are referred to as unaccounted for water, but in cities where the main supply and water delivered to consumers are fully metered, about 85% of the supply should be reasonably accounted for. If estimates of water for fire fighting are added, the use of 90% or more of the supply should be identified (Twort et. al, 1987).

Standard of Supply: The rate of use of water increases when the pressure increases in the distribution system. This is due to greater loss of water through leakages and wastages. Intermittent pumping leads to restricted hours of consumption. But it may not reduce consumption if it is for a short duration. Intermittent pumping is practiced very much in developing countries, mainly because of inadequacy of energy. Table 2.4 shows the

TABLE 2.4: Percentage of Water Consumed vs. Hours of Supply for Hong Kong

Hours per day	Percentage of 24 hours supply
4	66
6	73
8	79
10	83
12	87
14	98
16	94

Source: Twort et. al (1987)

observation made in Hong Kong. Supply is restricted and the percentage of what a full 24 hours of supply consumption would be is also indicated, allowing for variation of ± 7%.

2.2.2 Components of Domestic Water Demand and its Fluctuation

According to Twort et.al (1987), domestic water demand varies from one house to another and it includes:

In-house water demand;

Garden watering;

Lawn sprinkling;

Washing cars;

Cleaning down outside yards.

Domestic (in-house) water consumption in the typical middle class dwelling having a kitchen, a bathroom, and water-borne sewerage centres requires about 115-200 lcd. Asian (or squatting pan) toilets need only 7 lcd for flushing.

The demand for water in a household is never uniform. It changes from season to season in a year and hour to hour in a day. People use more water during holidays (especially for weekend washings) and in summer. Religious festivals also are the occasions when people require more water. A large number of people may require water at the same time mostly before going to their place of work. In a cosmopolitan city, the general requirement pattern does not show steep and sharp changes of demand, due to different working hours of the people in different jobs and trades. The size of the city has considerable influenced over the extent of hourly demand of water. The existence of an intermittent supply is another factor which affects the hourly fluctuation of demand.

2.3 Water Infrastructure

Water infrastructure consists basically of the headworks, the transmission storage and distribution. Complete housing entails provision of all basic social amenities particularly water which is a basic need of life. However many a times, especially in developing nations, arrangement of housing does not allow for proper layout of water distribution network as a result of irrational urban development.

An urban settlement can be defined culturally, politically, economically and demographically (Palen, 1987). However, urbanization is taking place at a more rapid rate in the developing countries than in the developed ones. Within the developing world, the growth of urban population is most acute in the poorest countries (Willis and Tiopple, 1991). In Africa, the rate of growth of the urban population is estimated to be the most rapid compared with other regions. In Nigeria for example, the rate of urbanization in many town is very high whereas expansion of water distribution infrastructure is quite low. It has also been observed that even the existing infrastructures are performing far below their design capacities.

However, if there is one overriding lesson from this survey of experiences, it is that the key to effective management of both resources and systems is the capacity (technical, institutional and financial) of 'local' level organizations (government, community and private sector). Much else is also important, an enabling environment, appropriate technologies, etc. but without greatly increased local level ability it will be impossible to make progress towards improved and sustainable management (IRC, 2002).

2.4 Importance of Adequate Water Supply

In their rapid urban growth, cities in developing countries have suffered a slow process of urban planning and zoning and these cities are generally characterized by an increasing shortage of urban services and infrastructure, which are accessible to a diminishing share of urban population. The high rate of urbanization in Nigeria for instance is not matched by a corresponding commensurate change in the rate of economic and technological development as well as social change (Onibokun, 1986).

The consequences of the poor socio-economic circumstances of adequate economic activities under which urbanization is taking place manifest in the highly problematic water provision for the teeming masses and a growing gap between employment opportunity and demand. Shortage of water has become a visible enduring feature of the urbanization process. The impact of rapid urbanization on social and economic conditions is enormous and is of immediate concern in Third World Countries since they are largely deficient in the economic, technological and managerial resources needed to maintain and improve complex urban environment (Pacione, 1981)

In most major cities of Nigeria, problems arising from rapid urbanization and urban growth are very apparent. These problems emanate form the concentration of a huge migrant population within urban centers where the existing urban water infrastructure is too limited with the demand growth. This slows down the pace of economic development by retarding growth of labour productivity. The problems can be broadly categorized into two: macro-problems (including food shortage, paucity of cheap energy and unemployment); and micro-problems (including housing, welfare and amenity provision). Micro-problems are symptomatic of the much broader macro-problems, which are associated with the mode of development in Third World Countries. This author further stressed that the implications of the macro-aspects of the rapid rate of urbanization are that the Third World Countries in the dilemma will have to slow down the urbanization process. This can be achieved through integrated rural development, agricultural intensification, intermediate technology, appropriate education, small industry and export promotion, employment generation, satisfying basic needs, income re-distribution and education, unstable employment or outright unemployment, low nutritional status, large families, lack of access to municipal services and poor housing conditions amongst others (Makinwa-Adebusoye,1988; Soyombo,1987), Poverty can thus be seen as a broad deprivation which can be defined as the inadequate command over economic resources and quantitatively a distance along a continuum towards a zero purchasing power. In this light, poverty is characterized by the lack of income needed to acquire the minimum necessities of life including shelter, medical care, clothing, food and education. The insufficiency in quantity and in quality of water has a lot of impact on the health of the consumers as indicated in Tables 2.5a and 2.5b.

Table 2.5a: Classification of Infective Diseases in Relation to Water Supply

Category	Examples	Relevant water improvement
(i) water – borne infections		
(a) Classical	Typhoid, Cholera	Microbiological sterility
(b) Non- classical	Infective hepatitis	Microbiological improvement
(ii) water – washed infection		
(a) Skin and eyes	Scabies, Trachoma	Greater volume available
(b) Diarrhoeal disease	Bacillary dysentery	Greater volume available
(iii) Water – based infection		
(a) Penetrating skin	Schistosomiasis	Protection of user
(b) Ingested	Guinea worm	Protection of source
(iv) Infections with water-related vectors		
(a) Bitting near water	Sleeping sickness	Water piped from source
(b) Breeding in water	Yellow fever	Water piped to site of use
(v) Infection primarily of defective sanitation	hookworm	Sanitary faecal disposal

(**Source**: Richard et al, 1980)

Table 2.5b: Main Effective Diseases in Relation to Water Supply

Category	Disease	Frequency	Severity	Chronicity	Percentage Suggested Reduction by Water Improvement
I_a	Cholera	+	+++		90
I_a	Typhoid	++	+++		80
I_a	Leptospirosis	+	++		80
I_a	Tularaemia	+	++		40
I_b	Paratyphoid	+	++		40
I_b	Infective hepatitis	++	+++	+	10
I_b	Some enteroviruses	++	+		10
I_a, II_b	Bacillary dysentery	++	+++		50
I_a, II_b	Amoebic dysentery	+	++	++	50
I_b, II_b	Gastroenteritis	+++	+++		50
II_a	Skin sepsis and ulcers	+++	+	+	50
II_a	Trachoma	+++	++	++	60
II_a	Conjunctivities	++	+	+	70
II_a	Scabies	++	+	+	80
II_a	Yaws	+	++	+	70
II_a	Leprosy	++	++	++	50
II_a	Tinea	+	+		50
II_a	Louse - borne fevers		+++		40
II_b	Diarrhoeal diseases	+++	+++		50
II_b	Ascariasis	+++	+	+	40
III_a	Schistosomiasis	++	++	++	60
III_b	Guinea worm	++	++	+	100
IV_a	Gambian sleeping	+	+++	+	80
IV_b	sickness	++	++	++	20
IV_b	Onchocerciasis	+	+++		10
	Yellow fever				

(**Source**: Richard et al, 1980)

2.5 Appraisal of Water Supply Problems in Some Countries

2.5.1 Bangladesh

In Bangladesh, a model called Dushtha Shasthya Kendra (DSK) was adopted to solve urban water supply problems. The DSK model approach aims to demonstrate how 'informal' communities can access 'formal' utility services. The key principle of the model is to respond to demand for water indicated by communities willingness to pay. According to this model communities willing to form self help groups are provided training on management and maintenance of water infrastructure (water points); health/ hygiene habits and behavioral change. However, the major problems of this model are that it's acceptance and level of success is a function of uniqueness of social dynamics of different communities; it is time consuming to yield desired results and requires stable governance and commitment of local government/ municipal authorities.

2.5.2 Bolivia

One of the major factors affecting urban water supply is rapid urbanization of the cities of developing nations. Curtis (1996) gave the experience in Bolivia as follows: The rapid growth of cities in the Western Hemisphere is leading to large increases in water demand, and many urban centers face a crisis in water supply. Water quality is poor, service is irregular, water wastage is high, many of the urban poor do not have piped water, and water agencies do not collect the funds needed for maintenance and operation. To address this crisis, many urban centers are investing in expensive water supply rehabilitation and expansion projects, often with the assistance of the World Bank and the Inter-American Development Bank. However, little is being done to protect the sources of water. Urban watersheds and their biodiversity are being degraded by uncontrolled use, resulting in poorer water quality, threats to human health, and seasonal water shortages. Aquifers are being polluted and depleted.

2.5.3 Kazakhstan

An Environmental Assessment of the World Bank North-eastern Kazakhstan Water Supply and Sanitation Project was carried out. (Murfitt, 2002). The main objective of the project was to improve the quality, reliability, efficiency, financial viability, and sustainability of the water supply and wastewater services in four cities: Karaganda, Temirtau, Kokshetau and Atyrau. Water users in these urban centres currently suffer from low levels of water and wastewater services, both in terms of quantity and quality. Problems include significant wastage of water, problems with continuity of service, and poor quality (including heavy metal pollution). The incidence of water-related diseases, such as hepatitis A and dysentery, is also on the rise. The project's objective is to replace and repair a significant amount of the existing water supply and wastewater systems. In addition, mechanisms are planned which will result in institutional reform and strengthening capacity building, and increased private sector participation.

2.5.4 Malawi

The Piped Supplies for Small Communities (PSSC) Project started in 1988 with a view of stimulating the development of more appropriate, sustainable and successful methods to plan, implement and manage piped water supplies with peri-urban communities. Among the main activities were the development of a methodology for improving planning and implementing piped supplies with communities through demonstration schemes; training; hygiene education and sanitation. A new strategy was developed to have more women occupying key positions in the water committees and it was proven that the quality of women's participation was more important than their quantity. Special training programmes were then organized. Malawi is still in an early stage of urbanization with 11% of the total population living in the urban centres. It is estimated that up to 60% of the urban population live in the fringes of the urban centres. One of the major factors contributing to the sustainability of the project is that the communities in the piped water supply demonstration schemes play a bigger role in the maintenance of the water supply system (Murfitt , 2002).

2.5.4 Namibia

Piped water supply schemes were successfully developed in 9 peri-urban neighbourhoods of cities. Community managed and maintained communal waterpoints were established. Committees were established to operate these facilities. Men and women were trained to manage water supply. Water point committees were paying water fees in time. Water point committees were making profits, part of which is used for maintenance and repairs. Despite rapid urban growth rates in the 1990s (more than 5 percent annually in Windhoek) Namibia is still a relatively un-urbanized country. Approximately 32 percent of the population live in urban centres, which is low compared to 80 percent in many developed countries and an estimated 58 percent in South Africa. Because the cities, especially Windhoek, continue to dominate the country's economic, legislative, administrative and financial activities, Namibians will continue to flock to urban areas in coming years. In fact, the rate of urbanisation in Namibia is expected to increase significantly before growth levels off (van der Merwe, 1999). Because urban services can be provided more efficiently and managed more effectively than rural services, higher urbanization rates can be beneficial to water demand management efforts. The City of Windhoek initiated a demand management programme in 1992, and by 1996, the city's aggressive conservation and pricing measures succeeded in reducing demand to 1989 levels, despite a 35 percent population increase since that time. Future water consumption in urban areas could be reduced by at least 30 percent through implementation of effect water demand management programmes. As table 2.6 shows, managed water demand in Windhoek in 2002 would be 30 percent or more lower than unrestricted demand.

Table 2.6 Unmanaged vs. Managed Water Demand in Windhoek, Namibia

Year	1995	2000	2002	2005	2010	2015
Unmanaged Mm^3/yr	21.1	26.7	29.86	34.6	44.4	57.1
Managed Mm^3/yr	17.9	19.74	20.75	24.1	27.9	32.3
% Savings	15	26	30	30.4	37.2	43.4

Source: van der Merwe, 1999

In the worst case scenario, Windhoek demand in 2001 was estimated to be 22.6 Mm^3/yr. If effective demand management measures were in place by then, demand could be reduced to 20.75 Mm^3/yr, a savings of 1.85 Mm^3/yr.

2.6 Techniques of Water Distribution Analysis

2.6.1 Early Approaches

Historically, the Hardy Cross iterative technique of 1936 was the first breakthrough in the solution of flow in water distribution network. Using this technique, computations made with a slide rule or calculating machine were the only available means for solving flow distributions in complex distribution networks. However the development of the Universal Mallory analyzer brought another dimension to the solution of network flow distribution (Bhave, 1985). Haimes (2002) work considered the complexity of water and related land systems, which is due primarily to their large number of constituencies and interdependent subsystems. This is familiar to all those practicing in the field. This complexity, however, had over the years been developed and adopted into relatively manageable models that often oversimplify some fundamental attributes of these systems. Most water distribution networks consist of a vast number of interconnected components, for example the distribution network, pumps, pipes, and treatment plants. In addition, a hierarchy of institutional and organizational structures such as federal, state, local government, and city is involved in the decision making process. The degree of physical and institutional coupling that exists among the subsystems (e.g., the budget constraint imposed on the overall system) further complicates their modeling as well as management. In the maintenance of water distribution systems, different replacement/repair strategies for varying subsystems often have unexpected impacts on the overall system; the demands for the resources and their appropriate allocations likewise have a diverse impact on the system's reliability.

In the area of water transmission, one of the earliest works was the work of Brandon (1984). The work using the available data on water, gas and oil pipeline studied the factor that influences the cost of transmission facilities. He identified the principal factors that influence cost as capacity, transmission distance, and method of transmission. They at the end developed a general expression using a standard minimization technique to obtain the most economical diameter for a particular capacity. They carried out a regression analysis on pipeline data for variables diameter, D, and capital cost, K in dollars per kilometer. The resulting expression for pipelines is

$$K = 1.890D^{1.29} \tag{2.1}$$

The correlation coefficient is 0.98 He also considered the operation and maintenance cost and obtained the unit energy cost as:

$$C_e = \frac{(\sum s)P_c}{E} \tag{2.2}$$

where C_e is the unit energy cost in dollars per 45000 litre/km hour, E is the efficiency factor and c - conversion factor. One of the shortcomings of this work is that adequate technical data must be available to carry out the analysis and the result is localized.

Cembrowicz et al (1983) introduced graphical techniques which were used to facilitate rapid balancing of the energy line elevation. This analysis was based on the concept that at a junction there is a common energy line for all types entering of leaving the junction. It was found that this technique leads directly to accurate solutions of flow distributions in branching networks. Further progress was made in this direction when the hydraulic-network design problem was formulated by Jacoby in 1968 as an optimization problem. Application of this computerized optimization procedure considerably reduces the amount of engineering labour in the design of hydraulic networks (Cembrowicz et al, 1983). It also allows deficiencies in the initial network design or geometry or both to be corrected automatically. A method which took full advantage of the Newton—Raphson method was presented by the same author to solve directly for combinations of unknowns in the water distribution networks. The formulation of this method resulted in solving for a steady state non-linear network, the solution of which requires a high-speed computer of large capacity. Other researchers namely, Bhave (1983) presented a linear programming model for the problem of selecting optimal pipe diameters and power requirements for water distribution networks. Their model was based on the assumption that the configuration of the pipes connecting the delivery points to the water source is decided a priority and that there is only one source.

The following quotes from Cenedese (1987) explains the system approach: 'In principle system approach consists of combining elements of several sciences to analyse large scale, complex, total problems in ways designed to provide objective, systematic, quantitative and explicit analysis. The general framework consists of iteration among formulations of clear goals, composition of conceivable alternatives, building of models and model measurements using carefully designed criteria".

While many intelligent planners and engineers have adopted this approach for years, much planning of complex systems has clearly not been done this way. However with the large memory high speed digital computers and the development of mathematical techniques of system analysis, more complex water supply systems can now be dealt with through system approach. The basic planning techniques include individual simulation, calculus of variations, queing theory, linear and nonlinear programming and dynamic programming.

Cembrowicz (1983) treated the optimal design of water supply systems with several specific parts of the overall systems. The cost-benefit criteria for water supply systems was expressed as the least expensive alternative supply using the Langrangian method and expressed optimaly as:

$$L\,(x, \lambda) = \sum_i \beta_i(x_i) + \lambda\left(\sum_i x_i + y - v\right) \tag{2.3}$$

Some necessary conditions considered. In the above equation,

x_i is the possible draft use

λ is the Langragian multiplier

β_i is the benefit function

v is the volume of existing reservoir

y is the unit of water supply required

He also considered aspects in several other ways. This equations can be difficult to interprete in practical sense. One of the earliest works done in this area of study was the simulation study carried out on Melbourne water supply scheme (Twort, 1987). According to Bellman's principle of optimality- 'An optimal policy has the property that whatever the initial state and initial decision, the remaining decisions must constitute that resulting from the first decision and must constitute on optimal from the first decision" (Goulter, et al, 1986).

Unfortunately direct optimization technique often requires the real system to be considerably simplified to fit the requirements of the particular approach under consideration. According to Morgan, et. al (1982), simulation can readily model the particular idiosyncrasies of a complex real system. However, it does not lead directly to an optimal solution. A trial and error search must be employed with the possibility of bypassing the best solution. Direct optimization techniques, such as linear and dynamic programming lead directly to an optimal solution in terms of the chosen objective function and are therefore very powerful analytical tools.

Because of the approach of sequential decision-making, dynamic programming is well suited to analysing the operation of water resource system because non-linear objectives and constants are readily handled. Despite this advantages a serious problem with dynamic programming is the amount of computer memory required to solve a multi reservoir system in which multiple decisions need to be made. This is termed "the curse of dimensionality" by Goulter et al , (1987)

In order that the dynamic programming can be applied, two conditions must be fulfilled, namely, separability and optimality, (Fujiwara, 1987). Separability condition governs whether the value of a state can be calculated by a recursive equation, given a fixed policy. Stated generally, this means "for every policy the value of every state must be expressible as a function of the immediate return and the value of the succeeding state" (Fujiwara, 1987).

The optimality condition, which relates to Bellman's principles of optimality state "for every state and action, the optimal policy is to consist of the given decision followed by the policy which is optimal with respect to the successor state (Fujiwara, 1987).

Another approach to the solution of this kind of problem is the use of stochastic dynamic programming. This approach allows the inclusion of the stochastic mature of stream flow. Goulter et al, 1985 used this approach to drive the optimal release rules for a single multipurpose reservoir operation problem. Also in 1985, Morgan et al using this approach employed an explicit probability constitution of inflows to take account of the stochastic nature of stream flow. The result was that the dimensionality of the dynamic programming formulation increases thereby presenting significant problems to its applications. This approach was earlier employed to solve two reservoir "series-parallel" systems, Bhave,(1985). The same approach was used by Kessler and Shamir (1989) to optimize the releases from two interlinked reservoirs. From the outcome, they proposed that the matrices of optimal release obtained from the dynamic programming calculation be combined with the inflow sequences to set up 12 monthly storage content transition matrices.

Goulter (1985) also applied stochastic dynamic programming in a backward looking manner to determine an optimal operating policy for a single storage supplying a hydroelectric power station. In his case, inflows were assumed to be independent random variables.

Quindry, et.al (1981) applied the stochastic technique to a conjunctive use surface and groundwater storage system in order to find the optimum operating policy. The state variables were the volume of water in both storage and the volume to replenish the groundwater system. Decisions were made on an annual basis hence the independent nature of the stochastic inflows was acceptable.

Nada (1995) applied stochastic dynamic programming in similar ways to the single storage example. Their approach included the use of an iterative dynamic programming technique in which the objective function included a penalty function to take account of under supply.

Bhave et al (1989) proposed a method based on dynamic programming while working on aqueduct route optimization for regional irrigation project. The dynamic programming formulation enables the search for and the costing of the aqueduct route and the application of the principle of optimality ensure the attainment of the overall optimum aqueduct route. However, this solution is only good for a first estimate of the strip of contours within which detailed investigations need to be carried out.

Some other authors also proposed a method based on dynamic programming theory and systems concept which made it possible to avoid the difficulty of making many simultaneous decisions. Instead, optimization proceeded by stage and the actual calculations were reduced to within the capacity of mostly computers. The result showed that the optimal design could be obtained readily (Bhave, 1985).

Savic and Walters (1997) also worked on sizing of pumped storage conduits. He presented direct solution for economic diameter which uses levelized operating and economic conditions and for which internal loads determine minimum thickness. The development allows for the difference in value of pumping and generating losses. The formulas may be simplified to size conduits with unidirectional flow and easily modified when external loading governs conduit wall thickness. The major limitations are that they are not applicable to manifold and transitions. Over the last decade research results in this area have improved drastically with the help of different techniques of mathematical programming. Chiplunkar, et. al., (1985) derived the minimum cost of a branch network using linear programming model. It was discovered however that these research results are not in general applicable in practice. Apart from preventive computational intricacy, any of the following drawbacks occurred :

i. The optimal design is based on continuous diameters instead of available standard sizes;

ii. Local optimal are derived instead of a global solution;

iii. Branch networks are used instead of existing loop grids.

Hansen et al (1991) present a linear programming approach which resulted into a global cost solution for standard diameters based upon free configurations of the network graph. His procedure was based on continuous diameter approach which disregarded topographical constraints. In this work, it was considered that the optimal network will be the one in which its layout is not fixed a priori but allowed to vary to obtain the optimal solution. Quindry (1981) proposed another model which is an improvement on his earlier work.

The objective function of that model is as presented below

$$\text{Minimize cost} = \sum_{ij} \beta_{ij} L_{ij} X_{ij} \qquad (2.4)$$

β_{ij} = cost per unit length from i to j

X_{ij} – unit of length

L_{ij} = total length of link ij.

In another work, (Bhave, 1985), a method of optimal expansion of water distribution networks subject to single loading pattern was developed. This method is iterative and converges rapidly to a local but fairly good optimal solution.

Another method reported in Bhave, (1983) looked into the optimal design of multi-source, looped, gravity-fed water distribution systems subjected to a single loading pattern.

This method was based on classical transportation problem principles. A theory was developed to formulate a linear programming (LP) problem for obtaining the design distribution graphically for the entire distribution system. An iterative method for the optimal expansion of water distribution systems subjected to a single loading pattern was developed. The method optimally decides the pumping heads at new source nodes, if any, and also optimally selects the links which need strengthening to improve the delivery capacity of the existing system.

A method based on critical path concept was developed for selection of the optimal sets of pipe sizes for optimization of branching system network by linear programming, (Bhave, 1989). The leading equation here is the equation of the slope, given by.

$$S_j = \frac{H_o - H_j^{min}}{Lp_j} \tag{2.5}$$

where S_j is the path slope, H_o and H^{min} are the hydraulic gradients at the source and terminal.

The final equation in this regard is

$$H^{min} = H_o - S_j L_e. \tag{2.6}$$

The weakness of this model is that it requires many more pipe sizes to achieve global optimality.

In solving the problem of optimal design for both collection and distribution networks applied Modular In-core Non-linear Optimization System (MINOS). The two models formulated are users oriented, arrive at discrete solutions and can handle a large sale network efficiently. Duan et al (1990) presented another model:

$$\text{MB}= \frac{(7.14N_pWf_i^y + 0.1134K_4)}{E_f}\left(\frac{2.315}{c}\right)^{1.85} Q^{2.85}\left(\frac{1}{D_1^{4.87} - D_2^{4.87}}\right) \geq \text{MC} \tag{2.7}$$

$$\text{MB} = K_1K_2Q^{2.85}\text{DC} \geq \text{MC} \tag{2.7a}$$

A model consisting of two linked linear programming (LP) was formulated by Goulter, et.al, (1985) for the least cost layout and design of looped water distribution networks. One linear program determines the least cost layout of a looped distribution network given an initial pressure distribution while another model determines the least cost component design given an initial flow pattern or pipe layout. One linear programme determines the least cost layout of a looped distribution network given on initial pressure distribution.

$$\text{Min. } \sum_{i,j} \beta_{ij} L_{ij} X_{ij} \tag{2.8}$$

where β_{ij} = cost per unit X_{ij} of pipe between node i and node j.

L_{ij} = total length of pipe required between nodes i and j and

X_{ij} = is the pipe size variable

The other linear programme determines the least cost component design given an initial flow pattern or pipe layout. Here the operation is of the form

$$\text{Minimize } \sum_{i,j} \sum_{m} COST_{ijm} X_{ijm} \tag{2.9}$$

with $\sum X_{ijm} = L_{ij}$ and minimum permissible head

$$\sum_{|ij\in p|} (\pm H\lambda_{ij}) + H_s \geq H \min_n \tag{2.10}$$

in which the expressions

$COST_{ijm}$ - cost per meter of pipe of diameter *m* used between nodes i and j.

Hl_{ij} - head loss in link between nodes i and j in direction of flow (m).

$Hmin_n$ = minimum head permitted at node n.

A modified linear programming gradient (LPG) method was used for solving looped water distribution network problems, together with a mathematically rigorous derivation of the LPG model (Fujiwara and Debashis, 1987). The initial equation is

$$\text{Min.} \sum_{(ij)}\sum_{m} C_{(ij)m} \ \text{x (ij)}m \tag{2.11}$$

and having three constraints revolving around maintenance of head at the nodes where $C_{(ij)m}$ is the cost of a unit length of pipe segment of the n^{th} diameter and $X_{(ij)}$ is the length of pipe segment of n^{th} diameter. One of the major problems of this model is that a pipe direction or coop orientation must be pre- specifed.

Morgan and Goulter (1985) employed a heuristic linear programming based procedure for the least cost layout and design of water distribution networks. The methodology is capable of analyzing a wide range of demand pattern and pipe failure combinations Hydraulic consistency is ensured throughout the procedure through the use of Hardy-Cross network solver technique. In another approach (Goulter et.al. 1986), the effects of varying the paths used to ensure adequate pressure throughout a water distribution network when using iterative linear programming-gradient search techniques for least cost solution of the network were analysed.

Fujiwara and Debashis (1987) modified the linear programming–gradient (LPG) algorithm in both direction of flow change and step size, proposed quasi-Newton search direction instead of original search direction and the step size was determined by a black-tracking line search method instead of fixed step size. The convergence rate of this modified LPG algorithm is improved to a super linear convergence rather than a linear rate of convergence.

The initial equation is:

$$\text{Min.} \sum_{(ij)} \sum_{m} C_{(ij)m} \ \text{x (ij)}m \qquad (2.12)$$

and having three constraints revolving around maintenance of head at the nodes where $C_{(ij)m}$ is the cost of a unit length of pipe segment of the n^{th} diameter and $X(_{ij})$ is the length of pipe segment of n^{th} diameter. One of the major problems of this model is that a pipe direction or orientation must be pre specifed.

Taher, et.al. (1996) presented a prototype decision support system called Water Distribution System Optimization Programme (WADSOP) to guide water distribution system design and analysis in response to changing water demands, timing and use pattern. WADSOP integrated a geographic information system (GIS) for spatial data base management and analysis with optimization theory to provide a computer aided decision support tool for water engineers.The objective function is

$$\text{Minimize} \sum_{j\varepsilon A} \left[K_{jdr} X_{jdr} + K_{jds} + PWF.hp_{ij} \right] \qquad (2.13)$$

in which the expressions

A =- a set of network link

X_{jdr} and X_{jds} = length of pipe j of diameter d replaced by a pipe of larger diameter r or smaller diameters, respectively;

K_{jdr} = unit cost of replacing pipe j of diameter d with pipe of larger Diameter i.

($K_{jdr} = c_r - c_d > 0$); and K_{jds} = unit cost of the same pipe if replaced with smaller diameter s ($K_{jds} = C_s - c_d < 0$) where C_d = unit cost of a designated pipe of the d^{th} diameter; and C_s = unit cost of a replacement pipe of diameter s, and hp_{ij} = change in head of pump j under demand load pattern L.

GIS was applied for accurate, spatially and referenced assessment of pipeline installation costsbased on landuse, soils, elevations existing pipe networks, and street network. The constraints considered include pressure, length and capacity.

In another study Fujiwara and Debashis, (1987), a model was presented in which the necessary and sufficient conditions that a given set of pipe diameters is in an optimal solution and as a special case shows that the adjacency property holds if and only if pipe costs are a strictly convex function of a power of pipe diameters.

Savic, et.al. (1997) presented the development of a computer model GANET that involved the application of an area of evolutionary computing, better known as genetic algorithms, to the problem of least-cost design of water distribution networks. The mathematical formulation is as follows:

$$F(D_1 \ldots\ldots\ldots\ldots\ldots D_n) = \sum_{i=1}^{N} c(D_i, L_i) \qquad (2.14)$$

in which (D_i, L_i) = cost of pipe i with diameter D_i and length L_i and N = total number of pipes in the system. The constraints are revolving nodal discharge Q and head loss h.

Kessler and Shamir (1989), gave analysis of he linear programming gradient (LPG) method for optimal design of water distribution networks. This method consists of two stages that are solved in alteration: (1) a LP problem is solved for a given feasible flow distribution and (2) a search is conducted in the space of flow variables based on the gradient of objective function (GOF). Due to interactive of the LPG the GOF was evaluated each time independently of the previous ones and so was self-correcting. It was however proved that the GOF value before and after a change in the loop/path constraints is unchanged. This was contrary to Goulter, et. al (1986) claims. This method was first presented by Bhave and Sonak (1992).

In another attempt by Kessler and Shamir (1989), a decomposition technique was suggested for optimal design of water supply networks. Here the general mathematical model was decomposed into two sub-models. This method was first suggested by Morgan and Goutler (1982) to determine the layout and pipe diameters while maintaining some degree of reliability. A slightly modified version of this was introduced by the same Morgan and Goutler (1985). The work which was based on reformulated Morgan and Goutler (1982) version, used new sets of variables and graph theory technique. This model is extended to handle reservoirs and pump station, where a variable speed pump was directly represented as a function of its impeller diameter and restricted to operate at its most efficient points.

Bogale (2002) used an object oriented programming approach to the development of hydraulic simulation models, which improved the capabilities of hydroinformatic systems.

Abebe and Solomatine (1998) employed global optimization in the design of water pipe networks. Using global optimization tool with various random search algorithms and a network simulation model that can handle both static and dynamic loading conditions, two algorithms, adaptive covering and genetic algorithms, yielded promising solutions enabling a choice between accuracy and required computer time. The proposed optimization setup can handle any type of loading condition and neither makes any restriction on the type of hydraulic components in the networks or does it need analytical cost functions for the pipes. The work deals with the determination of the optimal diameters of pipes in a network with a predating terminal layout. Although this model can handle both static and dynamic loading conditions, which is an enhancement, it is observed that all algorithms stopped near the optimum, in all cases with feasible and single diameter solution.

Kleiner, et. al. (2001) used dynamic programming approach combined with partial and implicit enumeration scheme to search the vast combinatorial solution space on water distribution network renewal planning. The outcome is a strategy that identified for each pipe in the network, the optimal rehabilitation/renewal alternative and its optimal time of implementation. It was observed however that the best current heuristic method is limited in practical studies to a network of up to 15-20 pipe links. This of course implies that a more efficient heuristic method is required for scale water distribution systems. It was also observed that this approach does not take into consideration other components of a distribution system.

Network hydraulic modeling software (specifically water CAD hydraulic programme) was applied in the design of water distribution networks (Ekenta, 2002). The programme simulated the flows and pressures in the pipe networks for given boundary conditions. The outputs are optimal pipe diameters, velocities and pressures. However, CAD programme requires adequate and accurate data to achieve the desired results, which are not readily available in most cases. Besides the model was not applied to real live-size networks to determine the actual performance

Abdel-Latif and Bani (2001), chose water distribution network as a case study to assess the hydraulic behaviour and evaluate their global performance by developing a computer model for a distribution network under actual existing and alternative conditions, especially involving intermittent supply. The network service area was divided into two separate zones, one supplied by gravity from an elevated tank and the other supplied from a pumping station, which draws water from a ground level tank. The simulation was conducted under real conditions applying to the operating pump schedule, the water tank levels, and the valve settings, using EPANET version II software. The output results from the model for pressures at nodes and flow rates in pipes were available for the evaluation and assessment of the hydraulic behaviour of the distribution network.

Since some data required for modelling calibration were not available, a sensitivity analysis was carried out to examine the precision of the model parameters and ascertain which of the input parameters have a significant effect on output results from the model. The results showed that the output results form the model are sensitive to the system demands and insensitive to the roughness coefficient factor.

The performance of the Bani-Suhila distribution network was evaluated from a hydraulic point of view using a systematic, engineering approach, and the results indicated that the performance was adequate and the system provided an acceptable level of service based on pressure considerations.

An object-oriented programming approach to the development of hydraulic simulation models is attracting increasing attention (Bogale, 2002). The potential of object-oriented technology lies basically on the promise to produce more maintainable and extendable information tools with less effort and expense. The implementation of an object-oriented modelling environment built upon the characteristics of object-oriented programming results in a simulation system that may greatly improve the capabilities of hydroinformatics systems. Current simulation systems deficiencies include lack of modularity, minimal provision for model component reuse, often limited flexibility, and complex model specification format. Object-oriented modelling systems provide improved capabilities in these areas while maintaining a comparable or reduced design effort.

In the study carried out by Shivaram (2002), the prototype architecture for a modelling environment has been designed. It includes graphical features for logical configuration of simulation entities and icon based capabilities for system definition. To quantify the purpose, prototype construction of a water distribution network simulation model was selected as a case study. The system has been implemented in Object-oriented Borland Pascal under Microsoft Windows operating environment. The choice of object-oriented methods for the development of modelling tools of this class proved to be justified, resulting in the open flexible architecture of modelling system, and the use of natural object-based representation of a water distribution network.

The consulting and research institute , WL Delft Hydraulics developed new decision – support technology for complex pipeline systems. The software is designed to optimize requirements from different disciplines and/ or company departments such as supply issue, available storage capacity, maintenance, manpower, material logistics, systems control, product quality, etc.

Constraint Logic Programming (CLP) provides a multitude of descriptive constraints some of which are at a high abstraction level to facilitate clear specifications of various applications, and solution. The problem with some of these models is that they cannot readily be applied to the networks that lack necessary data and logistics.

Nada (1995) developed and designed a modelling system for simulation and identification of parameters in water distribution systems. According to him, the solution to problem of steady-flow in a pipe network is obtained when the flows satisfy both the continuity equation at each junction and the energy equation in each pipe. The governing equations are nonlinear. Neither of the nonlinear systems of equations could be solved directly, so iterative methods were employed.

The choice of the method for linearizing the nonlinear terms is essential. In the Water Distribution Modelling System (WADES) which was developed in the framework of the study referenced above, the following methods have been implemented:

i. The Linear theory method
ii. The Newton-Raphson method
iii. The combination of the two previous ones
iv. The solution method based on unsteady state analysis

The method combines Newton-Raphson procedure with the linear theory method as a simple and robust starting procedure, shown to be especially robust and efficient, and guaranteed to converge in an expeditious manner.

Algorithm for identification of design, operating, or calibration parameters of the network, is also implemented in the modelling system. The algorithm is able to calculate directly the parameters required to meet specified operating conditions. Specified operating conditions represent stated minimum pressure and volumetric flow requirements at designated locations throughout the distribution system. Solutions are optimal in the sense that they exactly meet the specified constraints.

The feasibility, accuracy, and flexibility of WADES algorithms is examined using a variety of networks, including Hodeidah water distribution system in Yemen with 182 pipes and 142 demand nodes. The resulting Water Distribution Modelling System (WADES) is implemented in Borland Pascal programming language (around 4000 lines of Pascal code) under the Turbo Vision interface development framework, and allows for the graphical presentation of the analyzed networks.

2.6.2 Modern Approaches to Water Distribution Networks Analysis- Meta-Heuristic Optimization

There are relatively many applications of meta-heuristic optimization techniques especially for water distribution network problems including new design, expansion design, layout rehabilitation, pump operation, and calibration. In the developed countries, methodologies involving application of genetic algorithms (GAs) to the optimal design of water distribution networks started in the early 1990s (Geem, 2000). Recently, a meta-heuristic optimization algorithm was developed and applied to engineering, especially in the field of water resources planning and management. A new meta-heuristic optimization algorithm called Harmony Search which mimics musical performance was developed (Geem, 2000). Advantageous features of harmony search (HS) different from other meta-heuristic algorithms are that it makes a new vector the consideration of all existing vectors rather than consideration of only two (parents) as in the case of genetic algorithm. It also handles many solution vectors simultaneously; and it does not require the setting of initial values of decision variables e.g initial pipe diameters. Besides, HS produces a range of solutions such that a decision maker can choose between similarly priced alternatives.

In Sarbu (2002), an improved nonlinear method was developed, which has the advantage of using not only the cost criteria but also energy consumption, consumption of scarce resources, operating expenses and so on. The problem of nonlinear programming with equality constraints finally turned into a system of nonlinear equations to be solved by the gradient method.

Vairavamoorthy, et. al. (2002) developed international guidelines for the design of urban water distribution network systems in developing countries where intermittent supplies are unavoidable for foreseeable future, in order to sustain adequate and safe supplies. The work described a novel approach to design of water distribution systems in developing countries using modified network analysis simulation tools.

2.6.3 Multi- Objective Optimization Models

In years past and till today, a great deal of research has been conducted on optimally sizing water distribution systems i.e. selecting the least-cost combination of pipes, pumps, and tanks (Walski et.al, 1987), but to date, there have been few published applications of optimization models to sizing real water distribution systems.

While cost may be the most important objective of such models, there are generally a number of other factors which should also be taken into consideration in the search for the least cost design of the network. The problem may arise that most of these factors may not have direct monetary implications. Examples of such factors are reliability and safety issues which are though very important but difficulty to quantify in pure economic terms. Goulter, et. al. (1985) developed a model for the least cost layout and design of looped water distribution networks.

In another work, optimal design of closed hydraulic networks with pumping stations and different flow rate was considered, (Cenedese et al, 1987). The author proposed the use of multi-objective models as a reasonable solution to the problems of non-linearity of objective function and constraints equations and heterogeneity between construction costs and management costs which arise later are essentially related to the pumping energy. Cenedese et.al. (1987) expressed their mathematical models as

$$OF_1 = g^T - x \qquad (2.15)$$

$$OF_2 = x^T . U^T . Z . (q^* + E.x) \qquad (2.15a)$$

where OF_1 and OF_2 are objective functions 1 and 2

g^T is the weight of each section

Z is used to determine the branches that have pumping station

q* is the branch flow rate

E is the mesh of each branch. It has the value of ± 1 depending on the direction chosen as positive.

x is the vector of decision variables.

2.6.4 Reliability –Based Models

The problem of determining a least-cost network with acceptable reliability levels has received a considerable amount of attention over the past several years in a number of fields including municipal water supply. Two quantitative approaches to the incorporation of reliability measures in the least-cost design of looped water distribution networks were developed by Goulter et. al, (1986). The first approach addresses the probability of isolating a node through simultaneous failure of all links connected directly to that node while the second approach tends to recognize redundancy by minimizing the deviations in the reliabilities of all pipes connected to each node within the network. However, in both approaches, the probability of failure of individual link was modeled using the Poisson probability distribution. Su, et. al. (1987) presented the basic framework for a model that can be used to determine the optimal design of a water distribution system subject to continuity, conservation of energy, nodal head bounds and reliability constraints. They defined reliability as the probability of satisfying nodal demands and pressure heads for various possible pipe failures (breaks) in the water distribution network.

The most recent works in this direction include the development of probabilistic reliability measures for the performance of water distribution networks and the use of analytical methods for their computation (Wagner, et. al. 1988). This work classifies reliability analyses according to the level of detail with which the water system is modeled, and then concentrates on methods relevant to networks. Two algorithms for the computation were presented, one for series-parallel networks and the other for general networks. Wagner et al, 1988, further presented simulation as a complementary method for analyzing the reliability of water distribution networks. For this simulation, the distribution system is modeled as a network whose pipes and pumps are subject to failure, nodes are targeted to receive a given supply at a given head. If this head is not attainable, supply at the node is reduced. It was claimed by the authors that simulation enables computation of a much broader class of reliability measures tan do analytical methods but requires considerably more computer time and its results are less easy to generalize. They therefore commended that analytical and simulation methods be used together when assessing the reliability of a system and considering improvement.

2.6.5 Other New Trends in Water Supply Research

Kleiner, et. al. (2001) used dynamic programming approach combined with partial and implicit enumeration to search the vast combinatorial solution space on water distribution network renewal planning. The outcome is a strategy that identified for each pipe in the network, the optimal rehabilitation/renewal alternative and its optimal time of implementation. It was observed however that the best current heuristic method is limited in practical studies to a network of up to 15-20 pipe links. This of course implies that a more efficient heuristic method is required for scale water distribution systems. It was also observed that this approach does not take into consideration other components of a distribution system.

Network hydraulic modeling software, specifically water CAD hydraulic programme was applied in the design of water distribution networks by Ekenta (2002). The programme simulated the flows and pressures in the pipe networks for given boundary conditions. The outputs are optimal pipe diameters velocities and pressures. However, CAD programme requires adequate and accurate data to achieve the desired results, which are not readily available in most cases.

Geem (2001) used Harmony Search (HS) to solve four problems namely, a traveling salesman, a specific function, a hydrologic parameter calibration, and optimal design of water supply network. According to him the comparison of solutions obtained by HS and other optimization techniques shows that HS algorithms can find better solutions in less number of iterations and it can solve continuous variable problems without unnecessary restrictions imposed by split-pipe or linearizing assumption.

A new computer software programme, the "Rural Water Supply- Decision Support System (RWS-DSS)" was developed by the Institute for Water Research at Rhodes University in South Africa. According to the expectation of that project, families in village communities will receive clean water on a more sustainable long-term basis. The user-friendly programme, a 32-bit Windows-based interpreter supporting formula-solving interactive graphing, database access and standard interfacing using the Delphi 3.0 Developer object-oriented Pascal System, is a planning and design tool for rural water supply.

At present, about 80% of the rural population in South Africa lacks access to water supplies - despite the provision of standpipe facilities in the more densely populated regions - with those more inaccessible areas being the most poorly serviced.

According to IRC International Water and Sanitation Centre (2002), small towns are experiencing the same trends that have affected the rest of the water sector, particularly the worldwide move towards decentralization and increased involvement of the private sector, as part of which communities, local government and sometimes the private sector are given the ultimate responsibility for ensuring the upkeep and functioning of their infrastructure services.

2.7 Summary

From the literature, effective management of water resources and systems allocation depend on technical, organizational and financial strength of the government, community, and private sector involved. All the planning and implementation for efficient delivery depend on the availability of data, without which it will be impossible to make progress towards improved and sustainable water supply.

With respect to the water allocation and distribution analysis, majority of the above cited literature tend to oversimplify many of the important features of water supply networks, while employing sophisticated mathematical tools to achieve their objectives. They are only applicable to systems with adequate data. These conditions cannot obtain for Ondo and Ekiti water supply systems, where sufficient data for sophisticated model development are not available. There is therefore a need to develop appropriate allocation and distribution models which recognize the peculiar features of local water facilities.

CHAPTER THREE

RESEARCH METHODOLOGY

3.1 Background to Study Area

Ondo State (Figure 3.1) was originally created out of the old Western State in 1976 and Ekiti state was carved out from the old Ondo State on October 1, 1996 by the Late Gen. Sani Abacha led government with Navy Capt Onyarugbulem and Lt. Col. Barwa as the first Military Administrators of the two States respectively. The two states are part of the Odu'a states with a common general language, which is Yoruba. The major occupation of the people of the two states are farming and trading. Figure 3.2 is a map of the old Ondo State, the study area shows the map of the study area with the location of the major schemes. The study area lies within latitude 7° 15' - 8° 5' N and longitude 4° 15' - 4° 45' E. The two states are bounded in the west by Osun state, in the north by Kwara state, in the south and in the east by Kogi State (North-East) and Edo State (South East). Figures 3.3 and 3.4 show the location maps of Ondo and Ekiti States respectively.

Relief in Ondo and Ekiti states is generally flat lying to gently undulating with elevations increasing northwards from sea level to approximately 600 m. Three landscape types occur in Ondo and Ekiti states. The coastal area is covered with forested freshwater swamp with mangrove growing in the more saline areas nearest the coast. An extensive flat lying to gently undulating coastal plain extends northward to just south of Akure.

3.1.1 Topography

An Isenberg landscape has been developed over basement complex rocks in the northern half of Ondo and Ekiti states. Relief is undulating and marked by numerous domed-shaped hills and by occasional flat-topped ridges. Summits range between 600 and 900m.

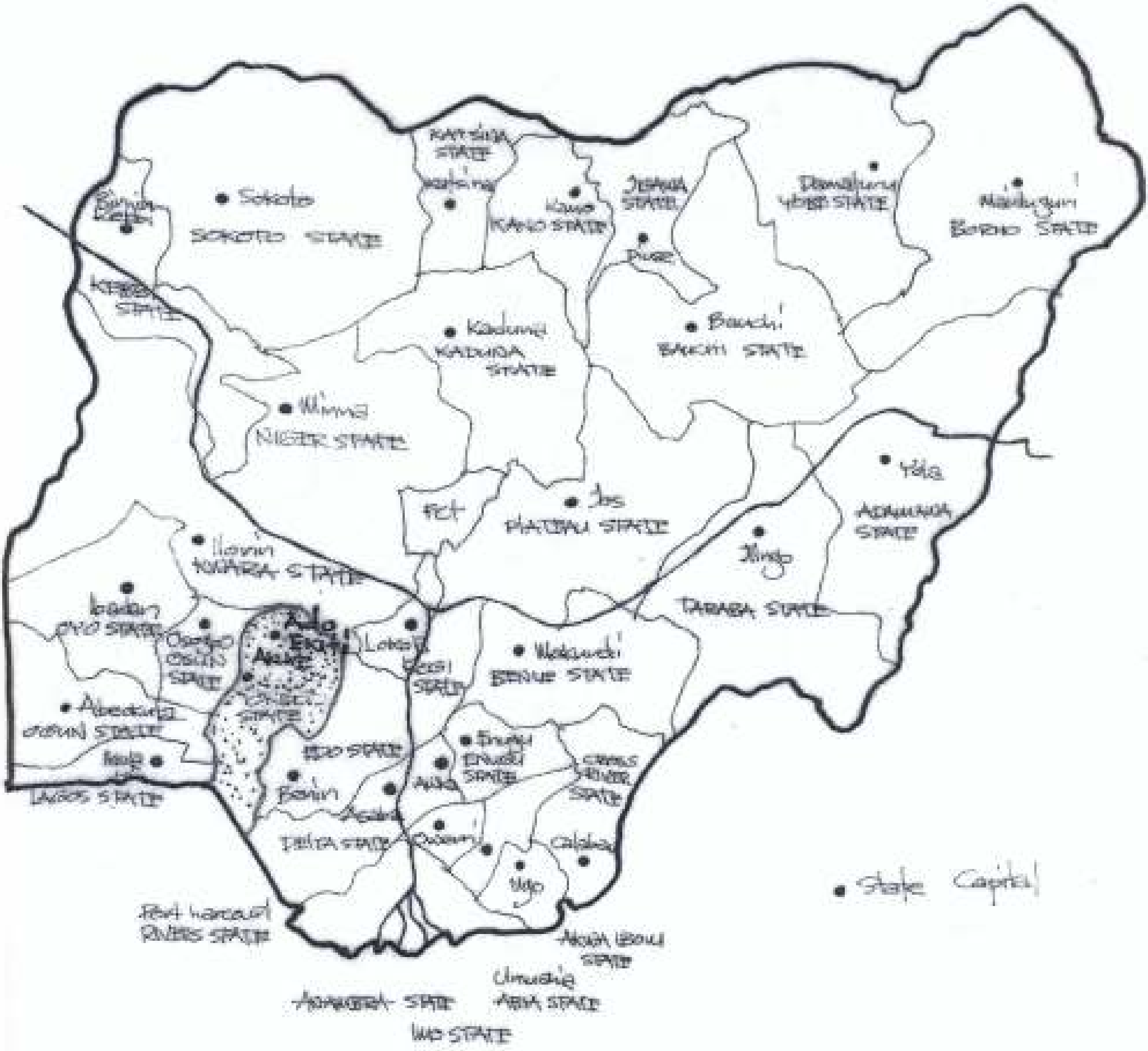


Figure 3.1. Map of Nigeria

Figure 3.2. Map of Old Ondo State Showing Some Important Schemes

3.1.2 Climate

The climate of Ondo and Ekiti states is tropical with distinct wet and dry seasons. The mean daytime temperature varies between 25^0 C in the north to 34^0 C in the south. The wet and dry seasons are associated, respectively with the prevalence of the moist maritime southwesterly monsoon from the Atlantic Ocean and the dry continental northeasterly harmattan from the Sahara Desert. From the coast to the interland, a distance of some 220 km, there is a steady decrease in the duration and intensity of the wet season.

Annual rainfall reaches 2500mm on the southeastern seacoast, but is only 1250mm in the northeast. The length of the rainy season decreases from more than 300 days on the coast to 240 days in the north. The distribution of rainfall is markedly bimodal, with a temporary decrease in July and August (Resource Inventory and Management 1992; Barbour et al. 1982). In the south, rainfall at the start and end of the rainy season is often marked by afternoon thunderstorms accompanied by heavy showers, tailing off in prolonged steady drizzle.

The lowest daily values of relative humidity are recorded in the afternoon, while the highest values are recorded in the early morning hours. Mean annual relative humidities vary from more than 80% in the south to less than 50% in the extreme north.

A widely extending high rocky plateau dominates the two states. The local crystalline rocks of the basement complex underlying the area consist of granites, gneiss and schist's. The basement has shallow ground water mainly along intermittent river course.

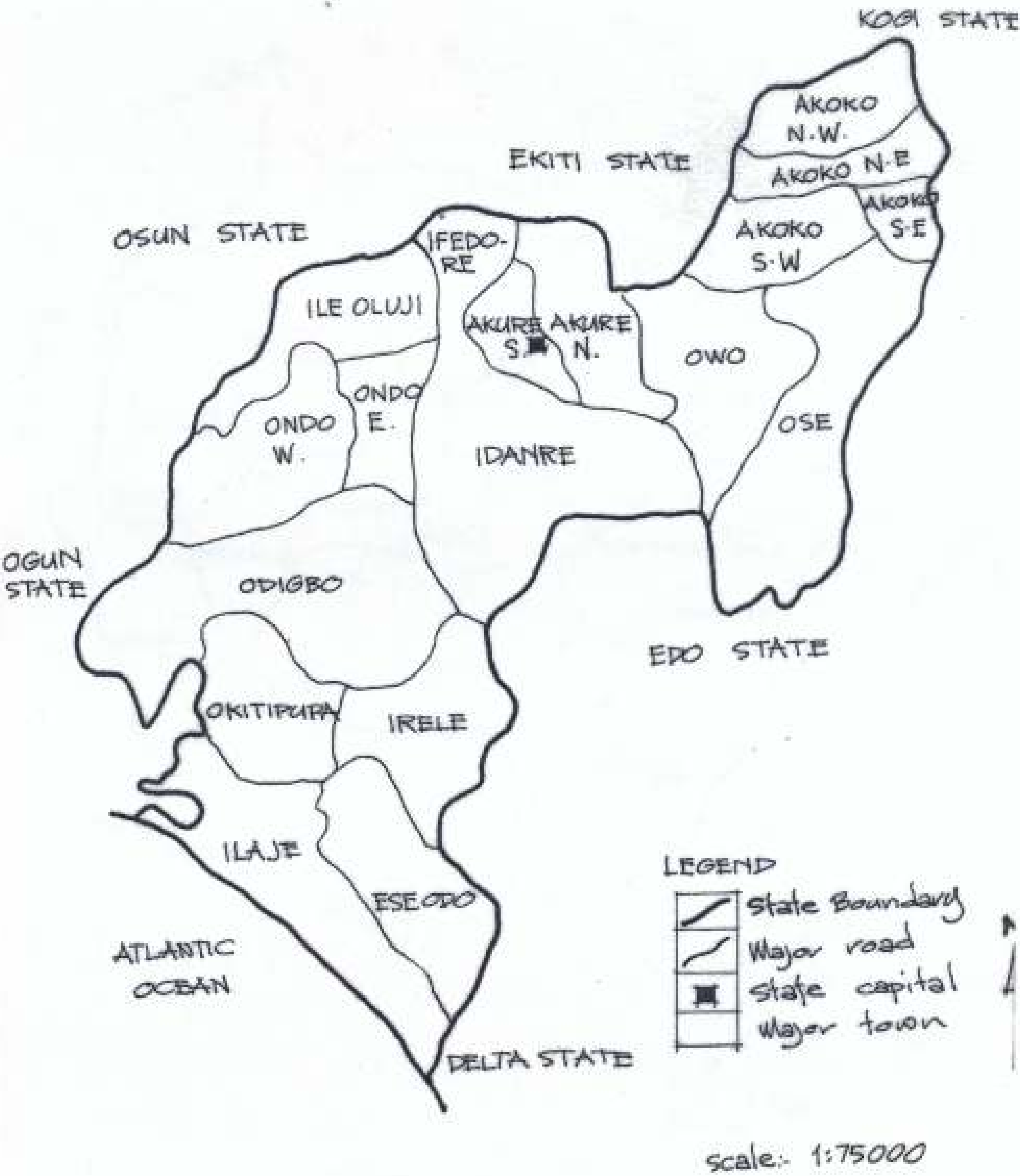


Figure 3.3. Map of Ondo State

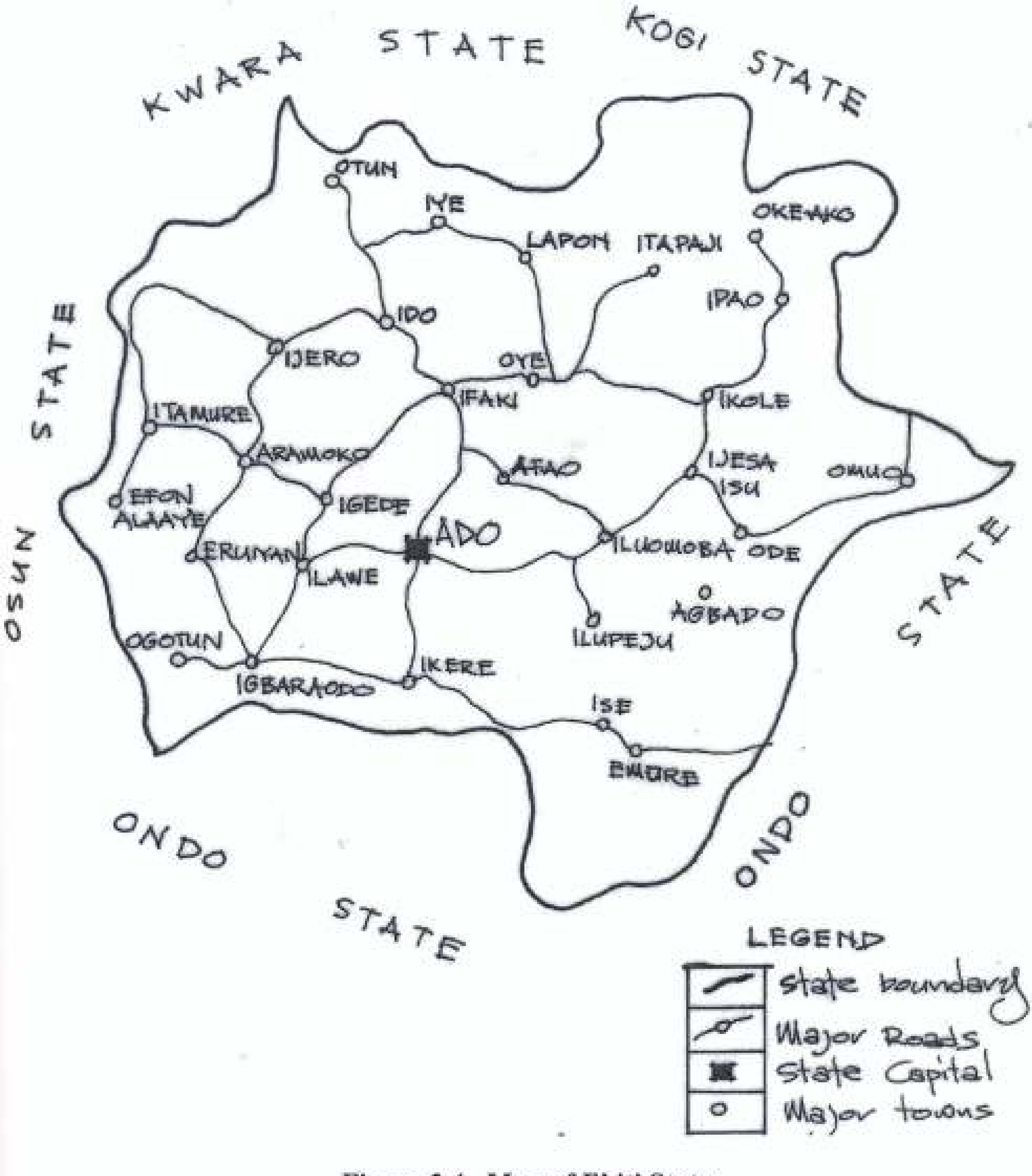


Figure 3.4. Map of Ekiti State

3.1.3 Population

According to the census report of 1993, results of which were yet to be legalized, the population of Ekiti state is approximately 1.7 million while that of Ondo state is 2.4 million. The bulk of the population of the two states is located in the rural area. However the current trend of urbanization in the study area indicates a high drift of the rural population to the urban centers in search of paid employment.

3.2 Data Collection

Data were collected through the field surveys carried in both Ondo and Ekiti States respectively. The data collected were technical data relating to the major water schemes and headworks, production and pumping records, data relating to consumers and the demand centers, data relating to other sources of water supply, and many others which are relevant to this research work. These data were collected through:

i. Appraisal of existing water schemes through visits and examination of design and operational documents;

ii. Field Measurements;

iii Administration of Questionnaires and Oral Interviews.

3.2.1 Technical Performance Evaluation

Performance evaluation was carried out for some selected headworks. Each of the selected headworks was visited and interviews were for the Headworks Superintendents/Engineers and other relevant staff. Available documents relating to operation, repair and maintenance were studied and data collected.

Types of questions that were asked include:

(i) How many headworks are failing to give satisfactory services and why?

(ii) Would it be worthwhile to improve their design to give an improved

(iii) service and how?

(iv) How much does it cost to supply a litre of treated water from the headworks ?

(v) How long can the system components be expected to last before they need replacement?

(vi) What more in maintenance do they require?

(vii) What parts of plant or equipment need servicing and replacement? Etc.

(viii) Which of the components of the headworks often breaks down and why ?

Two visits were made to each of the existing head works in the study area to carry out their technical and operational appraisal. Detail of the visits and activities involved as shown in Table 3.2

Design documents relating to the headworks and the pipe networks were examined at the Water Corporation Headquarters of the two states located at Akure and Ado – Ekiti respectively.

3.2.2 Field Measurements

The following measurements were carried out in the course of this study.

(i) Yield of some selected hand dug wells within Akure during the dry season of the year. Details of this are presented in the relevant chapter.

(ii) Distances and elevations along the selected routs for possible interconnection were carried out using Geographical Positioning System (GPS). This was achieved by traveling through the routes with the instrument.

Other field measurements carried out in this work include:

i) Public dependence on different sources of water

(ii) Closeness of well to septic tanks and toilets in household

(iii) Quality of water from some boreholes and well in Akure metropolis

(iv) Households with water consumption within some urban centers

(v) Pattern of water consumption within some urban centers

(vi) Factors responsible for the emergence / acceptance and rapid increase of packaged water producers were also measured

(vii) Percentage coverage of Water Network in some selected towns.

Results of all these are presented in Chapter 4 of this work

Table 3.2: Time of Visitation and Activities of Various Headworks.

Headworks	Date visited	Activities
Ero	21 - 10 - 2000	Pumping Records, Production Records, Maintenance
	08 - 02 - 2001	Records. Visit to the dams and treatment plants. Interview of
		the key personnel.
Egbe	04 - 12 - 2000	,,
	15 - 02 – 2001	
Itapaji	04 - 10 - 2000	,,
	20 - 12 – 2000	
Ureje	16 - 01 - 2001	,,
	09 - 02 – 2001	
Owena – Ondo	17 - 01 - 1999	,,
	21 - 02 – 1999	
Owena – Igbara	11 - 01 - 1999	,,
Oke	05 - 02 – 1999	
Little Osse	09 - 10 - 2000	,,
	15 - 01 - 2002	

3.2.3 Design and Administration of Questionnaire

Two types of questionnaires were designed for the study. Type I and type II had close-ended questions while the third mode of data collection is in the form of direct interview. This questionnaires were administered to collect information concerning funding level, policy and decision making as they affect water supply, private sector intervention in water supply and the factors propelling the rapid proliferation, willingness of the consumers to pay in case of regular supply, and efficiency of some small schemes serving one or two communities. TypeI questionnaire (see Appendix I) was designed to appraise the demand centres. The variables investigated in the questionnaire include available sources of water, performance of Water Corporation, adequacy of water and sources during rainy and dry season willingness to pay bid among others. The variables were expressed in question form to elicit information on the subject.

3.3 Data Analysis

Analysis of data obtained from the field survey was carried out using four statistical methods on three different tiers viz:

i. Uni-variate analysis using descriptive statistics;

ii. Bi-variate analysis which involves both non-parametic test of the chi-square test of independence, and the correlation analysis, a parametic test; and

iii. Multi-variate analysis on which the multiple regression was extensively employed (Drapper et.al, 1981).

In conducting comprehensive appraisal of existing water schemes in Ekiti and Ondo States, records of monthly production of some of the major schemes were collected and compared with designed production capacity to determine the efficiency of these schemes. From the data, the constraints hindering water production were identified. Accessibility of urban dwellers to good water supply was measured using the existing supply records.. Investigation of the level of dependency of the dwellers on other sources was carried out through field survey. Adequacy of the existing scheme to match the rate of urbanization and population growth was also investigated through data acquisition from the headworks, Area Offices, and the Headquarters of Water Corporations, and also from the field survey.

3.4 Modelling and Computer Programmes

Two mathematical models were formulated to describe (i) optimal allocation of water at the existing production level of the headwork to ensure equity in distribution within the urban centers and (ii) interconnection of some of the water schemes to allow for water transfer from a place of abundance to augment schemes of lower capacity and high demand respectively. Computer programmes were developed (using MATLAB and Visual Basic Application) to evaluate the model parameters and obtain their optimum values.

3.5 Constraints

This study, which involved analysis of water supply network, requires that data concerning daily water pumping, hours of pumping, energy consumption, apart from other basic parameters of the networks must be readily available. In our situation, some of these data were missing. However, for this work to progress such data were either generated or extrapolated using appropriate methods.

The extremely bad conditions of road leading to some of the major headworks and non-availability of communication gadgets made the appraisal process very difficult, notwithstanding, adequate visits were made and the appraisal was thoroughly carried out.

The creation of Ekiti State out of old Ondo State and the accompanied assess sharing problems affected the research somehow but with persistence efforts many of this problems were solved.

CHAPTER FOUR

APPRAISAL OF WATER SUPPLY SITUATION IN THE STUDY AREA

4.1 Effect of Economy on Water Supply

This chapter presents the results of the field survey of the water facilities in the two states. Information gathered in this exercise was later used as basis for the development of the models. The effects of the depressed economy on water supply are numerous and varied. There are those due to the producers (the State, Federal Government and private individuals) whether directly or indirectly, and those due to the users. The effects were considered from the point of view government control, water production, and insufficient data for planning and maintenance problem.

4.1.1 Government Control

Most water undertakings come under some form of public control. The control exercised by the government is usually of three kinds: financial, qualitative and administrative. Financial control usually comprises control of capital expenditure and rates of charge for water. Water supply is normally subsidized by government as a social service so that the masses who cannot afford to pay for clean potable water due to their low income can have access to it as "free" water for a token tariff. The government can no longer live up to its obligation because the sectoral allocation to water industries keeps decreasing instead of increasing. According to Olashore (1991), expenditure in this sector is usually classified under "others'' in the budget documents.

4.1.2 Water Production

The exchange rates of Naira to other currencies and inflation have a lot of influence on the expenditure in the water sector. Table 4.1 shows the prices of basic chemicals use for water treatment between the year 1993 and 1998 in Ondo State. The variation of these prices is more significant between 1994 and 1995.

Table 4.1: Prices of Basic Chemicals for Water Treatment.

Year chemical	Units	1993 ₦	1994 ₦	1995 ₦	1996 ₦	1997 ₦	1998 ₦
Aluminum Sulphate	metric tonne	15,500	20,000	30,000	38,000	41,000	30,000
High test Hypochlorite	drum	6,500	9,000	24,000	24,000	22,000	20,000
Chlorine gas 65/68kg	drum	5,000	9,000	12,5000	17,000	25,000	28,000
Chlorine gas 1,000kg	drum	105,000	140,000	260,000	360,000	360,000	360,000

Source: Ondo State Water Corporation.

It may be noticed that there are decreases in the prices of some of the water treatment chemical between 1997 and 1998. This was attributed to the following reasons:

(i) The fluctuating exchange rate of currency;

(ii) The lifting of the ban on importation of water treatment chemicals by the Federal Government and;

(iii) That some of their vendors waited to sell off old stock at 'give-away' prices.

The inevitable consequence of cutting down on the undertaking's expenditure is that the water may not be available at adequate quality due to reduction in quantity of treatment chemicals. The corresponding reduction in maintenance costs and personnel of the undertaking leads to increasing losses of water through damaged pipes.

4.1.3 Insufficient Data

One of the major problems in water sector is non-availability of continuous and reliable data. Many of the schemes in the study area are having this problem. In many cases, design documents of the schemes are not available. Reliable data for planning are usually not available. Records of operations and maintenance are usually not properly kept essentially because there is no continuity in the daily operation.

4.1.3 Maintenance Problem

Maintenance is not just a process of repairing bad plants/equipment but also a process of keeping them steady. Maintenance has become very costly due to increase in the prices of fittings and spare parts largely because they are not produced locally. Table 4.2 presents the annual price variations and their percentage increase from 1993 to 1998.

4.2 Appraisal of Existing Water Schemes

Next, the details of the appraisal carried out in this research work on the existing major schemes in both Ondo and Ekiti states are given while the summary is given in Table 4.3 in the appendix.

i) Owena/Ondo Water Supply Scheme

The general layout of Owena/Ondo distribution system shown in show in Figure 4.1is as follows: Water is supplied to the cities of Akure, Ondo, Idanre and Ile-Oluji using a system of transmission mains and booster stations.

Table 4.2: Some Pipe Fittings and Variation in their Prices

S/N	Name of Fitting	1993 ₦	1994 ₦	1995 ₦	1996 ₦	1997 ₦	1998 ₦
1	35mm Nipple	15.00	20% 18.00	22% 22.00	13.6% 25.00	40% 35.00	0% 35.00
2	13mm 19mm Reducing Nipple	15.00	20% 30.00	10% 20.00	50% 30.00	23.3% 37.00	8% 40.00
3	38.25mm Reducing Socket	25.00	20% 30.00	10% 33.00	9% 36.00	11% 40.00	12.5% 45.00
4	32mm Socket	15.00	33% 20.00	50% 30.00	6.7% 32.00	25% 40.00	10% 40.00
5	13mm Reducing Bush	20.00	25% 25.00	40% 35.00	7.5% 43.00	11.6% 48.00	40.3% 40.00
6	50M G.L	75.00	6.7% 80.00	12.5% 90.00	33.3% 120.00	16.7% 140.00	78.6% 250.00
7	25MM G.L	30.00	16.7% 35.00	12.5% 40.00	33.3% 42.00	7.1% 45.00	60% 160.00
8	25mm Union Connector	35.00	14.3% 40.00	12.5% 45.00	33.3% 60.00	25% 75.00	60% 120.00
9	13mm Ferrule	80.00	18.75% 95.00	5.26% 100.00	30% 130.00	38.5% 180.00	44.4% 260.00
10	38 *32mm Elbow	25.00	40% 35.00	8.6% 38.00	18.4% 45.00	10% 50.00	40% 70.00

Source: Ondo State Water Corporation. .

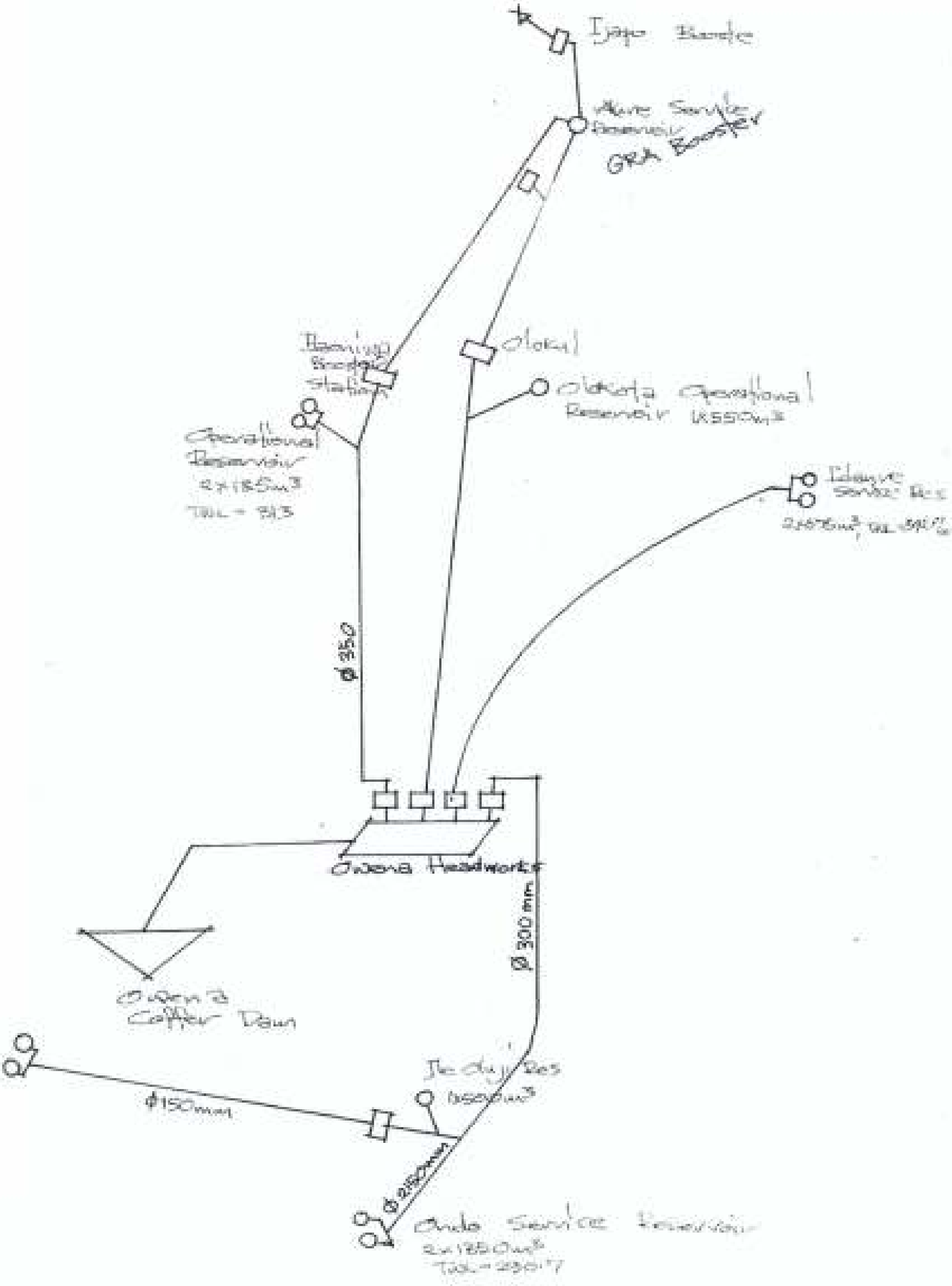


Figure 4.1. Akure–Ondo (Owena Water Supply Scheme)

Akure is supplied by mains consisting of the Akure old and new lines. Booster station located at Ita-Oniyan which is 13km away from Owena has two reservoir of capacity of $135m^3$ each with a total water level (T.W.L) of 313m. Olokuta Booster station is on the new line located 11.6km away form the headwork and has operational reservoir capacity of $550m^3$ with T.W.L. of 316.9m

ii) Ondo Distribution system:

Ondo transmission main consists of 300mm A.C pipe and has a length of 22.7km. There are two adjacent floating on line round level reservoirs each with capacity of $1350m^3$ and WL of 290.7m. Also, Ondo distribution receives a supplementary supply of about $610m^3$/day from Okeigbo headworks which is located on Oni river. This quantity is delivered to a booster station which feeds Ondo town system.

iii) Ile-Oluji distribution System

Ile-Oluji is supplied through a branch connection from the Ondo transmission main with an operational reservoir capacity of $500m^3$ and TWL of 251.5m. The Ile-Oluji booster station then pumps water form the operational reservoirs to two adjacent ground level reservoirs in Ile-Oluji town via a 150mm diameter A.C, pipes. The ground level reservoirs in Ile-Oluji provide a gravity supply to the town.

iii) Idanre Distribution System:

The Idanre transmission supplies water to Idanre directly from the Owena headworks (figure 4.1). Water is delivered to two adjacent ground level reservoirs in Idanre each with a capacity of $675m^3$ and TWL of 345m. The town's distribution system receives water by gravity from the ground level reservoir chemicals.

As at the time of this study, the availability of chemicals at Owena headworks was adequate to produce enough water to supply the demand centers. On transportation/communication, there is no specific vehicle attached to this scheme.

v) Awara Water Scheme

Awara water supply scheme (Figure 4.2) is located about 8km South – West of Ikare township. It was originally designed to provide treated water for consumers in Ikare, Arigidi, and Ugbe. The scheme was commissioned in 1958 with a target population of 164,436 according to 1963 census figure. The scheme was abandoned in 1989 when Little Osse Water Supply Scheme was commissioned to serve the people of Ekiti East and Akoko division in the old Ondo State. However with the creation of Ekiti State in October 1996, the Little Osse (Egbe Dam) which is located in Ekiti land mass ceased to supply water to Akoko division which is in the new Ondo State. This development compelled Ondo State government to reactivate the Awara scheme to supply water to an expanded area which covers 44 towns and villages of Akoko Division.

Awara dam was built on the outcrop of Asanodi river and it is an earth embankment of about 150m long and 8m high at its highest section. The impounded reservoir was augmented in the year 1980 to store about 2270m^3 of water. The spilling blocks have a height of 6.5m and a width of 8.5m.The outlet of the spillway is located in an inlet chamber through the draw off valves from where it is pumped to the treatment plant. There are two highlift pumps of the horizontal type. One newly installed and he other rehabilitated both discharging 190m^3/h. There are two booster stations located at Ikun and Eshe respectively. The pump at Ikun has been removed. There are three pumps at Eshe with a total discharge capacity of 170m^3/hr,

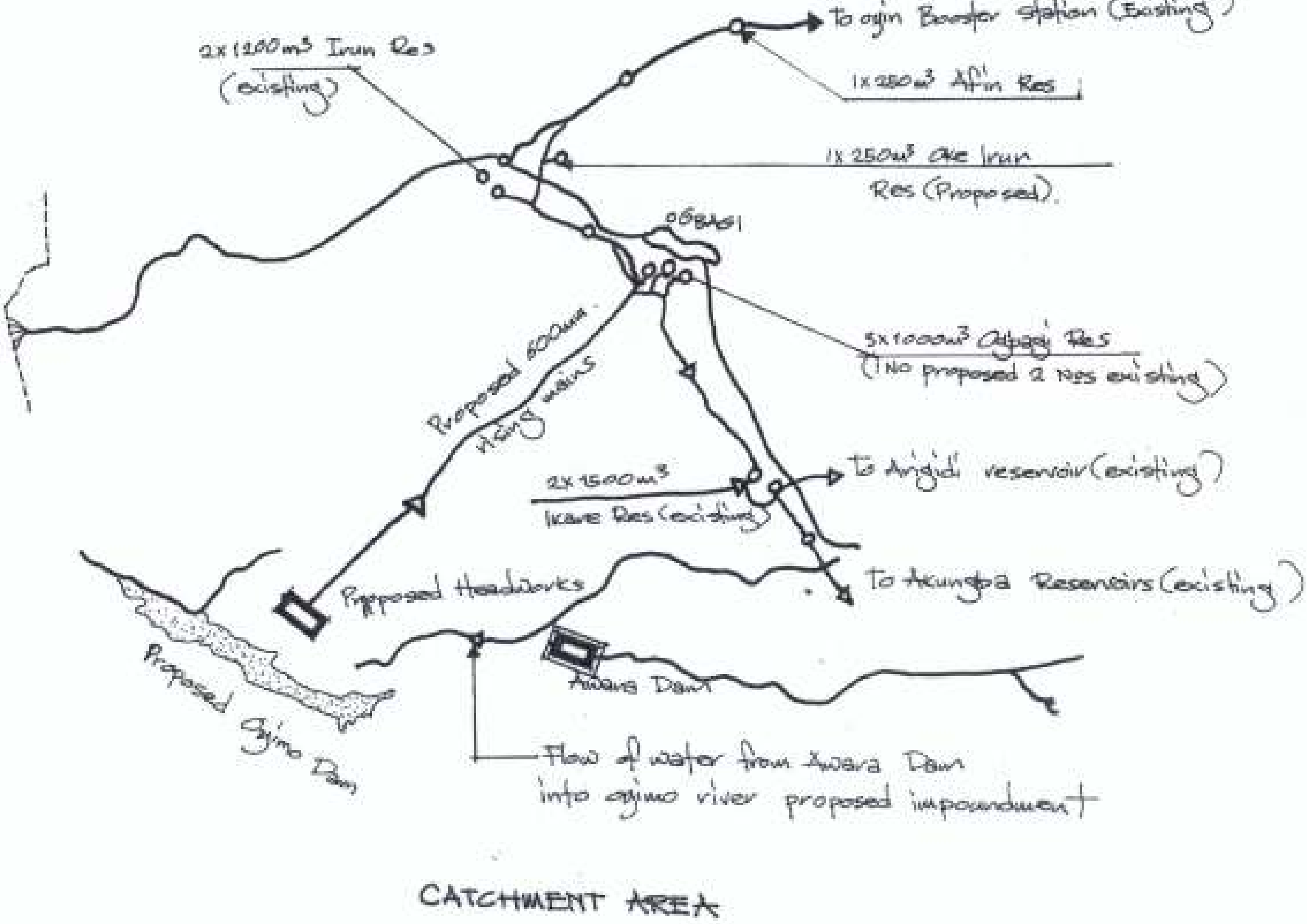


Figure 4.2. The Awara Water Scheme

total head of 155m and nominal power of 110kW. Besides, booster stations are proposed at Iwaro and Irun respectively. There are four concrete service reservoirs – three upper while one is at the ground level. Two of the reservoirs have 2500m^3 capacity each, while the other two have 1500m^3 and 1000m^3 capacities respectively.

Ikare, Arigidi, Ugbe and Akungba distribution networks are feed from rising mains of 9km from Awara headwork. Water is delivered to two reservoirs each with a capacity of 2000m3 and round lee reservoir that releases water to Ikare, Arigidi and Ugbe by gravity.

Chemicals generally are inadequate on this system. There is a rehabilitation work going on the scheme in which the treatment section is going to be affected. Chemical supply is a component in the contract. The headwork is not connected to NEPA grid. However it is operating on a 200KVA generator set.

vi) Osse/Owo Water Supply Scheme

The gravity concrete dam located on River Osse was built on an outcrop. There is a central spillway block with a vertical gate of 7.62m wide and 4.88m high located on the left and the intake chamber on the right. The gate bottom elevation is 229.97m, and the height of the concrete section is approximately 5.7m. The spillway length is 53.4m and the crest length is 15.2m to the left and 42m to the right of the spillway respectively. The crest elevation is 234.5m with a free board of 0.9m with respect to the steel footbridge. The steel footbridge mounted on piers has an elevation of 237.5m.

The intake chamber is provided with two 300mm diameter intake openings, one above the other and controlled by a sluice valve. A scour pipe of 200mm diameter to control the removal of sediment is provided. The designed volume of the reservoir is 420,000m^3 with flood discharge of 450m^3/sec. The Osse/Owo headwork initially was designed to treat

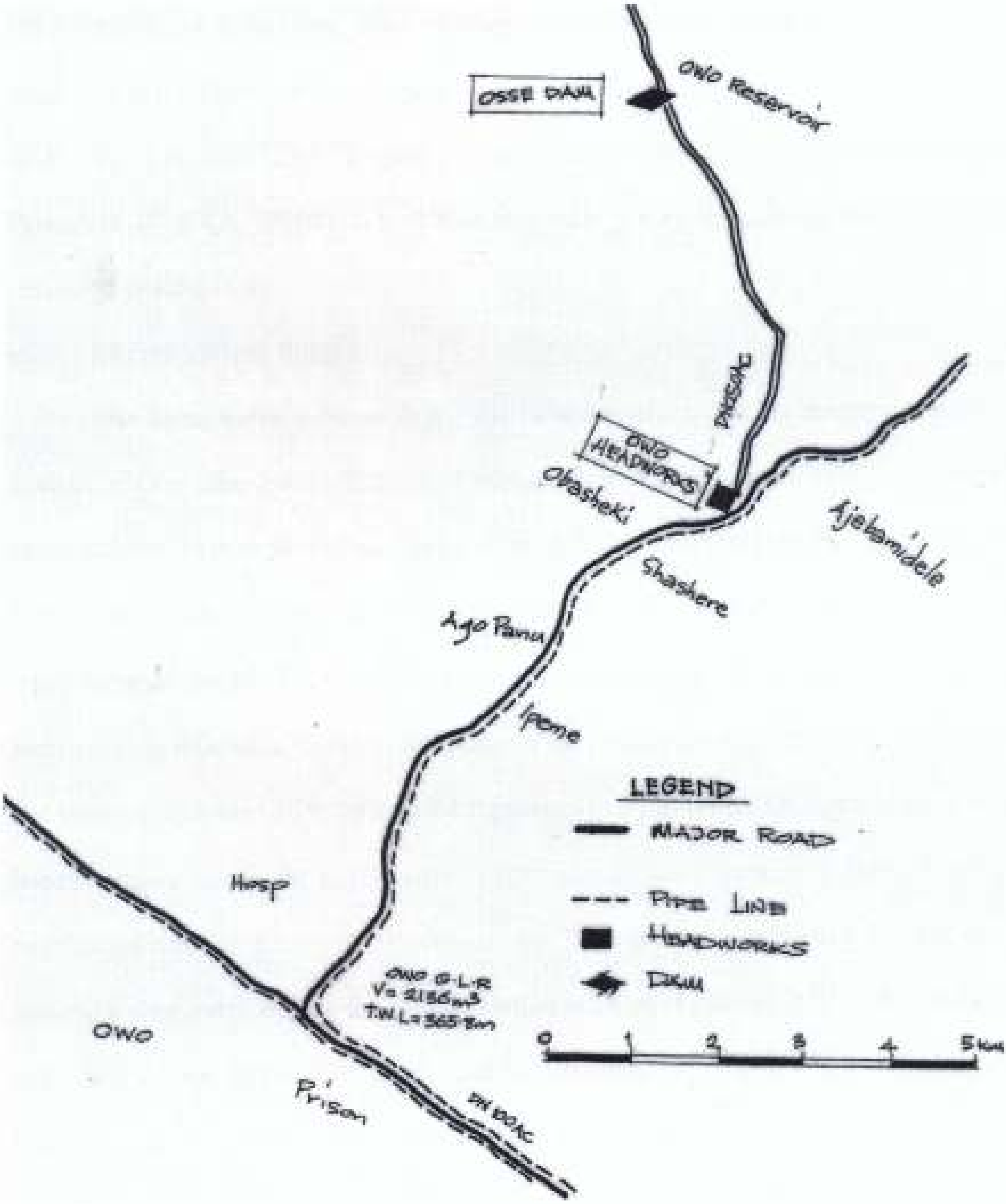


Figure 4.3. Owo Water Supply Scheme General Layout

$90m^3$ with the old conventional plant built in 1961. It was later adjusted to treat $120m^3/h$ in 1989. The high-lift pumping plant delivers water from clear water tank to Owo through rising main of 13km. There are two high-lift pumps provided with one newly installed in 1997. Each of the pumps is rated $123m^3/h$ against a head of 229m. Electric power is supplied through NEPA through a 500KVA, 33/041 KV. There is neither phone nor radiophone on this scheme. No vehicle is attached too.

vii) Okeigbo Water Supply

Oke-Igbo water scheme is a small scheme which has its source in River Oni at the outskirt of Oke-Igbo town. The intake was constructed in 1975. In 1979 a mini headworks was commissioned to provide the raw water treatment with reticulation network in the town. With the population of about 190,000, Okeigbo today is not enjoying good supply of water. The topography of the town coupled with the epileptic electric power supply have been the major barriers. This scheme is having a fair share of the on-going rehabilitation works. The headworks had been rehabilitated with three low-lift pumps (LLP) installed. Okeigbo scheme supplies Ondo booster station which the later boosts to the two number reservoirs at Barrack road (Ondo) for distribution through gravity to the consumers. The pipeline transporting potable water from the scheme's clearwater well to the booster station at Ondo is 200mm A/C. The pipeline is fatigued and often records frequent bursts. In fact investigation revealed that for some years now, water has not gotten to Ondo booster station majorly because of the series of leakages and bursts which has defied all repairs.

viii) Ondo Water Supply

Ondo town gets water form two sources Owena/ Ondo headworks and Okeigbo headworks. The Okeigbo line connects the (2 numbers) reservoirs ($1200m^3$) located at Barrack increase in the number of consumers along the rising mains, resulting in reduced head, the two reservoirs earlier mentioned rarely get water for years now. From Owena-Ondo headworks, there is a 300mm A/C pipeline up to Ile-Oluji from a 150mm A/C now links the reservoirs. However due to low hours of pumping as a result of erratic power supply, water from Owena-Ondo headworks hardly get to the township. The seeming beneficiary of the present dispensation is Stamark Nig. Ltd, a company located on Akure-Ondo road.

ix) Ile-Oluji Water Supply.

There is a 150mm pipeline branch to Ile-Oluji from the 300mm A/c rising main going to Ondo. A booster station is located at Ita-nla that receives water from the 150m A/c. From here water is boosted to Ile-Oluji. One of the major problems facing Ile-Oluji supply is the low hour of pumping from Owena-Ondo. The arrangement is such that supply to Ondo can be short for Ile-Oluji and vice versa. Water supply to Ile-Oluji is currently very poor. The major sources of water to the town are wells and boreholes.

x) Egbe Dam (Little Osse)

The scheme which started in 1977 was only commissioned on 25th January, 1989. It is an impoundment on Little River Osse. It has a design capacity of 66,000m^3/day and a life span of 50 years. Apart from little Osse, other sources include Eku, Ewu, Esu and Ero rivers. The main source Osse River is seasonal and has a safe yield of 94.3m^3/hr. Raw water from this reservoir is occasionally coloured and has a wide range of physical and chemical properties.

Egbe dam has a length of 4 km and a height of 54.4m. Its raw water pumping house has a capacity of 1200m^3/hr with 5 low-lift pumps, out of which only three were working at maximum capacity as at the time of this study. There are ten highlift pumps, five of which is serving Irun distribution line and only three out of this are working at full capacity with a capacity of 1040m^3/hr. Three pumps working at maximum capacity are stationed to serve Aisegba distribution line with a capacity of 510m^3/hr. There are two other pumps serving the pumps as u-wash pumps. The scheme has a canalized treatment plant which has an output of 2680m^3/hr presently.The scheme was designed o serve four booster stations. Due to the creation of Ekiti state from the old Ondo State, the scheme is presently not serving any of the booster stations at Irun, Eshe, Oyin and Odowara because they fall within the new Ondo State. Before the creation of state, the scheme was serving 39 community reservoirs out of which only eleven fall within the present Ekiti State.

The scheme which was designed to serve sixty-six (66) town and villages is presently serving only eleven (11) towns and villages in Ekiti State. These include Imesi- Lashigidi, Ode, Agbado Aisegba, Iluomoba, Ijan, Ise, Emure, Eporo. Distribution is done between the average of 5-10hrs daily. There is an acute shortage of chemicals generally at the scheme as at the time of this study.

The scheme has a power house equipped with three transformers for power supply from NEPA and two standby generating sets (Diesel engine).A pick-up (Peugeot 504) van is attached to this scheme but no communication equipment such as radiophone or telephone as at the time of this study (Year 2002).

xi) Ero Dam

Construction of the scheme started in 1977 by Nigeria Water Resources Corporation and was commissioned on the 15th of March, 1985. The multipurpose scheme has a design capacity of 104500m^3/day with a life span of 50 years. It is located on Ero river which is seasonal and usually gives low yield for the five months of the dry season. The cost of the scheme at the time of construction was put at N200 million. The dam has a length of 620 and a depth of 22m.

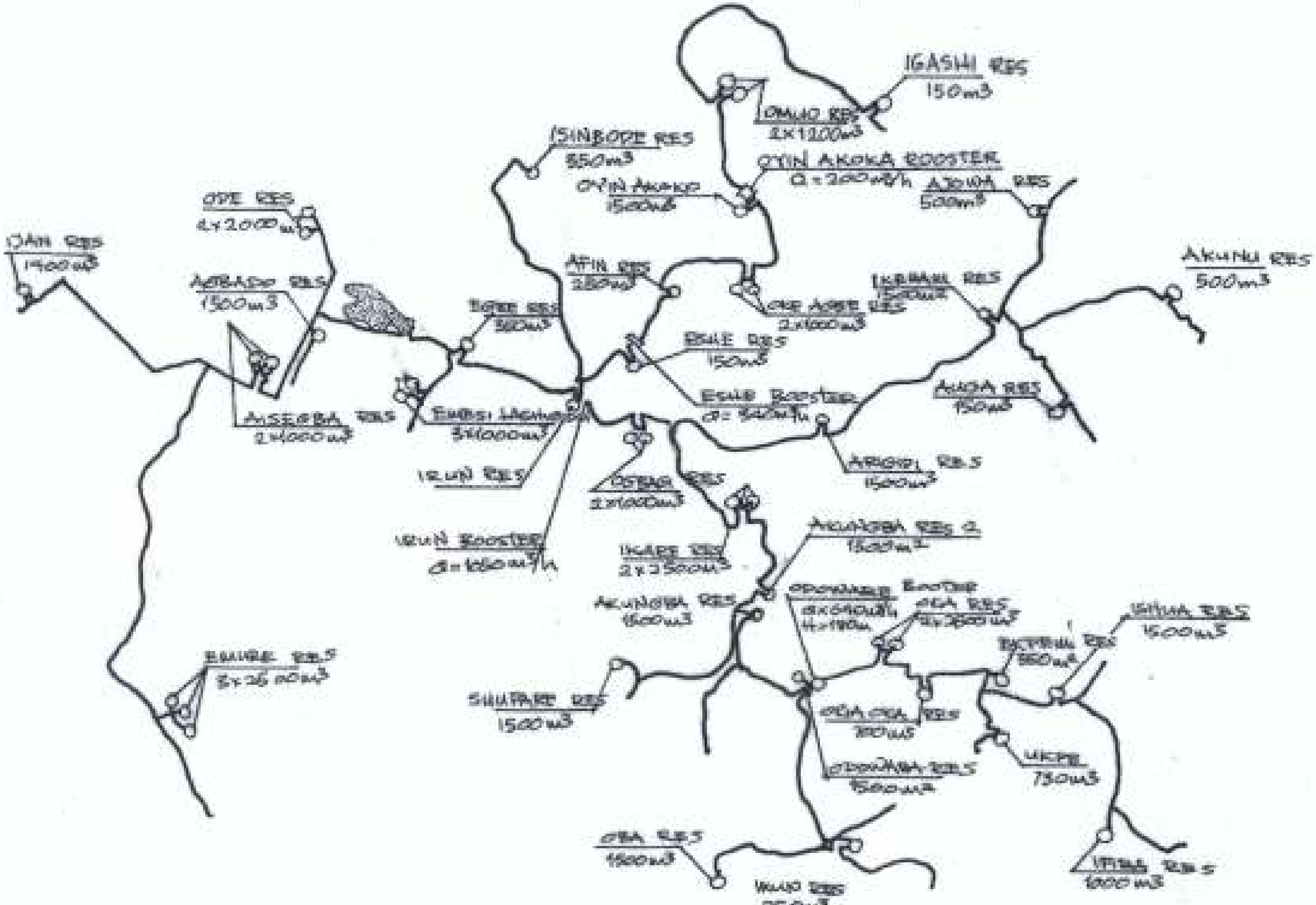


Figure 4.4. Egbe Water Supply Scheme

A 33m-length spillway with three radial gates which open up to 1.22m and clear water well of 125,000m^3 capacity are available in the scheme. There is a power house equipped with three transformers. An up wash tank for intermittent flushing of the filter bed is also available. One of the structures found on this headwork is dilapidated staff quarters. The scheme has a raw water pump-house which consists of low-lift pumps with a capacity of 1200m^3/hr out of which three are used at a time depending on the turbidity of the water.

At the treated water pump house, there are 11 high-lift pumps with six serving Otun distribution line with a capacity of 800m^3/hr, three serving Iye-Ekiti distribution line with a capacity of 270m^3/hr while the other two with a capacity of 100m^3/hr serve as dam wash-up pumps. The scheme has a centralized treatment plant which in turn has all the necessary components. In the treatment plant house, there is a laboratory for water quality analysis. The average production of the treatment plant is 2400m^3/hr

Booster stations: The scheme is designed to serve five booster stations viz: Ishan, Ayetoro, Igede, Aramoko and Orin. However, it is presently serving eight booster stations out of which Ero is one. The scheme is serving 56 reservoirs scattered all around the places which the scheme is supposed to serve. The scheme was originally designed to supply potable water to eight-towns and villages. Presently with a producion capacity of 545520m^3/month (approximately 1800m^3 day.), scheme serves in addition, part of Ado-Ekiti viz: University of Ado Ekiti and Barwa Low Cost Housing State.

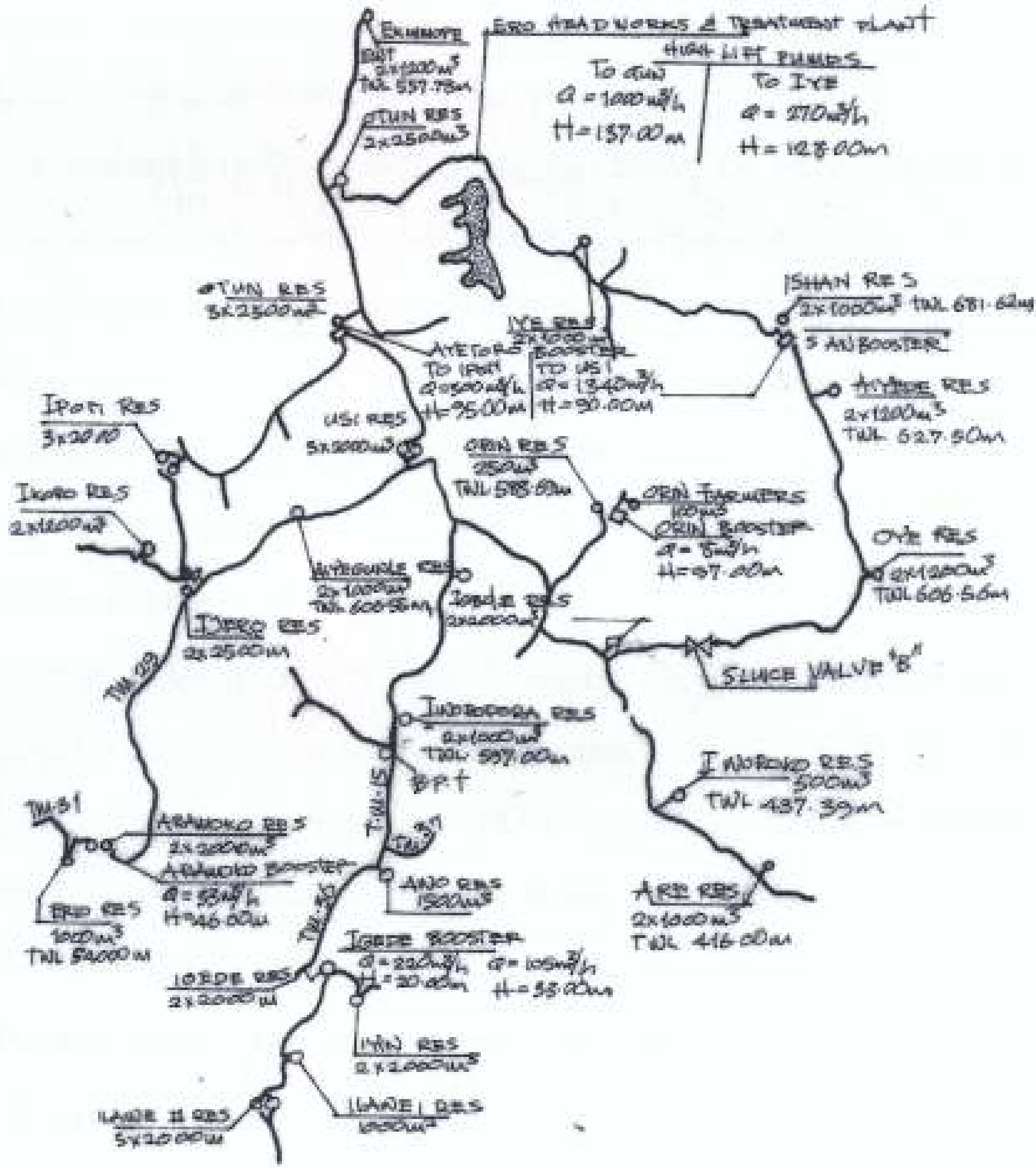


Figure 4.5. Ero Water Supply Scheme

As at the time of this study, the scheme was consuming an average of 230 50-kg bags of alum per month with 110 50-kg bags of lime and 4 drums of granular chlorine or 2 cylinders of chlorine gas per month. The schemes store of chemicals was still filled with different types of chemicals that can last for almost five years.

Power supply to this scheme is mainly from NEPA. The scheme has three transformers. Apart from this, the scheme has two stand-by diesel engine generating sets.

It is pertinent to mention that the major problem of this scheme is power supply. Power supply from NEPA is highly erratic. The corporations cannot cope with provision of diesel especially with its low level of funding neither can the revenue compensate for the huge operational expenses.

xii) Ureje Dam

Ureje dam (figure 4.6), is one of the oldest water supply schemes in Ekiti State and is located at Ado-Ekiti. The scheme was constructed in the 50's at the cost of N1.51million (year 2000 conversion rate) and commissioned in 1961. The scheme has a design capacity of 4930m^3/day. It was designed to serve a population of 213,036. The scheme utilizes Ureje River as its source of water supply by means of impoundment and intakes at the Ureje dam. This dam consists of an earth filled embankment and a 45m long concrete spillway which also serves as a washout. Fishing activities were conducted in the reservoir in the early years, making the scheme to be considered as multipurpose, small as it was. There are two low-lift pumps at the scheme for raw water with a capacity of 100m^3/hr at a head of 120m. All these pumps are new. There are also two new up-wash pumps with a capacity of 250m^3/hr.

Water treatment on this scheme is centralized. The treatment plant is very standard and new being part of the recently executed World Bank assisted National Water

Rehabilitation Project. The quantity of water currently being treated stands at $225m^3/hr$. The only booster station (Fajuyi booster station) is not functioning due to the obsolete pump and generating set. As at the time of this study, it was under rehabilitation.

Before, the scheme was serving four service reservoirs viz:-Ado-Ekiti service reservoirs ($1350m^3$), Iyin/Igede service reservoir ($450m^3$), Fagbote service reservoirs and Mary Immaculate Hill reservoir. Currently, the last two are not being served due to communal disagreement between Ado-Ekiti and Ikere-Ekiti. In their place is a new reservoir serving some part of Ado-Ekiti including GRA. The original distribution system of this scheme was designed to serve Ado, Ikere, Iyin, Igede, Awo and Aromoko communities. Currently the scheme is serving only Ado-Ekiti with an extension to Elekute and Bamgboye along Ikere road. Water is currently being distributed to each street once in a week on an average of 3-5 hours each day of service.

As at the time of the execution of this study, the scheme was equipped with assorted chemicals that can feed the scheme for two-three years at the present production capacity. The scheme has a powerhouse equipped with a transformer for supply from NEPA and a newly installed diesel engine electricity generating set. Beside this, there is old refurbished power generating set. There are three vehicles and two water tankers on this scheme apparently because it is serving the state capital. However there is no communication (telephone, radio etc) equipment.

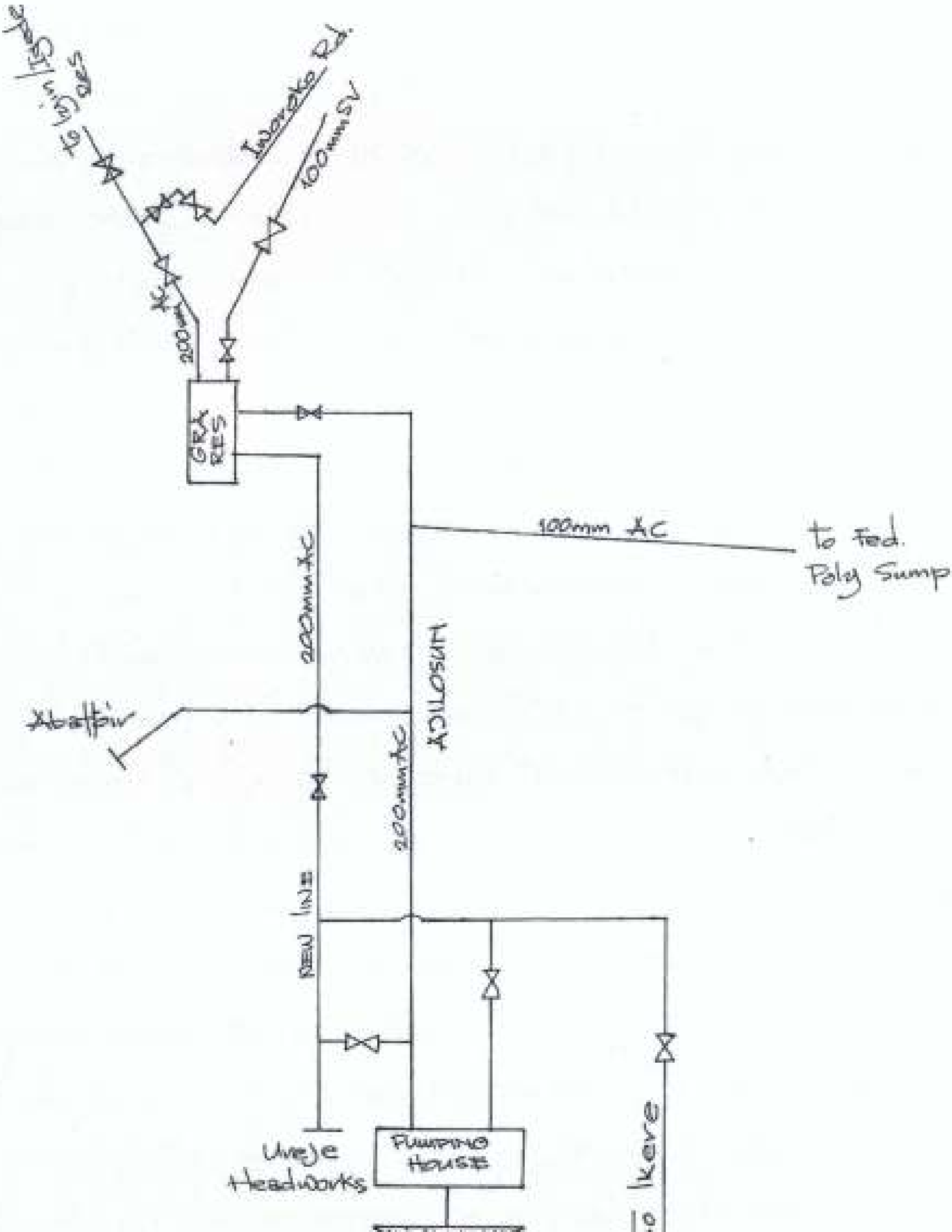


Figure 4.6 Ureje Water Scheme

xiii) Itapaji Dam

Itapaji dam located at about 4km from the actual town (Itapaji-Ekiti) is one of the oldest water supply schemes in the old Ondo State. It is a medium grade scheme whose construction started in the old Western State in 1973. The cost of the scheme then was $0.93 million, which is about N995million at the present exchange rate of N107 to $1. It has a design capacity of $4091m^3$/day. The scheme utilizes the Ele and Oye rivers as its sources by means of impoundment and intake at Itapaji . The scheme also consists of a 50m long spillway of rigid mass concrete (washout), clear water well of $160m^3$ capacity.

As at the time of this study, there are three new low-lift umps with a capacity of $150m^3$/hr each and only two at this are used at maximum production capacity. Also present are two high-lift pumps (one old, one new) with a capacity of $295m^3$ each to service Odo-Oro distribution line using one at maximum capacity. There are other two newly installed backwash pumps with capacity of $295m^3$/hr also. The scheme has a centralized standard treatment plant but the quantity of water that can be treated could not be ascertained as at the time of this study. The scheme is presently serving two-booster stations viz: Odo-Oro (now Araromi) and Ilasa booster stations. The scheme feeds seven reservoirs, located at These are Itapaji, Omuo, Aiyebode, Ilasa, Ipawo-Omu, Ikole and Ijelu all within Ekiti State. The scheme was designed to serve 14 towns and villages: Itapaji, Ikole, Itapa, Aiyegbaju, Ijesa-Isu. Aiyedun, Ara, Ode, ire, Ilupeju, Omuo, Ilasa, Ipawo and Aiyebode. It went out of production since 1994 and none of the above mentioned community had received water from it since then.

Power supply is achieved through a powerhouse having a transformer for power supply from NEPA. There is a new generating set installed apart from the existing old one which is not functional. There is no means of transportation provided on this scheme despite its long distance from Itapaji village. The alternative is trekking. However there is a radio receiver present on the scheme. As at the time of this study, the receiver is not functional because it has not been decoded from Ondo State communication code.

4.3 Socio-Economic Appraisal

4.3.1 Population Distribution per Household in State Capitals

Figure 4.7 shows the results of the survey carried out to test the population distribution per household within the two state capital cities. The result of this study is necessary to analyze the pattern of demand/consumption within the two cities.

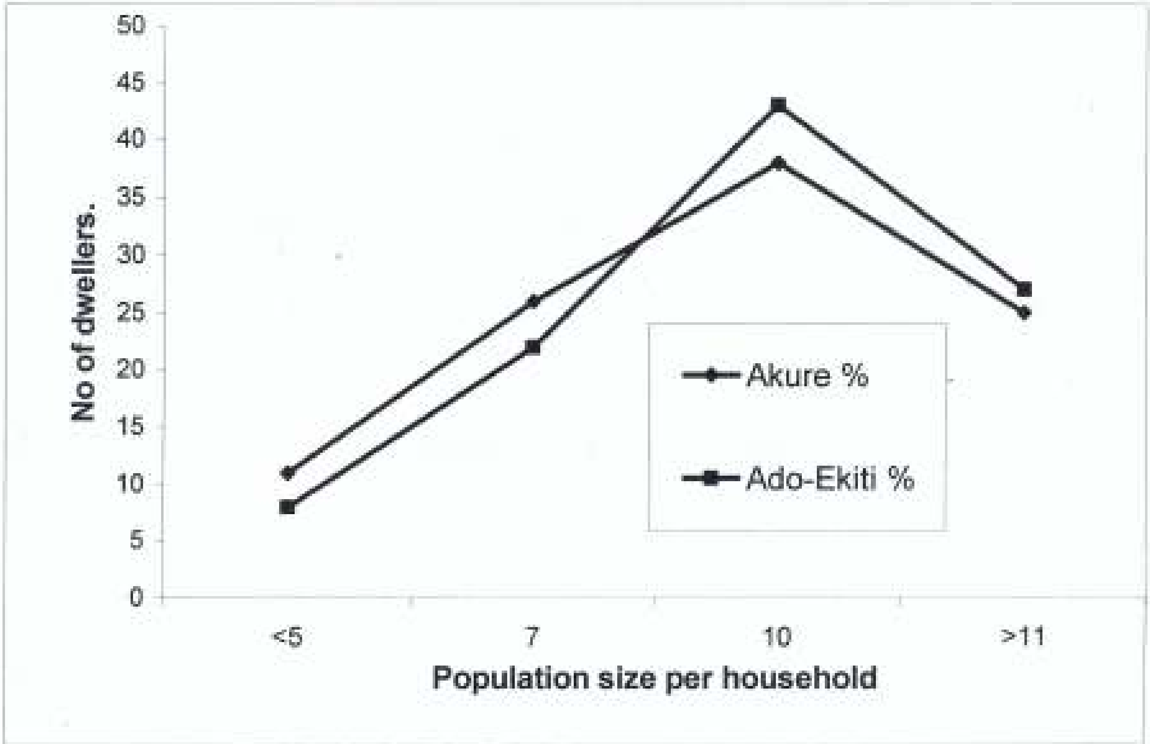


Fig 4.7: Household (size) in Two Urban Centers.

The figure shows that the population per household in the two capital cities is not statistically different. This also indicates that the pattern of living and that of water consumption in the two towns are similar.

4.3.2 Public Dependency on Different Sources of Water

Figure 4.8 presents the result of the survey carried out to study the level of dependency of demand centers (urban) on different sources of water. This was necessary to get the true picture of the consumers' dependency on different sources of water in the study area. The results revealed that:

i) The performance of the Water Corporation (the state's water agency) to distribute water to the populace is poor;

ii) There is heavy dependence of the populace on well water in many towns under consideration except in Okitipupa and Igbokoda;

iii) Wells thrive better in communities located in the basement complex than those in the sedimentary complex. This accounts for the reason why the level of dependency on boreholes in Okitipupa and Igbokoda is high (close to 70%);

iv) Level of dependence on other sources such as stream and rivers is very low in all the towns considered.

4.3.3 Closeness of Wells to Septic Tanks and Toilets in Households

Considering the level of dependency on well water in urban centers from the results of earlier studies, it is necessary to consider the quality of water from the well, which can be affected by their closeness to septic tanks/soakaways in households. This is more so important since many dwellers, apart from other uses, also drink the water from their wells without any serious treatment. A study of this nature was earlier carried out in Ibadan and some other communities in Oyo State (Sangodoyin, 1993). Figure 4.9 shows the results as obtained from the field studies. The following facts can be deduced the results:

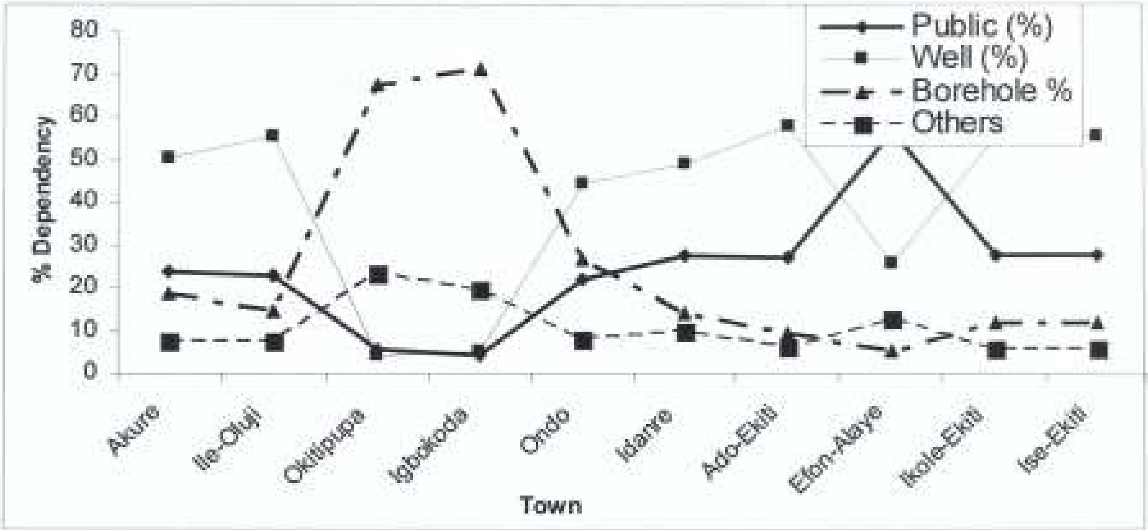


Fig 4.8: Sources of Water Supply and the Level of Dependency

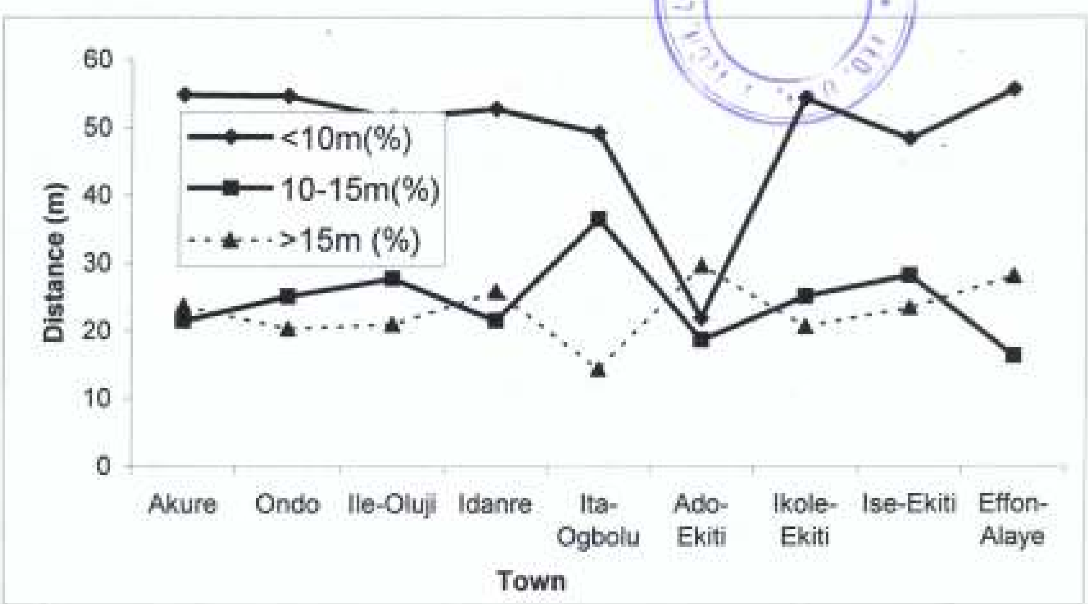


Fig 4.9: Closeness of wells to Septic tanks/Toilets in Households

i) The consumers are not aware of the possibility of their wells being polluted by water from soakaways.

ii) They are not aware of the WHO recommendation of 30m minimum distance between a well and a soakaway;

iii) Virtually all the wells are sited only with the aim of getting water and in good yield too;

iv) There is usually no communication or agreement between adjacent land owners as to how and where to site their wells and soakaways to avoid possible groundwater pollution;

4.3.4 Quality of Water from Some Boreholes and Wells in Akure Metropolis

Earlier studies revealed that there is heavy dependence of urban dwellers of the study area on water from wells and boreholes. Besides, one of the objectives of this work is to ascertain that the water gets to the consumers in good quality. These among others prompted the investigation of quality of water in the wells and boreholes within Akure metropolis. Similar exercise can be carried out in any other town within the study area The results of the water tests carried out on water samples from some boreholes and wells within Akure metropolis are as presented in Appendix IV. They are presented in form of average monthly variations of different water quality parameters. From the results of this study, the following deduction can be made:

i) Qualities of water in the boreholes do not vary much within the months under consideration and they are still within the limit of WHO recommendations;

ii) There are variations in the quality of water from well to well and from month to month;

iii) Water from many wells do not meet the required standard which indicates that pollution of ground water is taking place one way or the other.

4.3.5 Households with Water Connection.

In this study, the percentages of the urban dwellers actually connected to water distribution systems and as such are benefiting them and the number of taps available to them was determined. This is crucial to the planning, production, operation of any water agency and the expected revenue which is the backbone of the system's sustainability. The survey was limited to three towns in each state the state capital inclusive while others are local government headquarters. The picture is as shown in Fig 4.10.

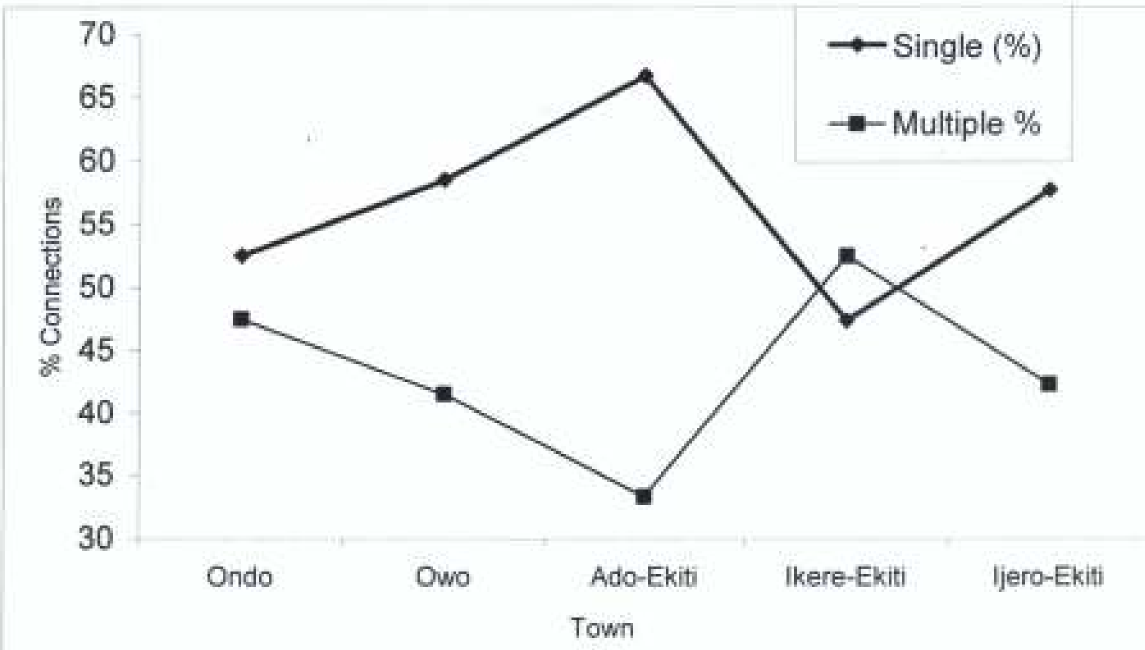


Fig 4.10: Percentage of Households with Water Connection in Some Urban Centers

The results of the study revealed that:

i) The highest percentage of house connection is found in Ado Ekiti and they are single-tap connections.

ii) In all the towns considered under this study, less than 50% of the houses have multiple connections.

4.3.6 Pattern of Water Consumption in Some Selected Urban Centers

A survey was carried out in some towns within Ondo and Ekiti states to examine the pattern of water consumption and the level of dependency on wells as sources of their supply. The results of this study are as shown in Table 4.3. The level of dependency was categorized as total, partial, and zero, while the quantity of water obtainable was measured in liters and ranged but the results of that study are expressed in percentages of respondents.

Table 4.3 -Pattern of Water Consumption Within Some Urban Centers

Town	No of households	Ave. No of dwellers	Percentage Dependence on well			Average Number of 10 litres bucket obtainable/day (%)			
			Total	Partial	Zero	>10	8	5	<5
Akure	1000	11	65	27	8	19	28	38	15
Owo	750	9	68	28	4	9	31	40	20
Ondo	750	9	61	29	10	10	25	40	25
Idanre	750	8	66	30	4	6	22	59	23
Ikare	750	10	58	36	6	15	24	40	21
Okitipupa	750	9	17	22	61	24	40	22	14
Ado-Ekiti	1000	10	67	28	5	28	41	33	11
Ilawe-Ekiti	750	10	63	34	3	9	26	48	17
Ijero –Ekiti	750	9	70	26	4	7	24	51	18
Emure –Ekiti	750	8	78	16	6	12	22	52	14
Agede – Ekiti	750	8	62	30	8	8	31	51	10
Ikere-Ekiti	750	9	63	30	7	10	28	49	13

Survey conducted - Year 2001 – 2002

The results obtained from this study revealed that:

The level of total dependence on well water is high in all the towns sampled except in Okitipupa ;

The average yield of all the wells sampled range between 5liters and 8liters per day;

The yield of individual well is too low for the average number of dwellers.

4.3.7 Emergence, Acceptance and Growth Rate of "Packaged Water" Business

The history of wrapping water in cellophane selected dated back to the time when it was no longer possible for people to easily get water from public taps to drink especially in the urban centers. This lack of public water led some people to start making a living by wrapping water and selling it to people on request. The situation of water supply in most urban centers gradually deteriorated further to a level where private individuals have now taken over the supply. Now, privately owned boreholes are very common in many urban centers, selling water either through tanker service, people coming there to buy on a fetch and pay basis or packaging of water in sachets or plastic bottles popularly known as "pure water or spring water". The sale of such "packaged water" was rare in the area of study up to the early 80's.

The results of field survey which was carried out in some urban centers in the study area and the analysis are shown in Figures 4.11 and 4.12.

i) Many of the respondents have accepted 'pure water' as a reliable alternative because Water Corporation has failed in it's duty;

ii) The emergence and rapid growth of 'pure water' business is as a result of poor economy;

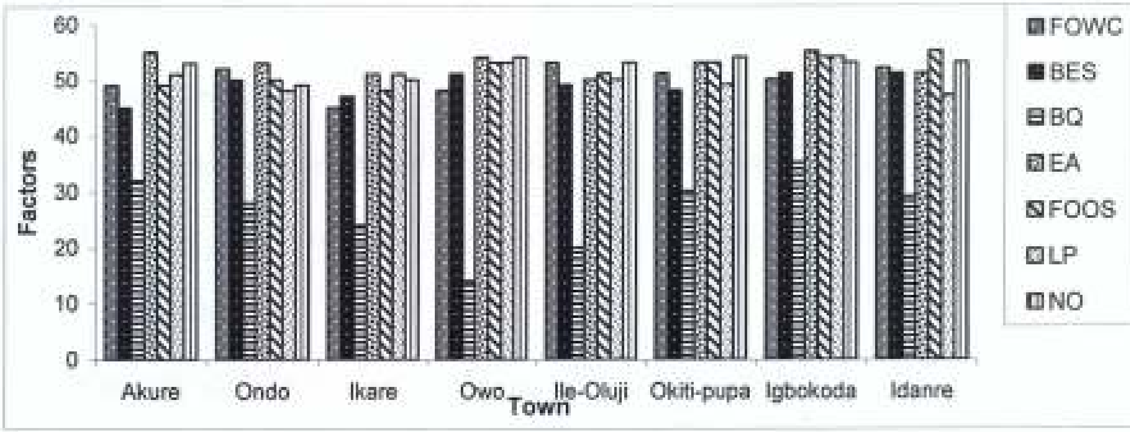


Fig 4.11: Factors Responsible for the Emergence/Acceptance of Packaged Water ("Pure water ")

FOWC-Failure of Water Corporation BES- Bad Economic Situation
BQ-Better Quality EA-Easy Access
FOOS-Failure of Other Sources LP-Low Price NO- No Option

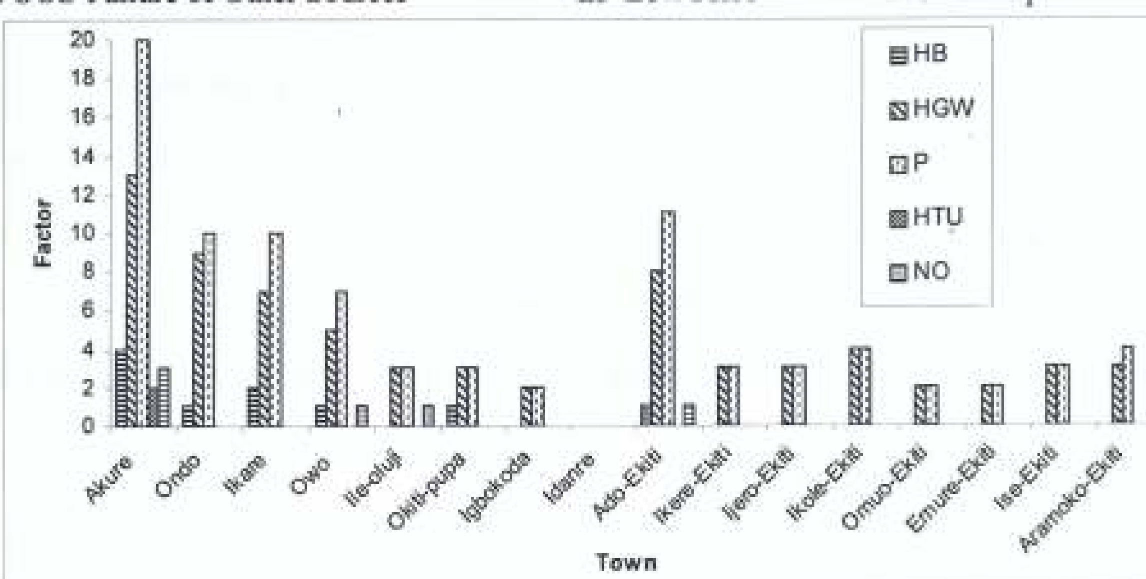


Fig 4.12 : Factors Responsible for Rapid Increase of "Pure Water" Producers.

HB-Have Borehole HGW-Have Good Well P-Profitability

HTU-Have treatment Unit NO-NAFDAC Number

iii) Acceptability of 'pure water' is as a result of its accessibility as a quick source of drinking water;

iv) Acceptance of 'pure water' is not because it has better quality;

v) The rapid growth of 'pure water' is largely due to its profitability and availability of good well;

vi) Majority of the producers do not have NAFDAC number.

4.3.8 Availability of Water Supply Networks In Selected Urban Centers

Availability of water distribution facilities was investigated in some selected towns of the study area. This was necessary to test the level of accessibility of the dwellers of such urban centres to pipe borne water and to determine the percentage of the consumers that can benefit from any improvement done to any of the headworks and booster stations. The survey was also important to determine the level of service that can be offered by the Water Corporation to those urban centres. All the towns selected for this study are important towns as far as the two states are concerned and they are relatively highly populated, the results of the survey are as presented in Table 4.4 .This survey was carried out with the assistance of staffers of Water Corporation in the Area Offices.

The results revealed that:

i) The present coverage of network in all the town sampled is generally low with Ikare having the highest of 56.4%.This indicates that less than 60% and in most cases less than 50% will effectively benefit from any improvement of the headworks. While urbanization is growing rapidly in virtually all the towns, not much is being done to expand the water supply networks.

Table 4.4- Percentage coverage of Water Supply Network in Some Selected Towns in Ondo and Ekiti States.

Town	Percentage coverage
Akure	48.6
Ondo	53.0
Owo	47.3
Ikare	56.4
Okitipupa	52.6
Ile-Oluji	46.6
Ado-Ekiti	44.4
Ikere-Ekiti	54.2
Ijero – Ekiti	41.7
Ikole – Ekiti	47.7
Ido-Ekiti	52.1
Ifaki-Ekiti	48.2

4.3.9 Operational Constraints of Major Schemes

Performances of some of the major schemes were examined using some operational constraint factors. Seven parameters were considered and the variables were either Yes (Y) or No (N). In this case, the schemes were visited and the water superintendent for each scheme was interviewed. The results of the interview are as shown in Table 4.5

From the results of this survey, it is clear that:

i) The problem of most schemes is not inadequacy of raw water to be treated.

ii) The major problem is operational and managerial.

iii) There is lack of adequate funding to acquire some necessary spare parts for routine maintenance.

iv) There is an irregular supply of energy from NEPA or generator due to high cost of fuel. The results as shown in Table 4.5 revealed that management problems are responsible for poor production and supply and not inadequacy in the availability of raw water at the sources.

4.3.10 Statistical Analysis of Field Data

Average Daily Yield of Selected Hand-dug Wells within Akure Metropolis (Nov. 2000 – April 2001)

The data collected on average daily yield of selected hand-dug wells within Akure metropolis between November 2000 and April 2001 were analyzed using Analysis of Variance (ANOVA). The aim is to investigate whether there is significant difference in the average daily yield of water per month and between one zone and the other. The result is presented in Table 4.6.

Table 4.5:Summary of Constraints Affecting major Water Scheme Performance

Scheme	Energy supply	Fault break down	Inadequate chemical store	Cost of fueling Gen. set	Lack of Adequate water to fetch	Man power inadequate	Lack of Comm.. equipment	Delay in repair/ spare parts
Owena /Igbara-Oke	Y	Y	N	Y	N	Y	Y	Y
Awara	Y	Y	Y	Y	N	Y	Y	Y
Ureje	Y	Y	N	Y	N	Y	Y	Y
Ero	Y	Y	Y	Y	N	Y	Y	Y
Egbe	Y	Y	Y	Y	N	Y	Y	Y
Okeigbo	Y	Y	Y	Y	N	Y	Y	Y
Owena – Ondo	Y	Y	Y	Y	N	Y	Y	Y

Y = Yes N = No

Table 4.6. ANOVA Table for average daily yield of water per month (6 months; - zone)

Source of variation	Sum of squares	df	Mean square	Fcal	F-tab
Month	1539.633	5	307.927	30.041*	2.21
Zone	1935.267	7	276.467	276.971*	2.01
Error	2326.833	227	10.250		
Total	**5801.733**	**239**			

*= ***Significant at 0.05 level***

From Table 4.6 it is observed that the calculated F-ratio (30.041) is greater than the tabulated F-ratio (F_α 0.05, 5,227, 2.21) for the month. Since the calculated F-ratio is greater than the tabulated F-ratio it means there is significant difference in the average daily yield of the wells on monthly basis. Similarly, for the investigation based on zonal basis the calculated F-ratio (26.971) is greater than the tabulated F-ratio (F_α0.05,7,227,2.01). Hence it can be concluded that there is significant difference in the average daily yield of the wells from one zone to another.

Duncan Multiple Range Test (DMRT)

a) Four Months

DMRT is employed to investigate further, which of the months has average daily yield of water significantly different from one another. This helps to classify the months into homogeneous groups or subsets. The result of the classification based on the DMRT is presented in Table 4.7.

The yields in the months of February through April are not significant from one another. By February, the groundwater table has gone down sufficiently for the well to be yielding at very low level as compared with November through January, when the previous raining season still manifest significant influence. The result also is likely to be a reflection of the fact that groundwater abstraction is high around that period as there is heavy dependence on well water by different households.

Table 4.7: DMRT of Average Daily Yield of Hand- dug Well In Different Months.

Month	Mean ± S.D	No of means
November	11.90 ± 3.64^{d}	5
December	10.20 ± 3.94^{c}	5
January	8.425 ± 4.30^{b}	5
February	6.125 ± 4.54^{a}	5
March	5.275 ± 4.51^{a}	5
April	5.275 ± 4.60^{a}	5

NB: Means that carries the same alphabet or superscript belong to the same homogeneous subset.

Zone

In order to investigate which of the zones has average daily yield of hand dug wells significantly different from one another; we employ DMRT for the analysis and classification. The result is presented in Table 4.8.

Based on Table 4.8, zones 1 – 4 which carry superscript "c" are homogeneous in their average daily yield, zone 5 with superscript "ab" is different from other zones, zone 6 with superscript "b" is also different from other zones. Similarly zones 7 and 8 with superscript "d" and "a" respectively are different from one another and from other zones in the average daily yield of the hand dug wells during the period under study.

Table 4.8: DMRT of Average Yield of Hand Dug-wells in Different Zones.

Zone	Mean ± S.D
Zone 1	9.27 ± 3.40^{c}
Zone 2	8.53 ± 4.98^{c}
Zone 3	8.03 ± 3.06^{c}
Zone 4	9.57 ± 4.34^{c}
Zone 5	4.93 ± 3.55^{ab}
Zone 6	6.00 ± 2.85^{b}
Zone 7	13.13 ± 5.93^{d}
Zone 8	3.47 ± 3.57^{a}

Note: Mean value with the same superscript are homogeneous and are not significantly different from one another.

From the engineering point of view the two situations above can be explained as follows: Firstly, raining season is gradually winding up from the end of October. The month of November (the beginning of the measurement) can be called the beginning of dry season. Replenishment of groundwater from rainwater reduces and abstraction of ground water increased due to increase both in demand and in dependency on wells for water supply. Secondly, the variation on zonal basis can be linked to water bearing capacities of the different geologic formations occurring in those zones as well as average depth of wells and water-table.

Apart from these, virtually every house around the selected wells has its own well where water abstraction was going on simultaneously. This close range concentration of wells will bring about interaction between the wells, which definitely will affect the water-table and their yield. All these can be subjected to further study.

t-test of Sources of Water During Raining and Dry Seasons.

In order to investigate whether the source of water by the respondents interviewed are statistically different between raining season and dry season, paired samples t-test was used. The result is presented in Table 4.9.

Table 4.9. Paired t-test of Difference Between Sources of Water During Raining and Dry Seasons.

Comparison	No of samples	df	t-calculated	t-tabulated	Remark
Source of water during raining season and during dry season	180	179	2.269*	1.645	significant

df= degree of freedom

* Significant at 0.05 level

Table 4.9. revealed that calculated t-ratio is greater than tabulated t-ratio (t_{α}0.05, 179, 1.645). Hence, it can be concluded that there is significant difference in the sources of water between dry season and source of water during raining season. Thus, throughout the years, consumers still result to all available sources of water supply.

Chi-square Test of the Relationship between Amount of Bill from Water Corporation and Some other Variables.

In order to investigate whether there is significant relationship between the amount of bill from Water Corporation and other variables such as frequency of settling bills, refusal to pay bills, willingness to pay etc chi-square tests were carried out. The result is presented in Table 4.10.

Table 4.10. Chi-square Test of Relationship between Amount of Bill and Other Variables.

Variable	No of samples	df	Calculated chi-square	Tabulated chi-square	Remark
a. Willingness to pay		2	2.162	5.99	NS
b. Amount consumer is willing to pay		6	6.103	12.59	NS
c. How often bill is settled		4	13.033*	9.49	S

df= degree of freedomS = Significant

NS = Not significant *= Significant at 0.05 level

From Table 4.10., it is revealed that only the frequency with which bill is settled has significant relationship with the amount of bill sent from Water Corporation. Willingness to pay and the amount the consumer is willing to pay have no significant relationship with the amount of bill sent from Water Corporation. Hence it can be interpreted that the corporation will obtain a larger revenue if bills are dispatched more regularly.

Chi-square Test of the Relationship between Settlement of Bill from Water Corporation and Some Other Variables

The relationship between the frequency of billing and other variables such as ownership status of the person paying the bill, frequency of meter reading, whether the house has meter or not, whether the meter is in good working condition or not was analysed. The result is presented in Table 4.11

Table 4.11: Chi-square analysis of the relationship between how often bills are settled and other variables.

Variable	No of sample	df	Calculated chi-square	Tabulated chi -square	Remark
Ownership status of the person that pays bill	180	4	9.566*	9.49	S
How often meter is read	180	2	7.020	5.99	NS
Availability of meter in the household	180	2	4.211	5.99	NS
Working condition of meter	180	2	3.466	5.99	NS

df = degree of freedom S = Significant

NS = Not significant* = Significant at 0.05 level

From Table 4.11, it is observed that only ownership status of the person that pays the bill has significant relationship with how often the bill is settled. Other variables like how often the meter is read, availability of meter in the household and the working condition of the meter have no significant relationship with how often the bill is settled.

Stepwise Regression Analysis Of Factors that Determine the Amount of Water Consumed in a Household per Day.

Stepwise regression analysis describes the relationship between a variable called dependent variable and other variables referred to as independent or explanatory variables, stepwisely considered. It is used to investigate how some groups of explanatory variables stepwisely explain the variations in the dependent variable. Regression model is a analytic tool often used for predictive purposes. The relationship between dependent variable and the independent variable could be expressed implicitly as follows.

$$Y = f(x_1, x_2, x_3, x_4 \ldots\ldots x_n) \quad (4.1)$$

With respect to the factors that determine the amount of water used by a household per day, the relationship could be stated explicitly as follows:

$Y = b_o + b_1 x_1$

$Y = b_o + b_1 x_1 + b_2 x_2$

$Y = b_o + b_1 x_1 + b_2 x_2 + b_3x_3$

$Y = b_o + b_1 x_1 + b_2 x_2 + b_3x_3 + b_4x_4$

$Y = b_o + b_1 x_1 + b_2 x_2 + b_3x_3 + b_4x_4 + b_5x_5$

$Y = b_o + b_1 x_1 + b_2 x_2 + b_3x_3 + b_4x_4 + b_5x_5 + b_6x_6$

$Y = b_o + b_1 x_1 + b_2 x_2 + b_3x_3 + b_4x_4 + b_5x_5 + b_6x_6 + b_7x_7$

$$Y = b_o + b_1 x_1 + b_2 x_2 + b_3x_3 + b_4x_4 + b_5x_5 + b_6x_6 + b_7x_7 + b_8x_8 \qquad (4.2)$$

In which,

Y = amount of water used by a household per day

b_o = constant term

$b_1 - b_8$ = regression coefficients (to be estimated)

x_1 = Availability of well

x_2 = No of people living the house

x_3 = Source of water

x_4 = Availability of WC

x_5 = Pattern of House

x_6 = Period of Service

x_7 = Performance tap close to House

x_8 = Availability of tap around House

e = error term

In each case, R^2 was evaluated to find out which of the variables is more predominant relative to Y. The empirical result of the stepwise regression models is presented in Table 4.12.

Table 4.12: Empirical Result of the Multiple Regression Model.

Variable	Co-efficient	Standard Error (b)	t-ratio
Constant	0.251	0.351	0.716
X_1	0.192	0.031	6.241*
X_2	0.235	0.054	4.383*
X_3	0.09749	0.033	2.943*
X_4	0.176	0.081	2.177*
X_5	0.132	0.051	2.572*
X_6	0.222	0.078	2.826*
X_7	0.09792	0.036	2.721*
X_8	0.0936	0.042	2.214*

* = significant at 0.05 level

$R^2 = 0.73$

$R^{2-} = 0.727$ (Adjusted R^2)

F ratio = F_α 0.05, 8,171, 9.219*.

The result presented in Table 4.12 reveals that adjusted R^{2-} is 0.727. This means that about 73% of the variations in amount of water consumed by a household per day are jointly explained by the six variables fitted into the model. Also, the table reveals that pattern of house, number of families living in the house, condition of the tap, availability of WC in the house and number of people living in the house is the significant determinants of amount of water consumed by a household per day.

The F-ratio (F_α0.05, 8,171,9,219) is significant at 0.01 level. This shows that the model derived could be used for prediction purposes with 99.9% confidence level.

The above stepwise regression models will be particularly helpful in policy formulation, planning and decision making. One only needs to take the appropriate census data along with various factors considered as independent variables to be able to determine accurately the water requirements for any particular community.

CHAPTER FIVE

DEVELOPMENT OF MATHEMATICAL MODELS

5.1 Preamble

The development of the models is aimed at providing an efficient tool of allocating water to individual consumers at the least cost and in a manner that maximizes the effectiveness of the entire water supply system.

For this study, two options are considered;

Option 1: Intermittent supply with rationing of water to consumers at minimum cost; and

Option 2: Supply on continuous basis through inter- water transfer between existing water schemes also at the least cost.

Under the first option, water is allocated to the consumers within the same demand centre (e.g a city) at different periods. In this case, a city can be divided into M demand centres i.e. (1, 2,.........m) and then the available water is allocated among them for ranges of time to be considered e.g. 4hrs, 8hrs, and 16hrs time respectively. This rationing of the available water can also be between cities as demand centers that are getting their water supply from a source but which cannot cope with simultaneous supplies to all demand centers. Allocation cost is a function of allocation option chosen.

The main objective here then is to find:

(a) The allocation of flow to demand centers, and

(b) The allocation option, which will give minimum operating, cost for pumping or energy.

In this model development, the notation and input data are as detailed:

Notations

a = trench width/pipe radius

b = trench depth/pipe radius

C = cost to construct one unit length of pipeline

C_e = cost to excavate one unit length of trench;

C_{HW} = Chezy Coefficient

C_p = cost of pipe in one unit length of pipeline

C_t = cost to excavate and backfill one unit length of pipe trench

$D_{p,}$ = design factors pipelines

$e_{p.}$ = excavation costs per unit volume for pipelines

E = Value of energy lost fluid friction in one unit length of pipeline per year;

f_p = fluid friction factors in Chezy formula for, pipelines, and

g = acceleration of earth gravity force;

H = hydrographic-shape factor for pressure pipelines

Hg = hydrographic-shape factor for gravity flow in pipelines

I = annual interest on construction cost plus annual payment needed to retire the debt during useful life of the structure;

k_p = thickness of pipe/radius

l_p = costs per unit volume of pipe lining

M = total cost of one unit length of pipeline per year;

n_p = number of kilogram-meters in 1 kw-hr.

n_t = number of seconds in 1 year;

p = value of unit of energy in flowing liquid, in Naira per kilowatt-hour;

Q = rate of fluid flow in pipeline

Q_a	=	average rate of fluid flow in pipeline
Q_m	=	maximum rate of flow in pipeline
q	=	rate of flow per year, in hectare meters per year;
R_e	=	Reynolds number for pipes;
R	=	hydraulic radius
r	=	radius of pipelines
s	=	energy gradient of pipelines
s_g	=	energy gradient of gravity flow
s_m	=	energy gradient when flow is maximum;
t	=	time, in years;
V_e	=	average volume of excavation in one unit length of trench
v	=	velocity of flow (average over the cross section);
v_m	=	maximum velocity of flow (average over cross section)
γ	=	specific weight of liquid being conveyed; and
v	=	kinematic viscosity of liquid being conveyed

Input Data

a	=	3	-	width/radius
b	=	3	-	height/radius
e_p	=	₦600	-	excavation/m^3
L_p	=	₦6, 000	-	pipe material /m^3
K_p	=	0.1	-	thickness/radius
I	=	0.22	-	debt service factor
C_{EN}	=	₦6.00/kW hr	-	Energy cost/kW hr

C_{HW} = 90 -150 - Hazen-Williams Coefficient.

c_u = 0.278 - unit coefficient

5.2 MODEL I – Mathematical Model Describing Allocation of Water to Consumers

The main objective of model 1 is to supply water to demand centers through allocation of flow at the least operating cost for pumping or energy.

The objective function in general is of the following form

$$\min C_T = \sum_{j=1}^{n} c(Q_j) \tag{5.1}$$

subject to the following constraints

$$g_{i(Q\,j)} \leq b_i \; (i = 1,2,\ldots\ldots\ldots\ldots\ldots,m). \tag{5.2}$$

$$f_{j(Qj)} \geq 0 \quad (j=1,\ldots\ldots\ldots\ldots n) \tag{5.3}$$

where C_T is the total cost of allocation, and

c is the unit cost of supplying Qj allocation.

$$Q_j = \Sigma q_j \tag{5.4}$$

Besides a minimum pressure head must be maintained at each demand centre or point i.e.

$$H_s \geq H_{min} \tag{5.5}$$

The following assumptions are made:

(a) Since the rate of pumping remains constant and the time lag between each period is the same, then supply to all the nodes remain constant i.e.

$$q_{1t} = q_{2t} = q_{3t} = \ldots q_{nt} \text{.} \tag{5.6}$$

(b) A day is divided into n time periods of equal intervals 1,2, 3…N

It is necessary to supply at least $q_i \geq q_{min}$ to each demand center *i* at minimum cost. As earlier mentioned, the mode of meeting this can be expressed as any of the following:

Total daily consumption per demand		Period of day		
		1	2	............n
Q_1	=	q_{11}	q_{12}	............q_{1n}
Q_1	=	q_{21}	q_{22}	............q_{2n}
.	.	.		.
.	.	.		.
.	.	.		.
Q_m	=	q_{m1}	q_{m2}	............q_{mn}

(5.7)

The supply is on continuous basis with equal allocations in each time period.

In order to make operational definition of pumping cost a variable Q_{ijt} can be introduced to indicate the rate of flow in a pipe link ij during time period t.

Pumping cost includes cost due to elevation head and cost due to frictional head loss. Any increase in Q_{ijt} will lead to increase in pumping cost. From here it can be deduced that pumping cost of each link is a convex function of the discharge Q_{ijt}.

This kind of problem can be solved using piecewise linearization (Loucks et al, 1981). Here there are three basic methods. Usually for concave functions, it is maximization while for convex function, it is minimization. The case under consideration is a convex function. Fig. 5.1 can be used to explain this fact.

The problem can be formulated as

$$\text{Min } C_T = \sum_i \sum_j \sum_t c_{ijt} \, Q_{ijt} \tag{5.8}$$

Where C_T is the total cost and c is the unit cost of supplying Q_j allocation. $Q_j = \Sigma q_i$

$$Q_{ijt} = a_1 w_1 + a_2 w_2 + a_3 w_3 + \ldots\ldots\ldots\ldots + a_n w_n \qquad (5.9)$$

With the weighing factor,

$$w_1 + w_2 + w_3 + \ldots\ldots\ldots\ldots w_n = 1$$

$$\text{or } \Sigma w_i \leq 1 \qquad (5.10)$$

it can otherwise be written as

$$\text{Minimize } C_T = \sum_i \sum_j \sum_t c_{ijt}\left[(a_1)w_1 + (a_2)w_2 + (a_3)w_3 + \ldots\ldots(a_n)w_n\right] \qquad (5.11a)$$

Where w_i will be the variable to be determined and a represent a flow at a_i particular period at a particular node for a stated period of time.

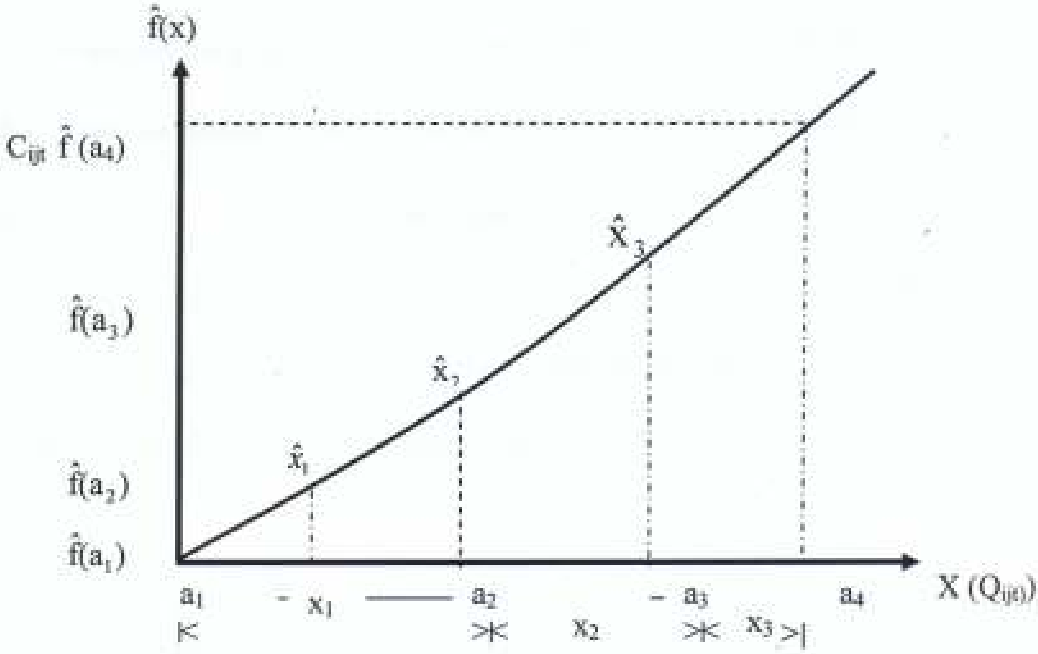


Figure 5.1: Graph of Convex Function

5.3 Scope of the Mathematical Model

It is expected that this model will provide the quantity of water to be supplied to each demand centre for each period with minimum pumping or energy cost. For the purpose of this work, the following variables have to be considered:

(i) the quantity of water to be supplied to each demand centre Qj is equals to the sum of small discharges,q_n i.e

$$R_j = \sum_{i}^{n} q_n \tag{5.11b}$$

(ii) the schedule for water allocation or rationing system;

(iii) the number of loops in the entire network;

(iv) the minimum target pressure for effective supply;

(v) the pumping head for each demand centre or reservoir height.

5.4 Characteristics and Constraints in Linear Equations

5.4.1. Flow Rate Constraints

This constraint requires that the sum of all flows to a given demand center j for any 24 hour period must not be less than total consumption (demand) at that centre (node), that is total flow to node j in period t is designated by Q_{jt} such that:

$$\sum_{t=1}^{24} Qjt \geq Q_j \qquad \forall (j = 1, \ldots\ldots J) \tag{5.12}$$

where Q_j is the expected consumption at node j

$$\sum_{t=1}^{24} Q_{jt} = \left[Q_{j1} + Q_{j2} + Q_{j3} + \ldots\ldots\ldots + Q_{jn} +\right] \geq Q_j \tag{5.13}$$

for any J-node network .the sets of linear equations below describe the physical characteristics of the flow systems

Node 1: $\qquad Q_1 \geq Q_{11} + Q_{12} + Q_{13} + \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots + Q_{1N}$ (5.14)

Node 2: $Q_2 \geq Q_{21} + Q_{22} + Q_{23} + \dots\dots\dots\dots + Q_{2N}$ (5.15)

: :

Node:j $Q_j \geq Q_{j1} + Q_{j2} + Q_{j3} + \dots\dots\dots\dots + Q_{jN}$ (5.16)

According to the law of conservation of mass

$$Q_1 + Q_2 + Q_3 + \dots\dots\dots + Q_j \geq Q_j \quad (5.17)$$

5.4.2. Continuity constraint

Flow received in a node j will be carried to another node j+1 through a link (pipe) and continuity equations will be written for each time period for which there is flow the link entering or leaving node j. The sum of water flows entering a node and the sum of flows leaving a node including the withdrawals during any time period are equal.

Figure 5.2: A Simple Flow Network

where D_j is the consumption at node j or generally,

$$\sum_{i=1}^{m} Q_{ijt} - \sum_{k=1}^{m} Q_{jkt} - D_{jt} = 0 \quad \forall j = 1........m. \quad (5.19)$$

$$t = 1..........24$$

with the non-negativity constraint

$$Q_{ijt} \geq 0; Q_{jkt} \geq O \quad (5.20)$$

However any of the flows can be zero e.g. $Q_{ijk} = O$, if there is no flow in the link during the time period t.

Each of the components of Equation 5.19 except D_{jt} can be written as follows

$$\sum_{i=1}^{m} Q_{ijt} = \begin{bmatrix} Q_{1j1} + Q_{1j2} + Q_{1j3} + \dots\dots\dots\dots + Q_{1j24} \\ + \\ Q_{2j1} + Q_{2j2} + Q_{3j3} + \dots\dots\dots\dots + Q_{2j24} \\ + \\ Q_{3j1} + Q_{3j2} + Q_{3j3} + \dots\dots\dots\dots + Q_{3j24} \\ \vdots \\ \vdots \\ + \\ Q_{mj1} + Q_{mj2} + Q_{mj3} + \dots\dots\dots\dots Q_{mj24} \end{bmatrix} \tag{5.21}$$

and also for $\sum_{k=1}^{m} Q_{jkt}$ as in Eq. 5.21 above.

The nodal demand, D_{jt}, is now given by

$$D_{j1}, D_{j2}, D_{j3} \dots D_{j24} \tag{5.22}$$

In general, the problem under consideration can be expressed as shown below

$$\text{Minimize} \quad \sum_{j} \sum_{e} \sum_{t} C_{ej_1} \ (a_1) w_1 + C_{ejt} (a_2)\, w_2 + C_{ejt} (a_3)\, w_3 + \dots \tag{5.23}$$

subject to

$$\underbrace{\sum_{t=1}^{24} Q_{jkt}} \ \geq \underbrace{\ Q_j \ } \qquad \forall j\ ; j = 1; \dots m \tag{5.24}$$

where Q_{jkt} is inflow into node j in time t.

Consumption at node j is given as

$$\sum_{t=1}^{m} Q_{ejt} - \sum_{k=1}^{m} Q_{jkt} - D_{jt} = 0 \quad \forall j\, t \tag{5.25}$$

$$Q_{ijt} = a_1\, w_1 + a_2\, w_2 \ \ a_3 w_3 \quad \forall\, ijt \tag{5.26}$$

$$w_i \leq 1 \tag{5.27}$$

$$Q_{ijt} \geq 0 \tag{5.28}$$

The objective of this model is to find the allocation of flows to demand centers an the best allocation option that will minimize the operating cost for pumping or energy.

To achieve this, the following data will be required:

(i) Location of supply sources and the demand centers;

(ii) Routes from the sources to the demand center;

(iii) Allocations to each demand center;

(iv) The distance and height from the centre.

In addition, other data, such as the friction factor (f) and the efficiency of the pumping system (i.e. motors and pumps) η_o as well as the unit cost of electricity (in ₦/kWh) must be known.

5.5 Cost Functions

Pumping cost: Pumping cost of water in distribution network is a function of the age of the pipe, the extent of corrosion damage in the pipes, quantity of water pumped, the length of pipe network, the diameters of the pipes, the head losses due to friction in the pipes and the head against which the water has to travel.

5.6 Cost of Operation

This can be obtained by equating the pumping power required for a link i.e. the total dynamic head (H_T) to that of an equivalent number of kilowatt-hours and then multiplying that by the corresponding unit cost

$$H_T = h_{f\,(i\text{-}j)} + h_{e}. \tag{5.29}$$

Where

h_e is the static or elevation that is required to lift the water form one node to another.

$h_{f\,(i\text{-}j)}$ is the loss due to friction between two nodes (i - j).

If the unit cost of overcoming friction between node i and j is C_{ij}, then Ci_j, $h_{f,(i\text{-}j)}$ is the total cost of friction head loss in link i-j assuming that C_{ij} $h_{f(e\text{-}j)}$ is continuous and monotone increasing as shown in Fig. 5.23below

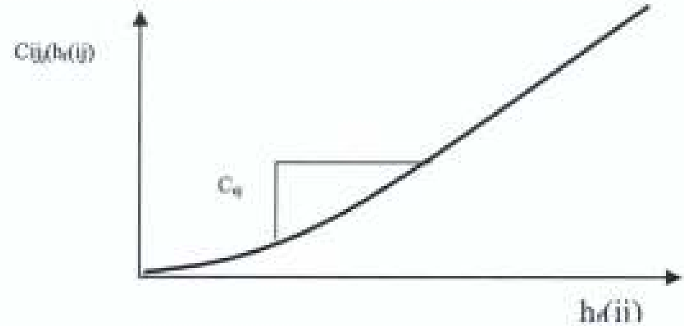


Figure 5.3: Graph of Cost Versus Head Loss

The implication of these assumptions is that the objective function is differentiable and convex in nature. Energy dissipation in a pipeline during flows is a function of the pipe roughness; diameter and discharge (Q).This can be expressed using Darcy-Weisbach equation

$$h_{f(i-j)} = \frac{16 fLQ^2}{19.6 D^5 \pi^2} = \frac{0.083 fLQ^2}{D^5} \qquad (5.30)$$

where

Q_{ij} is the flow between nodes i and j

f is the friction factor

L and D are the length and diameter of pipe between node i and j.

For $h_{f(i-j)}$ the horse power or energy for conveying a flow Q is given by

$$\Delta p = \frac{\gamma HQ}{19.6.D^5 \pi^2 1000\eta} \qquad (5.31)$$

where γ the specific weight of water kg/m^3

η - The efficiency factor of the pumping station

Putting the value of h_f into equation 5.31, we have

$$\Delta p = \frac{\gamma f\ LQ^3}{19.6.D^5 \pi^2.1000\eta} \qquad (5.32)$$

Daily pumping cost along link L is given by:

$$C_P = \frac{16\gamma fLQ^3 t.C_{en}}{19.6.D^5\pi^2 1000\eta} \tag{5.33}$$

Where C_{en} is the unit cost of electricity (tariff) in ₦/K WH. This value can be obtained from NEPA and it is N6/KWH (2002), t is the hours of pumping per day.

The overall pumping cost for all the links in the network can be written as:

$$C_T = \sum_{ij} C_p = \sum \frac{16\gamma fLQ^3.t.C_{en}}{19.6D^5.\pi^2.1000.\eta} \tag{5.34}$$

where γ is the specific weight of water (γ= 9810N/m^3)., and η is the pump efficiency

The option that was earlier mentioned can now be rewritten with 5.34, assuming the supply and consumption in each case is constant initially, that is,

$$A_1 = A_2 = A_3 = \ldots\ldots\ldots\ldots\ldots\ldots = A_n = A \tag{5.35}$$

where $A_1, A_2, \ldots\ldots A_n$ are the constants

To satisfy the consumption at each demand centre continuously over 24 hours per day, the equations can be written as

	t_1	t_2	t_3	t_{n-1}	t_n		consumption.	
Node 1	q_{111}	q_{112}	q_{113}	$q_{11(n-1)}$	q_{11n}	$\geq$	A	
Node 2	q_{121}	q_{122}	q_{123}	$q_{12(n-1)}$	q_{12n}	$\geq$	A	
	:	:	:	:	:	:	:	(5.36)
Node m	q_{1m1}	q_{1m2}	q_{1m3}	$q_{1m(n-1)}$	q_{1mn}	$\geq$	A	

where q_{1mn} indicates flow q for option 1, node m, time n. The flow constraints at network nodes for n-time periods can be written as follows:

First the flow in the links is:

	In period 1(t_1)	In period 2 (t_2)	In period n (t_n)
Along link S-1	$q_{111}+q_{121}+....q_{1m1}$	$q_{121}+q_{122}..+...+q_{1m2}$	$q_{11n}+q_{12n}+...+..q_{1mn}$

Further combination here will be a function of the network layout.

The optimization problem is to get the least total daily cost of pumping C_T

Recalling Eq. 5.35, this can be written as

$$C_T = \sum \frac{16\gamma fLQ^3.t.Cen}{19.6D^5.\pi^2.1000.\eta} = \sum Y\frac{(LQ^3)}{D^5} \tag{5.37}$$

where $Y = \dfrac{16\gamma ft.Cen}{19.6.\pi^2.1000.\eta}$

Now for policy 1,

$$C_T{}^1 = Y_1\left[L_{5-1}\frac{(q_{111}+q_{121}+....+q_{1mn})^3}{D^5{}_{s-1}} + \frac{L_{12}(q_{121})^3+}{D^5{}_{1-2}}.....+\frac{L_{(m-1)}-m(q_{1mn})^3}{D^5{}_{(m-1)-m}}\right]$$

$$+\left[L_{5-1}\frac{(q_{112}+q_{122}+....+q_{1mn})^3}{D_{1-2}{}^5} + \frac{L_{1-2}(q_{112})^3+}{D^5{}_{1-2}}.......+\frac{L_{(1-3)}(q_{132})^3+..+}{D^5{}_{1-3}}\frac{L(m-1)-m(q_{1mn})^3}{D^5{}_{(m-1)-m}}\right]$$

$$+\left[L_{5-1}\frac{(q_{11n}+q_{12n}+....+q_{1mn})^3}{D^5{}_{s-1}} + \frac{L_{1-2}(q_{12n})^3+}{D^5{}_{1-2}}.......+\frac{L_{(m-1)-m}(q_{1mn})^3}{D^5{}_{(m-1)-m}}\right] \tag{5.38}$$

It should however be noted that this arrangement can change depending on the network configuration.

The constraints are

$$\begin{aligned} &q_{111}+q_{112}+...+q_{11n} \geq A \\ &q_{121}+q_{122}+...+q_{12n} \geq A \\ &\vdots \qquad \vdots \qquad \vdots \qquad \vdots \\ &q_{1m1}+q_{1m2}+...+q_{1mn} \geq A \end{aligned} \tag{5.39}$$

It is observed that there are some terms in the objective functions that cannot be subjected to piecewise linearization such as $q_{111} + q_{121} + \ldots + q_{1mn}$. Therefore, some transformation is needed, thus:

$$q_A = q_{111} + q_{121} + q_{131} + \ldots + q_{11n}$$

$$q_B = q_{121} + q_{122} + q_{132} + \ldots + q_{12n}$$

$$\vdots \qquad \vdots \qquad \vdots \qquad (5.40)$$

$$\vdots \qquad \vdots \qquad \vdots$$

$$q_x = q_{1m1} + q_{1m2} + {}_{1im3} + \ldots + q_{1mn}$$

The above are the necessary transformations that are also constraints.

The above transformations are to be used in conjunction with piecewise linearization, method (Loucks et al, 1981) which was mentioned earlier.

A computer programme was written to solve this model and it has been tested. The details of this are given in the results section.

5.7 Model II– Mathematical Model Describing Inter-water Transfer between Existing Schemes.

The second model in this work is focused towards the analysis of relatively long distance transfer of water between two different schemes. Figure 5.3 shows a typical cross section of a pipeline.

In Nigeria, the main mode of large water conveyance from a reservoir to a treatment or distribution center is pipeline. This analysis therefore will be limited to pipelines.

In the analysis we shall consider the volume of excavation per unit length of pipeline trench (Ve)

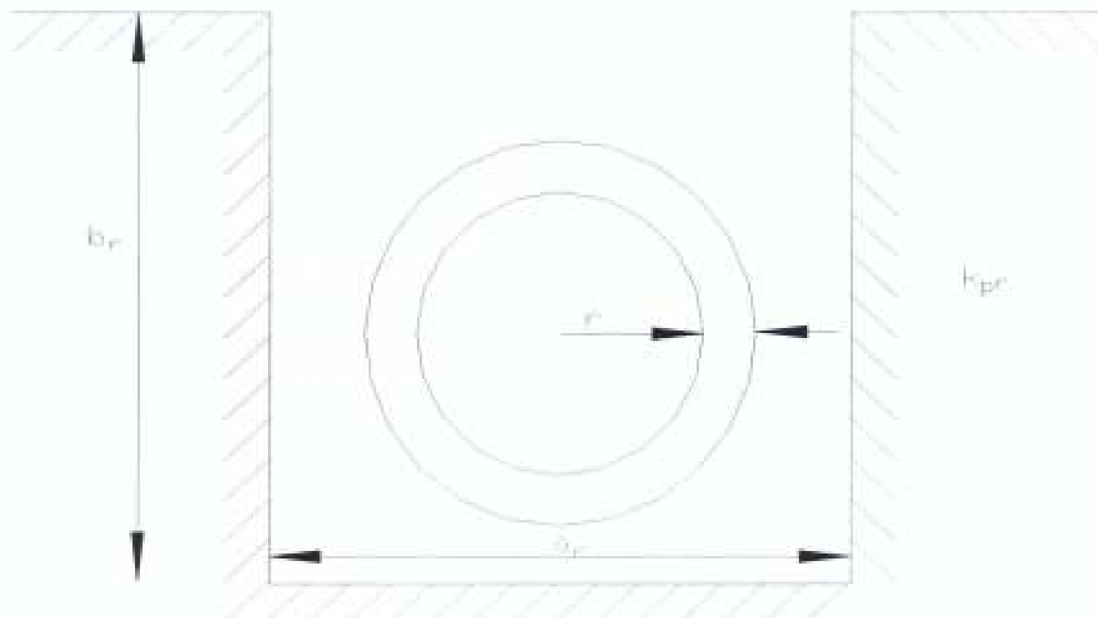


Fig 5.4: Cross – section of a Pipeline and Trench

From Fig 5.3, for a unit length

$$V_e = b.r \ \mathrm{x} \ a.r \qquad (5.41)$$

where br and ar are the depth and width of trench.

Cost of excavation from the above equation i.e cost of trench taking the cost per unit volume to be e_p is as stated below

$$C_t = e_p abr^2 \qquad (5.42)$$

Volume of material (V_p) in one unit length of pipelines equal

$$V_p = \pi \ (r + K_p r)^2 - \pi r^2 = \pi r^2 \ (2K_p + K_p^2) \qquad (5.43)$$

Let l_p be the total cost per unit volume of pipe in place in the trench including all coatings and so on. The total cost of pipe per unit length of pipeline is

$$Cp = l_p \ \pi r^2 \ (2K_p + K_p^2) \qquad (5.44)$$

The total construction cost per unit length of the pipeline is the sum of equations 5.42 and 5.44

$$C = C_t + C_p = e_p r^2 \left[\pi\left(2K_p + K_p{}^2\right)\frac{l_p}{e_p} + ab\right] \qquad (5.45)$$

Putting into consideration now the debt service factor (I) bearing in mind that most big projects are funded through loan, then the total annual cost is as follows

$$I\,C = e_p r^2 D_p I \qquad (5.46)$$

Where D_p is that part of Eq.5.45 in bracket. Now if we consider Q(m^3/s) as the instantaneous rate of flow or discharge and γ as the specific weight of water (Kg/m^3),

Taking n_s as the number of seconds in a year, then n_s, the instantaneous rate of energy dissipation in kilogram-meter per year per meter of pipe is given by $n_s \gamma QS$. If also n_p is the number of kg-meter in 1kw – hr, then, $\frac{n_s}{n_p}\gamma QS$ is the instantaneous rate of energy dissipation, in kW – hr per year per meter length of pipe. If C_{EN} be the unit cost of energy in the flow (C_{EN} /kw – hr), $C_{EN}\frac{n_s}{n_p}\gamma$ QS is the instantaneous rate of cost of energy dissipation.

Taking dE as the infinitesimal cost of fluid friction (C_{EN}/meter length of pipe) in an infinitesimal time dt (yrs),

$$\frac{dE}{dt} = \frac{C_{EN} n_s}{n_p}\gamma QS \qquad (5.47)$$

Usually the value of C_{EN} is a function of the class of consumer e.g private, public, institutional etc. This value of C_{EN} can be obtained from Water Corporation or NEPA

In order to replace the energy gradient (slope) S from 5.47 by a function of flow rate Q and the radius of the pipe, Hazen – William equation is valid for both pipeline and open channel. Thus, we can introduce

$$Q = 0.623\ CAR^{0.63}S^{0.54} \tag{5.48}$$

C is the Hazen – William coefficient which can be expressed as

$$C = \frac{R^{\frac{1}{6}}}{n} \tag{5.49}$$

By substituting the hydraulic radius $R = \frac{r}{2}$

and of energy gradient $S = \frac{E}{L} = \frac{H}{L}$, which can be written as

$$S = \left(\frac{Q}{0.623CAR \quad \left(\frac{r}{2}\right)^{0.63}} \right)^{1/0.54} \tag{5.50}$$

From this point, this model shall further be developed for pressure pipeline and gravity pipeline respectively

5.7.1 Pressure pipeline

By pressure pipeline is meant pipeline working under pumping conditions. The slope is represented by Spp.

Substituting Eq. 5.50 into Eq. 5.47 for S and simplifying

gives $$\frac{dE}{dt} = C_{EN}\frac{ns}{np}\gamma\left[\frac{Q}{\left(.623C\pi r^2(\frac{r}{2})^{0.63}\right)}\right]^{1/0.54} \tag{5.51}$$

Further simplification yields

$$\frac{dE}{dt} = 0.649 \ C_{EN}\frac{n_s}{n_p}\gamma \ C^{-1.85} \ r^{-4.48} \ Q^{2.85} \qquad (5.52)$$

Integrating eq. 5.52 over 1 year period yields

$$E \ = \ 0.649 \ C_{EN}\frac{n_s}{n_p}\gamma \ \ r^{-4.48} \ C^{-1.85} \int Q^{2.85}dt \qquad (5.53)$$

For convenience, the definite integrals in Eq. 5.53 will be replaced by equivalent expressions as bellows.

From earlier works on pipeline studies, for pipelines working under gravity, maximum flow (Qm) will be achieved when the pipe must flow full. This is to say that Q = Qm when the pipe is flowing full.

Therefore, the equivalent of Eq. 11 for gravity pipeline will be

$$E \ = \ 0.649 \ \ C_{EN}\frac{n_s}{n_p}\gamma \, r^{-4.48} \, C^{-1.85} \, Qm^{1.85} \int_0^t Qdt \qquad (5.54)$$

The above expression is necessary because a water distribution system usually contains both pressure and gravity flows.

For convenience, the definite integrals 5.53 and 5.54 will be replaced by equivalent expressions

$$\int_0^t Qdt = Qm2.85 \int_0^t \ \left(\frac{Q}{Q_m}\right)^{2.85} = Q_m^{2.85}H \qquad (5.55)$$

where $H = \left(\frac{Q}{Q_m}\right)^{2.85}$ is the hydrographic shape factor.

It may be evaluated numerically or graphically over one year by plotting Q as functions of time, in years and summing the area under the curve.

Now substituting Eq. 5.55 into Eq. 5.53 yields

$$E = 0.649\ C_{EN}\frac{n_s}{n_p}\gamma\ r^{-4.48}\ C^{-1.85}\ Q_m^{2.85}\ H \qquad (5.56)$$

Total cost C_T per year per meter of pipeline is obtained by adding the annual cost of construction Eq. 4.46 to Eq. 5.57.

$$C_T = e_p r^2\ D_p I + 0.649\ C_{EN}\frac{n_s}{n_p}\gamma\ \delta\ r^{-4.48}\ C^{-1.85}\ Qm^{2.85}\ H \qquad (5.57)$$

Best Economic Radius (Pressure Pipe)

An equation for the radius of least cost pipeline is obtained by differentiating Eq. 5.57 with respect to r and placing the differential quotient $dc_T/dr = 0$ i.e.

$$\frac{dc_r}{dr} = 2e_p r\ D_p\ \ I - 2.908\ C_{EN}\frac{n_s}{n_p}\gamma r^{-5.48}\ C^{-1.85}\ Q_m^{2.85}\ H = 0 \qquad (5.58)$$

From the above expression

$$2e_p r D_p I = (2.908\ C_{EN}\frac{n_s}{n_p}\gamma\ C^{-1.85}\ Q_m^{2.85}\ H)\ 1/r^{6.48} \qquad (5.59)$$

Now make radius (r) the subject

$$r^{6.48} = 1.454\ C_{EN}\frac{n_s}{n_p}\gamma\ C^{-1.85}\ Q_m^{2.85}\ HD_p^{-1}\ I^{-1} \qquad (5.60)$$

$$r = \sqrt[6.48]{\frac{1.56\ C_{EN} n_s \gamma C^{-1.85} Q_m^{2.85} HD_p^{-1} I^{-1}}{n_p}} \qquad (5.61)$$

$$\text{or}\ \ r = (1.454\ C_{EN}\frac{n_s}{n_p}\gamma C^{-1.85}\ Q_m^{2.85}\ HD_p^{-1} I^{-1})^{1/6.48} \qquad (5.62)$$

The velocity of flow is obtained by

$$V = \frac{Q}{\pi\left[(1.454C_{EN}\frac{n_s}{n_p}\gamma C^{-1.85}Qm^{2.85}HD_p^{-1}I^{-1})^{1/_{6.48}}\right]^2} \quad (5.63)$$

$$= \frac{Q}{\pi}\left(1.454\,C_{EN}\frac{n_s}{n_p}\gamma\, C^{-1.85}\, Q_m^{\ 2.85}\, HD_p^{\ -1}I^{-1}\right)^{1/_{6.48}}$$

Substituting Eq. 5.62 into Eq. 5.50 yields the maximum energy gradient, thus

$$S_{pp} = \frac{Q}{0.402\,C\pi(1.454C_{EN}\frac{n_s}{n_p}\gamma C^{-1.85}Q^{2.85}HD^{-1}I^{-1})^{-0.813}} \quad (5.64)$$

$$S_{pp} = \frac{Q^{-1.85}}{0.880C^{2.504}\left(C_{EN}\frac{n_s}{n_p}\right)\gamma HD_p^{-1}I^{-1})^{-0.813}} \quad (5.65)$$

For pressure pipeline all flow and gravity pipeline at maximum flow (Christensen, 1971),

Hydraulic radius is $R = \frac{r}{2}$

To relate the annual construction cost to the cost of fluid friction, Eq. 5.58 will be multiplied by r/2 and then transposed to yield.

$$e_p r^2 D_p I = \frac{2.908}{2}C_{EN}\frac{n_s}{n_p}\gamma\ r^{-4.48}\, C^{-1.85}\, Q^{2.85}\, H \quad (5.66)$$

The left hand side of Eq. 5.65 is the annual cost IC as shown in Eq. 5.46, the right hand component is 1.555 times the annual loss through fluid friction P, that is

$$IC = 1.555\ E \quad (5.67)$$

If Eq. 5.62 is substituted into Eq 5.45 for r, then construction cost per meter of pipeline

$$C_a = e_p\,(1.454\;C_{EN}\;C_{EN}\frac{n_s}{n_p}\gamma\;C_{-}^{1.85}\;{}_{Q}^{2.85}\;HD_p^{-1}I^{-1})^{1/3.24}\;[\,\pi\,(2k_p + k_p^2)\,.\,\frac{I_p}{e_p} + ab] \qquad (5.68)$$

When Eq. 5.68 is multiplied by debt service factor I, this yields

$$C_a = e_p\,(1.454C\;C_{EN}\frac{n_s}{n_p}\gamma\,C^{-1.85}\;{}_{Q}^{2.85}\;HD_p^{-1})^{0.308}\;I^{\,0.692}\;[\pi\,(2k_p + k_p^2).\frac{I_p}{e_p} + ab] \qquad (5.69)$$

Which is the annual construction cost per meter of pipeline.

Substituting Eq. 5.69 into Eq. 5.67 yield the following.

$$E = \frac{1}{1.555}\,e_p.(1.56\,C_{EN}\frac{n_s}{n_p}\gamma\;C^{-1.85}\;Q^{2.85}\;HD_p^{-1})^{\,0.309}\;\overline{I}^{\,0.692}\;[\pi\,(2k_p + k_p^2)\,.\frac{I_p}{e_p} + ab] \qquad (5.70)$$

Adding Eq. 5.69 and Eq. 5.70 will yield.

$$\frac{IC_a + E}{q} = \frac{2.555}{1.555}\,e_p(1.454\,C_{EN}\,\frac{n_s}{n_p}\gamma\;C^{-1.85}Q^{2.85}HD_p^{-1})^{0.309}I^{0.692}[\pi(2k_p{+}k_p^2).\frac{I_p}{e_p} + ab] \qquad (5.71)$$

which is the total annual cost

Taking q as the annual discharge in hectare meters ($10{,}000m^3$), then

$$q \;=\; \frac{n_t}{10{,}000}\int Qdt = \frac{n_t Q_a}{10{,}000} \quad \text{in which the average rate flow} \qquad (5.72)$$

To obtain the cost to convey $10{,}000m^3$ of water through 1m of pipeline divide Eq. 5.71 by Eq. 5.72 to obtain.

$$\frac{Ic_a + E}{q} = \frac{25550}{1.555 n_t Q_a}\,e_p\,(1.454\,C_{EN}\,\frac{n_s}{n_p}\gamma\,C^{-1.85}\;Q^{2.85}\;HD_P^{-1})I^{\,0.692}[\pi\,(2k_p + k_p^2)\frac{I_p}{e_p} + ab] \quad (5.73)$$

5.7.2 For Gravity Pipeline

By gravity flow is meant flow with a free surface in partially filled aqueducts. For example, all canal flow is gravity flow. Gravity flow must have an energy gradient equal to the slope of the aqueducts whereas the energy gradient for pressure flow is usually different than the aqueducts slope.

The slope for the pipeline under this condition of flow is represented as S_g. For gravity flow Eq.5.48 can only hold if Q is replaced by Q_m that is maximum flow because to be the most economical gravity flow pipeline, it must flow full but without pressure at maximum flow.

Therefore

$$S_g = \left(\frac{Q_m}{0.625\, C \pi r^2 \left(\frac{r}{2}\right)^{0.63}} \right)^{1/0.54} = \frac{Q_m}{60.402 C \pi r^{2.63}} \tag{5.74}$$

Substituting Eq. 5.74 into Eq. 5.47 for Sg and simplifying yields

$$\frac{dE}{dt} = C_{EN} \frac{n_s}{n_p} \gamma \frac{Q_m^{\;1.85}\, Q}{1.54 C^{1.85} r^{4.48}} \tag{5.75}$$

$$\frac{dE}{dt} = 0.649\ C_{EN} \frac{n_s \lambda}{n_p} \frac{Q_m^{\;1.85}}{C^{1.85} r^{4.48}} \tag{5.76}$$

Integrating Eq. 5.75 over a year gives

$$E = 0.649\, C_{EN} \frac{n_s}{n_p} \gamma r^{-4.48} C^{1.85} Q_m^{\;1.85} \int_0^1 Q dt \tag{5.77}$$

For convenience, the definite integral of Eq. 5.76 will be replaced by an equivalent expression i.e.

$$Q_m^{\;1.85} \int_0^1 Q dt = Q_m^{\;2.85} \int \frac{Q}{Q_m} dt = Q_m^{\;2.85} H_g \tag{5.78}$$

where H_g is the hydrographic factor for gravity flow and can be evaluated numerically or graphically over 1 year by plotting.

$\frac{Q}{(Q_m)^{2.85}}$ as a function of the time, in years and summing the area under the curve.

Substituting Eq.5.78 into Eq. 5.76 gives

$$E = 0.649 \ C_{EN} \frac{n_s}{n_p} \gamma \ r^{-4.48} C^{-1.85} Q_m^{\ 2.85} H_g \tag{5.79}$$

The total cost C_T per year per meter of pipeline is obtained by adding the annual cost of construction Eq. 4.46 to Eq. 5.79

$$C_T = e_p r^2 D_p I + 0.649 \ C_{EN} \frac{n_s}{n_p} \gamma \ r^{-4.48} C^{-1.85} Q_m^{\ 2.85} Hg \tag{5.80}$$

Introducing the principle of best economic radius, we can again differentiate Eq. 5.80 with respect to r and equate the differential quotient $\frac{dc_T}{dr}$ to zero.

$$\frac{dc_T}{dr} = 2e_p r \ D_p I - 3.11 \ C_{EN} \frac{n_s}{n_p} \gamma \ r^{-6.48} C^{-1.85} Q_m^{2.85} Hg = 0 \tag{5.81}$$

Making r the subject, as done in Eq. 5.59.

$$r = 6.48 \left(1.454 \, C_{EN} \frac{n_s}{n_p} \gamma \, C^{-1.85} Q_m^{2.85} Hg \, D_p^{-1} I^{-1}\right)^{1/6.48} \tag{5.82}$$

Subsisting velocity V_m for Q_m gives yields

$$V_m = \frac{Q_m}{\pi r} \ \left(1.454 \ C_{EN} \frac{n_s}{n_p} \gamma \ C^{-1.85} Q_m^{\ 2.85} Hg \, D^{-1} T^{-1}\right)^{-2/6.48} \tag{5.83a}$$

or

$$V_m = \frac{Q_m}{\pi r} \left(1.454 \ C_{EN} \frac{n_s}{n_p} \gamma \ C^{-1.85} Q_m^{\ 2.85} HgD^{-1} I^{-1}\right)^{-0.309} \tag{5.83b}$$

The maximum energy gradient Sg for gravity flow is then given by

$$S_g = \frac{Q_m}{0.402\,C\pi\,(1.454\,C_{EN}\,\frac{n_s}{n_p}\,\gamma C^{-1.85} Q_m^{\ 2.85}\, HgD^{-1}I^{-1})^{-0.813}} \tag{5.84a}$$

or

$$S_g = \frac{Q_m}{0.402\,C\pi\,(1.454\,C_{EN}\,\frac{n_s}{n_p}\,\gamma C^{-1.85} Q_m^{\ 2.85}\, HgD^{-1}I^{-1})^{-0.813}} \tag{5.84b}$$

According to Christensen (1971), for pressure pipeline and gravity pipeline, the hydraulic radius. $R = \frac{r}{2}$

To relate the annual construction cost to the cost of fluid friction. Equation 5.58 has to be multiplied with $\frac{r}{2}$ and also transposed to give.

$$e_p\, rD_pI \;= \frac{3.11}{2}\; C_{EN}\frac{n_s}{n_p}\gamma\, r^{-4.48}\, C^{-1.85}\, Q_m^{\ 2.85}\, Hg \tag{5.85}$$

Eq. 5.85 is similar to Eq. 5.67 except for Hg

A similar process is followed from Eq. 5.68 to Eq. 5.66 but Hg replaces H in each of the equations for gravity flow.

The final equation for gravity flow takes the following form.

$$\frac{IC+E}{Q} = \frac{2.555}{1.55n_tQ_a} - e_p\left[1.56\,C_{EN}\,\frac{n_s}{n_p}\,C^{-1.85}Q^{2.85}H_gD^{-1}\right]^{0.308} I^{\,0.692}\left[\pi(2K_p + K^2p)\frac{1_p}{e_p} + ab\right] \tag{5.86}$$

Eq. 5.86 is the final equation for which two computer programmes, one using MATLAB and the second using Visual Basic Application in Microsoft Access have been written. The application of these programmes will be demonstrated on computer systems.

CHAPTER SIX

IMPLEMENTATION OF THE MATHEMATICAL MODELS: EXAMPLES OF APPLICATIONS.

6.1 Application of Allocation Model

6.1.1 Example 1 – Owena–Igbara-Oke Scheme.

Criteria

(a) Allocation of water to demand centres is controlled using valves bearing in mind all constrains associated with pumping capacity and duration.

(b) Water is allocated to all demand centres at minimum cost.

To achieve this, the following were considered

i) Selection of flow rate of each demand centre;

ii) Grouping of the demand centres into zones first by contiguous arrangement and non-contiguous arrangement. Selection of the demand centers' discharges were selected close to the daily mean flow of the main pipelines. Thus,

$$\sum qi \leq Q \qquad (6.1)$$

where qi per zone and Q is the total flow expected from the source.

6.1.2 Assumptions

To simplify the mathematics of the model, the following assumptions were made:

i) The water distribution system is on the same elevation, neglecting the topography and therefore only pumping is required

ii) The discharges to the demand centres are equal;

6.1.3 Input Data

To apply the model, the following data are required:

i) The allocation to each demand centre (q_i) in m^3/day;

ii) The diameters of trunk mains (mm) and type;

iii) Pipe cost (₦) which is a function of diameter and material;

iv) Relative locations of sources and demand nodes;

v) The proposed routes from source to each demand centre;

vi) The distance through which water is to be transported;

vii) Minimum elevation required between the source and the demand centre;

viii) Unit cost of energy (electricity) in kWh.

The overall pumping cost as expressed in equation 5.34 is

$$C_T = \sum_{ij} C_p = \sum \frac{16\gamma fLQ^3.t.C_m}{19.6D^5.\pi^2.1000.\eta} \qquad (6.2)$$

6.1.4 Output

1 Owena-Igbara-oke and Owena–Ondo Water Supply Scheme

Owena- Igbara Oke and Owena Ondo schemes were selected for this analysis because of the relative availability of operational records.

Allocation of water from Owena –Igbara Oke scheme is allocated to 7 demand centers namely Owena, Igbara Oke, Ilara Mokin, Ijare, Shagari Housing Estate and Akure. From the results obtained, it appears there is no linear relationship between the friction head and pumping costs (Figure 6.1). A similar operation was carried out on Owena –Ondo scheme supplying basically Owena, Ondo, Ile-Oluji,and Akure using the current allocation ratio. The result is as shown in Figure 6.2.

2 Akure Zone

This model also used the existing nine zones in Akure plus Alagbaka (which is standing as a zone) to carry out the allocation analysis bearing in mind that Owena-Ondo which supplying the zone is only operating at approximately 60% of its full capacity. In this arrangement, equal volume is allocated to each zone to maintain equity except Alagbaka which is allocated 500m^3 and must flow on continuous basis. Table 6.1 shows a typical allocation arrangement.

Table 6.1 Typical Arrangement of Water Allocation

	PERIOD OF THE DAY				ALLOCATED	PUMPING
	t_1	t_2	t_3	t_4	$Q_I(M^3)$	COST (₦)
Zone 1	√				750	16,550
Zone 2	√				750	
Zone 3		√			750	21,270
Zone 4		√			750	
Zone 5			√		750	23,500
Zone 6			√		750	
Zone 7				√	750	26,800
Zone 8				√	750	
Zone 9	√	√	√	√	500	12,400

Since the major objective this model is to get water to every zone and at minimum pumping cost, three different arrangements were used to the programme. Summary of these arrangements is shown in Table 6.2.

Curves obtained from the results of this analysis are shown Figures 6.4. The best allocation arrangement is the one that gives the least pumping/energy cost.

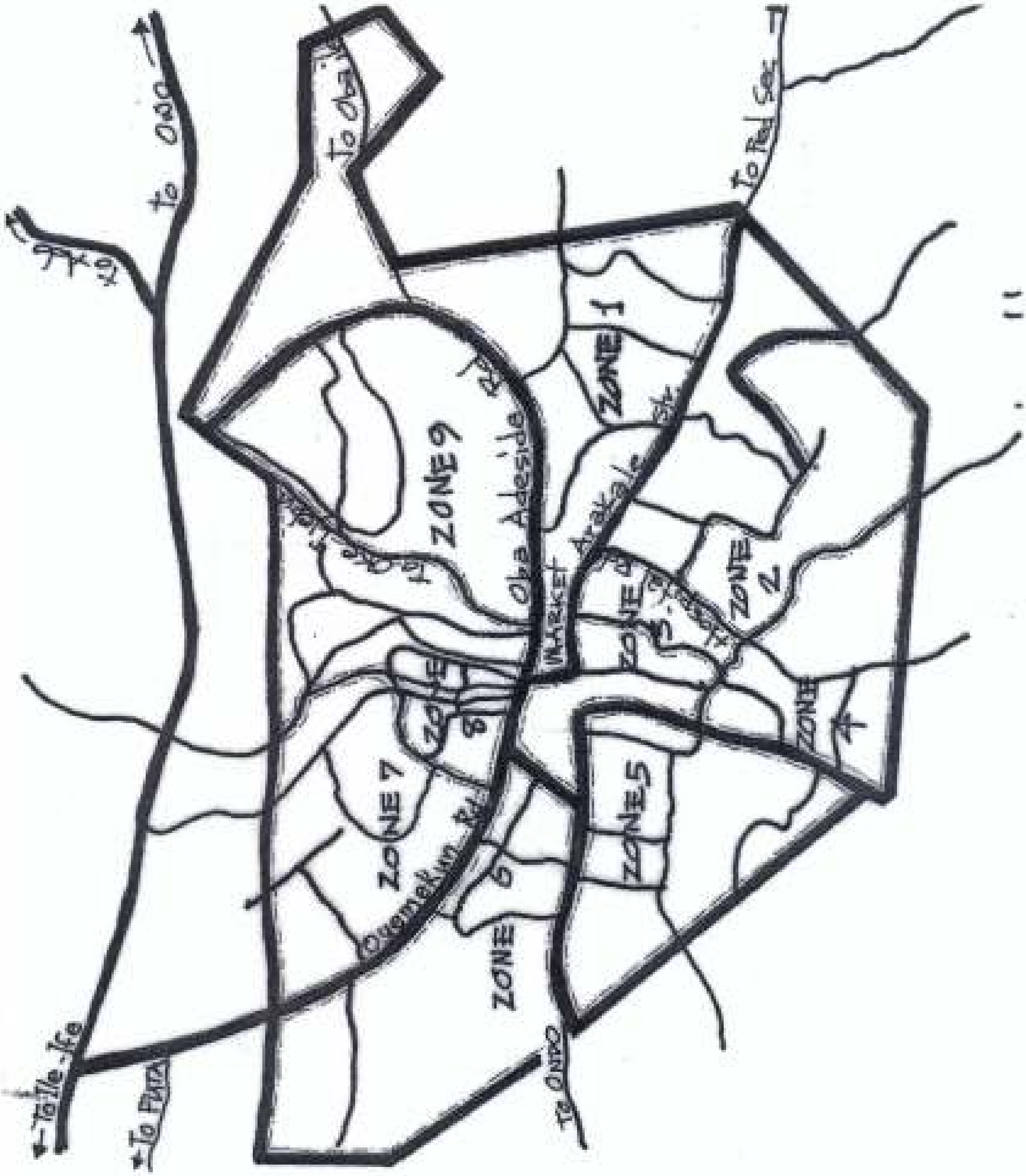


Figure 6.1. Zoning of Akure for Water Allocation

Table 6.2 Summary of Possible Allocation Arrangement

(a) 1st Arrangement			
ZONE/LOCATION		TOTAL FLOW M^3	PUMPING COST ₦
1	Zones 1 + 2	1500	16,550
2	Zones 3 + 4	1500	21,270
3	Zones 5 + 6	1500	23,600
4	Zones 7 + 8	1500	26,800
5	Zones 9	500	9,400
(b) 2nd Arrangement			
ZONE/LOCATION		TOTAL FLOW M^3	PUMPING COST ₦
1	Zones 1 + 2 + 3	22,500	13,600
2	Zones 4 + 5 + 6	22,500	16,400
3	Zones 7 + 8	1,500	17,400
4	Zones 9	500	9,400
(c)3rd Arrangement			
ZONE/LOCATION		TOTAL FLOW M^3	PUMPING COST ₦
1	Zones 1 + 3	1,500	25,100
2	Zones 2 + 4	1,500	16,600
3	Zones 5 + 7	1,500	17,500
4	Zones 6 + 8	1,500	18,100
5	Zone 9	500	9,400

6.2 Application of Model II – Water Transmission Model

6.2.1 Assumptions

It is assumed that the aqueduct consists of pressure pipelines that carry certain required flow. It is usually assumed that the water is delivered to the point of use which is at the same elevation as the sources in pumping operations.

For each set of computation, the two programmes were used to allow for comparison of results. Table 6.3 & 6.4 show the input as well as the calculated data for a particular run.

Table 6.3: Input Parameters:

Parameter	Value
Width / radius ratio, a	3
Weight / radius ratio, b	3
Thickness of pipe / radius	0.1
Cost per unit volume of trends excavation, e_p	₦ 1000,
Cost per unit length of pipe in trench L_p	₦ 600
Debt services factor	0.22
Hydrographic factor	4 /9
Hazen-William coefficient	150

Model are presented in Output from the Table 6.4.

Table 6.4: Output

Parameter	Values
Optimal radius, r	1.23 m
Maximum estimated velocity	0.12 m/s
Total construction cost per unit length of pipeline Naira/m/yr.	₦14,331
Annual cost of fluid friction	₦ 4,902.64
Annual cost of pipeline/conveyance	₦ 8,055.46

6.3 Influence of the Input Parameters

Sensitivity analysis was carried out to determine the influence of the parameters in Equations 5.73 and 5.86 on the result computed from the models, especially in reducing the cost.

Equation 5.86 was rearranged into four groups as shown in equation 6.3

$$\frac{M}{q} = \underbrace{\left[\frac{2550}{1.555} n_t^{-1}(n_p^{-1})^{0.308}\gamma^{0.308}.(\pi(2K_p + K_p^2) + ab\right]}_{\text{Group 1}} \underbrace{\left[e_p^{-1}(D_p^{-1})^{0.308} I^{0.692} l_p\right]}_{\text{Group 2}} \underbrace{\left[C_{Hw}^{-1.85}\right]^{0.308}}_{\text{Group 3}} \underbrace{\left[HQ_m^{2.85}.Q_a^{-1}\right]}_{\text{Group 4}} \quad (6.3)$$

6.3.1 Effect of Relative Cost of Excavation

Group 1 contains parameters, which are constants, and therefore have constant influence. Group 2 contains factors which contains annual cost of construction. Excavation cost can be reduced through negotiation or by selecting the contractor that submits the lowest value in bidding. Common values, from current construction practice are used for computation to demonstrate the influence of Group 2. For the design factor (D_p), the shape of the cross-section of the aqueduct must be considered. In Nigeria, the common shape of aqueduct is circular. The costs of pipes are related to materials and sizes (diameter). Figure 6.1 shows a graph of cost of excavation versus total construction cost per meter. All the curves show a similar pattern which is an indication of the fact that all pipe materials behave in the same manner under the same excavation cost. However, it was observed that the Total Construction Cost increases as the excavation cost increases. It reached the maximum at e_{p}= ₦ 900 and thereafter fell sharply at a higher value of e_p. Another important deduction from this analysis is that Total Construction cost is a function of the roughness of the pipe for all pipe materials.

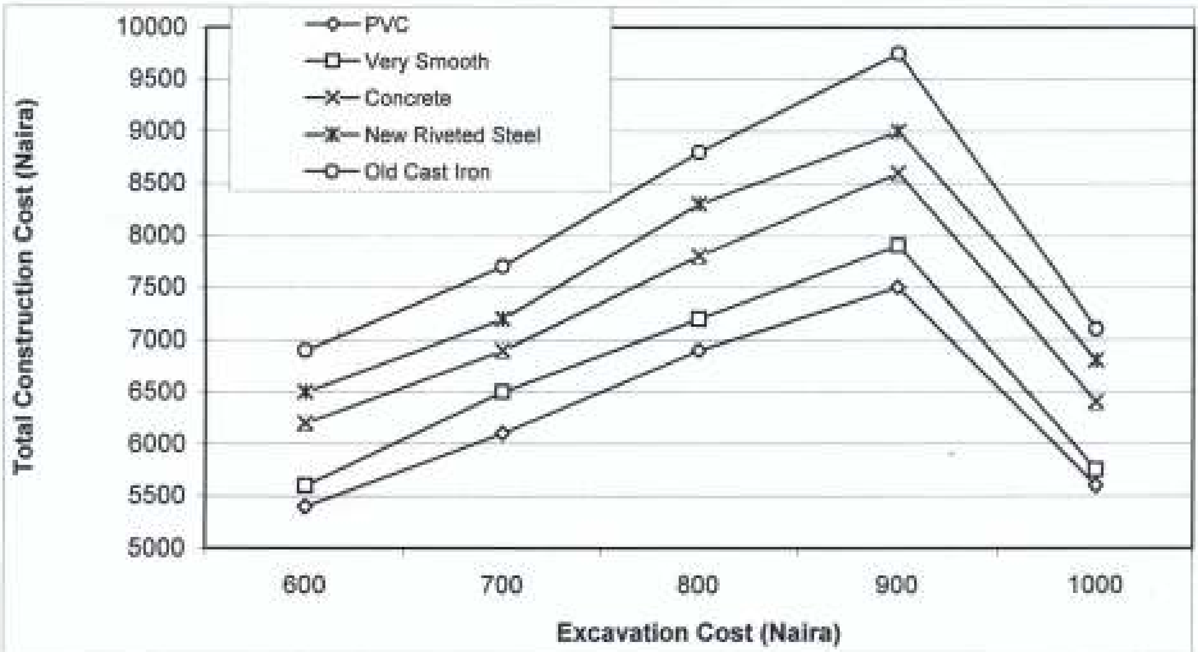


Fig. 6.2 Graph of Cost of Excavation vs Total Construction cost/m (for I = 0.2)

From Table 6.5, it can be seen that changes in the values of e_p have affected the various costs. Using values obtained for e_p is ₦ 1000 as the yardstick; we can compare values obtained for other values of e_p. For example the "Total link cost" for e_p value of ₦1000 in Table 6.5 of ₦707,827,243.95 while for e_p is ₦900 the link cost is ₦638,268,592.64. The savings that will be made for the reduction in e_p by comparing these two costs is ₦69,558,651.31.

Table 6.5: Effects of Excavation Cost (e_p) (for PVC pipe)

	Values e_p per unit cost ,₦				
Parameters	1000	900	800	700	600
Total construction cost per unit length of pipeline	14155.97	12764.85	11373.69	9982.50	8591.26
Annual cost of pipeline	3114.31	2808.7	2502.21	2196.15	1890.077
Annual financial cost per matter of pipeline	4842.76	4366.85	3890.94	3415.01	2739.07
Total annual cost of pipeline	7957.07	7175.12	6393.15	5611.17	4829.15
Total link-cost	707,827,244.00	638,268,593.00	568,708,633.00	499,146,812.00	429,582,218.00

6.3.2 Effect of Debt Service factor (I) on other costs

Debt service factor I is another factor that has great influence on the costs. In practice in Nigeria, the common rate of I is 22%. However, it is possible to negotiate the value down to about 15% and even lower. The values of I are also varied from 0.22 to 0.15 to examine this effect on other variables. Figure 6.2 shows the influence of debt service factor on total construction cost while Table 6.6 shows the various costs values obtained. However it can be observed in Figure 6.2 that sharply variation in Total Construction cost is between the values of 0.15 and 0.18 for I, thereafter the reduction followed the same pattern for all materials of pipe as the value of I increased.

Table 6.6: Effects of Debt Service Factor (I) on Costs

	Debt service factor (I)			
Calculated Parameters	0.22	0.20	0.18	0.16
Total construction cost per unit length of pipeline,₦	14906.16	15351.16	15858.57	16445.68
Annual cost of pipeline,₦	3279.36	3070.23	2854.54	2631.31
Annual friction cost per meter of pipeline,₦	5099.40	4774.21	4438.81	4091.69
Total Annual cost of pipeline,₦	8378.75	7844.44	7293.36	6722.99
Total link cost,₦	745,341,552.00	767,592,634.00	792,864,224.00	822,321,054.00

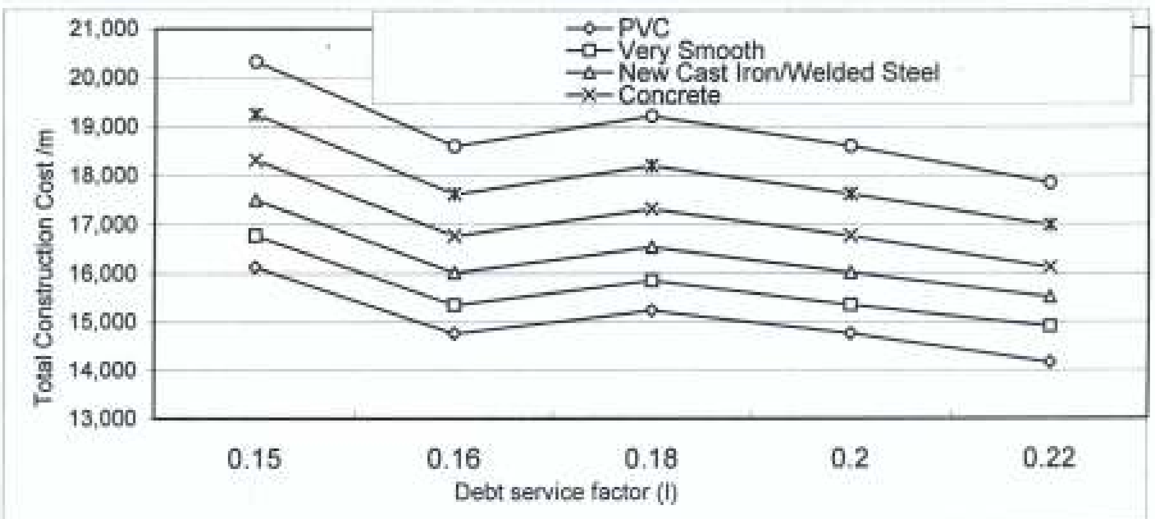


Fig 6.3. Influence of Varied Debt Service Factor on Total Construction Cost (Ep = N1000)

6.3.3 Effect of Pipe Materials on Costs

Group 3 in Equation 6.3 is the Hazen – William coefficient, which varies according to pipe material and age. Standard values of C_{HW} for different pipe materials which are available in standard text are used alongside with different values of excavation cost (e_p). The results are detailed in Table 6.7.

Table 6.7: Effect of Pipe Materials on Costs (e_p = ₦ 1000)

Computed Items / C_{HW}	140	130	120	110	100
Total construction cost per unit length of pipeline	14906.1	15512.68	16119.26	16982.02	17843.73
Annual cost of pipe line	3279.36	3412.79	3546.24	3736.04	3925.62
Annual friction cost per meter of pipeline	5099.40	5306.89	5514.40	5809.55	6104.34
Total annual cost of pipeline	8378.75	8719.68	9060.64	9545.59	10029.96
Total link cost	745341652.00	775668337.00	805996506.00	849136705.00	892222579.00

6.3.4 Effect of Operational Mode on Cost

Group 4 in Equation 6.3 consists of factors which result from operational mode. The method of operation had to be determined so as to allow for the evaluation of H and Hg. Assuming steady flow condition H = 1 and H_o = 1 that group will reduce to Q_o^{-1}. This implies that the cost per unit of water conveyed in steady flows varies inversely as Q_o. To achieve the steady state, the regulation works need to be investigated so as to reduce the values of H, Hg and Q. Figure 6.3 shows the relationship between the locationa head and annual friction cost for the different supplies of the Owena Scheme

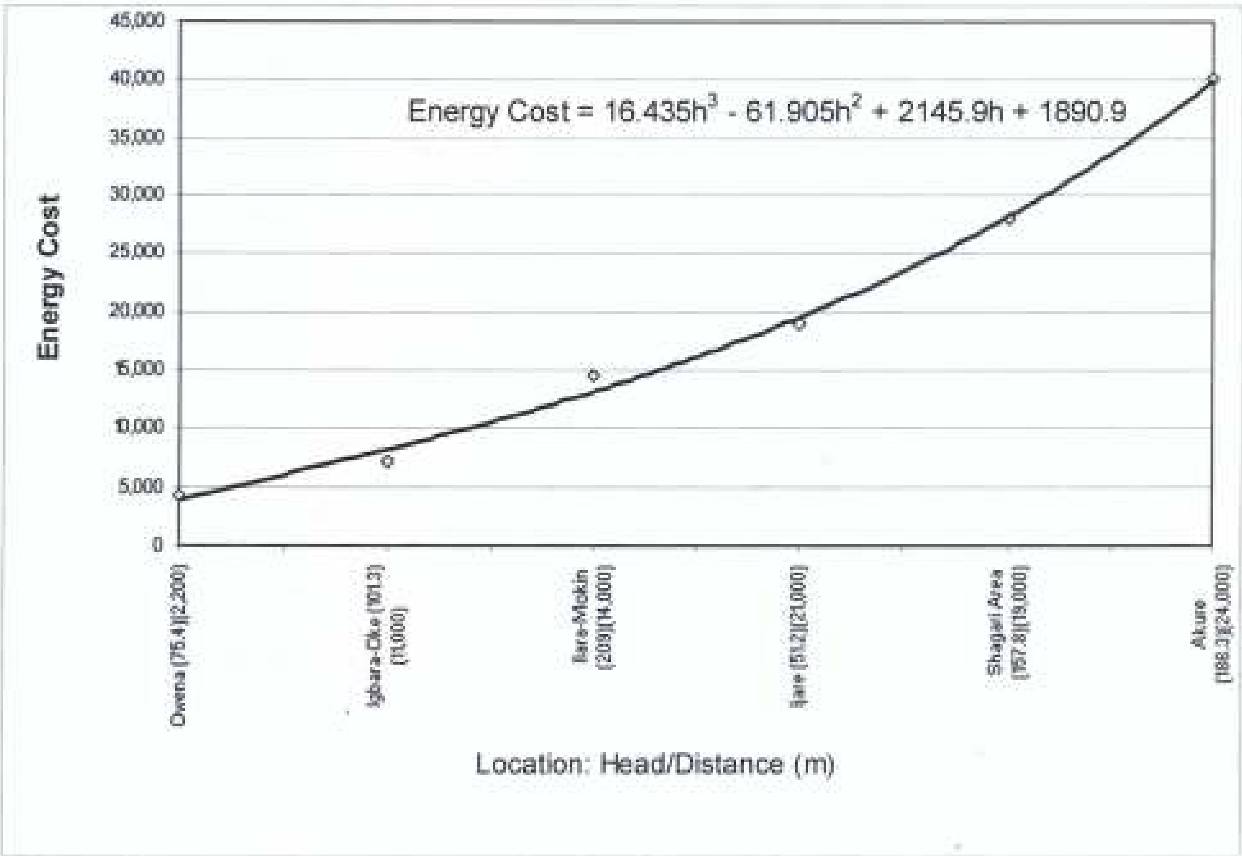


Fig.6.4. Relationship between Location Head and Annual Friction Cost

CHAPTER SEVEN

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The conclusions have been divided into three major complementary components namely:

i) The technical appraisal of the existing water supply situation in Ondo and Ekiti States and the socio-economic impact on the consumers, the governments providing the facilities, and the private sector;

ii) The formulation and implementation of the water allocation model, which is useful in simulating various combinations of providing water to various communities from available sources;

iii) The development of the hydraulic model for the conveyance of water through pipes (such as transmission mains) at optimal conditions including the minimizing of costs.

7.1.1 Socio-Economic Appraisal:

The socio-economic appraisal conducted, field observations and measurements indicate the following:

a.) No clear-cut policy for provision of water supply to urban and rural communities exist at all levels of government (Federal, State and Local). Majority of consumers cannot appreciate the exact roles of the various governments in providing municipal water supply. Yet, policy is the brainbox of any meaningful programme for the efficient management of municipal water supply.

b.) The rapid urbanization of demand centres for water supply has lead to the failure of existing water utilities in Ondo and Ekiti States to cope with the provision of water to various cities, towns and villages. This has led to private individuals and entrepreneurs initiating various water supply systems.

c.) The confusion created by duplication of efforts and lack of coordination by the three tiers of government NGO's, and private individuals in providing water to both rural and urban areas only compound the inefficiency and sometimes failure of the systems.

d.) The present situation of water supply through conventional means in the study area is not satisfactory. The level of performance of water corporation vis-à-vis the existing headworks are very low. Many of the existing beadworks are quite big and by the present level of technological development are difficult to maintain because of their costs, the funding level and lack of spare-parts.

e.) Unlike in electricity supply where people make personal efforts to get extension to their houses, the extension of water pipes are left to water corporations while people look for alternative sources of water.

f.) Records of operation, repairs and maintenance of water facilities are poorly kept by the operators and as such data for meaningful research are not usually easy to come by.

g.) The present charges for water supply services are unreasonable for sustainability compared with the production transmission costs. This leads to sustained conflict and irregularity. From the findings of this research, people in the study area are quite willing to pay for better and improved water supply service.

h.) Revenue generation and collection of public water utilities are quite low because of their poor services and regulation of charges by the government. Energy supply to lift water through NEPA is quite poor and erratic and therefore water supply services are poor. The alternative which is the usage of diesel is quite expensive apart from poor funding or the utilities and their inability to generate fund.

i.) Individual entrepreneurs are taking advantage of the present poor state of supply by investing in water supply through production of packaged water popularly called "pure water" as well as tanker services. This sector is fast growing with the proliferation of packaged water of different qualities in circulation.

j.) Presently, there is no proper monitoring of the location of household wells and septic tanks/soakaways. In many cases, WHO recommendations on this issue are not followed. This situation will likely lead to intrusion of polluted water into the wells from which the surrounding dwellers fetch for different purposes including drinking. Quality of water from many of these wells and boreholes are inadequate in comparison with the WHO standards.

7.1.2 Allocation Model

The allocation model has shown that a single source of water supply with limited production can serve several zones of demand which can be intra-city, or based on town to town. Efficient allocation is indicated by grouping of the zones in such a way to obtain minimum pumping costs, and hence minimum energy costs. The model can be used to specify the number and combinations of zones to be supplied at different periods of daily operations. Specific application of the model to Akure township water supply shows that the minimum costs of water allocation actually depend on the arrangement of zones at different period.

7.1.3 Transmission Model

The model developed for transmission of water through the mains incorporates several hydraulic parameters and cost factors to obtain the total construction cost per unit length of pipeline, amortized annual cost of the pipelines and cost of pumping against friction for different conditions of operation and materials. By simulating several conditions based on variations of the hydraulic and cost factors, best hydraulic radii and optimal cost of transmission could be obtained from one location to another. The most sensitive factors were the friction coefficient, which is a function of the pipe materials. In this respect, the best pipe material which is the PVC, must be considered. However, other cost factors can still mitigate against it's selection.

7.2 Recommendations

There is a need for continual appraisal of water supply facilities in Ondo and Ekiti States as well as in all states of Nigeria to determine the level to which the citizen's water requirements are met, considering the high rate of urbanization, and the increasing water demand. The data obtained in various appraisal studies are particularly useful in developing several other mathematical models to aid policy decision making in water supply. A substantial investment is required for data acquisition for the development of useful versatile mathematical models. The problem of data availability has been a clog in the wheel of research projects in Nigeria and other developing countries. It is important to encourage the operators of the public infrastructures to keep good records in a useful form.

Further work on the model for allocation may focus on the size of demand zones and the delineation accuracy along socio-economic factors such as low and high density dwelling areas, and also communities.

The transmission model can be further refined by considering both gravity and pumping scenarios together as these reduce the assumptions made to simplify the realities of the pipe systems and operations.

It is important that the government or any relevant agency implement these models on any of the existing schemes. This will give room for improvement of the models

References

Abdel-Latif, M.Y., 2001: Water Distribution Modelling with Intermittent Supply: Sensitivity analysis and performance evaluation for Bani Suhila City – Palestine Unpublished MSc Thesis. South Bank University, London.

Abebe, H. and Solomatine, W. F., 1998: Water Supply in Developing Urbans. Publication of CWRM. Madrid.

Alausa, S.B., 1997: Privatization of Urban Infrastructure: Issues, Myths and Realities Proceedings. Annual Conference of Nigeria Institute of Estate Surveyors and Valuers (NIESV), Ibadan, pp. 18 – 27.

Alavi, N.A.,1984: Proceedings of the International Seminar on Water Resources Management Practices. Ilorin, Nigeria. pp109.

Australian Academy of Technological Sciences and Engineering (1988). Urban Water Supplies. Australian Science And Technology Heritage Center.

Bhave, P.R., 1992: A Critical Study the Linear Programming Gradient Method of Optimal Design of Water Supply Networks. Water Resources Research 28(6), 1577-1584.

Bhave, P.R., 1983: Optimization of Gravity – fed Water Distribution Systems: Application. Journal of Environ, Eng. Div. Am. Soc. Civ. Eng. Vol. 107, No. 2, pp. 383 – 395.

Bhave, P.R., 1985: Optimal Expansion of Water Distribution Systems. Journal of Environmental Engineering Division. American Society of Civil Engineers. Vol. 111, No 2.

Bogale M.M., 1994: Design and Implementation of a modeling system for water Distribution Network: An object oriented Approach. (MSc thesis).

Brandon, T.W., 1984. Water Distribution Systems. Published by Institute of Water Engineers And Scientists, London. Pp71-75.

Cembrowicz, R.G., 1980: Least-cost Design of Large Scale Waste Water Disposal Systems. Proceedings of Conference on Hydraulic Research and River Basin Development in Africa. IAHR/UNESCO. Nairobi 11 -15 September 1980.

Cenedese, A, Gallarena, F. and Misti, A., 1987: Multi – Objective Analysis in Optional Solution of Hydraulic Networks. Journal of Hydraulic Engineering ASCE, Vol. 13, No 9, pp. 1133 – 1142.

Chiplunkar, A.V., Mehndiratta, S.L., and Khanna, P., 1985: Looped Water Distribution Optimization for Single Loading. J. of Envir. Engrg., ASCE, 112(2), 264-279.

Chopra, U.B., 1989: Dictionary of Civil Engineering 1st Edition. Pp. 295.

Clymo, J., 1989: The Role of Conventional Buses and Para – transit in Heraty, M.J. (Ed.-In–chief).'Developing World', Grosvenor Press International, London.Proceeding of the International Seminar on Water Resources Management Practices, Ilorin, Nigeria. Pp. 109.

Curtis, R., 1996: Water Utility Revenues for Watershed Protection. .Mtn Forum. Discussion Archive Article \\ 72396C.

Drapper, M. R. and Smith, H., 1981: Applied Regression Analysis. Second Edition. John Siley& Sons, Singapore.

Duan, N., Mays , L.W., and Lansey, K.E., 1990: Optimal Reliability-based Design of Pumping and Distribution Systems. J. Hydr. Engrg., ASCE, 116(2), 249-268.

Ekenta, O.E., 2002: .Modeling of Water Supply Distribution System for A semi-urban Town in South-East Nigeria. Journal of Civil Engineering. Publication of Nigeria Institution of Civil Engineers. 2nd Edition pp 18-24.

El – Bahrawy, A. and South, A. A., 1985: Application of MINOS to Water Collection and Distribution Networks. Civil Engineering System, Vol. 2.

Fujiwara, O. and Dey, D., 1987: Two Adjacent Pipe Diameters at the optimal solution in the water distribution network models. Water Resources Research. Vol. 23. pp. 1457 – 1460.

Geem, Z. W., 2001: Optimal Design of Water Distribution Networks Using Harmony Search. MSC Thesis Department of Civil Engineering, University of Toronto. Toronto.

Goulter, I.C., 1987: Current and future Use of Systems Analysis in Water Distribution Network Design. Civ. Engr. Syst. Vol. 4, pp. 175 – 184.

Goulter, I.C., Lussier B. M., and Morgan, D. R., 1986: Implication of Head Loss Path Choice in the Optimization of Water Distribution Networks. Water Resources Research, Vol. 22, No. 5, pp. 819 – 822.

Goulter, I. C., 1986: Multi – Objective Optimization of Water Distribution Networks. Civil Engineering System. Vol. 3, pp. 222 – 231.

Goulter, I.C. and Morgan, D.R., 1985: An Integrated Approach in the Layout and Design of Water Distribution Networks. Civil Engineering Systems, vol. 2.pp 105-113

Haimes, Y.Y., 2002: Water System Complexity and The Misuse of Modeling and Optimization, an Article Dedicated to Warren Hall, one of the UCOWR founder.

Hansen, C.T., Madsen, K., and Nielsen, H.B., 1999: Optimization of Pipe Networks. Maths Programming, 52(1), 45-58.

IRC Home, 2002: IRC International Water and Sanitation Centre.

IRC International Water and Sanitation Centre, 2002: Between Rural and Urban, Executive Summary.

Kessler, A and Shamir, U., 1989: Decomposition Technique for Optimal Design of Water Supply Networks. Faculty of Civil Engineering Technician – Israel Institute of Technology, Haifa 32000, Israel.

Kessler, A and Shamir, U, 1989: Analysis of the Linear Programming Gradient Method for Optional Design of Water Supply Networks. Water Resources Research, Vol. 25, No. 7, pp. 1469 – 1480.

Kleiner, Y., Adams, B. J. and Rogers, J.S., 2001: Water Distribution Network Renewal Planning Journal of Computing in Civil Engineering, Vol. 15, No 1 pp-15-26.

Loucks, D.P., Stedinger, J.R. and Haith, D.A., 1981: Water Resources Systems Planning and Analysis. Prentice-Hall Inc. Englewood Cliffs. New Jersey, U.S.A.

Mabogunje, A.L., 1993: Infrastructure: The Crux of Modern Urban Development. The Urban Age, Vol. 1, No. 3, pp. 2 – 3.

Makinwa-Adebusoye, P.K., 1998: 'Upgrading An Urban Squatter Settlement in Nigeria – The Experiment in Olaleye- Iponri' in Urban Services in Developing Countries, Rondinelli, Dennis A. and Cheema, G Shabbir(Eds);The Macmillan Press Ltd, London. Pp 175-194.

Morgan D.R. and Goulter, I.C., 1982: Least Cost Layout and Design of Looped Water Distribution System. Proceedings of Ninth International Symposium on Urban Hydrology, Hydrolics and Sediment Control, Univ of Ky., Lexington.

Morgan, D.R. and Goulter, I.C., 1985: Optimal Urban Water Distribution Design, Water Resources Rev. Vol. 21, No. 5, pp. 642 – 652.

Murfitt, S., 2002: Kazakhstan: Water Supply and Sanitation Project. For EU TACIS(JEP)/World Bank, 2001-2002.

Nada, T., 1995: Modelling System for Water Distribution Networks.

Olashore, O., 1991: Perspectives on Finance, Banking and Economic Policies in Nigeria. Fourtain Publication, Ibadan, pp. 259 – 267.

Onibokun, A.G., 1986: Understanding the Management of Urban Crisis in Nigeria: The Guardian, October 26, pp. 19 – 23.

Pacione, M., 1981: Problems and Planning in Third World Cities, Biddle Limited.

Palen, J. J., 1987: The Urban World; McGraw – Hill Inc, New York.

Quindry, G.E, Brill, E.D, Liebman, J.C., 1981: Optimization of Looped Water Distribution Systeme. Journal of the Environmental Engineering Division, ASCE,Vol.107, No EE4.

Richard F, Michael M and Duncan J, 1980: Water, Waste and Health in Hot Climates, John Willey & Sons, N.Y. pp 149-211

Sarbu, I, and Kalmar, F, 2002: Optimization of looped Networks. Periodidical Polytechnic Ser. Mech. Eng. Vol. 46, No 1. pp 75-90.

Savic, D.A. and Walters, G.A., 1997: Genetic Algorithms For Least-Cost Design of Water Distribution Networks. Journal of Water Resources Planning and Management., Vol. 67.

Scott, W., 1981: Concept and Measurement of Poverty, UN Research Institute for Social Development.

Sen, A., 1981: Poverty and Famine, Clarendon Press, U.K.

Shivaram, S., 2002: Optimization Models and Analysis of Routing, Location, Distribution and Design Problems on Network. (Unpublished MSc Thesis).

Sonuga, J.O., 1984: Key Note Address on Water Resources Management Problems in Developing Countries. Proceeding of the International Seminar and Water Resources on Water Resources Management Practices, Ilorin, Nigeria.

Soyombo, O., 1987: Some Issues in the Conceptualization and Theory of Urban Poverty in Nigeria. Makinwa, P.K. and Ozo A.O. (Eds) – The Urban Poor in Nigeria, pp. 1- 13.

Su, Y. and May, L. W., 1987: Reliability – Based Optimization Model for Water Distribution Systems. Journal of Hydraulic Engrg. 114, pp. 1539 – 1555.

Twort, A.C., Law F.M. and Crowley, F.W., 1987: Water Supply. Arnold International Student Edition (AISE). Edward Arnold Publishers Ltd, London. 23-40.

Van der Merwe, 1999. Community Water Supply and Sanitation, Indonesia Experience. (Internet Material).

Wagner J. M., Shamier, Uri and Marks David H., 1988: Water Distribution Reliability: Analytical Methods. Journal of Water Resources Planning and Management ASCE Vol. 114, No. 3, pp. 253 – 275.

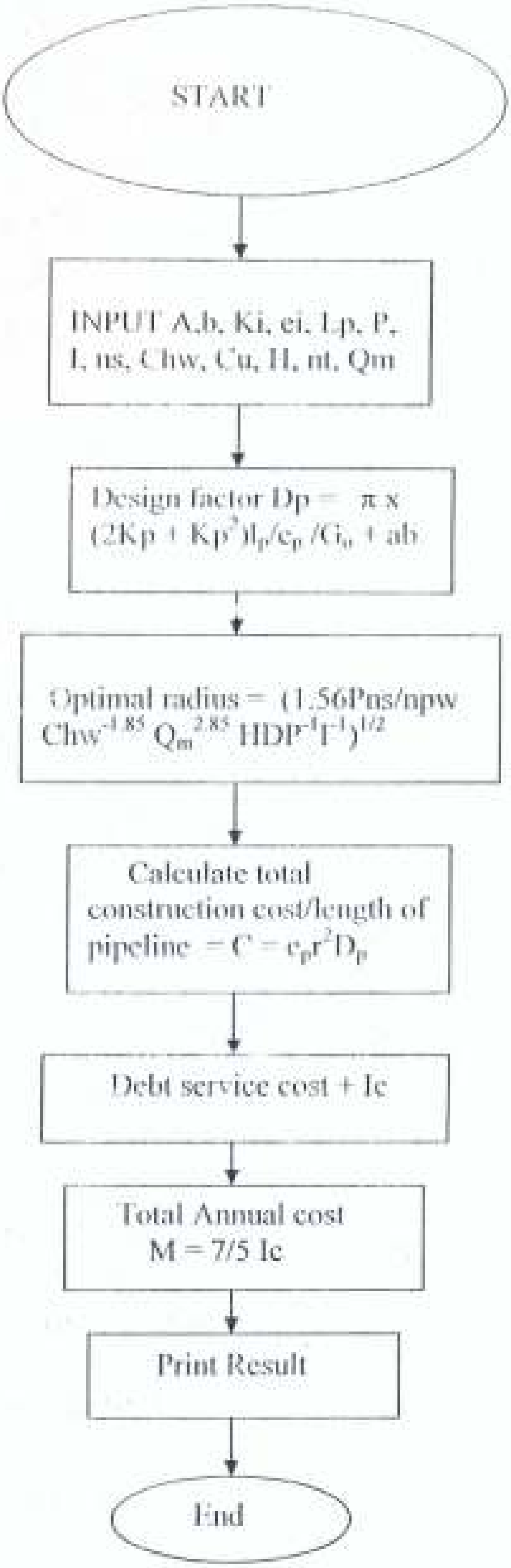
Walski, T.M., Brill, E.D., Gessler, J, Goulter, I.C, Jeppson, R.M, Lansey, K, Lee, H, Liebman, C.J, Mays, L ,Morgan D.R, and Ormsbee, L, 1987: Battle of The Network Models. Journal of Water Resources Planning and Management, ASCE. Vol. 113(2), 191 – 203.

World Bank, 1993: Restoring Urban Nigeria: A Strategy in Restoring Urban Infrastructure and Services in Nigeria. Washington, D.C. pp. 19 – 20.

World Water, 1998: Journal of American Works Association, 56(2), pp 1549-1560.

Wright P.H. and Ashford N.J., 1986: Transportation Engineering, Planning and Design (Third Edition). John Wiley & Sons, New York, pp. 234 – 254.

APPENDIX I – FLOWCHART OF PROGRAM



APPENDIX II PROGRAM LISTINGS

MATLAB

```
%PROGRAM TO ESTIMATE THE COST OF A PIPELINE

babinput

Dp=pi*(2*kp+kp^2)*lp/ep+a*b; % design factor

r=(1.56*p*ns/np*w*Chw^(-1.85)*Qm^2.85*H*Dp^(-1)*I^(-1))^(1/6.48);

C=ep*r^2*Dp %total construction cost per unit length of pipeline

IC=I*C % Annual cost of pipeline

E=1.555*I*C %Annual Friction cost per meter of pipeline

M=IC+E % Total annual cost

%S=4/1000;

%r1=(Qm/(0.63*Chw*pi*S^0.54*(1/2)^0.63))^(1/2.63)

%r=r1;

v=Qm/(pi*r^2)%maximum flow velocity

%C=ep*r^2*Dp %total construction cost per unit length of pipeline

%IC=I*C %Total annual cost of pipeline

%E=2/5*I*C annual cost of construction per meter of pipeline

%M=7/5*I*C % Total annual cost


q=np/10000;

M/q;

discharge=pi*r^2*v

cost=M/ns*L*D/discharge

%convey=M/q % Conveyance cost of the total volume

% add cost of debt servicing to total cost ******

% this part of the program writes the result to a file named babatola.txt

% it gives the option of appending the file or overwriting it.

fid=fopen('babatola.txt','w');
```

```
fprintf(fid,'COST ANALYSIS FOR DESIGN OF A PIPELINE \n');

fprintf(fid,'WRITTEN BY BABATOLA J. O \n');

D=now;

fprintf(fid,'THIS RESULT WAS GENERATED ON %s\n',datestr(now));


fprintf(fid,'INPUT PARAMETERS \n');

fprintf(fid,'-----------------------------------------------------\n');

fprintf(fid,'width/radius ratio a  = %6.4f  \n', a);

fprintf(fid,'height/radius ratio b  = %6.4f \n',b);

fprintf(fid,'thickness of pipe/radius ratio kp = %6.4f \n',kp);

fprintf(fid,'cost per unit volume of trench excavation ep =%6.4f

Naira\n',ep);

fprintf(fid,'cost per unit length of pipe in trench lp = %6.4f Naira

\n',lp);

fprintf(fid,'Energy cost of pump output naira p = %6.4f \n',p);

fprintf(fid,'Debt service factor I  =%6.4f \n',I);

fprintf(fid,' Hazen-William Coefficient = %6.4f\n',Chw);

fprintf(fid,'------------------------------------------------------------

------\n \n');


fprintf(fid,'CALCULATED PARAMETERS\n');

fprintf(fid,'Optimal Radius for Conveyance =  %10.6f meters \n',r);

fprintf(fid,'Maximum estimated velocity  = %10.6f  meter/seconds\n', v);

fprintf(fid,'Total construction cost per unit length of pipeline = %10.6f

Naira \n',C);

fprintf(fid,'Annual cost of pipeline = %10.6f Naira \n',IC);

fprintf(fid,'Annual Friction cost per meter of pipeline= %10.6f  Naira\n',

E);

fprintf(fid,'Total Annual Cost of Pipeline= %10.6f  Naira\n', M);
```

```
fclose(fid);
```

RESULTS

```
COST ANALYSIS FOR DESIGN OF A PIPELINE

WRITTEN BY BABATOLA J. O

THIS RESULT WAS GENERATED ON 14-Oct-2003 13:46:20

INPUT PARAMETERS

----------------------------------------------------

width/radius ratio a  = 3.0000

height/radius ratio b  = 3.0000

thickness of pipe/radius ratio kp = 0.1000

cost per unit volume of trench excavation ep =1000.0000 Naira

cost per unit length of pipe in trench lp = 600.0000 Naira

Energy cost of pump output naira p = 6.0000

Debt service factor I  =0.2200

 Hazen-William Coefficient = 150.0000

----------------------------------------------------------------------

 CALCULATED PARAMETERS

Optimal Radius for Conveyance =    1.234982 meters

Maximum estimated velocity  =   0.121736  meter/seconds

Total construction cost per unit length of pipeline = 14330.362260  Naira

Annual cost of pipeline = 3152.679697 Naira

Annual Friction cost per meter of pipeline= 4902.416929  Naira

Total Annual Cost of Pipeline= 8055.096626  Naira
```

VISUAL BASIC

'WRITTEN BY BABATOLA J.O.

Option Compare Database

```
Private Sub Calc_Click()

'*****************************************************************************
**
'Hydrographic Shape Factor
    HydShapeFact = (((FlowQtyMin / FlowQtyMax) ^ 2.85) * (7 / 12)) + (5 / 12)

    'HydShapeFact = (FlowQtyMin / FlowQtyMax) ^ 2.85
    'HydShapeFact = (((FlowQtyMin * 7) + (FlowQtyMax * 5)) / 12) ^ 3

'*****************************************************************************
**
'Design Factor
    DesFact = (WiTrench * Detrench) + ((22 / 7) * ((2 * PiThic) + (PiThic ^ 2)) * (CoPi /
CoEx))

'*****************************************************************************
**
'Best Economic Radius
    BestRad = 1.56 * EnCoPump * (31536000 / KKWHr) * FluidDen
    BestRad = BestRad * (HWc ^ -1.85) * (FlowQtyMax ^ 2.85) * HydShapeFact
    BestRad = (BestRad * (DesFact ^ -1) * (1 ^ -1)) ^ (1 / 6.48)
'BestRad = 1.073263
```

```
'Dim S, and S = Slope added to see what goes on in best Rad 16/10/2003

    'Dim S

    'slope = 4 / 1000

    'S = slope


    'BestRad = (FlowQtyMax / ((S ^ 0.54) * 0.63 * HWc * (((22 / 7) / 2) ^ 0.63))) ^ (1 / 2.63)


'*****************************************************************************
**
'Maximum velocity

    MaxVelocity = FlowQtyMax / ((22 / 7) * (BestRad ^ 2))


'*****************************************************************************
**
'Maximum Energy Gradient

    MaxEnG = 0.63 * HWc * (22 / 7) * (BestRad ^ 2)

    MaxEnG = MaxEnG * ((BestRad / 2) ^ 0.63)

    MaxEnG = (FlowQtyMax / MaxEnG) ^ (1 / 0.54)



'*****************************************************************************
**
'Cost of Construction

    ConCo = CoEx * (BestRad ^ 2) * DesFact
```

```
'*****************************************************************************
**
ConCoY = 1 * ConCo
'*****************************************************************************
**
   AnnConCost = (2 / 5) * ConCoY


'*****************************************************************************
**
'Cost of Fluid Friction
   FluFricCost = 1.555 * ConCoY


'*****************************************************************************
**
'Total Cost of Construction
   TotalCost = ConCoY + FluFricCost
'*****************************************************************************
**
TotalLinkCost = LinkLength * ConCo * 1000
'*****************************************************************************
**
DoCmd.Save
End Sub
```

APPENDIX III ORIGINAL AND PRESENT STATUS OF EXISTING SCHEMES

S/N	Schemes	Location	Design capacity (m3)	Present level (m3)	Constraints
1	Owena / Ondo	Akure / Ondo	19,800	11,880	Under Rehabilitat And unstable NEI Supply.
2	Uso / Ogbese	Ogbese	450	360	Fluctuating NEP Supply
3	Owena / Igbaraoke	Owena / Ilesha	5,480	1,644	-
4	Oke – Igbo	Okeigbo	2,275	910	Under Rehabilitat
5	Osse / Owo	Osse	1,860	472	Power supple & Fuelling
6	Alagbaka Spring Akure	Alagbaka	450	NA	NA
7	Ala Water Supply	Ala Akure	1,260	NA	Abandoned
8	Ukere Spring Water	Oshinle	50	40	NA
9	School of Agric Akure	Sch of Agric	675	3,375	NA
10	Oruju Ifon / Ikaro	Oruju Ifon	1,090	NA	NA
11	Idoani	Idoisale	990	900	Power Supply
12	Okitipupa	Okitipupa	1,000	NA	Burnt pump set
13	Araromi – Obu	Araromi	223	111.5	NA
14	Okeluse Borehole	Okeluse	680	NA	Abandoned
15	Imoru/Ijagba B/h	Imoru	695	NA	NA
16	Ute Borehole	Ute	100	NA	Faulty Gen set
17	Ugbonla Borehole	Ugbonla	918	900	Fuelling
18	Agadagba – Obon Borehole	Agadagba	810	800	NA
19	Bolowu - Zion Borehole	Bolowu	1,080	1,000	NA
20	Zion pepe	Zion pepe	648	NA	Abandoned

NA= Not Available

APENDIX IV RESULTS OF BOREHOLES AND WELL WATER TESTS

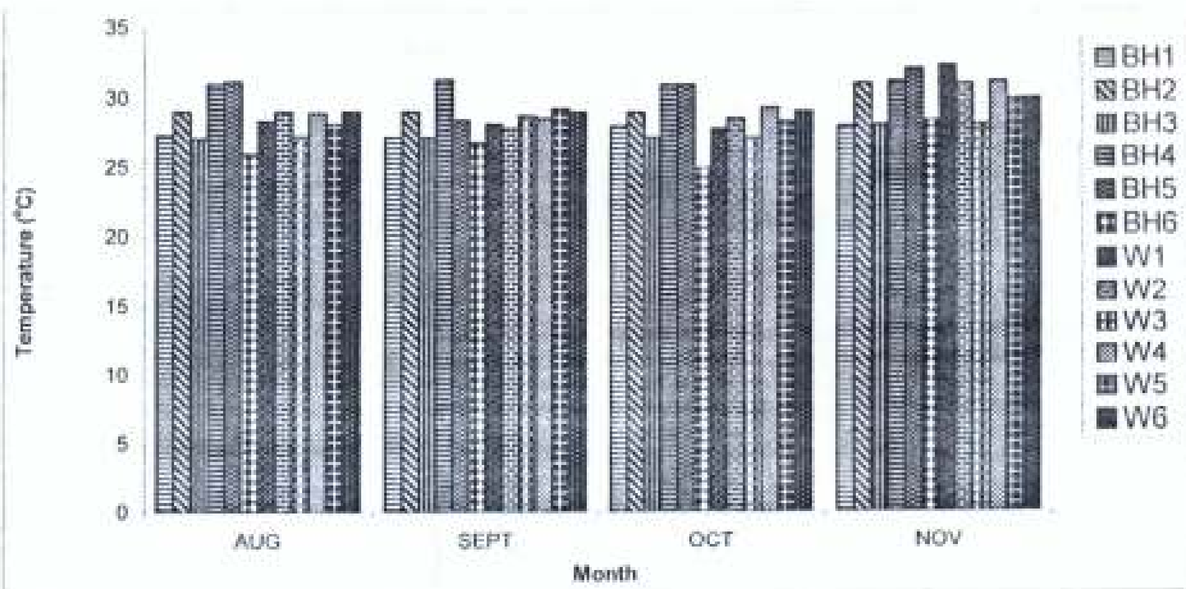


Fig 1: Average Monthly Variation of temperature (Aug-Nov)

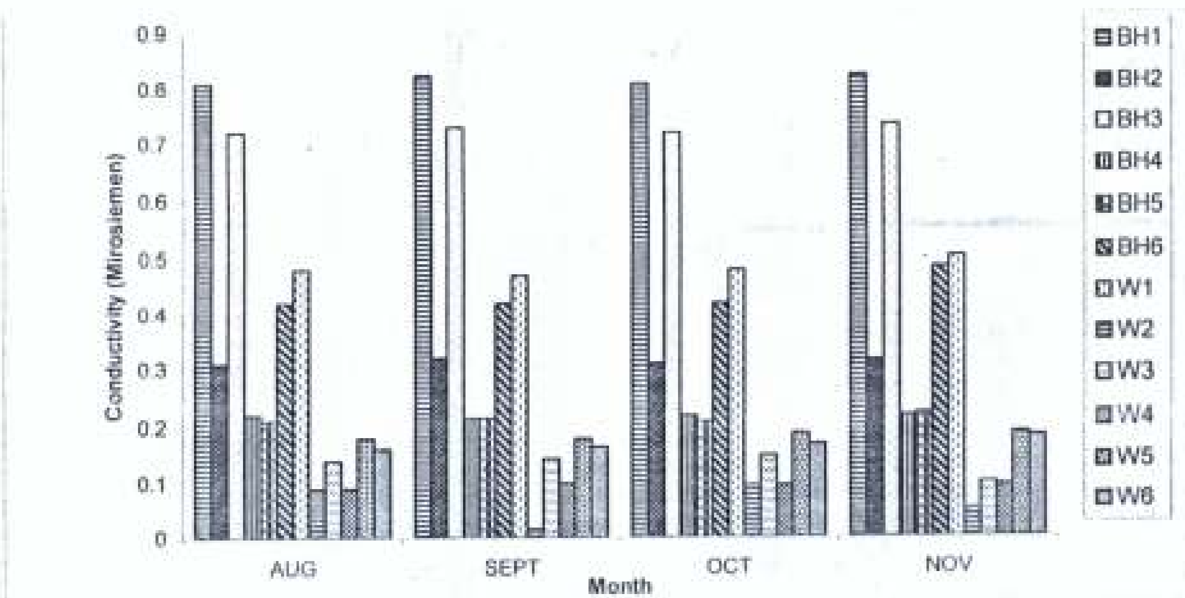


Fig 2: Average Monthly Variation for Conductivity

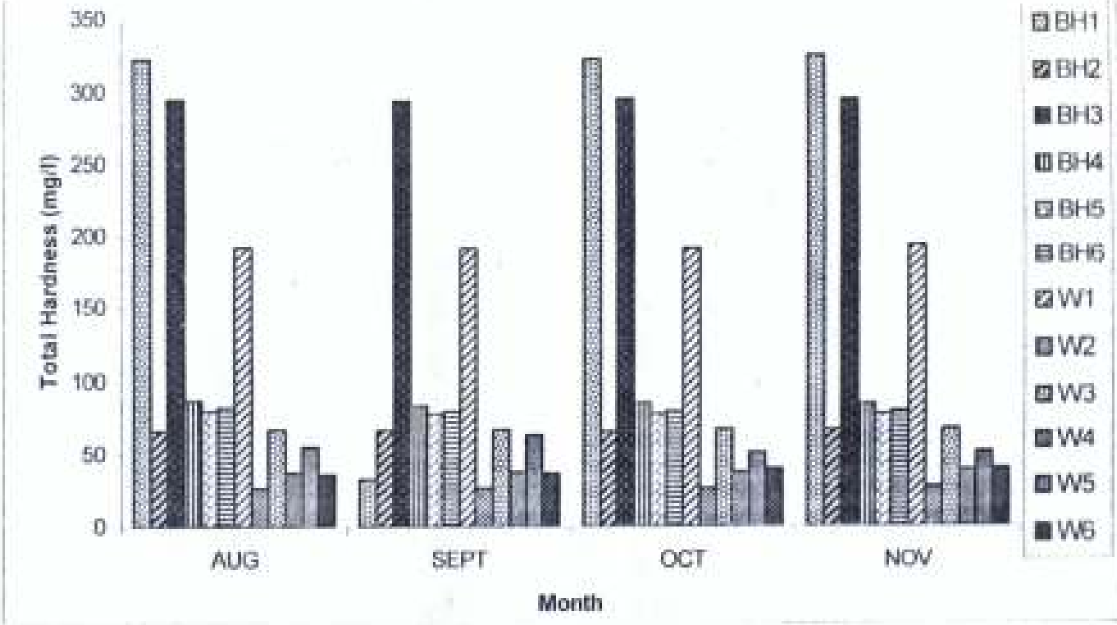


Fig 3: Average Monthly Variation for Total Hardness

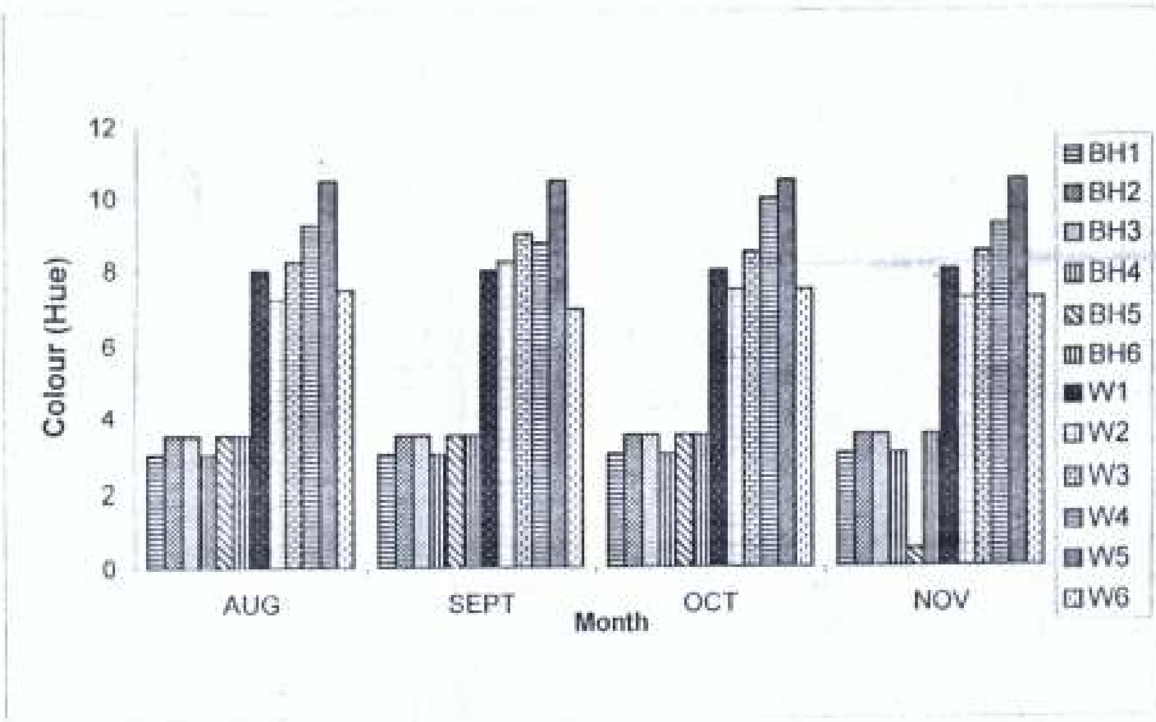


Figure 4: Average Monthly Variation for Colour

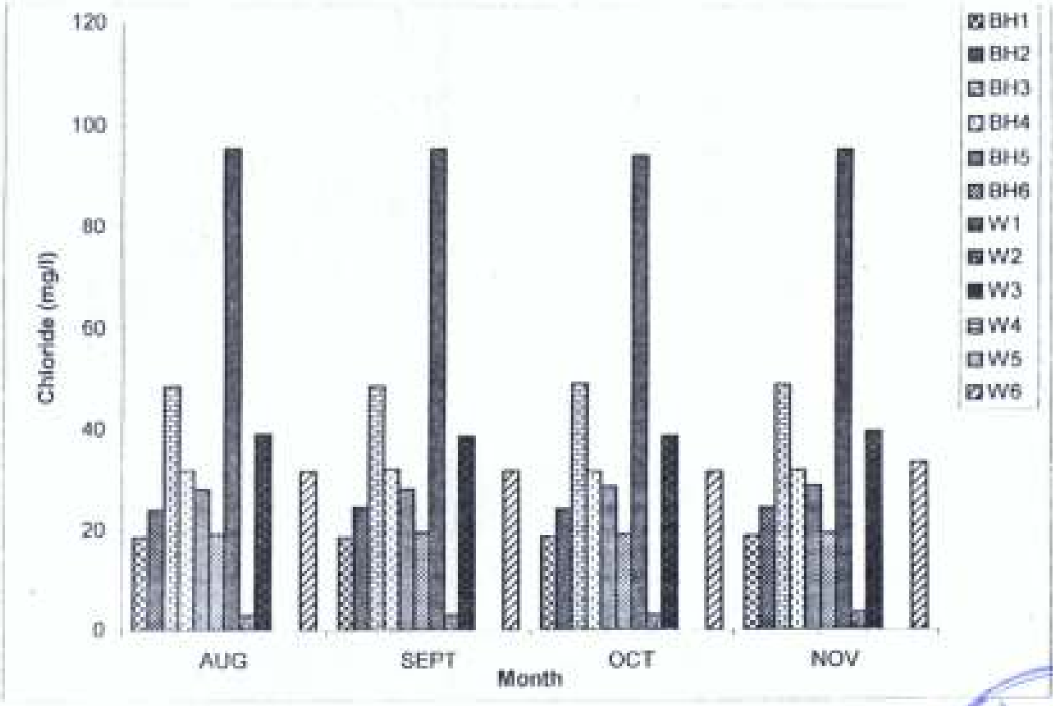


Figure 5: Average Monthly Variation for Chloride

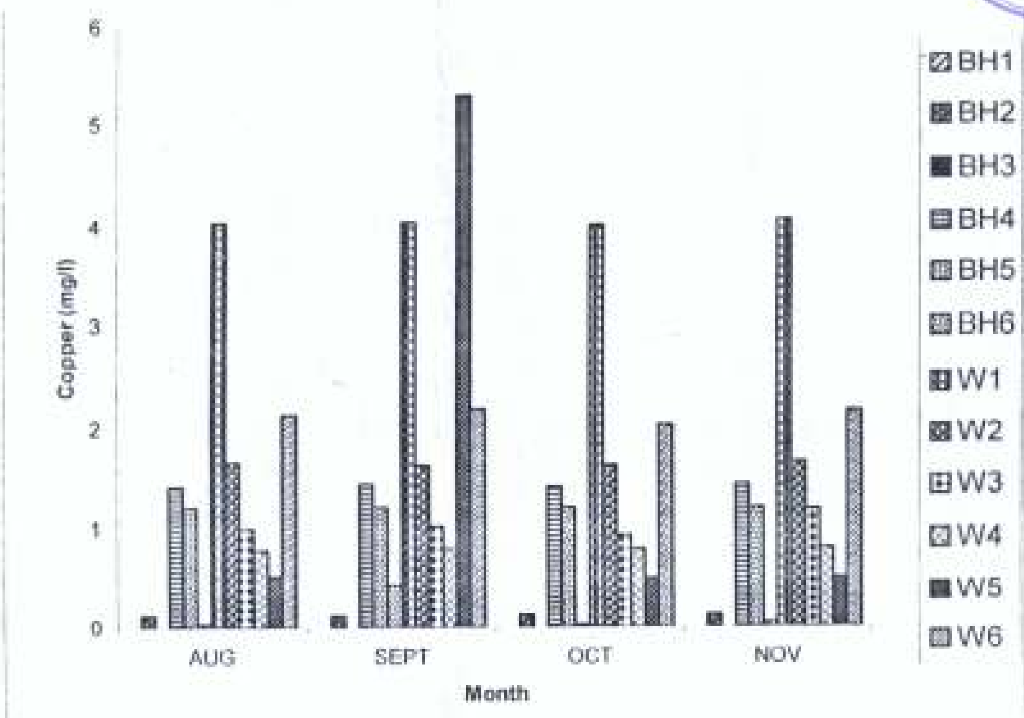


Figure 6: Average Monthly Variation for Copper

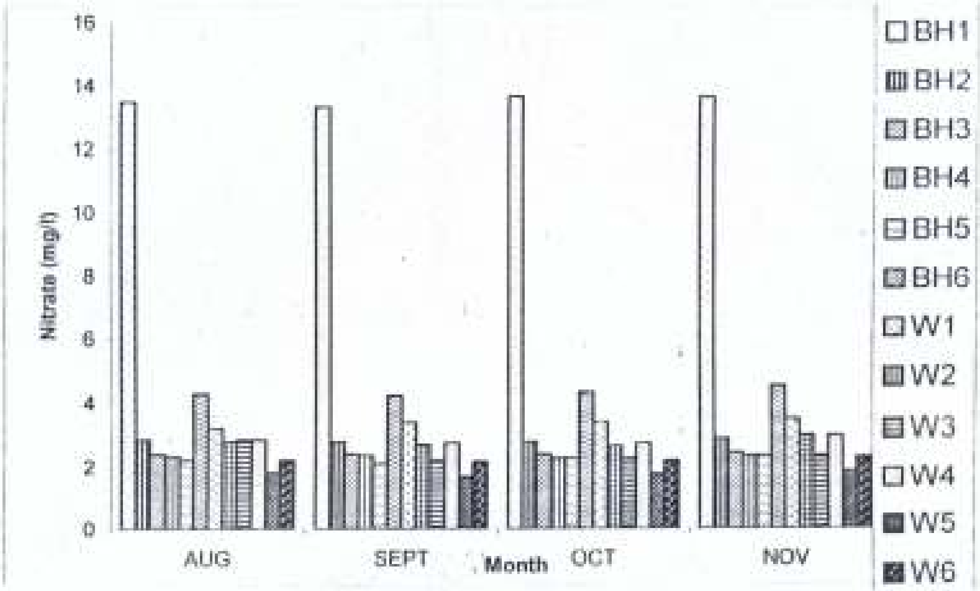


Figure 7: average Monthly Variation for Nitrate

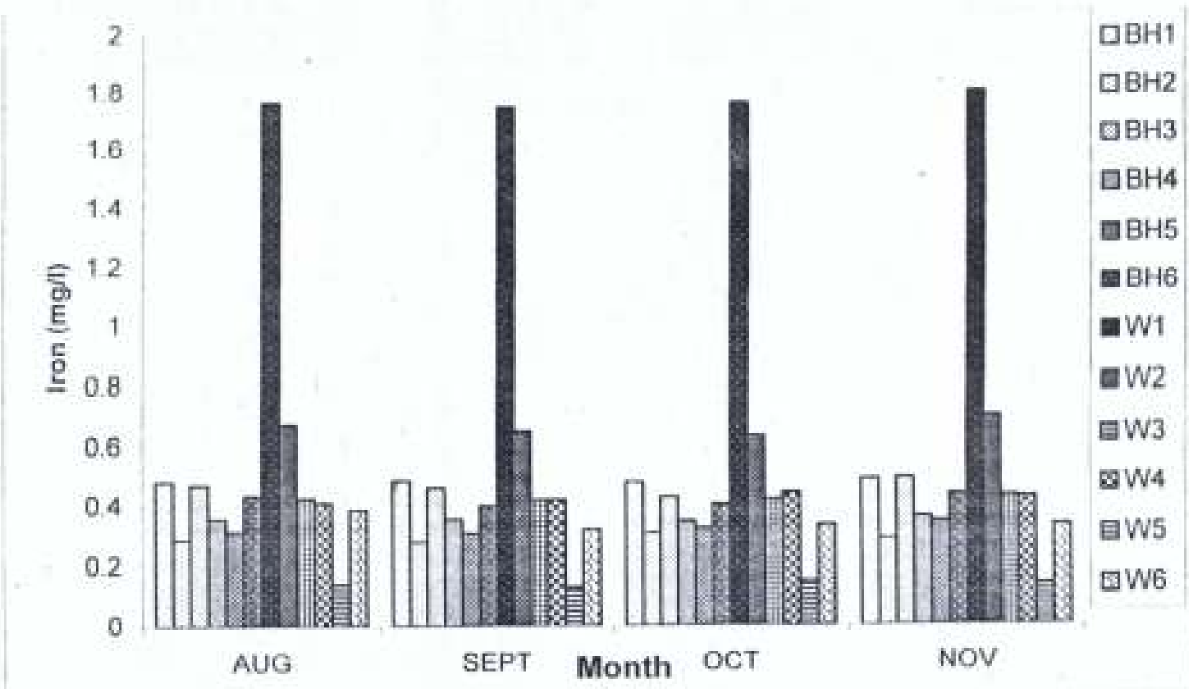


Figure 8: Average Monthly Variation for Iron

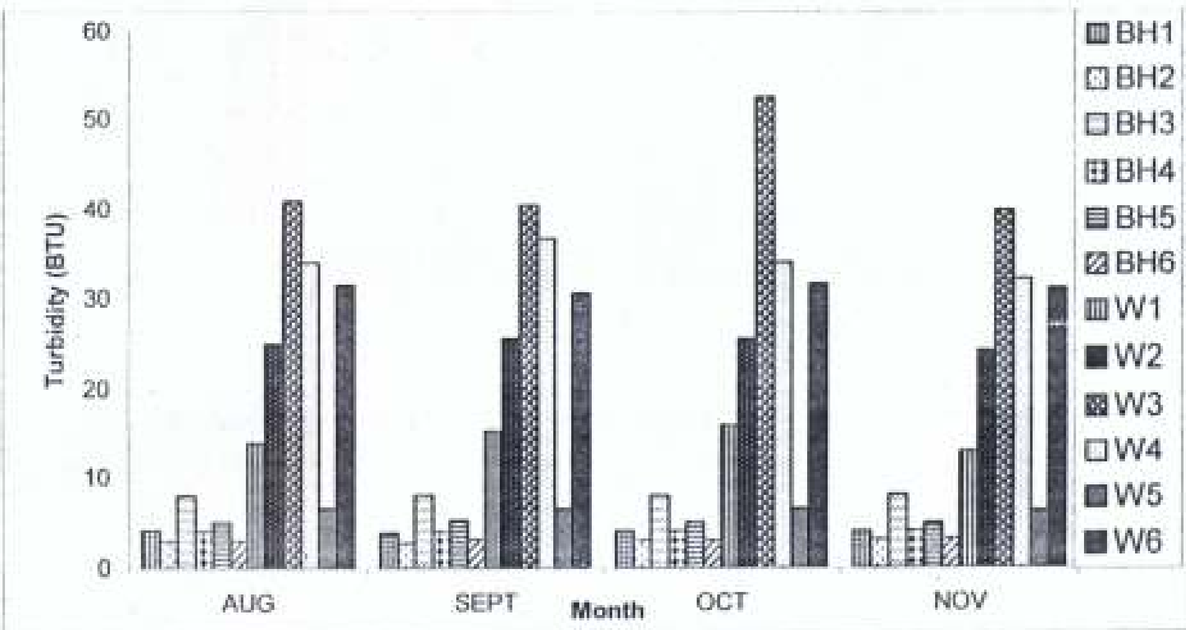

60
50
40
30
20
10
0
Turbidity (BTU)
AUG
SEPT
OCT
NOV
Month
BH1
BH2
BH3
BH4
BH5
BH6
W1
W2
W3
W4
W5
W6

Figure 9: Average Monthly variation for Turbidity

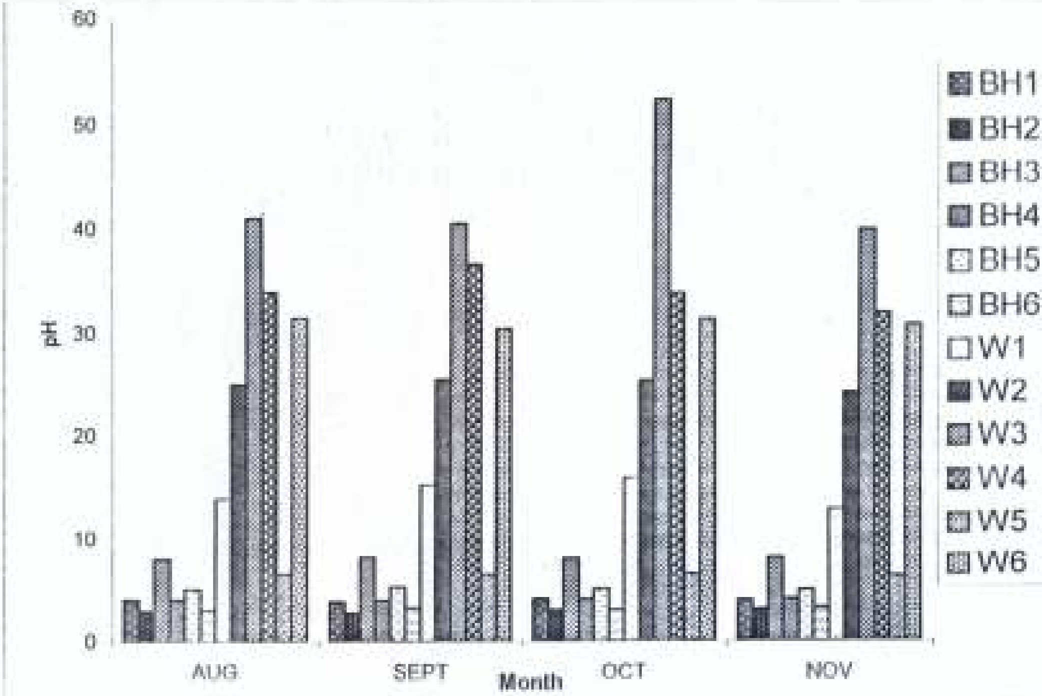


Figure 10: Average Monthly Variation for pH

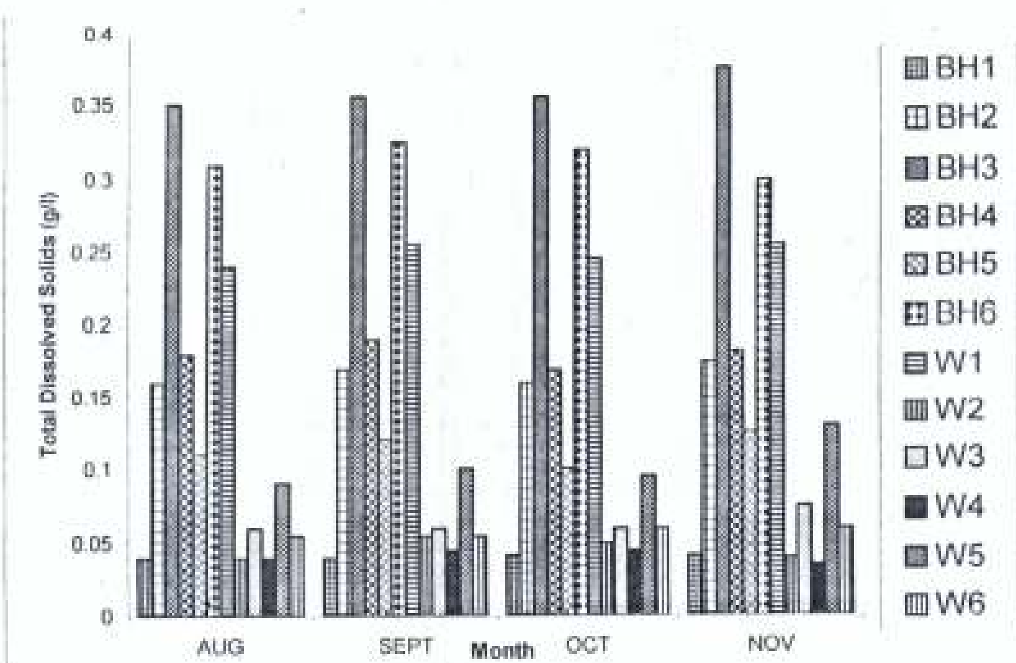


Figure 11: Average Monthly Variation Total Dissolved Solids

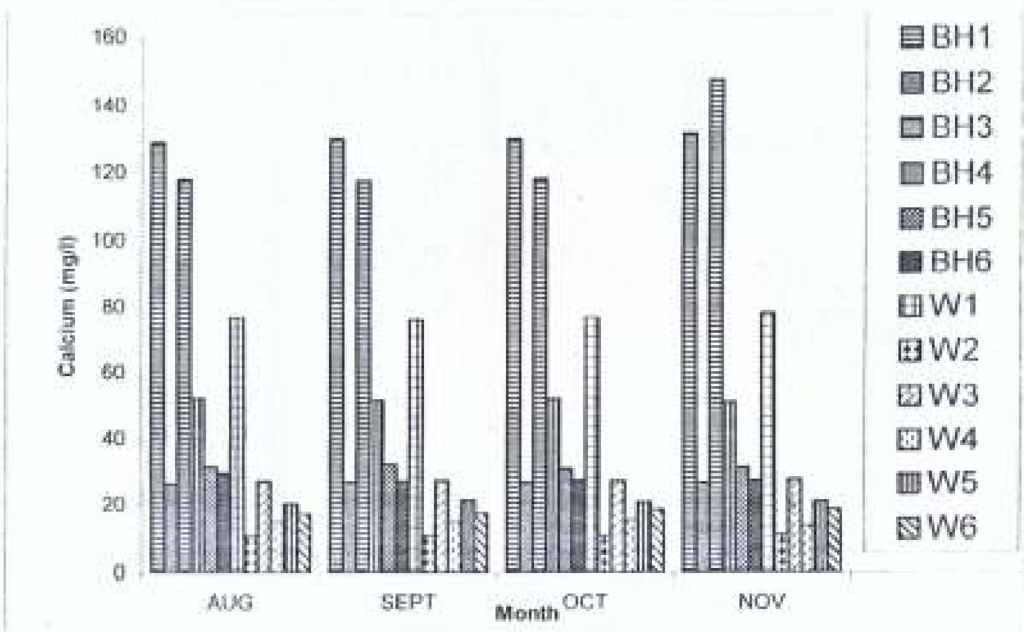


Figure 12: Average Monthly Variation for Calcium

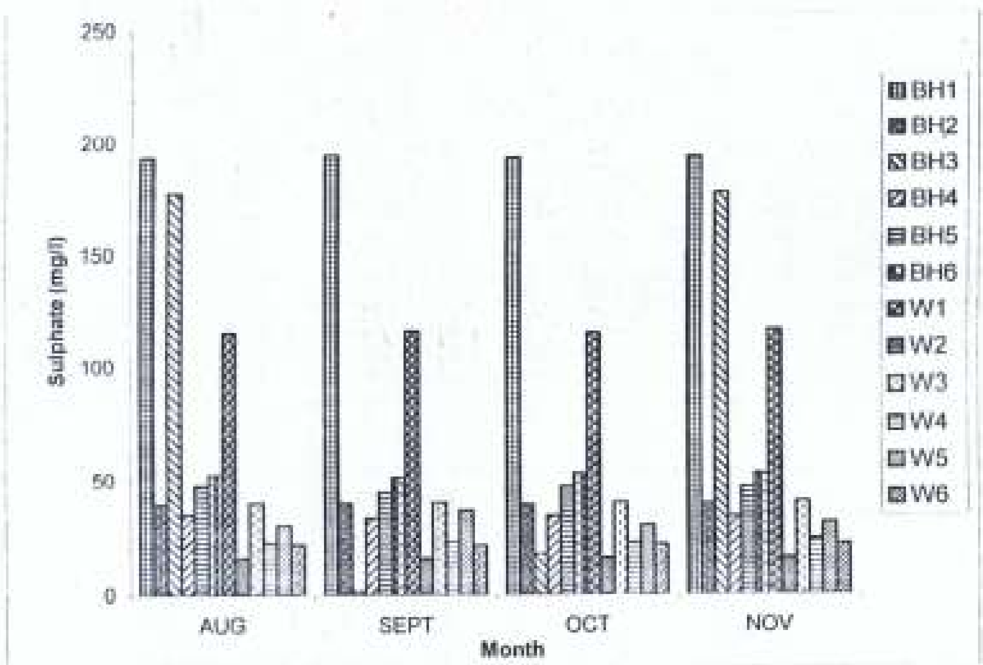


Figure 13: Average Monthly Variation for Sulphate

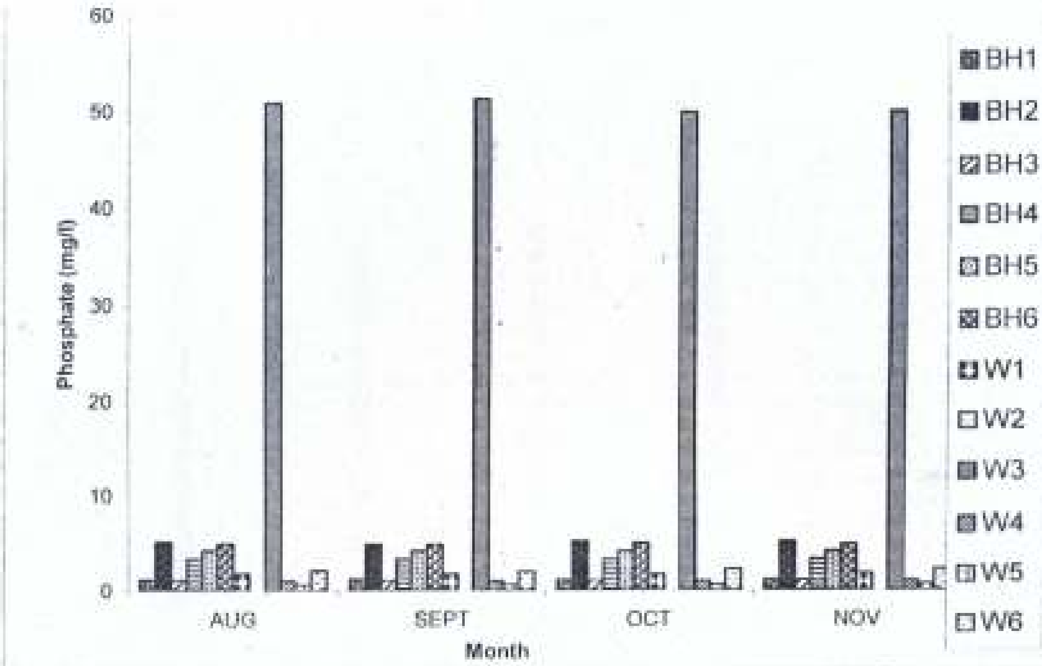


Figure 14: Average Monthly Variation for Phosphate

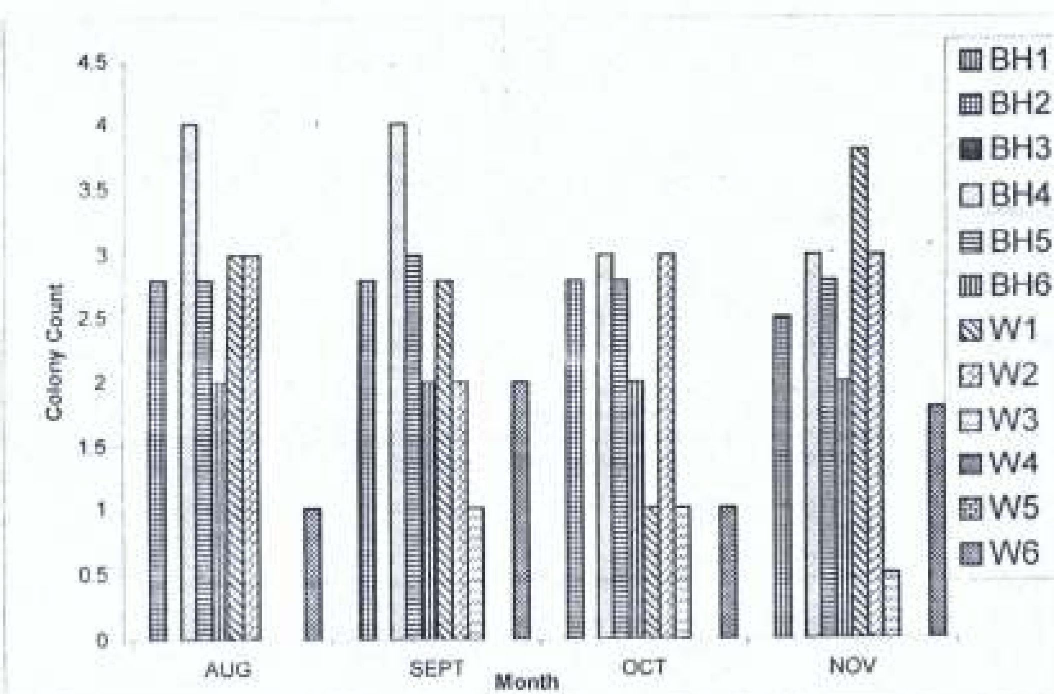


Figure 15: Average Monthly Variation Colony Count

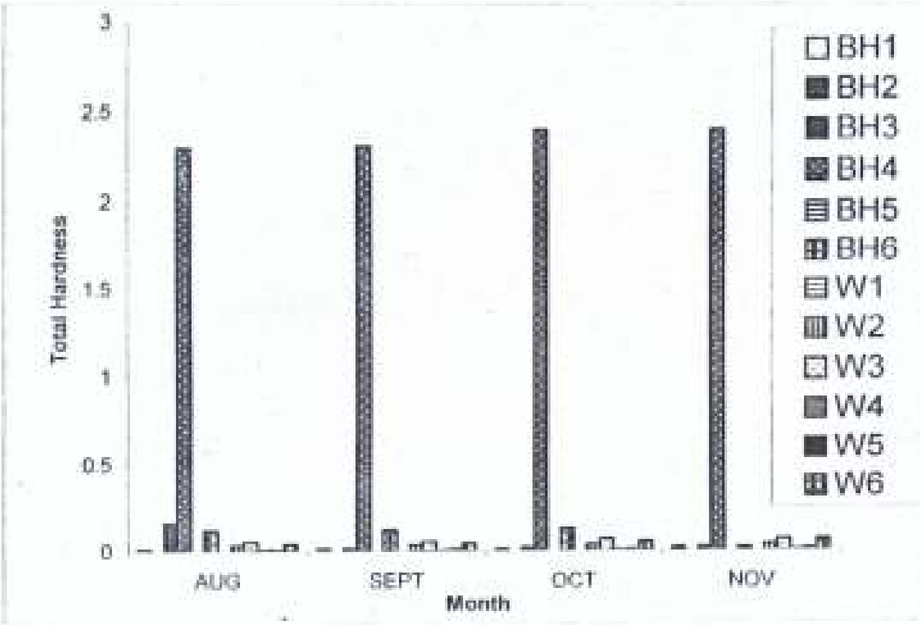


Figure 16: Average Monthly Variation for Total Hardness

APPENDIX Va –APPRAISAL OF DEMAND CENTRES

QUESTIONNAIRES

1. Name of town and state:

2. Population: 5,000

3. Sex of respondent: Male ☐ Female☐ Age < 30 yrs ☐ 30 – 50 yrs ☐
   >50 yrs ☐

4. Position of respondent in family, town/quarters: Head ☐ Tenant ☐ Chief ☐

5. Type of house: (Bungalow; 2 storey; multi-storey

6. Pattern of house (one, two, three, four) flats; Multi-room ☐

7. Number of families living the house One ☐ Two ☐ Three ☐ others ☐

8. Number of people living in the house 10☐ 11-20 ☐ 21 – 30 ☐ 3 0☐
   others ☐

9. Adults: Male................ Female:................

10. Sources of water: From where do you get water for domestic use during

	Raining season?	Dry season?
Water Corporation	☐	☐
Wells	☐	☐
Boreholes	☐	☐
Stream/Rivers/Spring	☐	☐
Others	☐	☐
Combination	☐	☐
None	☐	☐

11. If water Corporation: How often do you get water?
    24 hours ☐ 18 hours ☐ others ☐ None ☐

12. Do your community/town has a private mini dam/intake serving it alone? Yes ☐

No ☐

13. If yes, who runs it: Community ☐ Water Corporation ☐ Private body ☐

Others ☐

14. How old is this scheme?........, I don't' know:.........

15. Do you have water pipe-network in the town Yes ☐ No ☐

16. Does this network covers the whole town: Yes ☐ No ☐

17. Do you have a house connection? Yes ☐ No ☐

18. If your house is multi-family, how many taps do you have? One ☐ two ☐ three ☐

others ☐

19. What period of the day do you usually get water?

Usually morning ☐ Usually afternoon ☐ Usually night ☐ others ☐

20. Do you have a public tap around your house? Yes ☐ No ☐

21. If yes, is the tap working? Yes ☐ No ☐

22. If No, since when? Few months ☐ 1 year ☐ others ☐

23. How can you describe the water supply situation in your area during

Dry season Raining season

Good ☐ Critical ☐ Very critical ☐ Good ☐ Critical ☐ Very critical ☐

Information: From tankers: 500gal @ ₦800 – ₦1,000.

24. Is the tap working? Yes ☐ No ☐

25. If No, why? Technical problem ☐

Water cut off because water bill hasn't been paid ☐

Road construction ☐

Others ☐

26. If yes, how often do water flow during

	Raining season	Dry season
Every day	☐	☐
Once a week	☐	☐
Twice a week	☐	☐

27. How much of your household water would you say you obtain from supply of Water Corporation during

	Raining season	Dry season
Almost all	☐	☐
About half	☐	☐
Very little	☐	☐
None	☐	☐

28. Does your line / tap has a meter?

Yes ☐ No ☐ Don't know ☐

29. If yes, is the meter working

Yes ☐ No ☐ Don't know ☐

30. If yes, how often is the meter read? Once a month ☐

Others ☐

Don't know ☐

31. Do you obtain water from any other source? Yes ☐ No ☐

If yes, specify during

	Raining season	Dry season
Bore hole	☐	☐
Well	☐	☐
Stream	☐	☐
Water vendor	☐	☐

Rain water harvesting ☐ ☐

32. Do you pay for this water? Yes ☐ No ☐ If yes, how much per

	Raining season	Dry season
20 litres keg	☐	☐
25 litre keg	☐	☐
50 litre keg	☐	☐
Other ( )	☐	☐

33. How much is your water bill from Water Corporation per month?

₦250 / house ☐ Flat ☐

₦300 / house ☐ Flat ☐

₦500 / house ☐ Flat ☐

Others ☐

34. How often do you settle this bill?

Monthly ☐

Once in two months ☐

Others ☐

Don't pay ☐

35. If you hardly settle the bill or don't pay at all why?

Poor supply ☐

No supply ☐

Not willing ☐

36. If this house is not yours who pays?

Land Lord ☐

Share among occupants ☐

Nobody ☐

Others ☐

37. How much water do your household uses per day?

	Raining season	Dry season
50 litres keg	☐	☐
100 litre keg	☐	☐
150 litre keg	☐	☐
Other	☐	☐

38. Do you have any river/stream that can be develop for as source of water for your town? Yes ☐ No ☐

39. If yes, what is the name and nature name: ....

Nature: Perennial ☐ Annual ☐

40. Will your community allow this river/stream to be developed? Yes ☐ No ☐

Is there any shrine or taboo that can work against using this river/stream? No

41. Do you have public / private boreholes in your community?

Yes ☐ No ☐ with handpump/motorized pump.

42. If yes, what are their conditions? Functioning /Not-functioning

Public Private

43. Do you pay when you fetch water from such boreholes? Yes/No

Private:................ Public:..................

44. If yes, how much? Public:.............. Private: ....

45. Do you have a well in your house Yes / No

46. I f yes, is the well serving you throughout the year? Yes/No

47. If no, when does it usually dry/having low water?

From October, November:............. March/April:..............

48. Do you have a WC in your house? Yes/No

49. If yes, how many units?

50. Do you get water often to run them? Yes/No

51. If water is made available regularly, will you be willing to pay your water rate? Yes/ No

52. Like how much will you be willing to pay per month? ₦ as demanded by the Water Corporation.

53. If a river in your town is developed to supply water for your town/community, will you like it ?. Yes ☐ No ☐

54. If your town/community is given the opportunity to run this scheme to ensure regular water supply, will you be willing? Yes/No. Will you like to form water users Association in your town? Yes☐ No☐

55. Do you have any industry in your town? Yes ☐ No ☐

56. If yes, mention the ones you know? :...

57. How many primary schools are in your environment? One☐ Two☐ others ☐

58. How many secondary schools are in your environment? One☐ Two☐ others ☐

59. How many higher institutions (college of Education, Polytechnic, Universities)

One ☐ Two ☐ Three ☐

60. Finally, what do you think should be done to improve water supply in your town/community?

Privatization ☐

Build separate water-works for your town ☐

Drill more boreholes ☐

Privatization ☐

Build separate water-works for your town ☐

Drill more boreholes ☐

Develop separate source and treat in site ☐

Improve energy supply (NEPA ☐

Others: (i)

(ii)

(iii)

APPENDIX Vb – APPRAISAL OF SMALL SCHEMES SERVING ONE OR TWO COMMUNITIES

LOCATION:

1. Sex of respondent:................ Highest educational level:.........................
2. Age of respondent:...
3. Position or respondent:..
4. Location of scheme:...................... Population:.................................
5. When commissioned:..
6. Components of scheme: Dam, Reservoir, Intake, Treatment plant:.....................
7. Nature of treatment: Conventional, Unit process:...
8. Size of treatment plant: Big ☐ medium ☐ small ☐
9. Who is responsible for operation (a) Water Corporation (b) Community (c) both
10. No of low-lift pumps: One ☐ Two ☐ Three ☐ Others ☐
11. No of high-lift pumps: One ☐ Two ☐ Three ☐ Others ☐
12. Ages of pumps: Low –lift < 5 yrs ☐ 6-10 yrs ☐ 11-15 yrs ☐ others ☐

    High – lift < 5 yrs ☐ 6 – 10 yrs ☐ 11 – 15 yrs ☐ others ☐
13. Source of chemicals: Water Corporation ☐ Community ☐ Both ☐
14. No of worker on the scheme:
15. Rank of workers (1) Engineers ☐

    (2) Technicians ☐

    (3) Chemist ☐

    (4) Superintendent ☐

(5) Operators ☐

(6) Others ☐

16. Enumerate the common problems of the scheme in the raining season

(1) Epileptic power supply

(2) Generator often breakdown

(3) Pumps often breakdown

(4) Chemical inadequacy

(5) Consumers don't pay their bill

(6) Monthly charges not adequate

(7) Treatment facility too small for the town

(8) No fuel

(9) Poor staffing

(10) Others : Sources dry-up

(11) Source level become too low for low- lift pump.

17. Is there any pipe network in the town Yes ☐ No ☐

18. If yes, how old: 10 yrs ☐ 10 – 20 yrs ☐ 21 – 30 yrs ☐ 30 yrs ☐

19. What percentage of the town is covered by the network:

(1) < 50% ☐

(2) > 50% but less than 100% ☐

(3) 100% ☐

20. How often do you pump water?

(1) Daily ☐

(2) Twice a week ☐

(3) others ☐

21. How many hours of operation ? 5 hours

22. How many times has the pump been replaced since commissioned? Once ☐ Twice ☐

others ☐

23. Which month(s) of the year do regard as most critical in your operation:-------------

Why? (1)...................... (2)............................ (3)...................

24. On which river is your reservoir / intake located? --

25. If you are operating on a full capacity, can your system supply water to the whole town?

Yes ☐ No ☐

26. If No, how then do you meet the demand?

(1) By rationing through diversion ☐

(2) Continuous supply to all section ☐

(3) Some section don't get all ☐

27. What are your suggestions to improve your operation what is the source of energy for your operation:

(i) NEPA ☐

(ii) Generation ☐

(iii) Both ☐

28. If NEPA who settles the bill?

(a) Community ☐

(b) Local Government ☐

(c) Water Corporation ☐

29. What is your average daily production? ☐

30. How many towns do you supply

One ☐ Two ☐ Three ☐ Above three ☐

31. Can you say that your scheme is meeting the demand for the town? Yes ☐ No ☐

32. If No, what percentage (approximately) is the shortage

< 10% ☐ 11-15% ☐ 16-20% ☐ others ☐

33. What are your suggestions to improve your operation?

	Raining season		Dry season
Expand the scheme	☐		☐
Replace the pumps	☐		☐
Increase the charges to meet cost	☐		☐
Improve power supply	☐		☐
Improve chemical supply	☐		☐
Improve staffing	☐		☐
Improve funding	☐		☐
Handover to community to run fully	☐	partially	☐
Form Water Users Association	☐		☐
Replace generator	☐		☐
Improve education of the operators	☐		☐

Others (1) ______________________________

(2) ______________________________

(3) ______________________________

(4) ______________________________

APPENDIX VI – CORRELATIONS

Correlations

		v8a: Source of water: Raining Season	v8b: Source of water: Dry Season	v9: How often do you get water	v10: Does your community/town has a private mini dam/intake serving it alone	v13: Do you have water pipe network in the town	v14: Does this network cover the whole town	v15: Do you have a house connection	v18: Do you have a public tap around your house
v8a: Source of water: Raining Season	Pearson Correlation	1.000	.564**	.294**	-.072	.066	-.005	-.021	-.059
	Sig. (2-tailed)		.000	.000	.338	.379	.952	.780	.428
	N	180	180	180	180	180	180	180	180
v8b: Source of water: Dry Season	Pearson Correlation	.564**	1.000	.226**	.018	.080	-.005	.078	.036
	Sig. (2-tailed)	.000		.002	.806	.285	.946	.300	.631
	N	180	180	180	180	180	180	180	180
v9: How often do you get water	Pearson Correlation	.294**	.226**	1.000	.044	-.012	.084	-.066	.004
	Sig. (2-tailed)	.000	.002		.557	.869	.263	.379	.961
	N	180	180	180	180	180	180	180	180
v10: Does your community/town has a private mini dam/intake serving it alone	Pearson Correlation	-.072	.018	.044	1.000	.224**	-.021	.124	.221**
	Sig. (2-tailed)	.338	.806	.557		.003	.784	.096	.003
	N	180	180	180	180	180	180	180	180
v13: Do you have water pipe-network in the town	Pearson Correlation	.066	.080	-.012	.224**	1.000	.245**	.140	.202**
	Sig. (2-tailed)	.379	.285	.869	.003		.001	.060	.006
	N	180	180	180	180	180	180	180	180
v14: Does this network cover the whole town	Pearson Correlation	-.005	-.005	.084	-.021	.245**	1.000	.169*	.075
	Sig. (2-tailed)	.952	.946	.263	.784	.001		.024	.319
	N	180	180	180	180	180	180	180	180
v15: Do you have a house connection	Pearson Correlation	-.021	.078	-.066	.124	.140	.169*	1.000	-.068
	Sig. (2-tailed)	.780	.300	.379	.096	.060	.024		.365
	N	180	180	180	180	180	180	180	180
v18: Do you have a public tap around your house	Pearson Correlation	-.059	.036	.004	.221**	.202**	.075	-.068	1.000
	Sig. (2-tailed)	.428	.631	.961	.003	.006	.319	.365	
	N	180	180	180	180	180	180	180	180

**. Correlation is significant at the 0.01 level (2-tailed).

*. Correlation is significant at the 0.05 level (2-tailed).