DEVELOPMENT OF A MODEL FOR WATER SUPPLY AND DISTRIBUTION FOR A SMALL COMMUNITY

By

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CERTIFICATION

It is hereby certified that this is an original and independent research project carried out by Mr. Adesola A. Ilemobade in the Department of Civil Engineering of the Federal University of Technology, Akure in Partial fulfilment of the requirements for the award of Master of Engineering Degree.

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DEDICATION

This work is dedicated to

Adeoye and Olakitan Ilemobade,

&

The Green Olives

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Lord, How can I thank you enough for all the joy that you give me as I serve you. Unto you alone do I raise my 'Ebenezer'. For all the people, the places, the material, the strength, the patience, the knowledge, the understanding, and the grace, I am indeed grateful.

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NOMENCLATURE

C_{HW}	-	Hazen - William's coefficient
C_n	-	Critical Node
CPM	-	Critical Path Method
D	-	Diameter of pipe
D_{min}	-	Minimum Diameter
e	-	Wire to Water efficiency
E	-	Energy lost through friction
E_C	-	Energy cost
e_p	-	Pump efficiency
e_m	-	Motor efficiency
g	-	Acceleration due to gravity
h	-	Total dynamic head of pump
h_{ASP}	-	Actual Service Pressure head
h_{MSP}	-	Minimum Service Pressure head
H_F	-	Head loss
i	-	Node at beginning of pipe X
j	-	Node at beginning of pipe Y
L	-	Length of pipe
lcd	-	Litres per Capita per day
Me_C	-	Marginal Energy cost
Mp_C	-	Marginal Pipe cost
m	-	Manning's coefficient
n	-	Projected Period
N_C	-	Node consumption
S	-	Source node
P	-	Total number of pipes connecting pairs of nodes
P_C	-	Pipe cost
P_n	-	Projected population
P_p	-	Present population
P_X, P_Y	-	Pressure at X, Y
Q	-	Flow
S	-	Total number of Source nodes

S_C	-	Standard cost of pipe
S_P	-	Surplus pressure
t	-	time of operation during service time
U_J, U_I	-	Maximum quantity of water at node j, I
U_X, U_Y	-	Velocity of Unit mass of water at X, Y
V	-	Velocity
V_I, V_J	-	Minimum quantity of water at node j, I
x	-	Growth rate
Z_X, Z_Y	-	Height of X, Y above a given datum
λ	-	Friction factor
ΔH_{IJ}	-	Pipe pressure loss between node i and j
ρ	-	Unit weight of water
γ	-	Specific weight of water
f	-	Function

ABSTRACT

A model for the optimal design of simple conjunctive - source, looped, gravity - fed water distribution systems is developed and applied to the redesign of the water distribution system for a small urban estate. The model proposed is a modified version of that proposed by Hans Jorgen Rasmusen (1976) and involves two major steps; The Network Solver and The Pipe Modification Algorithm. The Network Solver utilises the Hardy Cross method for balancing flows and pressures. The Pipe Modification Algorithm then optimises the entire network of distribution pipes using a heuristic procedure on discrete pipe diameters until an optimal solution is arrived at. The variation of the combination of the diameter of the distribution pipes and the height of the overhead tank (pressure head) supplying the water is the decision variable. The model allows the optimisation of each pipe diameter to revolve around one of three choices; reducing to the next smaller available pipe diameter, leaving the diameter unchanged, or increasing to the next larger available pipe diameter. While the model is helpful for recommending optimal pipe sizes, the experience of the designer is most useful in the judicious selection of one of the alternatives proposed, which will best suit the required conditions on site.

A distribution system of four networks of 74 pipes is proposed for Ijapo estate, Akure city - Nigeria to replace the existing distribution system of three networks of 64 pipes. Population is expected to stabilize within Ijapo estate in twenty-five years. Hence, a design period of twenty-five years is used. Reliability of water flow to every node within the distribution networks despite breakage, blockage, or leekage in pipes is made possible by the introduction of redundant pipes. From the initial proposed system of four distribution networks, an efficiency of between 2 - 15% was achieved, reducing the total initial pipe and energy costs by about #200,000:00.

A conjunctive water supply system comprising of two imported sources of water (the Owena water supply scheme and, the College of Agriculture mini-water supply scheme) and four boreholes is recommended to supply the quantity of water required by the residents of Ijapo estate throughout the design period.

CHAPTER 1

INTRODUCTION

Ijapo estate, Akure was built about 20 years ago with imported water from the city's central water supply scheme as it's major source of potable water. During the planning, design and laying of the pipe system, the estate was divided into three(3) independent distribution networks with initially only one source(the Owena Water Supply Scheme through the overhead storage tank at Alagbaka estate, Akure), and then later, from two sources (the Owena Water Supply Scheme and the College of Agriculture Mini-Water Supply Scheme). Two main pipes of 250mm diameter supply the estate potable water from the storage tank at Alagbaka estate. They are presently laid along the two major roads(Oke-Ijebu road and Oba Adesida/Ado-Ekiti road) which boarder the estate at the east and west respectively. When the College of Agriculture Mini-Water Supply Scheme was constructed, the water supply pipe from that scheme was connected to the main pipe running from the storage tank to the estate at the western section. These two sources of potable water to the estate though once sufficient, are now inadequate.

1.0.i <u>The Owena Water Supply Scheme</u>

The Owena Water Supply Scheme was handed over to the Ondo State Government after the state's creation on the 3rd of February, 1976. The Owena water supply scheme comprises of two independent water supply facilities; the Owena water supply scheme along Ondo/Akure road and the Owena water supply scheme along Ilesha/Akure road. Presently, the largest quantity of water supplied into Akure city comes from the Owena water supply scheme along Ondo/Akure road. At that time a total of about 8 million litres of potable water was distributed to such cities as Akure; the state capital, Ikere-Ekiti, Idanre, Ile-Oluji, Ondo, Iju and Ita-Ogbolu. Since 1980, this supply of potable water to these cities have become grossly inadequate.

1.0.ii <u>The College of Agriculture Mini-Water Supply Scheme</u>

This scheme was commissioned by the then Ondo State governor on the 21st of November, 1988. It's major purpose was to compliment the existing central water supply scheme from the Owena river. It was built to supply potable water to Oba-Ile housing estate, Ijapo estate and the College of Agriculture .

It comprises one large and small dam(Upper and Lower dam respectively), a treatment plant, and a pumping plant. Two storage reservoirs were built within the School's compound with capacities of about 300 and 1,200 cubic metres. The larger storage tank is located on a hill adjacent to the scheme in order that water may be released by gravity.

Due to the growth of industries and commercial activities within the state capital, and the expanded public services at Federal, State, and Local levels, there has been an increasing influx of people into the city over the past few years with major settlements especially within the immediate environment of the major storage reservoir at Alagbaka estate. As a result, the Ondo State owned Water Corporation has had to divert the inadequate quantity of water originally meant for the entire city's Industrial, Commercial, Agricultural and Domestic consumption from the major storage tank to those now located within it's immediate environment. As a result of undue pressure, the officials of the Water Corporation stationed at the mini-water supply scheme at the College of Agriculture, have had to divert water meant for the three major consumers to one. The students of the College insist that they be given top priority twenty-four hours a day. This is obviously done to the detriment of the other two major consumers who at most times, have to source for their own water.

1.1 A BRIEF REVIEW OF RASMUSEN'S MODEL

Hans Jorgen Rasmusen (1976) proposed a heuristic optimal model for the analysis of a single or multi-source, water supply system consisting of looping or branching networks. The main computational problem of network optimisation rests upon the simultaneous solution of the hydraulic flow (balancing the network flows and headlosses) and the adjustment of the pipe diameters towards an optimal solution. Rasmusen therefore decomposed this problem in his model into two subproblems, namely;

(i) A hydraulic network flow problem which for any known values of pipe diameters and nodal consumptions can be readily solved by applying Kirchoff's first law of flow continuity and second law of energy loss around a network ; and

(ii) A diameter modification algorithm which attempts to modify the pipe diameters towards an optimal solution.

The decomposition of the model into two subproblems implies that the hydraulic flow and diameter modification must be solved iteratively until the optimal solution is found. Discrete pipe diameters are used, and therefore rounding-off continuous values of pipe diameter is eliminated. The diameter of each individual pipe is revolved around three choices; reduce to the next smaller available pipe diameter, leave it unchanged, or increase to the next larger available pipe diameter. This choice depends on the ratio between the prevailing marginal energy and pipe investment cost. Given that all inflows and outflows are constant, the least cost economic design is given by the pipeline diameter vector, D_p which minimises the investment in both pipe (installation and maintenance) and energy (installing, operation, and maintenance of pumps) costs. This model is constrained to water supply networks but the optimisation principle is adaptable to other flow problems.

1.2 PROBLEM DEFINITION

This thesis is therefore to address within Ijapo estate such problems as:

a) The need to improve the existing water supply and distribution system to accommodate the present and projected population for twenty-five years.

b) The consistent low pressures experienced at water supply input points into the estate from the existing water supply sources, as flow of water is either not experienced at all, experienced in trickles, or is restricted to the residential and commercial activities at the periphery of the estate.

c) The need for a centralised mini-water supply system which is able to supply adequate quantities of potable water and which will be easier to operate, monitor and maintain.

d) The under design of the present distribution system by failing to provide optimal pipe sizes for both large and small water consumers. Pipes of the same size were used throughout the present distribution system without distinction to secondary and small-distribution mains.

e) The large number of dead ends within the existing distribution networks causing large leakage of water during water flow.

The problems are therefore attacked at both the planning and operational level where the determination or modification of some or all of the numbers, capacities, locations and/or sizes of various components are required to improve the existing distribution at minimal cost. Recommendations are necessary to determine quantities of water expected from each source to meet daily or seasonal demands.

1.3 SCOPE OF WORK

As an engineering project, the scope of work was defined to include the following:

a) To review some relevant literature on water supply and distribution systems.

b) To cross-check preliminary and detailed designs and analysis of the existing water supply and distribution system within Ijapo estate.

c) To determine the schedules of water supply into the estate within a specified period from the relevant quarters, and verify same on site.

d) To carry out checks on leakage within the distribution system.

e) To check extent of water flow within the estate during water supply.

f) To verify types, sizes, and locations of pipes laid within the estate as exact as that on the plan.

g) To carry out all necessary physical measurements to obtain the necessary parameters for optimal design and analysis.

h) Consultation with relevant institutions e.g. Ondo State Housing Corporation and Water Corporation .

i) Consultation with experts on the physical characteristics of the estate e.g. Planners, Architects, Geophysicists and Geologists.

j) To submit a final project report covering the entire project.

1.4 METHODOLOGY

The following procedure was adopted for the research work:

(a) **Reconnaissance Survey:**

A Reconnaissance survey of the entire project area was carried out in the form of studies of all available maps(Geological, topographical, hydrogeological, e.t.c.), plans(layout, e.t.c.) and reports(Geophysical e.t.c.) of the estate. The study area was thoroughly investigated in order to be acquainted with the project for a fuller understanding of the existing and projected conditions of the estate.

(b) **Design Check**

A Design Check was carried out on the previous designs made for the estate's distribution system to verify deviations from prescribed standards.

(c) **Spot Levels**

Spot levels were carried out on the major streets and avenues within the estate to obtain exact elevations of all consumption points above the given datum and slopes of the relevant streets.

(d) **Sample Surveys**

Sample surveys were carried out within certain sections of the estate to obtain certain information vital for design such as:

(i) Average number of residents per plot.

(ii) Water demands.

(iii) Water consumption patterns.

(iv) Existent infrastructural developments originally not provided for, but existent on ground.

(v) Bore hole and hand-dug well locations and yields within the estate.

(vi) Other individual sources of water acquisition.

1.5 OBJECTIVES

Within the scope, the project focused on the following main objectives:

(a) To study the existing water supply and distribution system of Ijapo estate through analysis and field observations and to identify specific technical problems that require engineering solutions.

(b) To identify all existing and potential water supply sources within Ijapo estate considering their yields, suitability and locations.

(c) To recommend a conjunctive supply system which will include some or all of the existing and proposed water supply sources based on all-year round supply and least cost.

(d) To produce a design of an improved, effective distribution system for the estate with the source from a centralised conjunctive supply system using appropriate empirical models.

(e) To estimate the cost of the proposed improved water distribution system and compare with the existing distribution system, so as to determine the cost effectiveness and benefits.

CHAPTER 2

LITERATURE REVIEW

'Water is the best of all things'

Whichever way a body of water may exist on, above or below the surface of the earth, it is a basic human need and the procurement of it in sufficient quantities is a constant concern to people everywhere (Nwosu and Ojiako, 1989). In the same way as finances is the key to controlling a nation, so is food the key to controlling a people. Water is considered the least cost raw material from which most of our farm produce is made. It is essential for the growth of crops and animals and is a very important factor in the production of milk and eggs. Water constitutes 85% of milk, 66% of eggs, 77% of beef, 80% of fish and 75% of potatoes (Wright, 1977). Without potable water, life and civilisation cannot survive (Ayoade, 1988). For this reason, a nation is on the path to greatness if it can provide good enough water for it's inhabitants to drink, use in cooking, industry and sanitation

For the reason of locating potable water, communities in ancient civilisation moved from one place to the other. It is the one factor which has not only affected movement, but also the size of communities (Nwosu and Ojiako, 1989). As the positive aspects of water supply e.g. sanitation, health, crop and animal growth, have encouraged growth of communities, so have the negative aspects e.g. contribution to desease, discouraged and reduced community growth. Most cities which rapidly developed were situated near bodies of consumable water. Some of the catalysts for that growth include enhanced communication, navigation, commerce and trade, and protection from external aggression. It would be difficult to name any major town or city in the United Kingdom, Nigeria or elsewhere which either does not lie on the banks of a river or has easyaccess to other sources of potable water (Price, 1985).

The development of this resource is a major concern for Local, State, and Federal governments everywhere who constantly strive to make it readily available for consumption. Recent history reflects the basic role of water in socio-economic development (Cox, 1988). Howe emphasises that water is an important factor for regional growth. Though not the 'Index' for development in all regions, water is definitely one factor that cannot be ignored (Howe, 1968). Other factors which may dictate regional growth include environmental growth, human factors, and the state of physical and institutional infrastructure for the support of the development process e.g. transportation and favourable market conditions.

In all societies, water is considered as 'Top Priority'. This is so because water dependent activities are considered major catalysts for development and large sources of revenue. Where water is ignored or not given it's rightful place, development is retarded. Such water dependent activities include irrigation and industry. Therefore, water must be viewed as one of the several elements of the socio-economic growth process with potential to become a limiting factor (Cox, 1988).

2.1 SOURCES OF WATER

The three major sources of water are (a) Ground water, (b) Surface water, (c) Rain water. Other potential sources of potable water include, (d) De - Salination and (e) Waste - Water treatment.

i. Ground water

This is commonly referred to mean water occupying all the voids within a geographical stratum (Todd, 1980). Many cities around the world for reason of climate or geology have no permanent streams, rivers or ponds, yet have been settled for thousands of years because of their reliance on supplies of water from underground sources which are often a few metres below the earth's surface. The Persians about 3200 B.C used Kanat systems extensively to develop their ground water resources. Kanat systems were artificial underground channels which carried water over long distances either from a spring or water-bearing strata (Biswas, 1976). At present, certain settlements in the United Kingdom and Middle East still bear water related names. Such names either begin with "Bir -" or "Beer -" or end with "- well" (Price, 1985).

Ground water accounts for about 30% of the world's fresh water and is located within a depth of 200 - 600 meters below ground level (Chow, et.al.,1988). It is the largest accessible store of fresh water and is frequently of the best quality for domestic and agricultural usage in the third world. It is normally free of contaminants and can be consumed without treatment (Linsley, et.al., 1992). Ground water is replenished by meteoric precipitation, infiltration and percolation through the surface and sub-surface soil.

This resource may be tapped through the construction of wells. Classification is made according to the method of construction. They are as follows:

(a) Dug wells

(b) Driven wells

(c) Jetted wells

(d) Bored wells

(e) Drilled wells

(Wright, 1977)

Distinct advantages of ground water are:

- Constant temperatures which are maintained in the aquifer.
- Slow movement or seepage of water from the aquifer.
- Greatly reduced evaporation losses.
- Major disadvantages which occur as a result of over-extraction are;
- Salt water intrusion into the aquifer resulting in the deterioration of water quality (Kitching et. al., 1975).
- Land subsidence; examples have been measured in Thailand - Bangkok between 1969-85 at 800mm/yr. and Maiduguri - Nigeria at 6m/yr. (Ogwude and Oteze, 1988)
- Water logging on an area during an excess recharge situation.
- Falling water tables (Aron and Scott, 1971).

ii Surface water

This is water which exists on the surface of the earth in different forms such as streams, oceans, rivers, lakes and ponds. Fresh surface water accounts for 0.266% of the world's fresh water of which 0.006% is in rivers, and 0.26% in lakes. The oceans of the world are the largest containers of water(96.5%), but they are saline in nature (Chow,et.al., 1988).

Many communities in ancient civilisation had surface water as the major and only source of consumable water. Till present, many cities and towns are dependent on fresh surface water because of it's cheapness and accessibility.

Surface waters are highly susceptible to pollution in form of waste effluents from domestic, industrial, commercial, or natural activities from it's immediate environment e.g. storm water, vegetation and animal activity.

The major source of surface water is precipitation. This is, either directly, from storm water or underground seepage. Surface waters are susceptible to higher evaporation rates than groundwater.

iii. **Rain water**

Pacey and Cullis (1986) provides a comprehensive reference on the use of water collected during rainfall. The method of collection is called Rainwater Harvesting and is defined as the gathering and storage of water running off surfaces on which rain has directly fallen. It should not be mistaken as the harvesting of valley floodwater or stream flow collection. It is commonly practised in the rural areas in Nigeria. As the easiest sources of collecting water, it requires minimal treatment. This method of water collection is most effective where rain falls on an average of 8 - 10 months in a year, or where rainfall intensity is high.

It is a fact that on average, river flows together with the annual turnover of ground water account for less than 40% of the rain and snow which fall on the world's land surfaces. This shows that the quantity of water which can be effectively collected from this resource should not be underestimated.

Productive agriculture extended much further into the semi-desert areas of Arabia, Sinai, North Africa and Mexico in earlier civilisation than in most recent times because of the system of rain water harvesting. The most notable evidence of such a technique in ancient civilisation stems from the Negev desert in Southern Israel where rain water provided a livelihood for a considerable population.

The emphasis then, is the collection and conservation of rain water at as early a stage as possible in the hydrological cycle to ensure the best use of rainfall before it has run away into rivers and groundwater or has disappeared as evaporation. Collection centres include:

(a) Artificial surfaces at ground level

(b) Land surfaces with slopes less than 50 / 150.

(c) Roof tops

In the present contemporary society, method (c) of collecting rain water is more popular than (a) or (b). However, use of asbestos roofing sheets in urban areas have disqualified this method. This is because asbestos fibres in water are feared to cause intestinal cancers. Advantages of rainwater harvesting include not only the supply of potable water, but also the reduction in soil erosion in regions where runoff is high and the soil has little cohesive force, and reduction in damages caused by flooding.

2.2 CONJUNCTIVE USE

The various sources of water outlined above may be adequate for the demands of a community or region at certain periods or seasons of the year if utilised individually. On the other hand, the source being used may have originally been adequate for the purpose for which it was tapped, but as a result of the growth of population, industry, commerce, e.t.c., this source has become inadequate. Individually, water sources are important components of the total Water Resources Management problem. So, comprehensive analysis of Regional Water Management began considering the integrated/conjunctive use of as many sources of potable water (Biswas, 1976).

Optimal beneficial use can be obtained by conjunctive use, which involves the co-ordinated and planned operation of both surface and/or ground water and/or any other source to meet prevailing and future demands in a manner whereby water is conserved. It is usually based on maximum utilisation of surface/rain water during periods of above-normal precipitation, and ground water during periods of draught (Todd, 1980).

Several advantages of conjunctive use abound such as, greater water conservation, smaller surface storage, smaller surface distribution, smaller drainage system, greater flood control, smaller evapo-transpiration losses, and reduction in weed seed distribution.

Disadvantages attributed to conjunctive use of water sources are less. They include hydro-electric power, greater power consumption, greater water salination, complex project operation, and danger of land subsidence.

It is therefore wise to incorporate the use of as many sources as possible, so that when one source fails, the other(s) will still have something in hand (Twort et. al., 1987).

2.3 DISTRIBUTION SYSTEMS

Distribution systems can be dated back to the time when men sought for ways to channel water from source to place of demand. The system was then limited to transporting the resource from it's source to a central location within the community where inhabitants will come to fetch (McGhee, 1991). Progressively, as water sourcing methods improved, distribution systems also improved. The Egyptians as far back as 3200 B.C were known to have built intricate water resource networks for municipal use and irrigation. Since then, distribution systems have evolved to include several other components and functions and are now usually connected in limitless number of configurations.

According to Walker (1978) and Walski (1987), distribution systems include a Source(s), central water treatment plant(s), pumping station(s), Reservoir(s), valve(s) and booster pump(s) pressure reducing station(s), piping, chlorination, metering, instrumentation, fire protection, and public relations. Unlike water sources, distribution systems are unique to the community for which it is built. It's configuration is dependent on: (a) Water source, (b) Service area topography and, (c) History of the existing water distribution system(if any). Though mostly buried under the ground, distribution systems form a large part of the total expenditure in sourcing for potable water.

To effectively design distribution systems through knowledge of flow rates and pressures within the pipe system, several advances were made especially within the engineering environment. Isaac Newton's Laws of Motion and Conservation of Mass, Momentum and Energy are the fundamental principles on which these advances are based. An expression of Conservation of Energy frequently used in hydraulics is the Bernoulli equation. It states that "the sum of the Kinetic, Potential and Pressure energy in a flowing fluid at an upstream point X within pipe, is equal to the sum of the Kinetic, Potential and Pressure energy and energy lost through friction at a downstream point Y some distance A away from X". It is written mathematically as;

$$\frac{U_X^2}{2g} + P_X + Z_X = \frac{U_Y^2}{2g} + P_Y + Z_Y + E \quad \text{.........................2..1}$$

where Ux, Uy - velocity of unit mass of water at X and Y.

P_X, P_Y - Pressure at X and Y.

Z_X, Z_Y - Height of X And Y above given datum.

g - Acceleration due to gravity.

E - Energy lost through friction between X and Y.

Though piped distribution of water has been applied for centuries, it is only recently that engineering attention has been turned towards optimisation of entire systems. Several known procedures are available for the solution of the hydraulic state of the distribution network with known pipe dimension and nodal consumption. Such procedures are frequently used to check whether a particular layout and design satisfy engineering requirements (Watanatada, 1973). The flow through a pipe is therefore a function of the Pressure(head) difference between the ends of the pipe and pipe wall characteristics, which is in turn a function of pipe characteristic, diameter and length. Several formulas have been formulated and are used to calculate head losses. They are:

a. The Darcy - Weisbach formula

Headloss, $$h_f = \lambda \frac{L}{D} \cdot \frac{V^2}{2g} \quad \text{.............................2.2}$$

λ - friction factor, L - Length, D - Diameter, V - Velocity and

g - Acceleration due to gravity .

b. The Hazen - William's formula

Headloss, $$h_f = \frac{10.70 . L}{C_{HW}^{1.852} . D^{4.87}} Q^{1.852} \quad \text{........................2.3}$$

C_{HW} - Hazen William's coefficient, Q - Flow and L & D as denoted above.

c. The Manning's formula

Headloss, $$h_f = \frac{m^2}{0.397^2} \cdot \frac{L.V^2}{D^{\frac{4}{3}}} \quad \text{..........................2.4}$$

Where m = Manning's coefficient and L,V&D as denoted in equation 2.2.

Procedures used to solve flow distribution in pipe networks include:

i. The Newton - Raphson method

ii. The Linear Theory method

iii. The Hardy Cross method

Capital and operational costs associated with the design of the distribution system are normally not included in these calculations. Recently, several models have been proposed in order to provide design engineers with ready tools for economic and technical evaluation of large network problems while still satisfying engineering requirements. These models are highly valuable because water supply and distribution systems require massive investments in pipelines and installations and so, operational and installation costs may soar higher if networks are ill-designed. The advancement of Operations Research and computation techniques provided an efficient way of helping the designer solve large scale supply and distribution problems with the least cost (Rasmusen, 1976). These models solve the hydraulics of large distribution and supply networks while optimising installation and operational costs. Several literature which present such models include:

Optimisation of Gravity-Fed Water Distribution systems : Application (Bhave, 1983);

Simplified Optimisation of Water Supply Schemes (Rasmusen, 1976);

Cost and Pricing Relationships in Water Supply (Clark, 1976);

Least Cost Design of Water Distribution Systems (Watanatada, 1973);

Reliability Based Optimisation Model for Water Distribution Systems (Su, et.al., 1987); and

Optimal Expansion of Water Distribution Systems (Bhave, 1984).

2.4 WATER RESOURCES MANAGEMENT

Water Resources Management was therefore born out of the need to effectively utilise these supply and distribution systems built and to improve on the resource's quality and quantity. Other earlier known civilisations in the valley of the Tigris and Euphrates(Ancient Mesopotamia), Indus(India), and Yellow rivers(Huang-Ho, China) had water resources management playing a central role in their development (Cox, 1988).

In recent times however, broader definitions have been given to Water Resources Management which include: Water supply; treatment and distribution, waste water collection; treatment and disposal, drainage and flood management, and perhaps recreation and aesthetics (McPherson, 1970); Decisions about how and to whom water uses are allocated as well as the technical and economic aspects of physical water management (Crigg, 1973); Relationships between water, the organisms that live within and around, and it's immediate environment (Biswas, 1976); Development, control, treatment, utilisation, allocation and reuse (Sonuga, 1984); and the continuous use of available water, surface and underground conjunctively or separately in order to meet various demands, implying uninterrupted supply of water for whatever need viz.: domestic supply, effluent disposal, industry, agriculture, navigation, power or recreation (Offodile, 1988).

These definitions, because of their increasingly complex nature, have greatly contributed to two new developments in Water Resources Management namely; The application of Systems Analysis techniques, and Interdisciplinary involvement, that is, the direct participation of specialists in the fields of Ecology, Biology, Chemistry, Sociology, Law, Mathematics, Geography, Political Science, e.t.c. (Biswas, 1976)

2.5 SYSTEMS ANALYSIS

Systems analysis is of recent origin and was virtually non-existent until the second world war when complex problems arose as a result of the deployment of large armies, navies and airforces on a world-wide scale. There was need for an overall plan of supplying them, controlling and co-ordinating their tactical and strategic operations. The 'Rule of Thumb' was not adequate to handle the complex situation. As a result, new mathematical based planning and forecasting methods were proposed (Templeman, 1982). Selected mathematical methods were chosen out of those proposed and used to analyse the overall operation and then used to predict the performance of the system. From the many alternatives, the solution which best satisfied the needs of the war was chosen.

This analytical study that helps decision makers to identify and select a preferred course of action among several feasible alternatives is called 'Systems Analysis'. It basically has to do with finding the right scientific method or technique which will best model a real - world situation or system, with the objective of experimenting with the model to gain an insight into the real - world situation (Biswas, 1976).

Systems analysis approach to water resources management is therefore an attempt to look at the problems of the water resource system on an interdisciplinary basis by systems analysis techniques. In summary, systems analysis answers the following question - "which of the infinite variety and variations of possible water resources systems and policies should be implemented ?" (Loucks et. al., 1981).

Broadly speaking, the analysis of water resource systems involves the following stages:

(i) Identification and explicit statement of objectives;
(ii) Translation of objectives into measurable criteria;
(iii) Identification of alternative courses of action which will satisfy the criteria;
(iv) Determination of the consequences that follow from each alternative; and
(v) Comparative evaluation of the consequences of the alternatives in terms of certain criteria.

(Biswas, 1976)

The comprehensive nature of each of these stages mentioned above would be dependent upon:

(i) The objectives of the designer;
(ii) The data available to evaluate the model;
(iii) The time, money, data and computational facilities available for the analysis; and
(iv) The designer's knowledge and skill. (Loucks et. al., 1981)

CHAPTER 3
DESCRIPTION OF STUDY AREA

3.1 AKURE CITY

Ondo state was created on the 3rd of February, 1976 and carved out of the former Western state of Nigeria. Akure the capital city, is located around Latitude 7°15' North and Longitude 5°10' East. It has an area of approximately 23.02 square kilometres and is rapidly expanding. Population within Akure Local Government area is estimated at 316,925 according to the 1993 provisional census. It is therefore one of the fast growing urban settlements in south-western Nigeria with influx of peoples of diverse occupations from many parts of the country.

Akure city is situated in the humid tropical region of Nigeria where daily temperatures within the city range between 20 - 35°c during the year. Mean annual rainfall is measured at over 1500mm Depending on the season, humidity ranges between 50 - 90% with an estimated annual mean maximum humidity of 71.85%.. Akure experiences the south westerly winds which are moisture laden warm winds from the Atlantic ocean from March to October bringing the rainy season, and the north-east continental dry and cold winds from the Sahara between November and February bringing the dry season.

3.2 IJAPO ESTATE

Ijapo estate is one of the oldest estates inherited by the state capital, Akure from the former Western state of Nigeria. It is well planned and probably the best planned residential estate within Akure city. It is situated at the north-east section of the city. Patterned after the old Bodija estate, Ibadan, Oyo state, Ijapo has an estimated areal extent of 1.17 square kilometres of which about 70% is allocated to residential developments. The remaining 30% is allocated to related service oriented facilities such as petrol station, police station, recreation center, primary and high school, clinics, e.t.c. Area composition of the different facilities within the estate is shown in Table 3.1. Politically, the estate as part of Akure city falls under the administration of the Akure Local Government although the residential plots are privately owned.

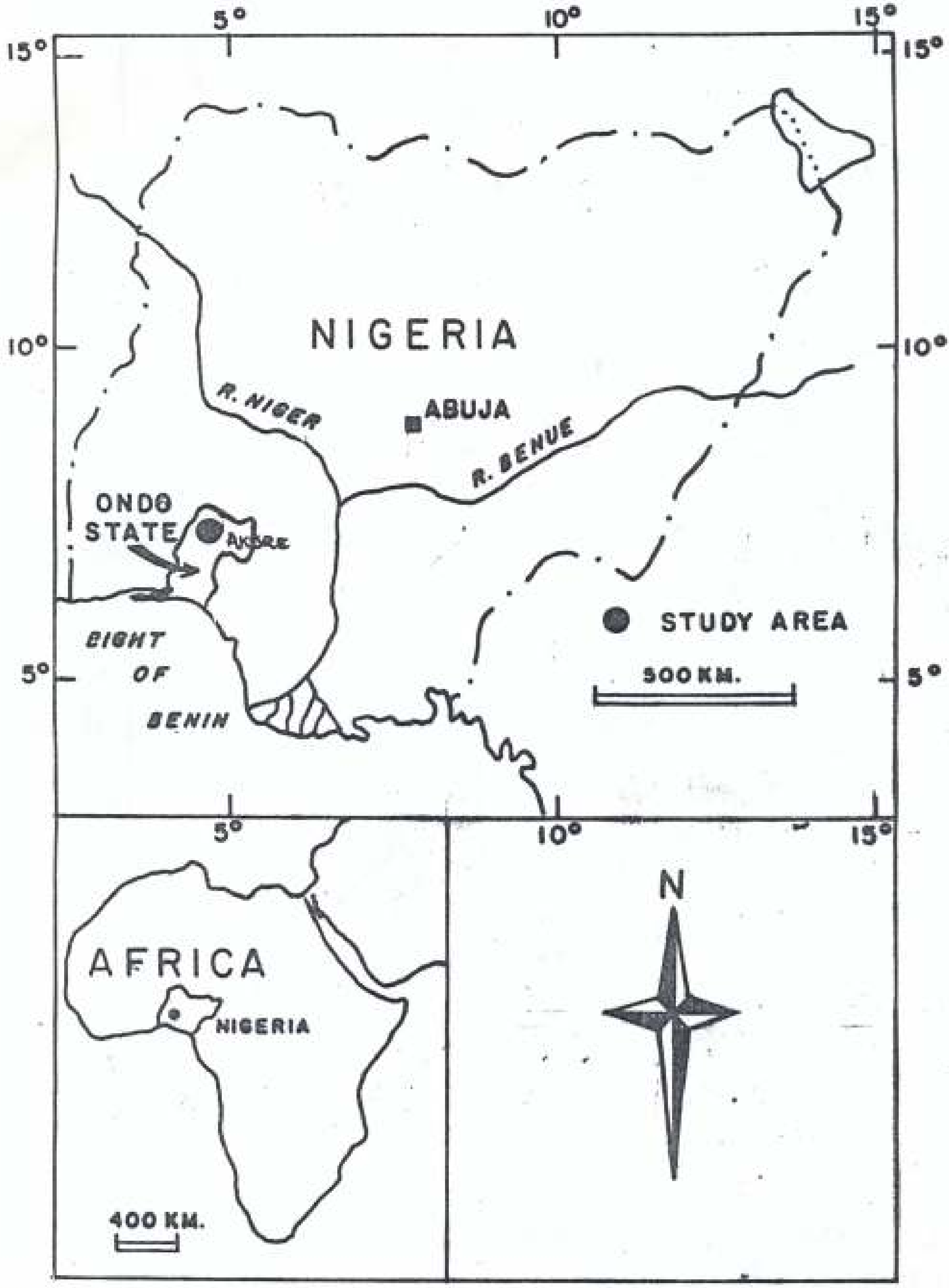


FIGURE 3-1; LOCATION MAP OF STUDY AREA.

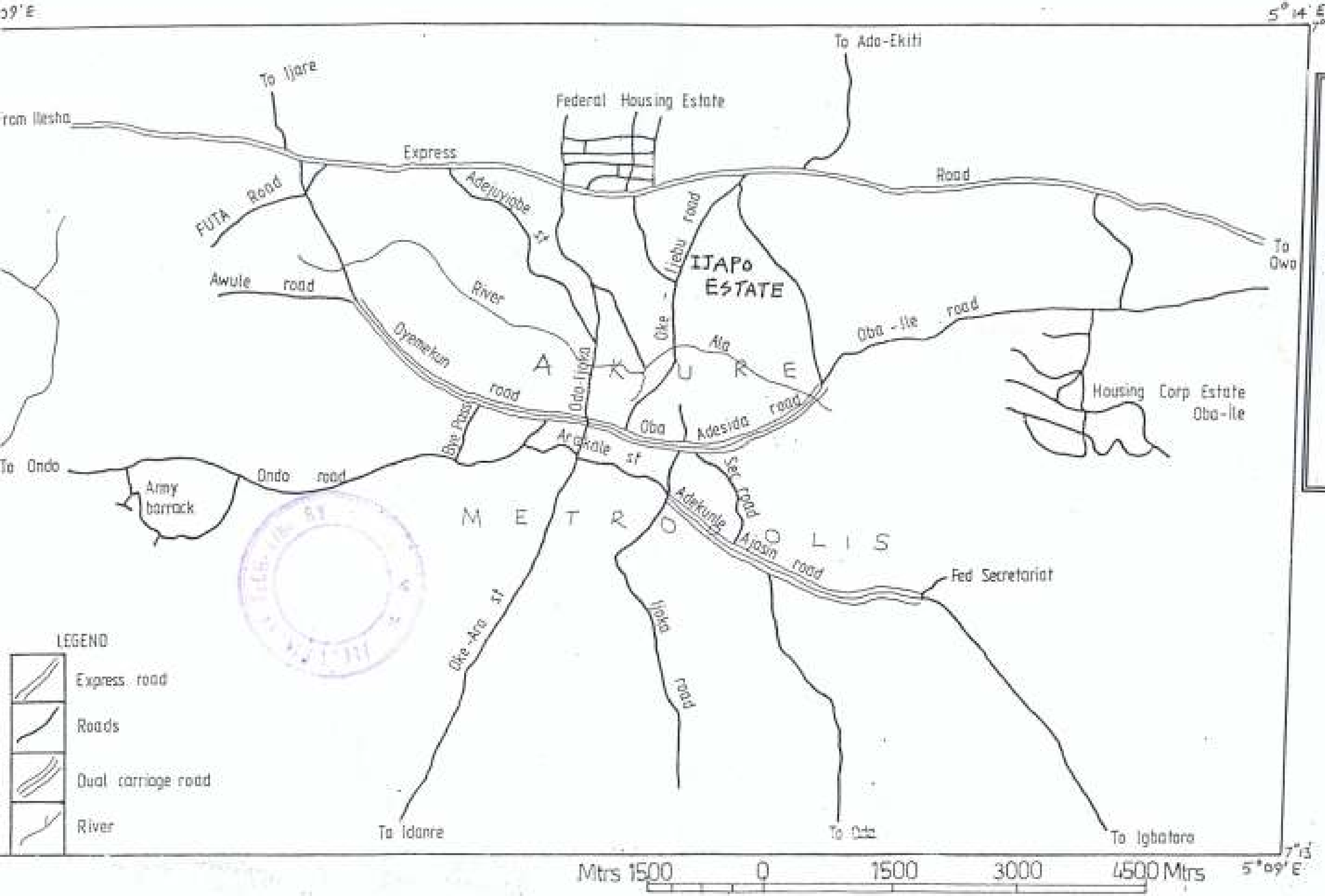


Fig. 3.2 : Street guide map of Akure city. (Ref: Akintunde, 1997)

Fig. 3.3 : Layout plan of Ijapo estate, Akure city.

Table 3.1

Area composition of Ijapo estate (Akure city, Ondo State)

S/No	Description	Units	Area of unit(sq. m.)	Total Area (sq. Km.)	% Area
1	Low density plots	68	36 x 40	0.136	11.7
2	Medium density plots	322	30 x 36	0.486	41.5
3	High density plots	233	22 x 30	0.214	18.3
4	Roads	-	-	0.051	4.4
5	Watershed and Stream area	-	-	0.042	3.6
6	Ijapo Primary School Ijapo High School	1 1	22400 31300	0.0750	6.4
7	Recreation center	1	-	0.0597	5.12
8	Utilities: Clinic(2), Petrol station(1), Shopping center(1), & Police station(1)	-	-	0.065	5.6
9	Unusable areas	-	-	0.040	3.4
			TOTAL	1.17	100

3.2.1 Population

There are 46 blocks and 623 residential plots within Ijapo estate. Presently, about 85% of these plots have been fully developed and accommodate a high percentage of the middle and high income class of the society. About 17 of the 623 plots meant for residential purposes have been converted for other purposes viz. public and commercial purposes. The estate's present residential population is estimated at 6,060, with an average of 10 people per plot. Using the National Growth Rate factor of 2.5%, Steel and McGhee (1979) proposes an Arithmetic model which is used to project the estate's population for the next 25 years. The model is written as;

$$P_n = P_p(1 + x)^n \quad \text{.................................3.1}$$

where P_n = Projected population in n years

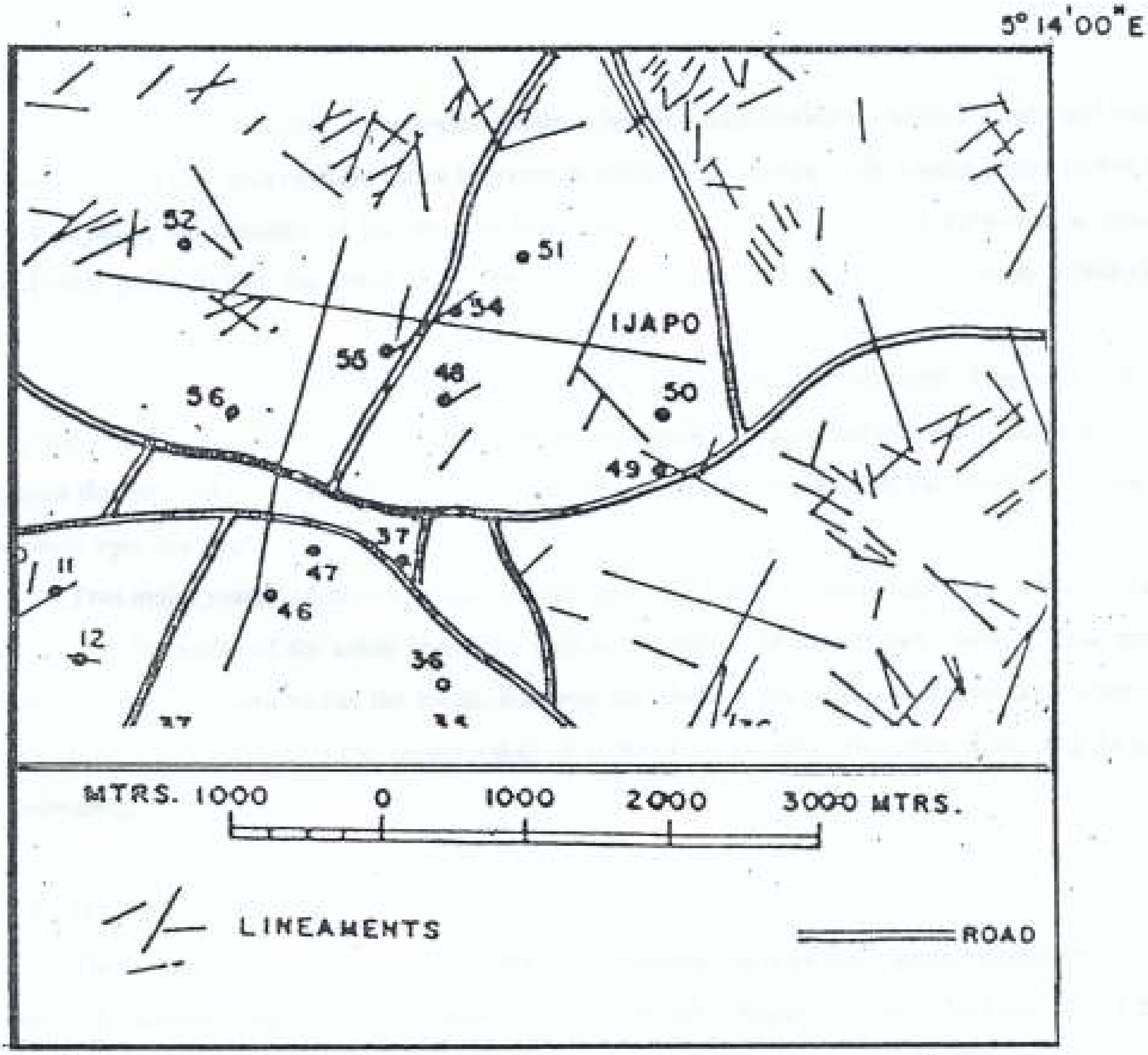


Fig. 3.4 : Lineament plan of Ijapo estate.

(ref: Owoyemi, 1996)

P_p = Present Population, n = Projected period, and x = Growth Rate

Therefore, $P_{25} = 6{,}060\,(1 + 0.25)^{25} = 11{,}235$

From this model, there will be an average of 19 people per plot within the estate in 25 years. Total population is estimated at 11,235.

3.2.2 Geology

Rock types found within the estate include porphyritic granite in the east of the estate, granite gneiss in the center and charnockites in the west. At certain locations, pegmatite gneiss occurs as outcrops from veins within the earth's surface.

3.2.3 Topography

Akure in general is situated within a lowland surrounded by towering hills and rocky mountains. The land area of Ijapo estate is generally sloppy and uneven . The lowest points within the estate lie along the direction of the streams flow. The vertical height difference between the highest and lowest points within the estate from the spot levels taken, is approximately twenty - two (22) metres.

Osisi stream, a tributary of Ala river, divides the estate into two sections. This stream flows from the northern section of the estate to join at the center, a major natural storm water drainage channel flowing from the north-west section. The stream then flows through the southern section of the estate into Ala river.

Two major roads (Moses Orimolade-Ondo street and Henry Fajemirokun street-Owo Avenue) run through the width of the estate from one gate to the other. All other roads connect these major roads. The road network within the estate, accounts for about 4.4% of the total land area. Over the years, these roads have become unmotorable as a result of erosion, excessive wear, and lack of maintenance.

3.2.4 Hydrogeology

The estate is underlain by the crystalline pre-Cambrian rocks of the Nigerian basement complex. Lineament maps of the area show that the rocks are compact although weathered above the ground surface. A lineament map compiled and drawn by Owoyemi(1996) shows three major fractures within the estate. Comparison between this map and the layout plan seems to indicate that the Osisi stream is structurally controlled. This is evident in the direction of flow and path along which the stream flows.

The water table is found at depths of about 2.4m at high elevation points and 0.91 - 1.52m at valley sections of the estate,

3.2.5 Water supply Sources

Apart from the two imported sources of water identified in Section 1.0.i and 1.0.ii, which are the Owena Water Supply scheme and the College of Agriculture Mini - Water Supply scheme, there are also other potential sources of water which can be tapped and utilised either separately or in conjunction with those above. They are as follows;

i. Groundwater Supply:

Several boreholes are being utilised extensively by individuals and corporate bodies within the estate. Of particular note are certain boreholes (precisely two) within the range of 0 - 100 metres from the Osisi stream. When drilled to a depth of about 60m, good quality water at very high pressures were found. In fact, one of these boreholes located at block XXXIV, plot 451 at the north-east section of the estate was found to be artesian in nature and flows out of the ground under very high pressures. The other borehole located at block I, plot 19 at the south-east section of the estate supplies water to the public unceaselessly as long as there are no power cuts.

Owoyemi's work on Akure Metropolis proposes that two major fractures occur within the bedrock in Ijapo estate (Fig. 4). This fractures have a meeting point somewhere within the area enclosed by the Shopping Center. The two boreholes mentioned above are located about 50 - 100 meters away from these fractures and therefore may be a confirmation of Owoyemi's survey study . (Owoyemi, 1996). Suggestions have been made to imply that the Osisi stream is structurally controlled by one of the major fractures running from the northern part of the estate.

ii. Surface Water:

The Osisi stream runs through the estate and is a potential source of potable water if it can be ponded and treated before distribution. Because of it's source (Ala river) and it's small nature, the stream is perennial. Quite a number of farmers fish along the stream and channel it's waters for irrigation. Because of it's proximity to both residential and public buildings, it is always subject to pollution.

iii. Rainwater Harvesting:

This is a less expensive venture provided the facilities needed for collection are on ground e.g. central drainage system, storage facilities, treatment plants and a large collection center.

Presently, there is none of the above facilities on ground within the estate. It is therefore an expensive venture in terms of instalment and maintenance, although quite a large quantity of water may be collected. It would therefore be most effective when implemented in homes.

CHAPTER 4
ANALYSIS

4.1 Existing Water Supply and Distribution System

As previously mentioned, Ijapo estate was divided into three(3) separate independent distribution networks to allow effective water flow to the different sections of the estate. Two main pipes of 250mm diameter supply these networks with water, one main pipe running along each of the two main roads bordering the estate at the east and west. The pipe running along the eastern section of the estate, supplies water to two of the distribution networks which have their source points along the Oba Adesida/Ado-Ekiti road (Fig. 4.1). This supply pipe runs from the elevated tank at Alagbaka estate through two booster stations; one along Oba Adesida road and the other just before the estate, along the Oba Adesida road/Ado-Ekiti road. The other main pipe runs from the elevated tank at Alagbaka estate through the Oba Adesida road to Oke-Ijebu road which borders the estate at the east. One source of water is the Owena Water Supply scheme through the city's overhead tank while the other source is the College of Agriculture mini-water supply scheme through it's overhead tank. The water from the College of Agriculture mini-water supply scheme flows to Ijapo estate through the pipe mains running along the Oba Adesida/Ado--Ekiti road.

Present demands for water are summarised in the following tables for a residential population of 6,060 people, for public and commercial purposes. Wright (1977) and Steel and McGhee (1979) put domestic demands between 75 - 380 litres per capita per day and 112 - 177 litres per capita per day respectively, depending on the locality, season, available water supply sources and several other factors, while Twort et. al. (1987) puts commercial and public demand at 330 litres per square meter per day and 40 litres per square meter per day respectively.

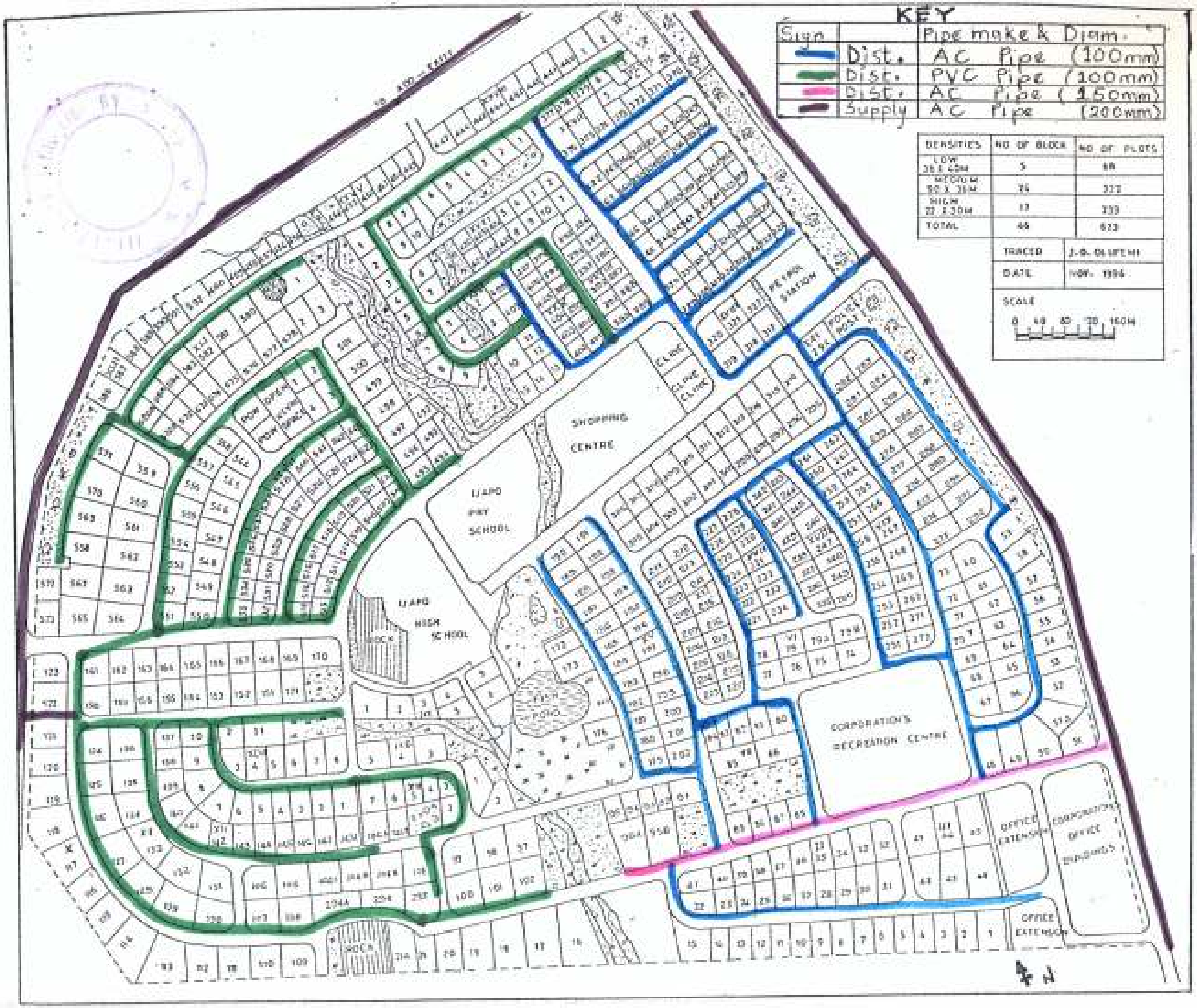


Fig. 4.1 : Present distribution networks within Ijapo estate.

Table 4.1

Estimates of Present Domestic demand(per capita consumption)*

S/No.	Description	Demand (litres)*
1.	Toilet flushing	27
2.	Kitchen use (cooking, drinking, washing, miscellaneous)	25
3.	Bathroom use (bathing, washing, etc.)	15
4.	Laundry	5
5.	Car washing, plant watering, etc.	5
Consumption per capita per day in litres		77
Peak factor		**1.50**
Consumption per capita at peak period		**115.50lcd**
Therefore present domestic consumption for 6,060 people is estimated at 699.93 cubic metres.		

*(Ref : Wright (1977) & Steel and McGhee (1979))

Table 4.2

Commercial demands for facilities provided for in the Existing layout*

S/No.	Facility	Area(square metres)	Demand (cubic metres/day)*
1.	Ijapo shopping center	600.00	198.00
2.	AP petrol station	150.00	49.50
3.	Corporation's recreation center	250.00	82.50

*A standard demand of 330 litres/square metres/day is used to estimate total demand

(Ref : Twort et. al. (1987))

Table 4.3

Commercial demands for facilities not provided for in the Existing layout*

S/No.	Facility	Area(Square metres)	Demand (cubic metres/d)*
1.	Owena Bank training school	200.00	66.00
2.	Modulore guest house	250.00	82.50
3.	Bolington guest house	250.00	82.50
4.	Cosmic guest House	250.00	82.50
5.	Network hospital	350.00	115.50
6.	Royal Birds motel	300.00	99.00
7.	Joe-Jane hospital	500.00	165.00
Total Commercial Consumption in cubic metres per day			**1023.00**
Peak factor			**1.50**
Consumption at peak period in cubic metres per day			**1534.50**

*A standard demand of 330 litres/square metres/day is used to estimate total demand (Ref : Twort et. al. (1987))

Table 4.4

Public demands for facilities provided for in the Existing Layout*

S/No.	Facility	Area (square metres)	Demand(cubic metres/day)*
1.	Ijapo estate primary school	600.00	24.00
2.	Ijapo estate high school	600.00	24.00
3.	Ijapo estate police station	100.00	4.00
4.	Housing Corporation offices	1000.00	40.00

*A standard demand of 40 litres per square metres per day is used (Ref : Twort et. al. (1987))

Table 4.5

Public demands for facilities not provided for in the Existing Layout*

S/No.	Facility	Area (square metres)	Demand (cubic metre/d)*
1.	Cabataf nursery and primary school	500.00	20.00
2.	Mount Olives nursery and primary school	250.00	10.00
3.	The Redeemed Church of God	450.00	18.00
4.	Triumphant Christian Assembly	300.00	12.00
5.	Spring of life Bible Church	300.00	12.00
6.	Reconciliation House	300.00	12.00
7.	St. Andrews Anglican Church	350.00	14.00
8.	St. John Boscoe Catholic Church	350.00	14.00
9.	Grail Center	400.00	16.00
Total Public Consumption in cubic meters per day			**220.00**
Peak factor			**1.50**
Consumption at peak period in cubic metres per day			**330.00**

*A standard demand of 40 litres per square metres per day is used (Ref : Twort et. al. (1987))

Total Present demands are:

	Type of Demand	**Volume (m^3/day)**
1.	**Domestic demand:**	**699.93**
2.	**Commercial demand:**	**1534.5**
3.	**Public demand:**	**330.00**

Total estate demand = 2563.80

There is therefore a present estimated demand of approximately 2600.00 cubic metres of water per day or 0.03000 cubic metres per second within Ijapo estate.

4.2 Projected Water Demands

Projected demands for water within the estate for a residential population of 11,235 people, public and commercial purposes are summarised in the following tables:

Table 4.6

Proposed future Domestic demand(per capita consumption)

S/No.	Description	Demand (litres)
1	Toilet flushing	27.00
2	Kitchen use (cooking, drinking, washing, miscellaneous)	33.00
3	Bathroom use (bathing, washing, etc.)	20.00
4	Laundry	10.00
5	Car washing, plant watering, etc.	10.00
Consumption per capita per day in litres		**100.00**
Peak factor		**1.50**
Consumption per capita at peak period		**150.00lcd**
Therefore future domestic consumption for 11,235 people is estimated at 1685.25 cubic metres		

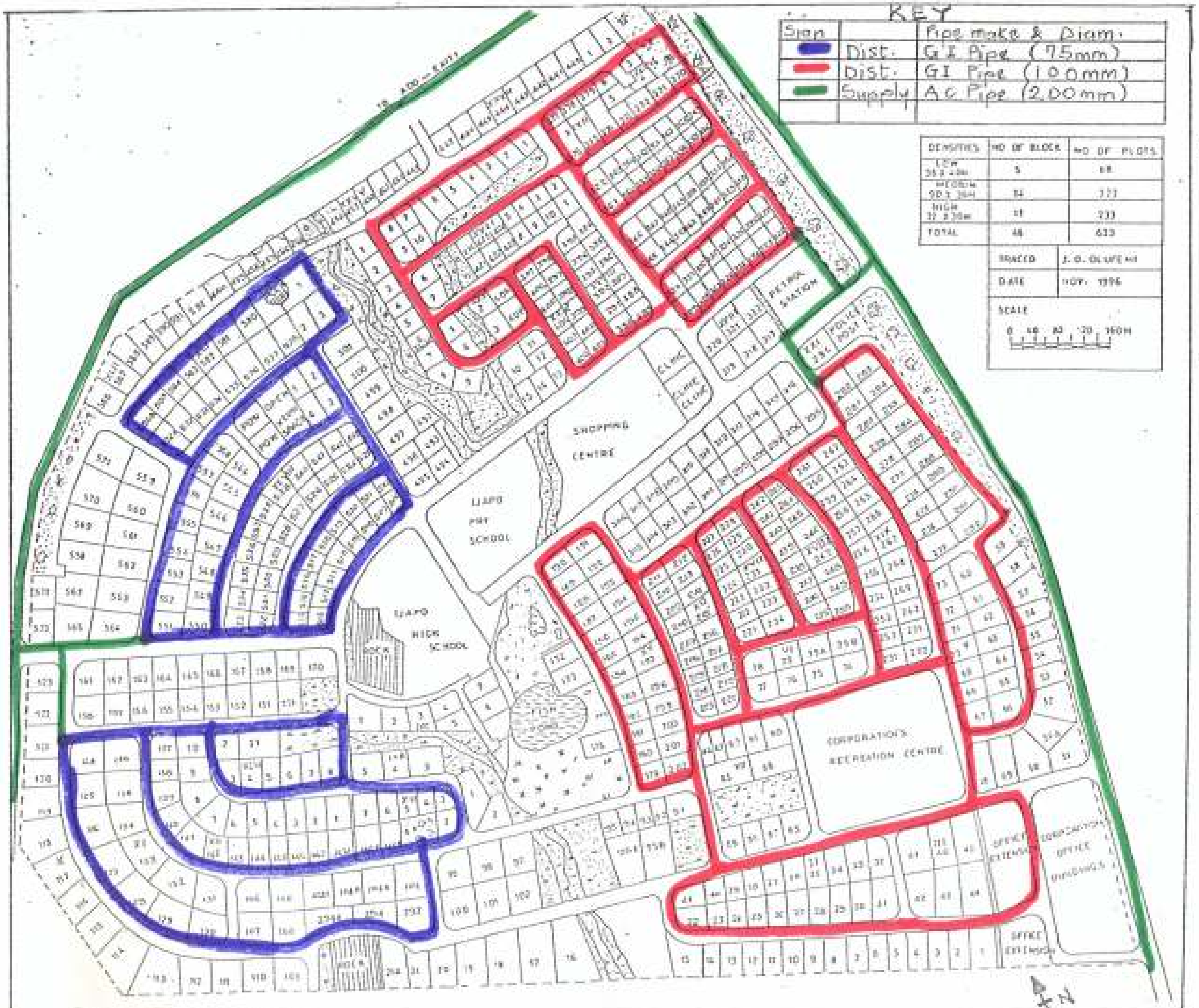


Fig. 4.2: Proposed distribution networks for Ijapo estate.

Table 4.7

Commercial demands for existing and future developments*

S/No.	Facility	Area(square metres)	Demand (cubic metres/day)*
1.	Ijapo shopping center	1800.00	684.00
2	AP petrol station	300.00	114.00
3	Corporation's recreation center	750.00	285.00
4	Owena Bank training school	400.00	152.00
5	Modulore guest house	450.00	171.00
6	Bolington guest house	400.00	152.00
7	Cosmic guest House	400.00	152.00
8	Network hospital	700.00	266.00
9	Royal Birds motel	600.00	228.00
10	Joe-Jane hospital	900.00	342.00
11	Two Clinics(future development	600.00	228.00
Total Commercial Consumption in cubic metres per day			**2774.00**
Peak factor			**1.50**
Consumption at peak period in cubic metres per day			**4161.00**

*A proposed standard demand of 380 litres per square metres per day is used to estimate commercial demand

Table 4.8

Public demands for existing facilities*

S/No.	Facility	Area (square metres)	Demand (cubic metres/day)*
1.	Ijapo estate primary school	1200.00	108.00
2.	Ijapo estate high school	1200.00	108.00
3.	Ijapo estate police station	300.00	27.00
4.	Housing Corporation offices	2000.00	180.00
5.	Cabataf nursery and primary school	850.00	76.50
6.	Mount Olives nursery and primary school	500.00	45.00
7.	The Redeemed Church of God	900.00	81.00
8.	Triumphant Christian Assembly	450.00	40.50
9.	Spring of life Bible Church	450.00	40.50
10.	Reconciliation House	600.00	54.00
11.	St. Andrews Anglican Church	700.00	63.00
12.	St. John Boscoe Catholic Church	700.00	63.00
13.	Grail Center	650.00	58.50
Total Public Consumption in cubic meters per day			**945.00**
Peak factor			**1.50**
Consumption at peak period in cubic metres per day			**1417.50**

*A proposed standard demand of 90 litres per square metres per day is used to estimate public demand

Total Projected demands are:

	Type of Demand	Volume (m^3/day)
1.	**Domestic demand:**	**1685.25**
2.	**Commercial demand:**	**4161.00**
3.	**Public demand:**	**1417.50**

Total estate demand = 7263.75

There is therefore a projected demand of 7300 cubic metres of water per day within Ijapo estate in the next 25 years. That will bring water demand to 0.08500 cubic metres per second.

4.3 DESIGN CHECK

The existing distribution system within the estate was analysed to check whether the system on ground satisfies all engineering requirements and hydraulic flow conditions, and to allow for an effective comparison between the existing and proposed distribution systems. The diameters of pipes presently on ground, required quantities of water expected for consumption, and present pipe costs were used for this design check. The Hazen - William's equation for headloss was utilised in the manual calculations.

The results of the design check, and other factors affecting the workability of the existing system include the following:

(i) Incessant power cuts within the city and the estate's environs frustrate a consistent pumping pattern of water into the storage tanks from the different sources and water pressure boosting at the booster stations.

(ii) Minimum service pressure heads for the various consumption nodes/points within the estate are at most times not attained because of the low input pressures experienced at the source node. This low input pressure at source node is primarily because of the long distance between the estate and the storage tank at Alagbaka estate, the uneven topography within the city, and the numerous other consumers who also collect water from the same pipe mains running from the storage.

(iii) There is presently no water flow to the entire north - west section of the estate because of the total collapse of the PVC pipes laid along Ondo street.

(iv) When water is released from both storage systems, actual quantities of water supplied to the estate cannot be verified. The schedules of releasing water to the estate are highly erratic and so cannot be depended upon.

(v) Water demands calculated for the estate in Tables 4.1, 4.2, 4.3, 4.4 & 4.5, show that estimated present water supply, when possible are inadequate and will not meet the needs of the residents.

(vi) Several of the plots within the estate are now being used for purposes for which they were not originally planned and designed for. In most cases where this is so, water demands are presently higher than the original estimate.

(vii) Several AC and PVC pipes laid within the estate have been either broken or dug up by the residents. This results in large water losses during water flow.

(viii) Several residents within the estate have constructed either wells or boreholes to cater for their individual or commercial needs.

(ix) Excessive leakage have been noticed within the individual distribution systems when water flows as a result of the large number of dead ends which through excessive pressures over the years have worn out.

4.4 OPTIMAL DESIGN

The optimal model proposed modifies the model proposed by Rasmusen (1976) for multiple or single source, branched or looped water supply systems to a model which optimises single (conjunctive) source, looped, gravity - fed water distribution systems. This modified model uses the optimisation principles employed in the Rasmusen's model. The model is decomposed into two sub - problems; The Hydraulic Network Flow problem and the Diameter Modification Algorithm. The Network Flow problem employs the Hardy Cross iterative method for balancing pipe flows and headlosses around a loop while the Diameter Modification Algorithm employs Rasmusen's optimisation principle of sequentially adjusting pipe diameters either one diameter value upwards, downwards, or leaving it unchanged. This choice of modification is dependent upon the ratio between the prevalent marginal energy cost and the pipe investment cost.

Optimisation is then achieved by varying the combination of both the pipe diameter and pressure head (the height of the overhead tank) at the source node, while at the same time accommodating the minimum service pressure requirements of all the consumption nodes within the distribution network at the least possible cost. Because of the tight interdependencies between flows within a network, an optimal size of a pipeline cannot be calculated individually. It must be found as optimal in relation to other pipes within the network.

Although this model is of a Mixed Integer Linear Programming (MILP) sort, it is best solved by a heuristic engineering problem solving technique other than a standard MILP technique. This is because a standard MILP technique cannot handle non - linear constraints, and there is the need to exercise practical engineering judgement during the analysis of the model, other than a straight optimisation process which cannot take into consideration some peculiarities of the distribution system.

A distribution network for the estate is proposed for a service time of 25 years. The network utilises and modifies two of the three existing layouts, and breaks the third existing layout into two, making the total number of independent networks four. This is done for ease of analysis, simplicity, reduction in pipe costs, effective water flow, independence and to aid operation and maintenance. The four networks proposed involve none of the pipes of the existing networks but the introduction of new pipes of different diameters where applicable. As much as is possible, dead ends are avoided to reduce possible leakage and discourage stagnant bodies of water within the system.

The sequential steps taken to optimise the distribution system are as follows;

(1) Calculate the hydraulic state of the network.

(2) Locate the critical node.

(3) Search for all the critical links and choose the most appropriate.

(4) Compute the Actual Service Pressure heads.

(5) Calculate the total costs.

(6) Calculate marginal energy and pipe costs.

(7) Modify all pipes sequentially.

(8) Repeat steps (1) to (7) until no further cost improvement is found possible.

The details of the above steps are provided in the sub-section below.

4.4.1. ***Calculation of the hydraulic state of the network***

After the proposed network layout has been decided upon, pipe dimensions are chosen for each of the pipes within the network and would be dependent on the designer's specifications. Pipe lengths would be known from the proposed layout. Water demands can be deduced from the targeted consumers and summed up to obtain the total demand of water within the distribution system.

Flow distributions are assumed for each pipe within the network subject to Kirchoff's first which state's that " the total quantity of water leaving a node must total that entering it". This implies that the total amount of water stored at a node within a distribution system must be equal to zero. This law is stated mathematically as;

$$\Sigma Q_{In} = \Sigma Q_{Leaving} + Q_{Consumption} \quad \text{.......................} 4.1$$

$$\text{where } \Sigma Q_{storage} = 0$$

Where Q_{In} = Quantity of water entering the node, $Q_{Leaving}$ = Quantity of water leaving the node,

$Q_{Consumption}$ = Quantity of water consumed , and $Q_{Storage}$ = Quantity of water stored at the node.

The flow of water through any pipe X is a function of the pressure difference between i and j, the diameter of that pipe, and the coefficient of pipe resistance.

The Hazen - Williams's formula is used in this model to calculate head losses within pipes. The Hazen - William's coefficient of pipe resistance, C_{HW} has several values depending on the type of pipe chosen within the network. The Hazen - William's equation for flow and headloss are stated mathematically as;

Flow,

$$Q = 0.849 C_{HW} (\frac{\Pi D^2}{4})(\frac{D}{4})^{0.63}(\frac{H_i - H_j}{L}) \quad ....4.2$$

Headloss,

$$H_F = \frac{10.7 L Q^{1.852}}{C_{HW}^{1.852} D^{4.87}} \quad ....4.3$$

Where C_{HW} = Hazen William's coefficient, D = Diameter of pipe, H_i, H_j = Pressure Head at node i, j,

Q = Flow, H_F = Headloss within pipe, and L = Length of pipe.

Diameter, D_X of any pipe X used must be equal or greater than the minimum required size, D_{Min} recommended for such a distribution system. That is,

$$D_X \geq D_{Min} \quad ....4.4$$

The Hardy Cross method is then used after calculating pipe headlosses, to iteratively correct pipe flows within the loops chosen by the designer. The correction of pipe flows are subject to Kirchoff's second law, which states that "the sum of headlosses around a loop must be equal to zero". It is written as;

$$\Sigma H_F = 0 \quad ....4.5$$

Where ΣH_F = Sum of Headlosses around a loop

Practically, a value of up to 0.5 is still considered negligible. Values higher than this warrant that the entire Hardy Cross process of pipe correction be carried out again from the first to last loop within the distribution network using the already corrected pipe flows. This process is repeated over and over again until headlosses within each pipe are negligible.

4.4.2. *Location of the Critical node*

A Critical node Cn is chosen for the network after the hydraulic flow conditions have been satisfied. The choice of which node should be chosen is dependent on certain conditions as stated below;

i. The physical elevation of the nodes;

From the spot levels taken on ground, the node with the lowest elevation within the distribution network is the best choice for Critical node. This node does not necessarily have to be the node farthest from the Source node.

ii. The network layout(showing flow patterns);

Peradventure there are two or more nodes within the distribution network with the same elevations, the flow patterns within the network can decide on which node should be chosen. A careful study of the flow patterns may indicate that one particular node at the lowest elevation is located at a point within the network where it is the recipient of the last drop of water when water flow ceases. This node is considered the best choice amongst the several alternatives which may exist.

iii. The Service Pressure head requirements;

This condition cannot be isolated from i and ii above and should be considered if i and ii are not sufficient enough to choose a critical node. The node amongst the possible number, with the lowest numerical minimum service pressure head is most suitable. The critical node is assigned the value of it's minimum service pressure head. That is, for the Critical node,

$$h_{ASP} = h_{MSP} \quad \text{.......................} 4.6$$

Where h_{ASP} = Actual Service Pressure Head , and h_{MSP} = Minimum Service Pressure Head. The actual service pressures of the other nodes within the network will then be calculated with reference to the Critical node.

4.4.3. *Searching for all Critical links and choosing the most appropriate*

All paths which show the flow of water from source node to critical node through so many pipes are potential critical paths. The pressure drop along the critical path from source node to critical node will determine the input pressure (actual service pressure head) at source node necessary to maintain the pressure requirements of each node within the network. Actual service pressure head values less than that of the critical node, recorded for other nodes while running the program is an indication that the path chosen is not critical.

The Critical Path Method (CPM) normally used in network analysis is adopted to determine which of the alternative paths are critical. Applying the CPM method to distribution networks show certain basic relationships.

(i) Arrows within the distribution network represent direction of water flow instead of job sequence as in CPM.

(ii) Arcs or straight lines in the distribution network represent individual pipes instead of individual jobs in CPM.

(iii) Nodes within the distribution network represent consumption points instead of specific points in time which mark the completion of one or more jobs in the CPM project.

(iv) Flows within each distribution pipe represent activity durations in CPM.

(v) The process of determining the critical path begins at Source node and ends at Critical node.

(vi) The earliest time of node j, in CPM is represented by the maximum quantity of water, U_j which can be obtained at that node assuming there is no consumption. This means that if there are two pipes S and T supplying node j with various quantities of water, say 0.005 and 0.008 m^3/s respectively, pipe T with 0.008 m^3/s will be chosen. This computation is carried out from Source node to Critical node and is denoted as:

$$U_j = \text{Maximum}\,(U_i + Q_{ij}) \quad \text{........................4.7}$$

Where U_j = Maximum quantity of water which can be obtained at node j.

U_i = Maximum quantity of water which can be obtained at node i.

Q_{ij} = Quantity of water flowing from node i to node j.

(vii) The latest time of node i in CPM is represented by the minimum quantity of water, V_i which can be obtained at that node assuming there is no consumption of water at the node i. This computation on the other hand is carried out from Critical node to Source node after U_j has been computed. In general,

$$V_i = \text{Minimum}\,(V_j - Q_{ij}) \quad \text{..........................4.8}$$

Where V_j = Minimum quantity of water which can be obtained at node j.

V_i = Minimum quantity of water which can be obtained at node i.

The basic assumption therefore is that the choice of a Critical path in the distribution network is proportional to the quantity of water flow within the pipes. Any path at the end of the computation

U_j and V_i which has a zero float is then chosen as the critical path. This also implies that the surplus pressures, S_P of each pipe on the critical path is equal to zero.

Therefore, for node i, $Float_i = U_j - V_i$ 4.9

and Surplus Pressure, $S_P = h_{ASP} - h_{MSP}$ 4.10

Paths apart from the critical path are labelled non - critical. They feed water to parts of the network where the actual service pressure heads in the nodes exceed the required minimum service pressure heads.

4.4.4. *Computation of Actual Service Pressure heads of all nodes*

Flow through any pipe is a function of the pressure head difference between the ends (node i and j) and the flow resistance, while flow resistance is in turn a function of pipe wall characteristics, pipe diameter, D and pipe length, L. Pressure losses for each pipe in the network are therefore calculated using equation 4.2 and is given as;

$$\Delta H_{ij} = H_i - H_j = \left[\frac{QL^{0.54}}{0.277C_{HW}D^{2.63}}\right]^{1.8519} \quad ....4.11$$

Where ΔH_{ij} = Pressure Head difference between nodes i and j

Since the critical node has been assigned the value of the node's minimum service pressure head, it becomes the reference point for the calculation of all other nodes.

(i) Critical Path

The actual service pressure head for any node i on the critical path adjacent to any node j whose actual service pressure head is known, may be calculated by adding the actual service pressure head of node j to the pressure loss within the pipe X that links the two nodes. This is done for all the nodes on the Critical path in a contrary direction to the flow of water. For the Critical path therefore,

$$h_{ASP(i)} = h_{ASP(j)} + \Delta H_{ij} \quad ....4.12$$

(ii) Non-Critical Path

The nodes on the Critical path are linked through one or more pipes to the nodes on the Non - Critical path. Since actual service pressure heads of nodes on the Critical path have been computed, actual service pressure heads of nodes on the Non - Critical Path can also be computed.

Service Pressure heads of each node j is calculated by subtracting the value of the pressure loss of the linking pipe from node i whose actual service pressure head is known. It is given as;

$$h_{ASP(j)} = h_{ASP(i)} - \Delta H_{ij} \quad ....4.13$$

As previously mentioned, any final values of actual service pressure heads which are less than that of the Critical node indicates that the path chosen is not critical and should be changed to another.

4.4.5. ***Calculation of total costs***

The total cost of a distribution network will be the sum of all pipe and energy costs.

$$\text{Total cost} = \Sigma\text{Energy cost} + \Sigma\text{Pipe cost}$$

Pipe and energy costs are expressed in this model as capitalised present values. Pipe cost per meter, P_c which is a function of pipe diameter, D includes cost of pipe per unit length, installation, location and maintenance. From equation 4.11, pipe diameter may be expressed in terms of difference in pressure head loss, ΔH_{β}.

The total cost of pipes of length, L within a distribution system is given as;

$$\Sigma\text{Pipe cost} = \sum_{p=1}^{P} P_c \, . \, L \quad \text{.............................4.14}$$

Energy cost , E_C is defined as the cost of power needed to elevate one cubic metre ($1m^3$) of water one meter (1m) continuously over the service time. Littleton (1962) derives cost of energy as follows;

Power is given as $= \gamma \, Q_S \, H$ (N.m/s) 4.15

Where $\gamma = \rho g =$ Specific weight of water $= 9.81 \times 10^3 = 9810 N/m^3$

Q_S = water supplied through the source node (m^3/s)

H = total dynamic head of pump (m)

Actual Power consumed is then given as;

$$\text{Power consumed} = \frac{\gamma Q_S H}{e} (W)$$

Where e = Wire to Water efficiency = $e_p \times e_m \cong 75\% = 0.75$

$$\text{Power consumed} = \frac{9.81 x 10^3 Q_S H}{0.750} (W)$$

$$= 13.08 \, Q_S H \, (KW)$$

Cost of Power consumed during service period is given as;

$$E_C = 13.08 Q_S H \times t \, (KW - hr) \quad \text{.............................4.16}$$

Where t = time(hours) of operation during service period

For a network with expected flow, Q_S and head, H_S at source node, total Energy cost for service period is given as;

$$\Sigma\text{Energy cost} = E_C.Q_S.H_S \quad \text{..........................4.17}$$

4.4.6. ***Calculation of marginal energy and pipe costs***

At any particular step of the optimisation, the diameter modifications are limited to one of three options; the pipe diameter may be reduced to the next smaller available diameter, the next larger available diameter or the diameter may be left unchanged. The decision to reduce, increase or keep unchanged the diameter of a particular pipe depends upon the ratio between the prevailing marginal energy and pipe investment costs.

(i) Marginal Energy Costs

As written in equation 4.17, total Energy cost is given as $E_CQ_SH_S$. The energy cost per unit head (Marginal Energy cost, Me_C) will be E_CQ_S. Since there is only one source, Marginal energy will then be constant along the Critical links.

Pressure head drop along pipe X with diameter D is given as ΔH_X^D. Pressure head drop along pipe X for the next larger available diameter D + 1 is ΔH_X^{D+1} and the next smaller available diameter is ΔH_X^{D-1}. An increase of pipe diameter from D to D + 1, will result in a head loss equal to ΔH_X^D - ΔH_X^{D+1} - S_P, whereas a reduction in pipe diameter from D to D - 1 will result in a head loss equal to ΔH_X^{D-1} - ΔH_X^D - S_P. Surplus Pressures, S_P have been calculated in equation 4.10.

The general equation for Marginal Energy cost will be;

- Marginal Energy cost saving for increase in pipe diameter

$$= Me_C(\Delta H_X^D - \Delta H_X^{D+1} - S_P) \quad \text{..........................4.18}$$

- Marginal Energy additional cost for decrease in pipe diameter

$$= Me_C(\Delta H_X^{D-1} - \Delta H_X^D - S_P) \quad \text{............................4.19}$$

The energy cost along pipe X will then be $Me_C.Q.\ \Delta H_X$..............................4.20

(ii) Marginal pipe costs

Pipe costs as computed earlier is given in equation (4.14). Marginal Pipe costs will then be the additional costs or savings arrived at by either increasing or decreasing the diameter of the pipe. In this case, pipe costs increase with increasing diameter because of the linear relationship above. The general equation for Marginal pipe cost for any pipe X will be;

- Additional cost for increase in pipe diameter from D to D + 1 is

$$= L_X(P_C^{D+1} - P_C^{D}) \quad \text{.......................4.21}$$

- Savings in cost for decrease in pipe diameter from D to D - 1 is

$$= L_X(P_C^{D} - P_C^{D-1}) \quad \text{.......................4.22}$$

4.4.7. ***Modification of all pipes sequentially***

For a pipe X, the energy cost is a function of the headloss between the nodes at the ends i and j, ΔH_X (equation 4.20) and pipe cost is a function of diameter, D of pipe (equation 4.11), which in turn is a function of headloss, ΔH_X (equation 4.14). The minimum cost of that pipe X in terms of both energy and pipe costs can be computed by differentiating with respect to the difference in pressure head, ΔH_X. This gives;

$$\frac{d\{Total\cos t(\Delta H_X)\}}{d(\Delta H_X)} = \frac{d\{Energy\cos t(\Delta H_X)\}}{d(\Delta H_X)} + \frac{d\{Pipe\cos t(\Delta H_X)\}}{d(\Delta H_X)} = 0 \quad \text{..........4.23}$$

When plotted as curves, the value of the energy cost E_C is an increasing function of pressure head loss, ΔH_X , while Pipe cost P_C is a decreasing function of pressure head loss, ΔH_X . In other words, an increase in pipe diameter would result in a decrease in both pressure head loss and energy cost. Whereas, a decrease in pipe diameter would result in an increase in both pressure head loss and energy cost.

Applying the principle of Turning points for curves (Stroud 1982);

If $\frac{dy}{dx} = 0$ and $\frac{d^2y}{dx^2} = -ve$ then the function is at it's maximum.

If $\frac{dy}{dx} = 0$ and $\frac{d^2y}{dx^2} = 0$ then the function is at the point of inflexion.

If $\frac{dy}{dx} = 0$ and $\frac{d^2y}{dx^2} = +ve$ then the function is at it's minimum.

The second differential of equation 4.23 produces the marginal energy and pipe costs. Applying this principle for each pipe within the network gives the basis for optimisation. Two tests (First and Second) are used to distinguish the decision process.

For the first test of optimisation, the diameter of a pipe X is reduced from D to D - 1. If the sum of the additional marginal energy cost and savings in marginal pipe cost as a result of reducing that pipe diameter from D to D - 1 gives a negative value, this means that the sum is at it's maximum value and implies that the pipe diameter should be reduced from D to D - 1. If the value is positive, then the second test of optimisation is performed. The mathematical expression below explains the first test of optimisation;

If diameter D is reduced to D - 1 and

$$L_X(P_C^D - P_C^{D-1}) + Me_C(\Delta H_X^{D-1} - \Delta H_X^D - S_P) = -ve \text{ ....4.24}$$

OR

$$L_X(P_C^D - P_C^{D-1}) > Me_C(\Delta H_X^{D-1} - \Delta H_X^D - S_P) \text{ ....4.25}$$

<u>**IMPLICATION:**</u> **Diameter, D is at it's maximum value.**

<u>**RECOMMENDATION:**</u> **Reduce pipe diameter from D to D - 1.**

But, If diameter D is reduced to D - 1 and

$$L_X(P_C^D - P_C^{D-1}) + Me_C(\Delta H_X^{D-1} - \Delta H_X^D - S_P) = +ve \text{ ....4.26}$$

<u>**IMPLICATION:**</u> **Diameter, D is not at it's maximum value.**

<u>**RECOMMENDATION:**</u> **Leave Diameter as D and then perform the second test of optimisation.**

The second test of optimisation involves the increasing of the diameter of pipe X from D to D + 1. If the sum of the savings in marginal energy cost and additional marginal pipe costs as a result of increasing the pipe diameter from D to D + 1 gives a positive value, this means that the pipe diameter, D should be increased to D + 1. If on the other hand, the value is negative, this implies that the pipe is at it's point of inflexion and should be left unchanged. The mathematical equation is given below;

If diameter D is increased to D + 1 and

$$L_X(P_C^{D+1} - P_C^D) + Me_C(\Delta H_X^D - \Delta H_X^{D+1} - S_P) = +ve \quad \text{.............4.27}$$

OR

$$L_X(P_C^{D+1} - P_C^D) < Me_C(\Delta H_X^D - \Delta H_X^{D+1} - S_P) \quad \text{..................4.28}$$

<u>**IMPLICATION:**</u> **Diameter, D is at it's minimum value.**

<u>**RECOMMENDATION:**</u> **Increase pipe diameter from D to D + 1.**

But, If diameter D is increased to D + 1 and

$$L_X(P_C^{D+1} - P_C^D) + Me_C(\Delta H_X^D - \Delta H_X^{D+1} - S_P) = -ve \quad \text{.............4.29}$$

<u>**IMPLICATION:**</u> **Diameter, D is at it's point of inflexion.**

<u>**RECOMMENDATION:**</u> **Diameter remains D.**

4.4.8 *<u>Repetition of section 4.4.1- 4.4.7 until no further cost improvement is found possible</u>*

The process of optimisation requires that the seven steps (1) - (7) on section 4.4 be repeated over and over again until there are no further cost improvements. The designer is then left with several alternatives to choose from which will depend upon the requirements of the client.

4.5 THE OBJECTIVE FUNCTION

Given the summary of steps above, the least cost economic design of the distribution system is given by the variation in both the pipe diameter, D and pressure head, h_{ASP} at the source node which minimises the objective function:

$$Cost(D_p, h_{ASP}) = \sum_{p=1}^{P} (P_C . L) + [Q.h_{ASP}]_S . E_C \text{ ....4.30}$$

Pipe cost, P_C is a function of the diameter of the pipe, which in turn is a function of the headloss, ΔH

The Objective function is solved subject to the following constraints;

(i) Minimum permissible Diameter,

$$D_p \geq D_{Min} \text{ ....4.31}$$

(ii) Supply conditions at node N,

$$Q_{in} = \Sigma Q_{Leaving} + Q_{Consumption} \text{ ....4.32}$$

(iii) Headloss requirements around a loop;

$$H_F = 0 \text{ ....4.33}$$

Where Headloss, H_F is a function of Q and D and is given as;

$$H_F = \frac{10.7 L Q^{1.852}}{C_{HW}^{\ 1.852} D^{4.87}}$$

(iv) Minimum Pressure head requirements at any node N,

$$h_{ASP} \geq h_{MSP} \text{ ....4.34}$$

(v) Flow equation,

$$Q = f(D_P, L, C_{HW}, \Delta H) \text{ ....4.35}$$

Where flow Q is given as

$$Q = 0.849 C_{HW} \left(\frac{\Pi D^2}{4}\right)\left(\frac{D}{4}\right)^{0.63}\left(\frac{H_1 - H_2}{L}\right)$$

CHAPTER 5

DISCUSSION OF RESULTS

5.1 SELECTED PIPE SIZES AND COSTS

The case study considered in this thesis has a total number of 76 pipes in four networks which are independent of one another. This proposed optmial system of four networks of 76 pipes is to replace the existing system of three networks of 64 pipes. The proposed optimal system contains some redundant pipes which serve to improve the reliability of water flow to every node within each network in case one or more pipes which normally carry a given quantity of water for one reason or the other are unable to do so. The redundant pipes then serve as alternatives. The distribution networks are labelled Network A, B, C, and D, and are shown in figures 5.1, 5.2, 5.3, and 5.4. Network A has a total number of 23 pipes, Network B; 29 pipes, Network C; 10 pipes and Network D; 14 pipes.

Commercial pipe diameters used in the model are restricted to certain sizes because of the size of the estate and water requirements . These sizes are 75mm, 100mm, 150mm and 200mm. As a result of the restricted diameter of pipes available in the market, Galvanised iron pipes of 75mm and 100mm diameters and Asbestos cement pipes of 150mm and 200mm are chosen, and to be used either separately or in combination within the four distribution networks.

Cost of electricity from equation 4.16 is calculated as N11.00 per Kw. Therfore, Marginal Energy cost for the period of twenty-five years is One million, five hundred and fourteen thousand naira, based on 1996 year of market survey taken at Akure city, Nigeria.

The Results section is divided into four according to the number of distribution networks. Table 5.1 shows the present commercially available pipe sizes and their costs in the market.

TABLE 5.1 Pipe Types and Costs(According to 1996 market survey at Akure city - Nigeria)

PIPE DIAMETER, D (m)	TYPE OF PIPE	PRESENT STANDARD COST, PER METER (NAIRA)
0.075	GALVANISED IRON	282.52
0.100	GALVANISED IRON	364.54
0.150	ASBESTOS CEMENT	937.38
0.200	ASBESTOS CEMENT	1,101.43

5.2 NETWORK A

Table 5. 2 shows all the characteristics of the pipes within the network. The pipe label, length, L (meters), initial diameter, D (meters), initial assumed flow, Q (m^3/s), Hazen - William's coefficient, C_{HW} and nodes at beginning and end of the pipes are shown. The criteria which determines beginning or end of a pipe is the direction of pipe flow.

Table 5 3 shows all nodes within the network and their consumption requirements along with the minimum service pressure head requirements.

Pipes **T, R, P, N, L, and K** are all on the Critical Path

Table 5 4 gives the corrections made to pipe flows from the initial assumed flow values of each pipe to the last iteration performed on the network, while Table 5.5 shows the modifications of pipe diameter from the initial assumed pipe diameters to the last iteration along with the total pipe and energy costs. Table 5. 6 shows the actual pressure heads experienced at each node within the network and the pressure at the source node required to ensure flow of water to every other node within the network during each iteration procedure.

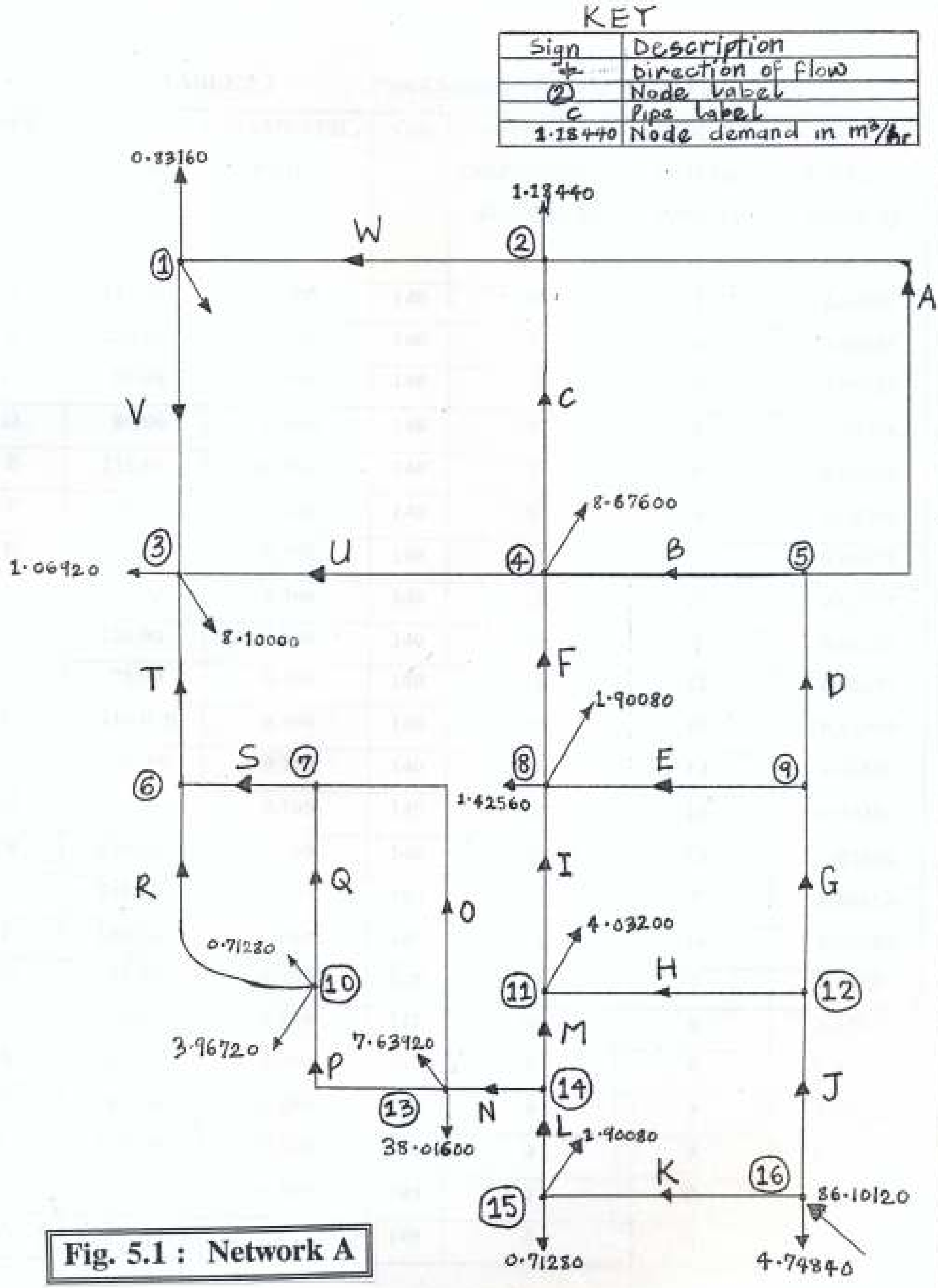


Fig. 5.1 : Network A

TABLE 5.2 Pipe Characteristics and Initial Flows

PIPE LABEL, P	LENGTH, L (m)	DIAMETER , D (m)	C_{HW}	NODE AT BEGINNING OF PIPE, P_b	NODE AT END OF PIPE, P_e	ASSUMED INITIAL FLOW,Q (m^3/s)
A	315.00	0.100	140	5	2	0.00050
B	220.00	0.100	140	5	4	0.00036
C	80.00	0.100	140	4	2	0.00055
D	80.00	0.100	140	9	5	0.00086
E	215.00	0.100	140	9	8	0.00092
F	90.00	0.100	140	8	4	0.00300
G	115.00	0.100	140	12	9	0.00179
H	210.00	0.100	140	12	11	0.00112
I	120.00	0.100	140	11	8	0.00300
J	75.00	0.100	140	16	12	0.00291
K	210.0 0	0.100	140	16	15	0.01969
L	40. 00	0.100	140	15	14	0.01896
M	38.00	0.100	140	14	11	0.00300
N	110.00	0.100	140	14	13	0.01596
O	250.00	0.100	140	13	7	0.00050
P	185.00	0.100	140	13	10	0.00050
Q	70.00	0.100	140	10	7	0.00050
R	175.00	0.100	140	10	6	0.00097
S	135.00	0.100	140	7	6	0.00100
T	80.00	0.100	140	6	3	0.00197
U	310.00	0.100	140	4	3	0.00040
V	90.00	0.100	140	1	3	0.00017
W	305.00	0.100	140	2	1	0.00073

TABLE 5.3 Nodal Consumption and Minimum Service Pressure head Requirements

NODE, N	NODAL CONSUMPTION, Q (m^3/s)	MINIMUM SERVICE PRESSURE HEAD REQUIREMENT, h_{MSP} (m)
1	0.00056	20.00
2	0.00032	20.00
3	0.00254	20.00
4	0.00241	20.00
5	0.00000	20.00
6	0.00000	20.00
7	0.00000	20.00
8	0.00092	20.00
9	0.00000	20.00
10	0.00130	20.00
11	0.00112	20.00
12	0.00000	20.00
13	0.01268	20.00
14	0.00000	20.00
15	0.00072	20.00
16	-0.02259	0.00

TABLE 5.4 NETWORK A : Corrected Flows

	PIPE FLOW Q (m^3/s)		
PIPE LABEL, P	INITIAL	1ST CORRECTED	2ND CORRECTED
A	0.00050	0.00043	0.00138
B	0.00036	0.00250	0.00187
C	0.00055	0.00045	0.00066
D	0.00086	0.00294	0.00325
E	0.00092	0.00277	0.00320
F	0.00300	0.00475	0.00386
G	0.00179	0.00342	0.00417
H	0.00112	0.00973	0.00497
I	0.00300	0.00520	0.00387
J	0.00291	0.01214	0.00813
K	0.01969	0.01045	0.01446
L	0.01896	0.00972	0.01373
M	0.00300	0.00240	0.00583
N	0.01596	0.01213	0.01270
O	0.00050	0.00283	0.00309
P	0.00050	0.00327	0.00358
Q	0.00050	0.00099	0.00024
R	0.00097	0.00176	0.00204
S	0.00100	0.00282	0.00234
T	0.00197	0.00185	0.00242
U	0.00040	0.00440	0.00266
V	0.00017	0.00001	0.00116
W	0.00073	0.00055	0.00171

TABLE 5.5 NETWORK A : Pipe Diameter Modification

	DIAMETER OF PIPE, D (m)		
PIPE LABEL, P	INITIAL VALUE	1ST ITERATION	2ND ITERATION
A	0.100	0.075	0.075
B	0.100	0.075	0.075
C	0.100	0.075	0.075
D	0.100	0.075	0.075
E	0.100	0.075	0.075
F	0.100	0.100	0.100
G	0.100	0.075	0.075
H	0.100	0.100	0.100
I	0.100	0.100	0.100
J	0.100	0.100	0.100
K	0.100	0.150	0.200
L	0.100	0.150	0.200
M	0.100	0.100	0.100
N	0.100	0.150	0.200
O	0.100	0.100	0.100
P	0.100	0.100	0.150
Q	0.100	0.075	0.075
R	0.100	0.100	0.100
S	0.100	0.100	0.100
T	0.100	0.100	0.100
U	0.100	0.100	0.100
V	0.100	0.100	0.100
W	0.100	0.075	0.075
PIPE COST (#)	**1,282,452:00**	**1,373,846:00**	**1,538,880:00**
ENERGY COST (#)	**1,288,991:00**	**1,103,458:00**	**1,059,230:00**
TOTAL COST (#)	**2,571,443:00**	**2,477,305:00**	**2,598,110:00**

TABLE 5.6 NETWORK A : Actual Service Pressure heads

	ACTUAL SERVICE PRESSURE HEADS, h_{ASP} (m)		
NODE	STARTING CONDITION	1ST ITERATION	2ND ITERATION
1	35.15432	27.65531	27.27260
2	35.17848	28.45226	27.82231
3*	30.00000	30.00000	30.00000
4	35.18283	28.48795	27.82864
5	35.46811	29.16012	28.66182
6	30.05913	30.09746	30.13534
7	30.17631	30.24728	30.27697
8	35.27139	28.05854	28.68907
9	35.60721	29.84262	29.47339
10	30.17718	30.25212	30.36533
11	32.51616	30.42510	30.00536
12	35.87187	31.39388	30.50165
13	30.56915	30.71652	30.48430
14	33.20878	31.11561	30.58945
15	33.84660	31.28334	30.64474
16**	37.67548	32.25261	30.95987

* Represents the critical node ** Represents the source node

5.3 NETWORK B

Table 5.7 shows all the characteristics of the pipes within the network. The pipe label, length, L (meters), initial diameter, D (meters), initial assumed flow, Q (m3/s), Hazen - William's coefficient, C_{HW} and nodes at beginning and end of the pipes are shown. The criteria which determines beginning or end of a pipe is the direction of pipe flow.

Table 5.8 shows all nodes within the network and their consumption requirements along with the minimum service pressure head requirements.

Pipes **X,W, B,C,D,E, and F** are all on the Critical Path

Table 5. 9gives the corrections made to pipe flows from the initial assumed flow values of each pipe to the last iteration performed on the network, while Table 5.10shows the modifications of pipe diameter from the initial assumed pipe diameters to the last iteration along with the total pipe and energy costs. Table 5.11 shows the actual pressure heads experienced at each node within the network and the pressure at the source node required to ensure flow of water to every other node within the network during each iteration procedure.

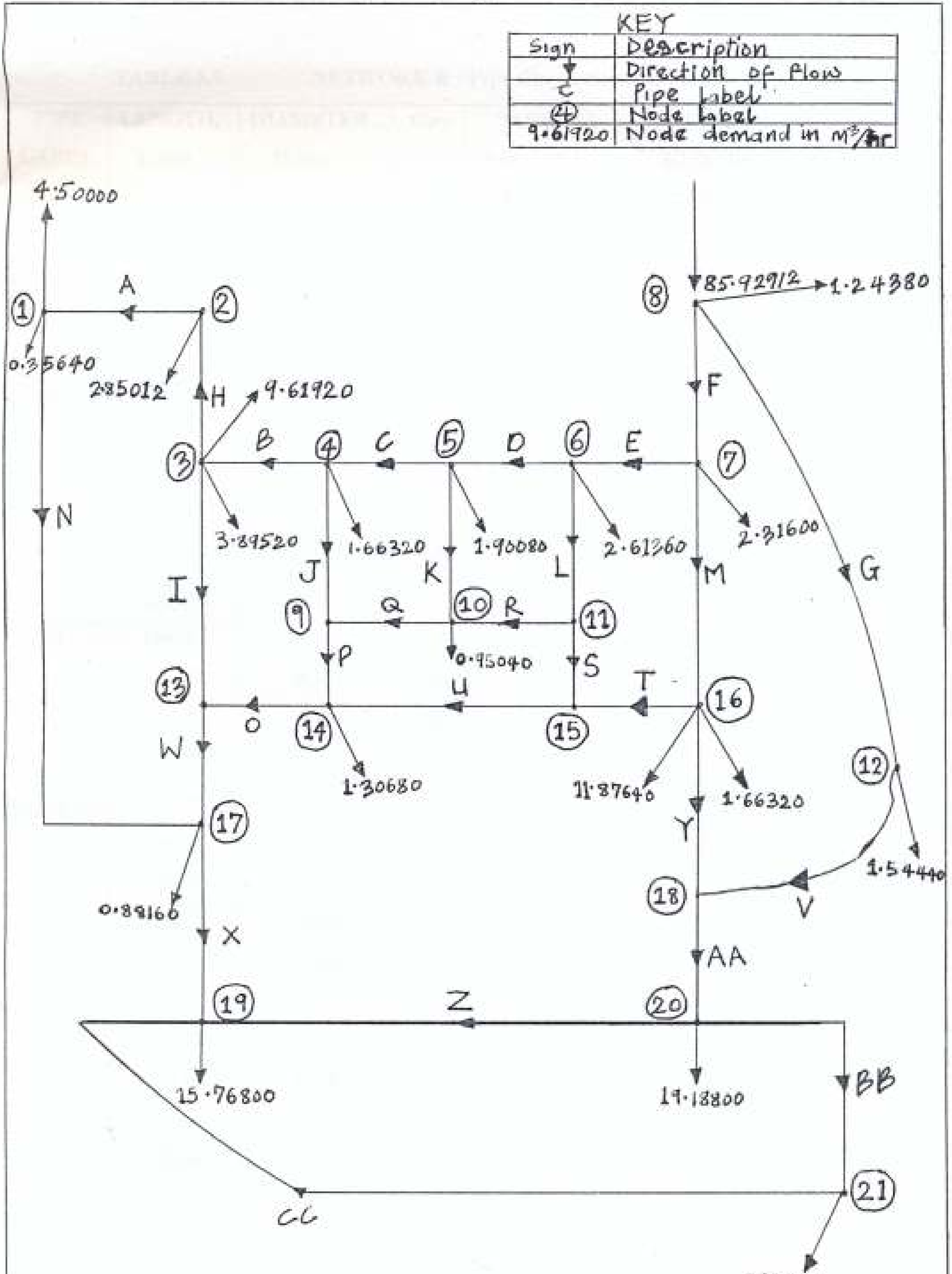


Fig. 5.2 : Network B

TABLE 5.7 NETWORK B : Pipe Characteristics and Initial Flows

PIPE LABEL, P	LENGTH, L (m)	DIAMETER , D (m)	C_{HW}	NODE AT BEGINNING OF PIPE, P_b	NODE AT END OF PIPE, P_e	ASSUMED PIPE FLOW,Q (m^3/s)
A	90.00	0.100	140	2	1	0.00196
B	120.00	0.100	140	4	3	0.00850
C	90.00	0.100	140	5	4	0.00926
D	90.00	0.100	140	6	5	0.01007
E	100.00	0.100	140	7	6	0.01139
F	95.00	0.100	140	8	7	0.02226
G	445.00	0.100	140	8	12	0.00125
H	100.00	0.100	140	3	2	0.00275
I	230.00	0.100	140	3	13	0.00200
J	180.00	0.100	140	4	9	0.00030
K	220.00	0.100	140	5	10	0.00027
L	260.00	0.100	140	6	11	0.00060
M	390.00	0.100	140	7	16	0.01020
N	80.00	0.100	140	1	17	0.00061
O	115.00	0.100	140	14	13	0.00100
P	90.00	0.100	140	9	14	0.00061
Q	105.00	0.100	140	10	9	0.00031
R	90.00	0.100	140	11	10	0.00030
S	110.00	0.100	140	11	15	0.00030
T	110.00	0.100	140	16	15	0.00045
U	190.00	0.100	140	15	14	0.00075
V	420.00	0.100	140	12	18	0.00082
W	70.00	0.100	140	13	17	0.00300
X	105.00	0.100	140	17	19	0.00338
Y	120.00	0.100	140	16	18	0.00600

Z	415.00	0.100	140	20	19	0.00070
AA	60.00	0.100	140	18	20	0.00682
BB	250.00	0.100	140	20	21	0.00079
CC	680.00	0.100	140	21	19	0.00030

TABLE 5.8 NETWORK B : Nodal Consumption and Minimum Service Pressure head Requirements

NODE, N	NODAL CONSUMPTION, $Q\ (m^3/s)$	MINIMUM SERVICE PRESSURE HEAD REQUIREMENT, h_{MSP} (m)
1	0.00134	20.00
2	0.00079	20.00
3	0.00375	20.00
4	0.00046	20.00
5	0.00052	20.00
6	0.00072	20.00
7	0.00066	20.00
8	-0.02351	0.00
9	0.00000	20.00
10	0.00026	20.00
11	0.00000	20.00
12	0.00042	20.00
13	0.00000	20.00
14	0.00036	20.00
15	0.00000	20.00
16	0.00376	20.00
17	0.00023	20.00
18	0.00000	20.00
19	0.00438	20.00
20	0.00533	20.00
21	0.00049	20.00

TABLE 5.9 NETWORK B : Corrected Flows

	PIPE FLOW (m^3/s)			
PIPE LABEL	INITIAL	1ST CORRECTED	2ND CORRECTED	3RD CORRECTED
A	0.00196	0.00215	0.00176	0.00112
B	0.00850	0.00579	0.00650	0.00715
C	0.00926	0.00608	0.00666	0.00757
D	0.01007	0.00694	0.00884	0.00911
E	0.01139	0.01340	0.01289	0.01259
F	0.02226	0.01686	0.01891	0.02071
G	0.00125	0.00665	0.00460	0.00280
H	0.00275	0.00294	0.00255	0.00191
I	0.00200	0.00090	0.00199	0.00328
J	0.00030	0.00017	0.00110	0.00136
K	0.00027	0.00033	0.00214	0.00150
L	0.00060	0.00573	0.00332	0.00276
M	0.01020	0.00279	0.00535	0.00744
N	0.00061	0.00080	0.00041	0.00022
O	0.00100	0.00338	0.00247	0.00151
P	0.00061	0.00234	0.00157	0.00120
Q	0.00031	0.00251	0.00188	0.00125
R	0.00030	0.00244	0.00048	0.00050
S	0.00030	0.00328	0.00283	0.00226
T	0.00045	0.00123	0.00153	0.00152
U	0.00075	0.00140	0.00126	0.00067
V	0.00082	0.00622	0.00417	0.00237
W	0.00300	0.00248	0.00267	0.00300
X	0.00338	0.00305	0.00285	0.00254
Y	0.00600	0.00092	0.00317	0.00528
Z	0.00070	0.00715	0.00126	0.00147

AA	0.00682	0.00090	0.00735	0.00765
BB	0.00079	0.00715	0.00075	0.00085
CC	0.00030	0.00042	0.00026	0.00036

TABLE 5.10 NETWORK B : Diameter Modification

PIPE LABEL	DIAMETER OF PIPE , D (m)			
	INITIAL VALUE	1ST ITERATION	2ND ITERATION	3RD ITERATION
A	0.100	0.100	0.100	0.100
B	0.100	0.150	0.200	0.200
C	0.100	0.150	0.200	0.200
D	0.100	0.150	0.200	0.200
E	0.100	0.150	0.200	0.200
F	0.100	0.150	0.200	0.200
G	0.100	0.100	0.100	0.100
H	0.100	0.100	0.100	0.100
I	0.100	0.100	0.150	0.150
J	0.100	0.075	0.075	0.075
K	0.100	0.075	0.075	0.075
L	0.100	0.100	0.100	0.100
M	0.100	0.100	0.150	0.200
N	0.100	0.075	0.075	0.075
O	0.100	0.100	0.100	0.100
P	0.100	0.075	0.075	0.075
Q	0.100	0.075	0.075	0.075
R	0.100	0.075	0.075	0.075
S	0.100	0.100	0.100	0.100
T	0.100	0.075	0.075	0.075
U	0.100	0.075	0.075	0.075
V	0.100	0.100	0.100	0.100
W	0.100	0.10 0	0.150	0.150
X	0.100	0.100	0.150	0.150
Y	0.100	0.100	0.150	0.150
Z	0.100	0.100	0.100	0.100

AA	0.100	0.100	0.150	0.150
BB	0.100	0.075	0.075	0.075
CC	0.100	0.100	0.100	0.100
PIPE COST (#)	1,975,807:00	2,151,506:00	2,791,230:00	2,855,210:00
ENERGY COST (#)	*1,407,149:00*	*1,137,912:00*	*1,086,623:00*	*1,088,332:00*
TOTAL COST (#)	*3,382,956:00*	*3,289,418:00*	*3,877,853:00*	*3,943,542:00*

TABLE 5.11 NETWORK B : Actual Service Pressure heads

	ACTUAL SERVICE PRESSURE HEADS, h_{ASP} (m)			
NODE	STARTING CONDITION	1ST ITERATION	2ND ITERATION	3RD ITERATION
1	30.06780	30.27505	30.00052	30.03989
2	30.15552	30.33572	30.02679	30.06143
3	30.32972	30.46976	30.10529	30.13150
4	31.06310	30.59569	30.14232	30.17369
5	31.66510	30.69446	30.17317	30.20778
6	32.43307	30.86145	30.21664	30.25279
7	35.31916	31.23432	30.30465	30.33695
8*	39.51557	31.95484	30.51454	30.56255
9	31.06146	30.38679	29.83422	29.90752
10	31.65844	29.82482	29.72315	29.88385
11	30.87474	30.29437	29.81429	29.96889
12	36.00150	30.18038	29.80374	30.18849
13	30.28498	30.27466	30.03698	30.03670
14	30.95868	30.18610	29.71195	29.85517
15	30.63971	30.11539	29.69674	29.88520
16	34.70292	29.17645	29.77906	30.16902
17	30.19593	30.17276	30.01941	30.01855
18	34.67848	28.93545	29.69342	30.05494
19**	30.00000	30.00000	30.00000	30.00000
20	34.13746	28.36597	29.60814	29.96839
21	34.08714	28.22219	29.42883	29.78238

* Represents the source node ** Represents the critical node

5.4 NETWORK C

Table 5.12 shows all the characteristics of the pipes within the network. The pipe label, length, L (meters), initial diameter, D (meters), initial assumed flow, Q (m^3/s), Hazen - William's coefficient, C_{HW} and nodes at beginning and end of the pipes are shown. The criteria which determines beginning or end of a pipe is the direction of pipe flow.

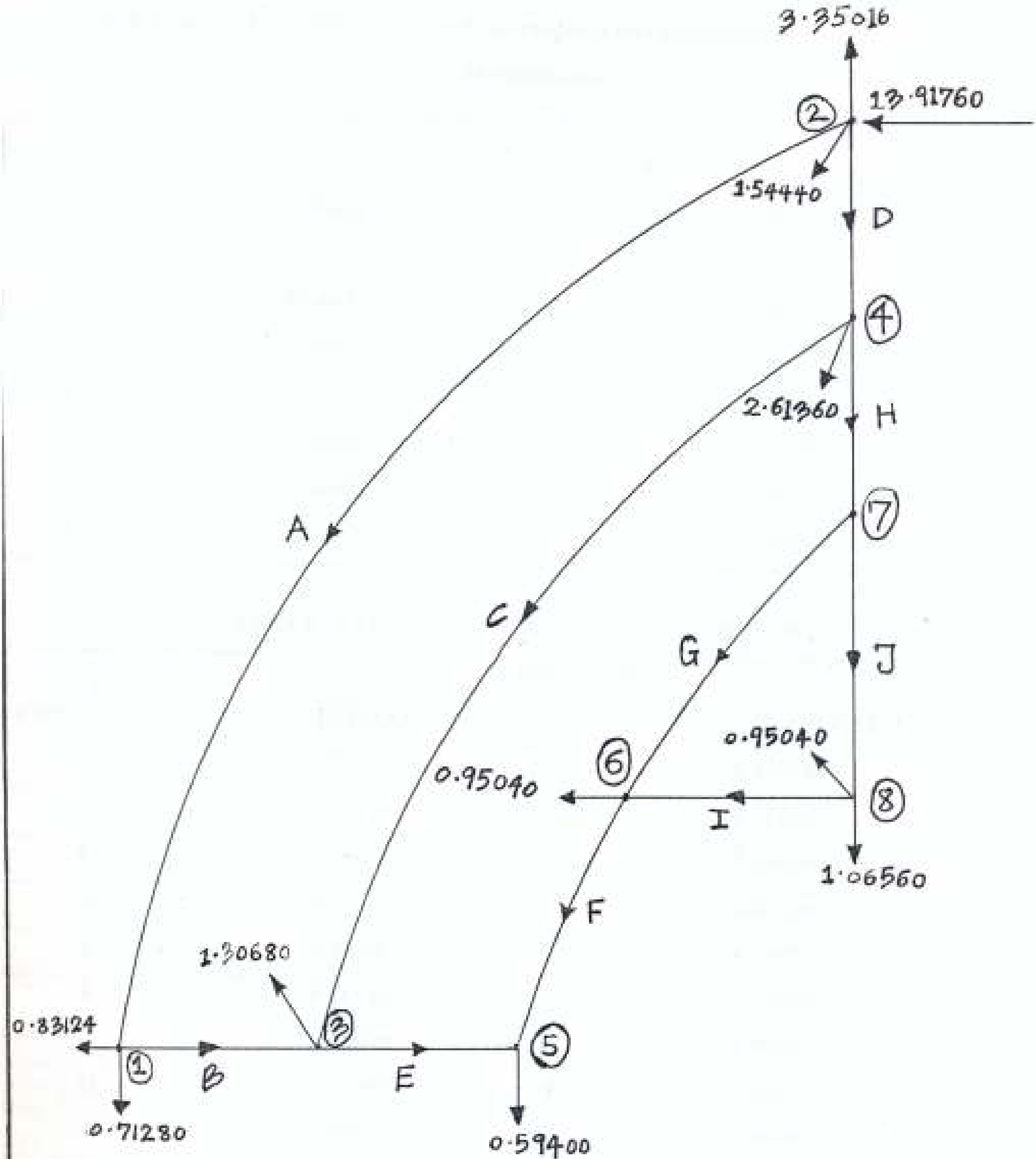
Table 5.13 shows all nodes within the network and their consumption requirements along with the minimum service pressure head requirements.

Pipes F,I ,,J ,H, and D are all on the Critical Path.

Table 5.14 gives the corrections made to pipe flows from the initial assumed flow values of each pipe to the last iteration performed on the network, while Table 5.15 shows the modifications of pipe diameter from the initial assumed pipe diameters to the last iteration along with the total pipe and energy costs. Table 5.16 shows the actual pressure heads experienced at each node within the network and the pressure at the source node required to ensure flow of water to every other node within the network during each iteration procedure.

TABLE 5.12 NETWORK C : Pipe Characteristics and Initial Flows

PIPE LABEL, P	LENGTH, L (m)	DIAMETER , D (m)	C_{HW}	NODE AT BEGINNING OF PIPE, P_b	NODE AT END OF PIPE, P_e	ASSUMED PIPE FLOW,Q (m^3/s)
A	795.00	0.075	140	2	1	0.00053
B	595.00	0.075	140	1	3	0.00010
C	130.00	0.075	140	4	3	0.00033
D	90.00	0.075	140	2	4	0.00197
E	150.00	0.075	140	3	5	0.00006
F	170.00	0.075	140	6	5	0.00010
G	290.00	0.075	140	7	6	0.00020
H	100.00	0.075	140	4	7	0.00092
I	130.00	0.075	140	6	8	0.00016
J	200.00	0.075	140	7	8	0.00072



KEY

Sign	Description
↓	Direction of flow
③	Node Label
G	Pipe Label
0·95040	Node demand in M^3/hr

Fig. 5.3 : Network C

TABLE 5.13 NETWORK C : Nodal Consumption and Minimum Service Pressure head Requirements

NODE, N	NODAL CONSUMPTION, Q (m^3/s)	MINIMUM SERVICE PRESSURE HEAD REQUIREMENT, h_{MSP} (m)
1	0.00042	20.00
2	-0.00231	20.00
3	0.00036	20.00
4	0.00072	20.00
5	0.00016	20.00
6	0.00026	20.00
7	0.00000	20.00
8	0.00056	20.00

TABLE 5.14 NETWORK C : Corrected Flows

	PIPE FLOW (m^3/s)	
PIPE LABEL	INITIAL	1ST CORRECTED
A	0.00053	0.00058
B	0.00010	0.00015
C	0.00032	0.00046
D	0.00197	0.00191
E	0.00006	0.00026
F	0.00010	0.00009
G	0.00020	0.00034
H	0.00092	0.00072
I	0.00016	0.00017
J	0.00072	0.00038

TABLE 5.15 NETWORK C : Diameter Modification

	DIAMETER OF PIPE , D (m)	
PIPE LABEL, P	INITIAL VALUE	1ST ITERATION
A	0.075	0.075
B	0.075	0.075
C	0.075	0.075
D	0.075	0.100
E	0.075	0.075
F	0.075	0.075
G	0.075	0.075
H	0.075	0.100
I	0.075	0.075
J	0.075	0.075
PIPE COST(#)	748,678:00	764,261:00
ENERGY COST(#)	115,295:00	114,376:00
TOTAL COST(#)	863,973:00	878,638:40

TABLE 5.16 NETWORK C : Actual Service Pressure heads

	ACTUAL SERVICE PRESSURE HEADS, h_{ASP} (m)	
NODE	STARTING CONDITION	1st ITERATION
1	30.09657	29.97708
2*	30.38101	30.13877
3	30.06205	30.00802
4	30.09254	30.05676
5**	30.00000	30.00000
6	30.00221	30.00310
7	30.03961	30.04306
8	30.00728	30.00883

* Represents the source node ** Represents the critical node

5.5 NETWORK D

Table 5.17 shows all the characteristics of the pipes within the network. The pipe label, length, L (meters), initial diameter, D (meters), initial assumed flow, Q (m^3/s), Hazen-William's coefficient, C_{HW} and nodes at beginning and end of the pipes are shown. The criteria which determines beginning or end of a pipe is the direction of pipe flow.

Table 5.18 shows all nodes within the network and their consumption requirements along with the minimum service pressure head requirements.

Pipes J, I, and D are all on the Critical Path.

Table 5.19 gives the corrections made to pipe flows from the initial assumed flow values of each pipe to the last iteration performed on the network, while Table 5.20 shows the modifications of pipe diameter from the initial assumed pipe diameters to the last iteration along with the total pipe and energy costs. Table 5.21 shows the actual pressure heads experienced at each node within the network and the pressure at the source node required to ensure flow of water to every other node within the network during each iteration procedure.

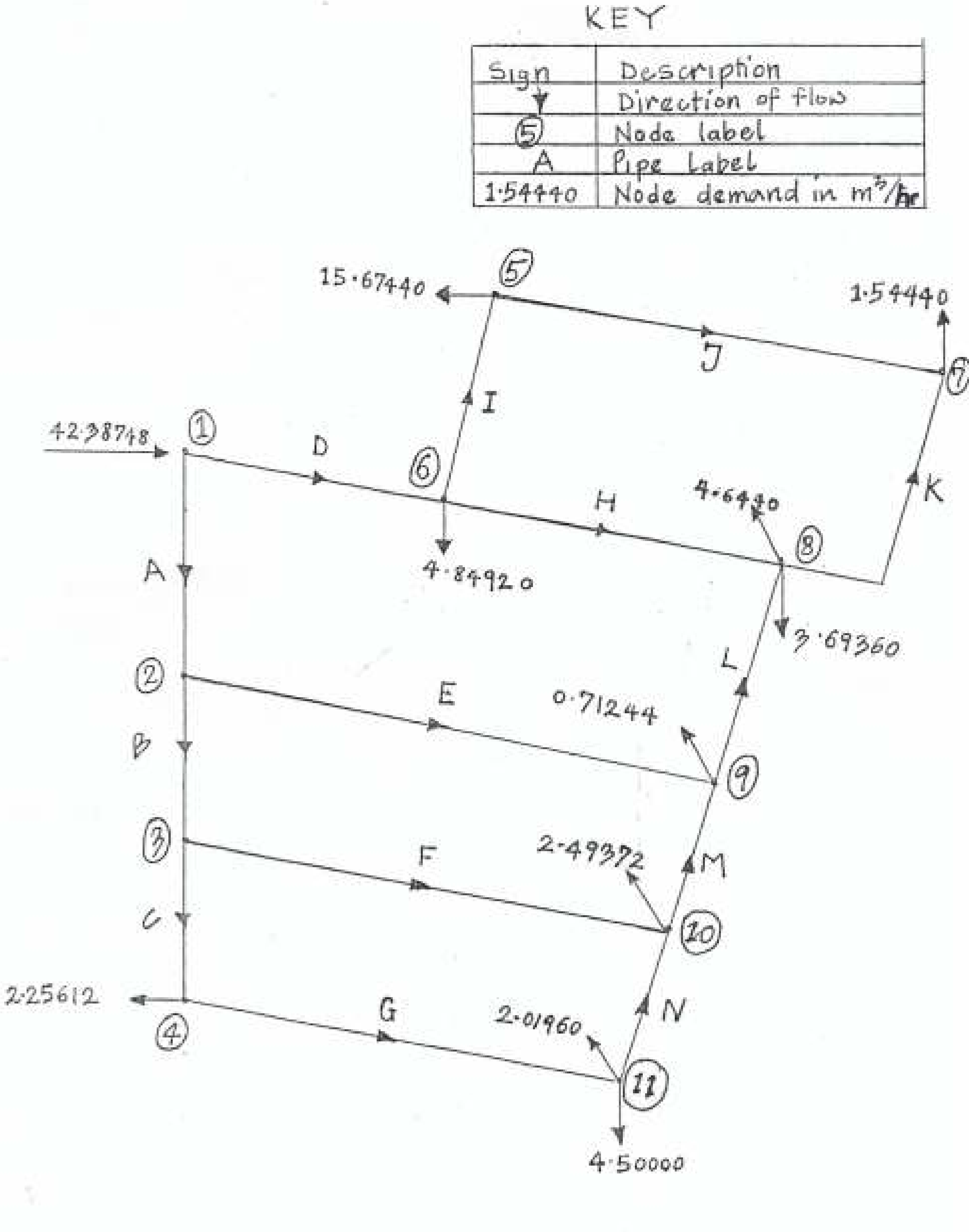


Fig.5.4 : Network D

TABLE 5.17 NETWORK D : Pipe Characteristics and Initial Flows

PIPE LABEL, P	LENGTH, L (m)	DIAMETER , D (m)	C_{HW}	NODE AT BEGINNING OF PIPE, P_b	NODE AT END OF PIPE, P_e	ASSUMED PIPE FLOW, Q (m^3/s)
A	120.00	0.075	140	1	2	0.00384
B	90.00	0.075	140	2	3	0.00344
C	60.00	0.075	140	3	4	0.00274
D	250.00	0.075	140	1	6	0.00793
E	375.00	0.075	140	2	9	0.00040
F	275.00	0.075	140	3	10	0.00070
G	210.00	0.075	140	4	11	0.00211
H	230.00	0.075	140	6	8	0.00200
I	140.00	0.075	140	6	5	0.00458
J	380.00	0.075	140	5	7	0.00022
K	210.00	0.075	140	8	7	0.00020
L	120.00	0.075	140	9	8	0.00051
M	90.00	0.075	140	10	9	0.00031
N	60.00	0.075	140	11	10	0.00030

TABLE 5.18 NETWORK D : Nodal Consumption and Minimum Service Pressure head Requirements

NODE, N	NODAL CONSUMPTION, Q (m^3/s)	MINIMUM SERVICE PRESSURE HEAD REQUIREMENT, h_{MSP} (m)
1	-0.01177	20.00
2	0.00000	20.00
3	0.00000	20.00
4	0.00062	20.00
5	0.00435	20.00
6	0.00134	20.00
7	0.00042	20.00
8	0.00231	20.00
9	0.00019	20.00
10	0.00069	20.00
11	0.00181	20.00

TABLE 5.19 NETWORK D : Corrected Flows

	PIPE FLOW (m^3/s)		
PIPE LABEL	INITIAL	1ST CORRECTED	2ND CORRECTED
A	0.00384	0.00584	0.00504
B	0.00344	0.00255	0.00343
C	0.00274	0.00196	0.00151
D	0.00793	0.00593	0.00672
E	0.00040	0.00328	0.00161
F	0.00070	0.00058	0.00191
G	0.00211	0.00134	0.00089
H	0.00200	0.00186	0.00258
I	0.00458	0.00271	0.00279
J	0.00022	0.00209	0.00217
K	0.00020	0.00166	0.00159
L	0.00051	0.00251	0.00172
M	0.00031	0.00057	0.00145
N	0.00030	0.00046	0.00001

TABLE 5.20 NETWORK D : Diameter Modification

	DIAMETER OF PIPE, D (m)		
PIPE LABEL, P	INITIAL VALUE	1ST ITERATION	2ND ITERATION
A	0.075	0.075	0.100
B	0.075	0.075	0.100
C	0.075	0.075	0.075
D	0.075	0.100	0.150
E	0.075	0.075	0.075
F	0.075	0.075	0.075
G	0.075	0.075	0.075
H	0.075	0.075	0.100
I	0.075	0.100	0.150
J	0.075	0.075	0.075
K	0.075	0.075	0.075
L	0.075	0.075	0.075
M	0.075	0.075	0.075
N	0.075	0.075	0.075
PIPE COST (#)	737,377:00	769,365:00	1,028,861:00
ENERGY COST (#)	690,967:00	601,890:00	562,153:30
TOTAL COST (#)	1,428,345:00	1,371,256:00	1,591,015:00

TABLE 5.21 NETWORK D : Actual Service Pressure heads

	ACTUAL SERVICE PRESSURE HEADS, h_{ASP} (m)		
NODE, N	STARTING CONDITION	1ST ITERATION	2ND ITERATION
1*	38.76180	33.76475	31.53558
2	35.73993	31.45838	31.09301
3	35.25038	30.61145	30.88715
4	35.04861	30.48679	30.68755
5	31.43372	31.53145	31.18024
6	32.28702	31.75264	31.20539
7**	30.00000	30.00000	30.00000
8	31.58780	30.47544	30.65173
9	32.48227	30.58482	30.73149
10	35.15303	29.73392	30.36233
11	34.70109	30.32391	30.34552

* Represents the source node ** Represents the critical node

5.6 SUMMARY OF COSTS

A summary of the results of the four(4) networks optimised in the previous section are as follows;

Table 5.22 NETWORK A: Summary of costs

	TOTAL COSTS (Naira)			Naira	
OPTIMISATION	PIPE	ENERGY	Σ(PIPE + ENERGY)	SAVINGS FROM TOTAL PREVIOUS COST	ADDITION TO TOTAL PREVIOUS COST
STARTING PROCESS	1,282,452:00	1,288,991:00	2,571,443:00		
1ST PROCESS*	1,373,846:00	1,103,458:00	2,477,305:00	94,138:00	
2ND PROCESS	1,538,880:00	1,059,230:00	2,598,110:00		120,805:00

*** Optimal arrangement of pipes**

The network under the different pipe sizes in the three processes satisfy standard engineering requirements of flow and headloss as dictated by the Kirchoff's first and second laws.

The network under the 1ST process of optimisation is recommended as the optimal arrangement of pipes for Network A. It is noted that the arrangement of pipes in this stage results in the least overall cost amongst the others, although the total sum of pipe costs exceed that of the INITIAL arrangement. On the other hand, the Energy cost for this stage is least when compared to the other two arrangements. It is therefore implied from the recommendation that both Galvanised iron pipes of diameter 75mm and 100mm along with Asbestos cement pipes of diameter 150mm would be used within the network.

The minimum recommended pressure head expected at the source node which will ensure flow of water to all nodes within the network is 32.25 meters.

Table 5.23 NETWORK B: Summary of costs

OPTIMISATION	TOTAL COSTS (Naira)			Naira	
	PIPE	ENERGY	Σ(PIPE + ENERGY)	SAVINGS FROM TOTAL PREVIOUS COST	ADDITION TO TOTAL PREVIOUS COST
STARTING PROCESS	1,975,807:00	1,407149:00	3,382,956:00		
1ST PROCESS*	2,151,506:00	1,137,912:00	3,289,418:00	93,538:00	
2ND PROCESS	2,791,230:00	1,086,623:00	3,877,853:00		588,435:00
3RD PROCESS	2,855,210:00	1,088,332:00	3,943,542:00		43,542:00

* Optimal arrangement of pipes

For Network B, the four(4) different processes labelled INITIAL, 1ST, 2ND and 3RD satisfy standard engineering requirements of flow and headloss as dictated by the Kirchoff's first and second laws.

The arrangement of pipe sizes in the 1ST process is recommended as the optimal. It is the least in Total cost as compared to the arrangement of pipes in the INITIAL, 2ND and 3RD stages. A close look however shows that both the pipe and energy cost in this stage are not the least as compared to the others. It is recommended therefore that Galvanised iron pipes of 75mm and 100mm diameter along with Asbestos cement pipes of 150mm diameter be used for distribution.

The minimum required pressure head expected at the source node that would ensure an even distribution of water to all nodes in the network is 31.95 meters.

Table 5.24 NETWORK C: Summary of costs

OPTIMISATION	TOTAL COSTS (Naira)			Naira	
	PIPE	ENERGY	Σ(PIPE + ENERGY)	SAVINGS FROM TOTAL PREVIOUS COST	ADDITION TO TOTAL PREVIOUS COST
STARTING PROCESS*	748,678:00	115,295:00	863,973:00		
1ST PROCESS	764,261:00	114,376:00	878,638:40		14,665:40

* Optimal arrangement of pipes

The two arrangements of pipe sizes above satisfy standard engineering [illegible]
and headloss as dictated by the Kirchoff's first and second laws.

The INITIAL arrangement of pipe sizes above is recommended as the optimal design for Network C. The INITIAL arrangement of pipes shows the least Total cost amongst the two, although the energy cost computed for the 1ST process is less than that for the INITIAL process. Galvanised iron pipes of 75mm diameter alone will be used in this distribution network.

The minimum required pressure head expected at the source node that would ensure the flow of water to all nodes within the network is 30.30 meters.

Table 5.25 NETWORK D: Summary of costs

	TOTAL COSTS (Naira)			Naira	
OPTIMISATION	**PIPE**	**ENERGY**	**Σ(PIPE + ENERGY)**	SAVINGS FROM TOTAL PREVIOUS COST	ADDITION TO TOTAL PREVIOUS COST
STARTING PROCESS	**737,377:00**	**690,967:00**	**1,428,345:00**		
1ST PROCESS*	**769,365:00**	**601,890:00**	**1,371,256:00**	57,089:00	
2ND PROCESS	**1,028,861:00**	**562,153:30**	**1,591,015:00**		219,759:

*** Optimal arrangement of pipes**

The three arrangements of pipe sizes above satisfy standard engineering requirement of flow and headloss as dictated by the Kirchoff's first and second laws.

The 1ST arrangement of pipe sizes is recommended as the optimal design. This process ha the least Total cost amongst the three processes, although the pipe and energy costs are not least whe compared with the others. Galvanised Iron pipes of 75mm and 100mm diameter alone a recommended for used in this network.

The minimum required pressure head expected at the source node which would ensure t flow of water to all parts of the network through all the nodes is 33.76 meters.

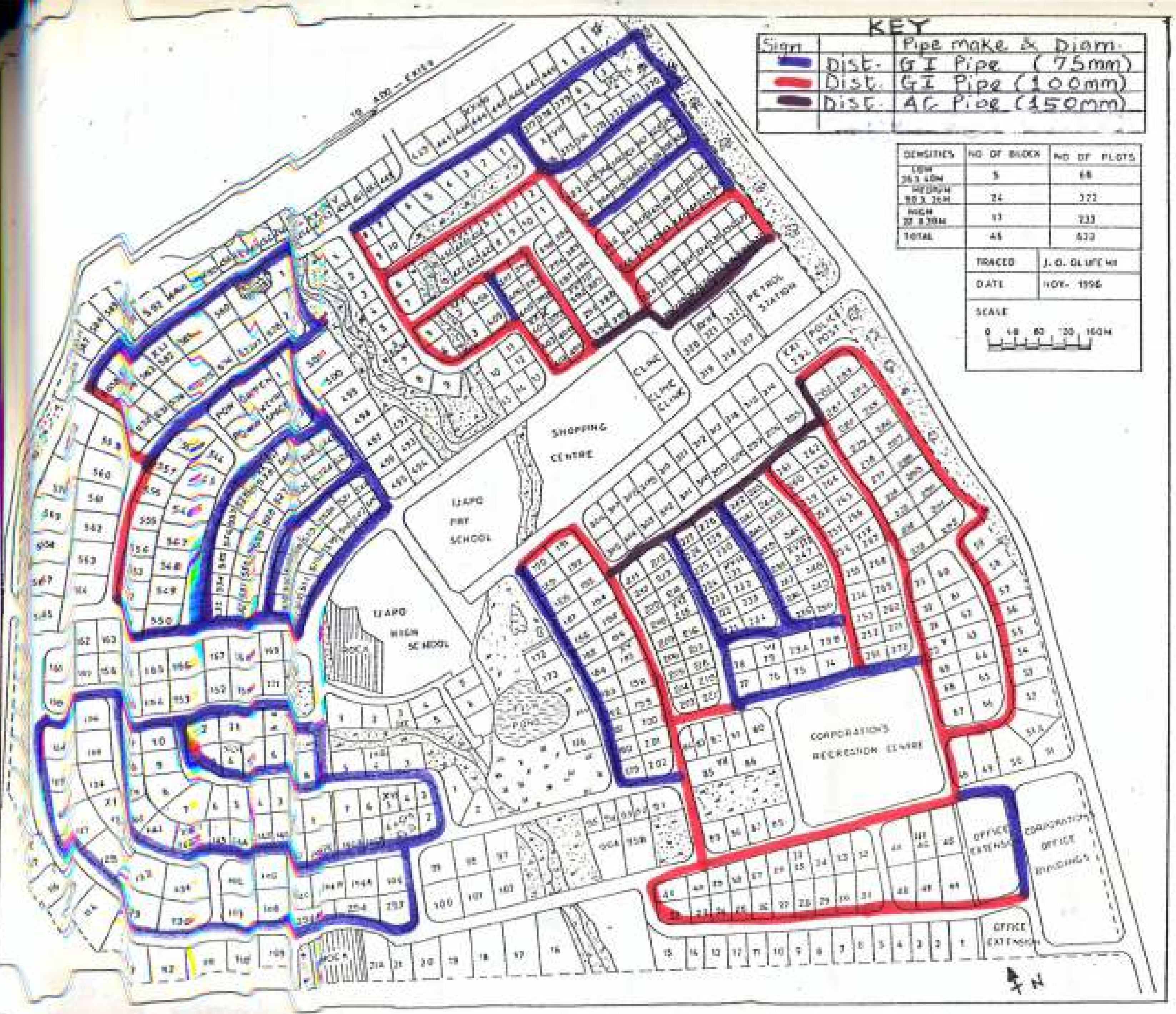


Fig. 5.5 : Optimal distribution networks for Ijapo estate.

5.7 THE OVERALL COST OF DISTRIBUTION SYSTEM

A total sum of the four networks of different pipe arrangements optimised and recommended in the tables above is summarised below;

Table 5.26 Overall cost of distribution system

	TOTAL COSTS(Naira)		
	PIPE	ENERGY	Σ(PIPE + ENERGY)
NETWORK A	1,373,846:00	1,103,458:00	2,477,305:00
NETWORK B	2,151,506:00	1,137,912:00	3,289,418:00
NETWORK C	748,678:00	115,295:00	863,973:00
NETWORK D	769,365:00	601,890:00	1,371,256:00
Σ (#)	**5,043,395:00**	**2,958,555:00**	**8,001,952:00**

A sum of about Eight million, one thousand, nine hundred and fifty-two naira (N8,001,952:00) is therefore needed in order to install an effective distribution system of four networks within Ijapo estate for a design period of twenty - five years.

5.8 THE COMPUTER MODEL

The FORTRAN77 programming language is used to run the model formulated because of it's universality and preference to all others, while the WATFOR Compiler was used to write and debug the model.

3 files (1 output and 2 input) are utilised within the program. They are opened simultaneously as the program is being run. The input files contain the bulk of the input data necessary for computation whereas, the output file collects all the results needed by the designer. This reduces the running time of the program and helps to avoid the tedious effort of entering data each time it is requested by the program.

CHAPTER 6

RECOMMENDATIONS AND CONCLUSIONS

6.1 RECOMMENDATIONS

6.1.1 OPTIMAL DESIGN APPLICATION

The pipes within the three networks presently on ground are made of either Asbestos cement (AC) or Unplasticized Polyvinyl Chloride (PVC) and are all of 100mm diameter. In the north - western section of the estate, opposite the estate's primary and high school, it is reported that most of the PVC pipes laid there have collapsed. Within the estate also, several other distribution pipes have either been dug - up or broken due to one reason or another. Therefore, replacement of all the pipes laid within the estate with the proposed optimal pipe arrangements in section 5.6 above, becomes a viable option that, should be implemented as early as possible for maximum benefit.

6.1.2 THE SERVICE STORAGE SYSTEM

An overhead storage tank is recommended at locations 5 (L5) and 6 (L6) (Figure 6.1). Each overhead tank is expected to contain a volume of at least 75 cubic meters. A circular shaped tank is geometrically the most economical shape. It consumes a lesser quantity of construction materials, and discourages leakage when placed in comparison with other shapes for a given volume and depth. Therefore, two conical shaped overhead storage tanks of reinforced concrete material are recommended. The depth of the frustum should be between 2.50 - 3.50 meters, with a base and top diameter of 3 and 5 meters respectively. The height of the bottom of the frustum of both overhead tanks above ground level should be at least 15 meters.

Overhead tank A at location 5 is expected to supply water directly to Networks C and D only. While overhead tank B at location 6 will supply water directly to Networks A and B only. Overhead tank B at location 6 is situated within the property of the College of Agriculture adjacent to the eastern section of the estate along the Oba - Adesida/Ado - Ekiti road. It is found to be the best location for an overhead tank which will supply water to Networks A and B considering the required minimum pressure heads.

A surface storage tank of at least 1000 m^3 capacity is recommended at location 7 which is situated within the area occupied by the shopping center. The surface storage tank will supply water to both overhead tanks A and B.

Ideally, it is expected to contain at least the total daily water consumption of the community which it services. This volume would ensure a balance to the fluctuating demand from the distribution system against the output from the source, and would also act as a safeguard for the continuance of the supply of water should there be any breakdown at the source or on the main trunk pipelines. Supply of water from the conjunctive source is expected to be continuous. A surface storage tank expected to contain a volume of 7000m^3 for a small community as Ijapo estate, is unfeasible. The space it is expected to occupy will be almost an entire plot, and the cost would be too high for the inhabitants to bear. Therefore, a storage volume of not less than 1,000m^3 is recommended for short-time emergencies. The surface storage tank should be constructed of reinforced concrete material.

6.1.3 THE WATER SUPPLY SYSTEM

For the provision of adequate potable water to consumers within Ijapo estate, four potential sources of water have been identified (Section 3.2.4). They are, (a) imported water (2 in number) (b)Groundwater, (c) Rainwater Harvesting, and (d) Surface water (Osisi stream).

The present sources of imported water (Owena water supply scheme and the College of Agriculture mini - water supply scheme), have become inadequate to cater for the water demands of the consumers within the estate. Therefore a conjunctive water supply scheme is proposed.

Osisi stream, a non-perennial stream within the estate is <u>not</u> recommended as one of the sources in the conjunctive supply system. It's development would require the construction of a dam - reservoir system or a weir across. The proximity to public, commercial and residential infrastructures, makes it inadvisable. Heavy rains may cause flooding, and this may become potentially dangerous to its surroundings and the inhabitants. In addition to a weir or small dam, there would also be the need to construct storage and treatment facilities. The overall cost for the development of this source per household will be too high for the inhabitants of the estate.

Rainwater harvesting which is also a good source of potable water is <u>not</u> recommended as part of the conjunctive supply system. Although quite a large volume of water may be collected from this source, the facilities for collection, treatment and storage are presently not on ground. The cost of constructing this facilities per household will also be too high for the inhabitants of the estate to carry.

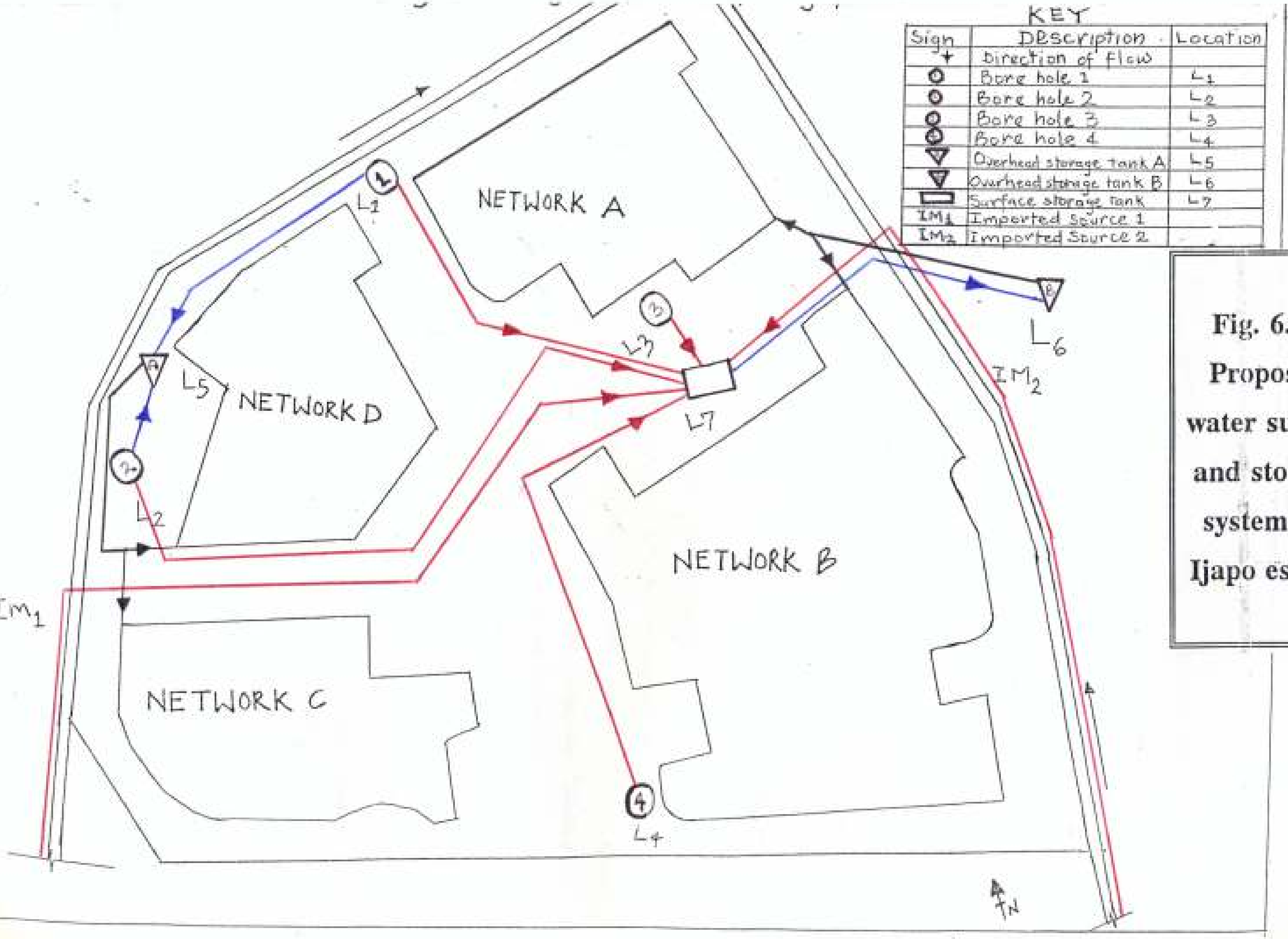


Fig. 6.1 : Proposed water supply and storage system for Ijapo estate.

The proposed conjunctive water supply system within Ijapo estate, therefore comprises imported sources of water and groundwater through four (4) boreholes. The four boreholes proposed will complement the proposed imported water supply sources.

It is recommended that the mini-water supply scheme at the College of Agriculture be expanded. This would require the following:

- Increasing the present capacity of the two existing reservoirs by the construction of bigger storage facilities,
- Purchasing bigger pumps to handle the large volume of water and pressures demanded by the estate's population and the other two major consumers,
- Constructing a bigger dam within the vicinity of the scheme and possibly dredging the two existing dams within the college premises, and
- Expanding the treatment facilities .

Presently, the scheme pumps about 300 m^3 and distributes the larger portion of it to the college population. An expansion of this scheme would benefit the estate considerably.

Presently, the Owena dam water supply scheme along the Ondo/Akure road supplies the largest quantity of water (18 million litres) to Akure city. There is the need to improve the Owena dam water supply scheme. The facilities of the scheme for supply and major distribution along both the Ondo/Akure road and the Ilesha/Akure road should be expanded to increase the overall supply to Akure Metropolis, and the estate in particular. The second phase of the Owena dam project presently being developed should be encouraged and completed on time. All imported sources of water into the estate from inlet points X and Y (Fig. 10) are channelled into the surface storage tank at location 7.

The four boreholes proposed are meant to complement the imported sources proposed above at occasions when water supplied from the imported source is less than the demand, or when flow is stopped for one reason or another. Borehole 1 should be constructed somewhere around plot 455/456, Block XXXV (location 1) at the northern section of the estate. As shown in Figure 10, Borehole 2 should be constructed around plot 558 (location 2) at the eastern section of the estate. Borehole 3 should be constructed around plot 16, Block X (location 3) at the south-eastern section of the estate, and Borehole 4 should be constructed within the area enclosed by the shopping center (location 4) at the center of the estate. Boreholes 1 and 2 would supply water directly to both the overhead tank A at location 5 and the surface storage tank at location 7. Boreholes 3 and 4 would supply water to both overhead tanks A and B through the surface storage tank at location 7.

A regional evaluation of Akure Metropolis by Akintunde (1996) affirms that Ijapo estate lies within the High Groundwater Potential Zone.Using Akintunde's classification, it is expected that a typical yield from a borehole within that area would be greater than 11.52 m^3/hour (confirmed yield from a borehole within the F.U.T.A. premises. F.U.T.A is situated within the Medium Groundwater Potential Zone). Pumping for at least 15 hours a day from that borehole will produce a volume of about 173m^3. It is expected that a carefully constructed borehole at any of the recommended locations within Ijapo estate at a depth of not less than 100 meters will produce a yield of at least 250m^3/day. Four boreholes pumping for 15 hours a day will then produce at least 1,000m^3/day. The proposed total cost of N8,001,952:00 summarised in section 5.7 excludes the cost for both the construction of the four boreholes and the cost of energy needed to pump water from these boreholes.

Potable water pumped into overhead tanks A and B are released into the four Networks (A, B, C, and D) by gravity.

6.2 FURTHER WORK

The optimisation procedure proposed in this thesis is constrained to simple, single-source water distribution systems with fixed water supply and nodal consumptions. An obvious extension of this procedure will be to develop a model which will accommodate multi-source distribution systems with variable flow rates at both the supply and consumption nodes while still maintaining the minimum service pressure requirements.

6.3 CONCLUSIONS

A model which optimises a simple, conjunctive - source, looped, gravity fed distribution system was designed and applied to the existing distribution system within Ijapo estate, Akure city, Nigeria. The model comprises of two major steps. The first step involves the balancing and correction of flows using the Hardy Cross method, while the second step optimises the distribution network by revolving each pipe diameter around one of three decisions which are, reducing to the next smaller available pipe diameter, leaving the diameter unchanged, or increasing to the next larger available pipe diameter. Discrete pipe diameters are used, and the entire process is iterative until no further cost improvement is ascertained.

Ijapo estate's present distribution system of three networks of 64 pipes is replaced by an optimal distribution system of four networks which comprises 76 pipes, made of both Galvanised Iron pipes of

75mm and 100mm diameter and Asbestos Cement pipes of 150mm and 200mm diameter. The initial total pipe and energy costs for the four proposed distribution networks within the estate are N4,744,314:00 and N3,502,402:00 respectively, a total sum of N8,246,716:00. The optimal system of four networks gives total pipe and energy costs as N5,043,395:00 and N2,958,535:00 respectively, or a sum of N8,001,930:00. The model therefore achieved a reduction of between 2 - 15% from the total initial pipe and energy costs of each of the four proposed distribution networks. For larger and more complex networks however, savings in costs of both pipe and energy will be particularly significant.

REFERENCES

1. Akintunde, O. M. (1996) : Hydro - geophysical evaluation of the groundwater potential of Akure metropolis. M. Tech. Thesis. Department of Applied Geophysics. Federal University of Technology, Akure. Nigeria.

2. Aron, G., and Scott, M. Verne, (1971) : " Dynamic Programming for Conjunctive Water Use". Journal of Hydraulics Division. Proceedings of The ASCE. Volume 97, Number HY5. pp 705 - 721.

3. Ayoade, J. O., (1988) : Tropical Hydrology and Water Resources. McMillan Publishers Limited. Gordon and Basingstoke. pp 8.

4. Bhave, R. Pramod, (1984) : "Optimal Expansion of Water Distribution Systems". Journal of the Environmental Engineering. Volume 111, Number 2. pp 177 - 197.

5. Bhave, R. Pramod, (1983) : "Optimisation of Gravity - Fed Water Distribution Systems : Application". Journal of the Environmental Engineering. Volume 109, Number. 2. pp 383 - 395.

6. Bhave, R. Pramod, (1978) : "Non - Computer Optimisation of Single - Source Networks". Journal of the Environmental Engineering. Proceedings of The ASCE. Volume 104, Number EE4. pp 799 - 814.

7. Biswas, K. Asit, (1976) : Systems Approach to Water Management. International Student Edition. McGraw Hill. Kogakusha Limited. pp 1 - 12.

8. Chadwick, A. J., and Morfett, J. C., (1986): Hydraulics in Civil Engineering. Allen and Unwin (Publishers) Limited. London, United Kingdom. pp 346 - 358.

9. Chow, Te. Ven, Maidment, R. David, Mays, W. Larry, (1988): Applied Hydrology. Civil Engineering Series. McGraw Hill Book Company. pp 1 - 5.

10. Clark, M. Robert, (1976) : "Cost and Pricing Relationships in Water Supply". Journal of the Environmental Engineering Division. Volume 102, Number EE2. pp 361 - 373.

11. Cox, E. William, (1988) : "Water and Development : A Complex Relationship". Journal of Water Resources Planning and Management. Volume 114, Number 1. pp 91 - 98.

12. Crigg, S. Neil, (1973) : "Legal Constraints on Urban Water Systems". Journal of Urban Planning and Development Division. Volume 99, Number UP2. Ithaca, New York.

13. Daugherty, L. Robert, Frazini, B. Joseph, (1977) : Fluid Mechanics with Engineering Applications. International Student Edition. Seventh Edition. McGraw Hill International Book Company. Kogakusha. pp 503 - 520.

14. Evett, B. Juck, Liu, Cheng, (1987): Fundamentals of Fluid Mechanics. Civil Engineering Series. McGraw Hill Book Company. Singapore. pp 189 - 209.

15. Howe, C. W., (1968) : "Water Resources and Regional Economic Growth in the United Stat 1950 - 1960". The Southern Economic Journal. 34(4). pp 477 - 489.

16. Kitching, R., Rushton, K. R., and Wilkinson, W. B. , (1975) : "Groundwater Yield Estimatio from Models in Engineering Hydrology today". Proceedings of the International Hydrological Decade Committee and Institute of Hydrology. London. pp 101 - 112.

17. Linsley, K. Ray, Frazini, B. Joseph, Freyberg, L. David, and Tchobanoglous, George, (1992) Water Resources Engineering. Civil Engineering Series. Fourth Edition. McGraw Hill Book Company. pp 497 - 506.

18. Littleton, T. Charles (1962): Industrial Piping. McGraw Hill Book Company. New York.

19. McGhee, J. Terence (1991): Water Supply and Sewerage. Civil Engineering Series. Sixth Edition. McGraw Hill International Edition. Highstown, New Jersey, USA. pp 104

20. McPherson, M. B., (1970) : Prospects for Metropolitan Water Management. The ASCE. New York, USA.

21. Morgan, D. R., and Goulter, I. C., (1985) : "Optimal Urban Water Distribution Design". Water Resources Research. Volume 21, Number 5. pp 642 - 652.

22. Nwosu, A. D., and Ojiako, G. U., (1989) : "A Transportation Model Approach for water Supply Development and Distribution in Nigeria : A Case Study". Water resources. Volume 1, Number 2. pp 176 - 180.

23. Offodile, M. E., (1988) : "Water Resources Management and The Nigerian Groundwater Systems". Water Resources. Volume 1, Number 1. pp 63 - 70.

24. Ogwude, G. N., and Oteze, G. E., (1988) : "A Geodetic Method for Monitoring Land Subsidence due to Fluid Abstraction in Nigeria". Water Resources. Journal of the Nigerian Association of Hydrogeologists. Volume 1, Number 1.

25. Owoyemi, F. B., (1996): A Geological - Geophysical investigation of rain - induced erosional features in Akure metropolis. M. Tech. Thesis. Department of Applied Geophysics, Federal University of Technology, Akure. Nigeria. pp 73.

26. Pacey, A., and Cullis, A., (1986): Rainwater Harvesting: The Collection of Rainfall and runoff in Rural Areas. Intermediate Technology Publications. 103 - 105, Southampton row, London. pp 1 - 22

27. Price, Michael, (1985): Introducing Groundwater. George Allen and Unwin (publishers) Limited. London, United Kingdom. pp 170 - 172.

28. Raghunath, H. M., (1990): Groundwater. Second Edition. Wiley Oastern Limited. New Delhi, India. pp 1.

29. Rasmusen, J. Hans, (1976) : "Simplified Optimisation of Water Supply systems". Journal of the Environmental Engineering division. Proceedings of The ASCE. Volume 102, Number EE2. pp 313 - 327.

30. Sonuga, J. O., (1984) : "Water resources Management in Developing countries". Proceedings of the International Seminar on Water Resources Management Practices. Ilorin, Kwara State, Nigeria. pp 15 - 26.

31. Steel, E. W. And McGhee, J. Terence (1979). Water Supply and Sewerage. Fifth Edition. McGraw- Hill Incorporated. Pp 8, 11.

32. Stroud, K. A. (1982) : Engineering Mathematics : Programmes and Problems. Second Edition. McMillan Press Limited, London and Basingstoke. pp 367.

33. Su, Y., Mays, W. Larry, Duan, N., and Lansey, E. Kevin, (1987) : "Reliability - Based Optimisation Model of Water distribution Systems". Journal of Hydraulic Engineering. Volume 114, Number 12. pp 1539 - 1556.

34. Templeman, Andrew, (1982): Civil engineering systems. The McMillan Press Limited. London and Basingstoke. pp 1 - 37.

35. Todd, K. David, (1980): Groundwater hydrology. John Wiley and Sons incorporated. New York. pp 1 - 19.

36. Twort, A. C., Law, F. M., and Crowley, F. W., (1987): Water Supply. Third Edition. Arnolds International Student Edition. pp 369 - 378, 400 - 405.

37. Walker, Rodger, (1987): Water supply, Treatment and Distribution. Prentice - Hall Incorporated. Englewood Cliffs, New Jersey. pp 1 - 14.

38. Walski, M. Thomas, (1987): Analysis of Water distribution systems. CBS Publishers and Distributors. Delhi, India. pp 30 - 36, 76 - 90.

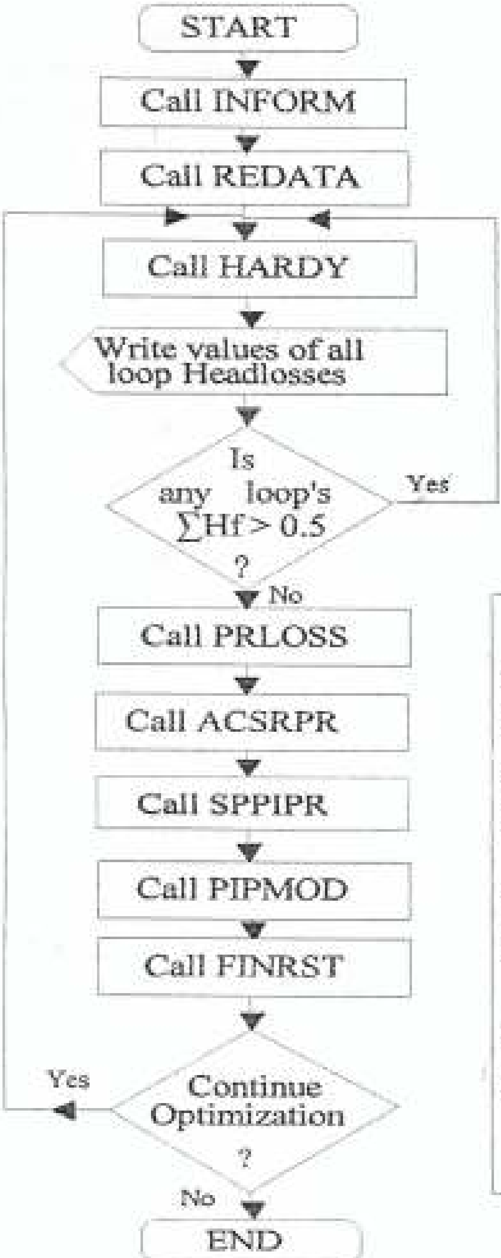
39. Watanatada, Thawat, (1973) : "Least - cost Design of water distribution systems". Journal of the Hydraulics Division. Proceedings of The ASCE. Volume 99, Number HY9. pp 1497 - 1513.

40. Wright, B. Forrest, (1977): Rural water Supply and Sanitation. Third edition. Robert E. Krieger

APPENDIX A

Figure A1: Summary of Main Program

This flowchart represents a detailed summary of all the operations within the optimisation model. The computer model is broken into eigth (8) subroutines, each representing a major operation. The details of each operation is shown in the subsequent flowcharts.



LEGEND

INFORM = Information Subroutine

REDATA = Read Data Subroutine

HARDY = Hardy Cross Subroutine

∑Hf = Summation of all Headlosses within loop

PRLOSS = Pipe Pressure Head Loss Subroutine

SPPIPR = Surplus Pipe Pressure Head Subroutine

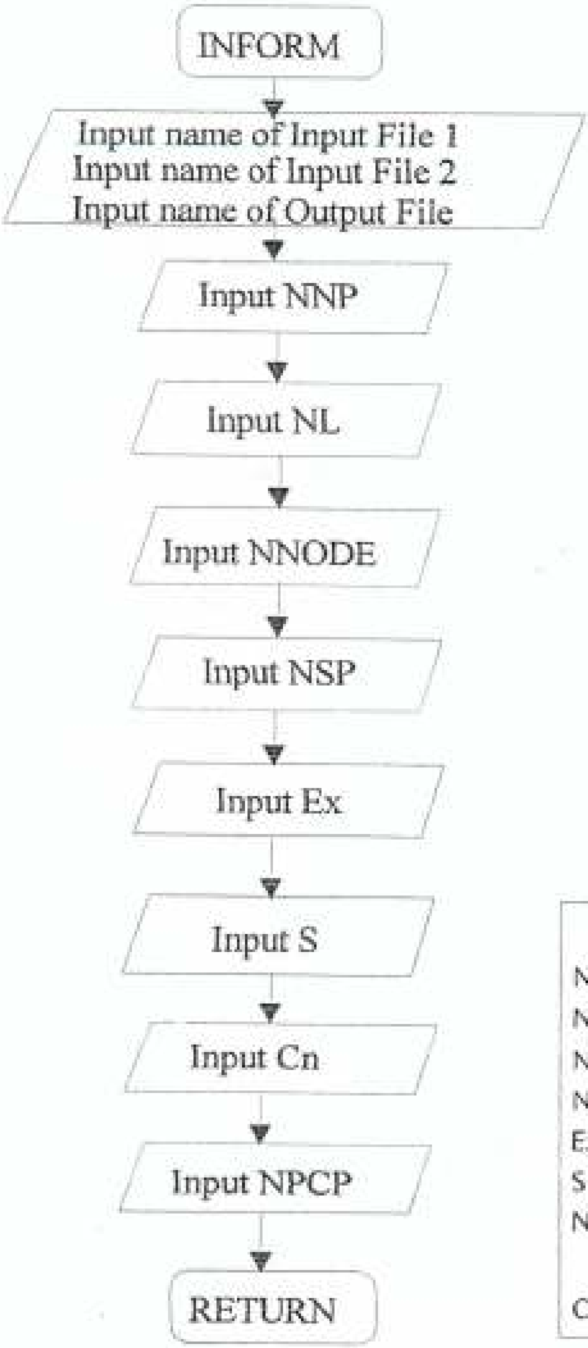
PIPMOD = Pipe Modification Subroutine

ACSRPR = Actual Service Pressure Head Subroutine

FINRST = Final Result Subroutine

Figure A2: Information Subroutine

In this subroutine, the quantities of all the various data to be processed (e.g. number of pipes), along with the names of the output and input files are requested.



LEGEND

NNP = number of Pipes in Network

NL = Number of loops in Network

NNODE = Number of Nodes in Network

NSP = Number of available commercial pipes

Ex = Energy cost over an x year period

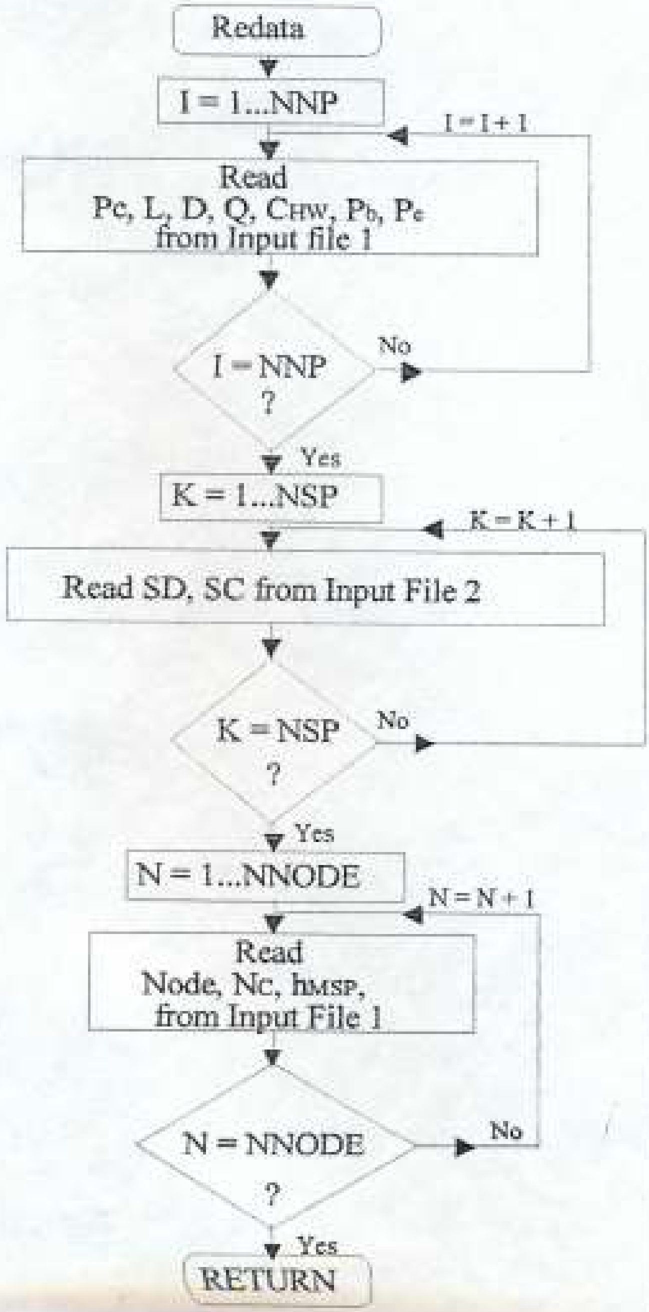
S = Source Node

NPCP = Total number of pipes on the Critical Path

Cn= Critical node

Figure A3: Read data Subroutine

This subroutine requests the details of all the data given in the Information subroutine, and transfers some of these details from the system's temporary memory to the output file.

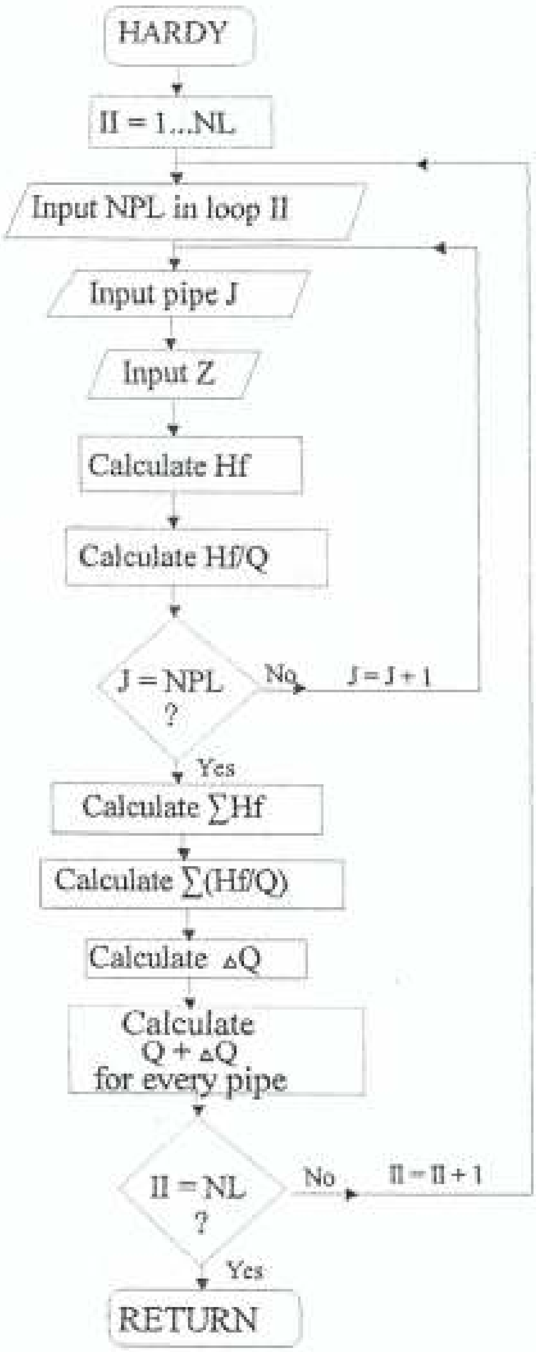


LEGEND

P_c = Pipe label
L = Length of Pipe
D = Diameter of Pipe
Q = Pipe Flow
C_{HW} = Hazen William's coefficient
P_b = Node at Pipe beginning
P_e = Node at Pipe end
I = Pipe counter
N = Node counter
S_D = Available Standard commercial diameter
S_C = Standard cost of Pipe
N_C = Node Consumption
h_{MSP} = Node Minimum Service Pressure head
NODE = Node label
K = Standard commercial pipe counter

Figure A4: Hardy Cross Subroutine

The Hardy Cross iterative procedure for correcting and balancing headlosses and flows around loops is shown in this flowchart.

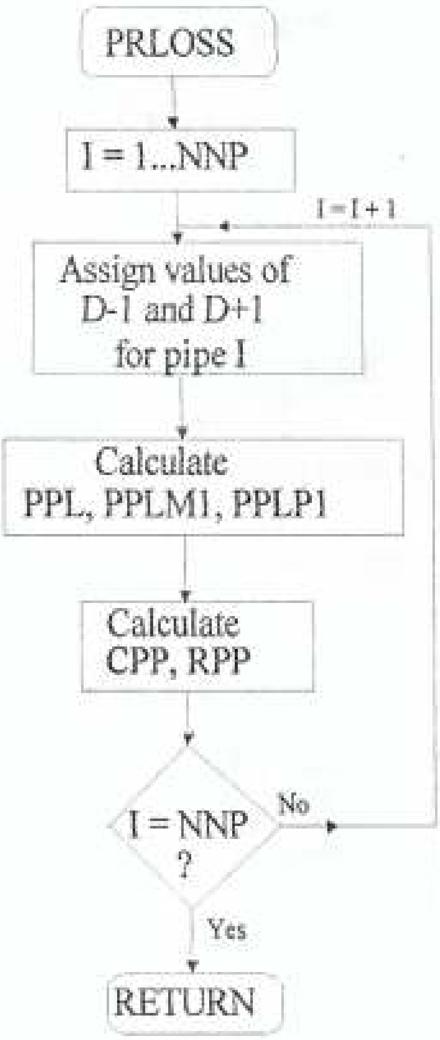


LEGEND

II = Loop counter
J = Pipe in loop
NPL = Number of pipes on loop II
∑ = Summation
Hl = Headloss in pipe
Z = Direction of flow of water in pipe
ΔQ = Headloss correction factor

Figure A5: Pressure Loss Subroutine

This subroutine computes the pressure head losses for all the pipes within the network for initial diameter, D, next smaller available diameter, D-1, and the next larger available diameter, D+1.



LEGEND

D-1 = Next smaller available commercial pipe diameter

D+1 = Next larger available commercial pipe diameter

PPL = Pipe pressure head loss for diameter, D

PPLM1 = pipe pressure head loss for diameter, D-1

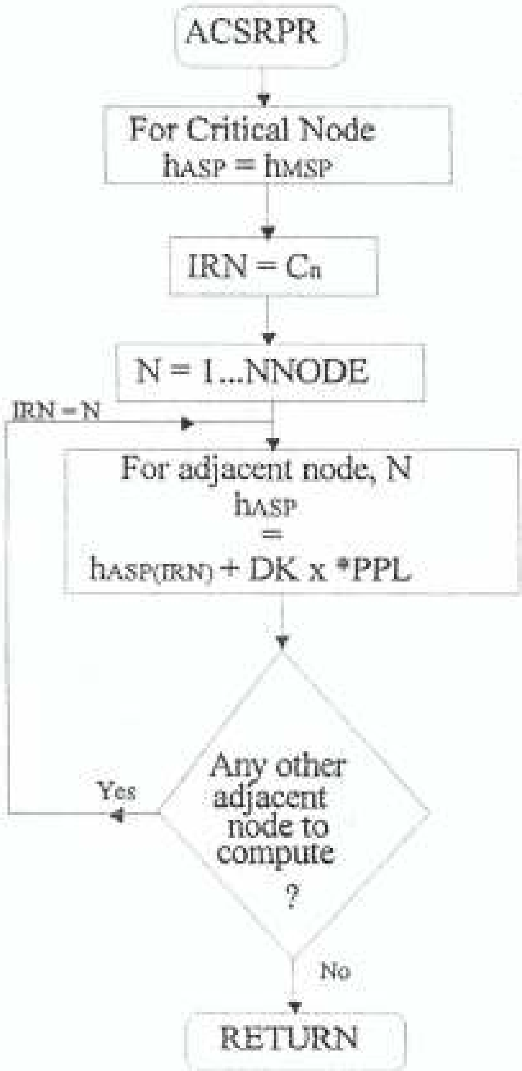
PPLP1 = Pipe pressure head loss for diameter, D+1

CPP = Increased pipe diameter

RPP = Reduced pipe diameter

Figure A6: Actual Service Pressure Head Subroutine

This subroutine calculates the actual service pressure head at all nodes within the distribution network beginning from the critical to source node.



LEGEND

hASP = Actual Service Pressure head

IRN = Arbitrary variable

**DK = Direction of pipe flow

**Sign convention

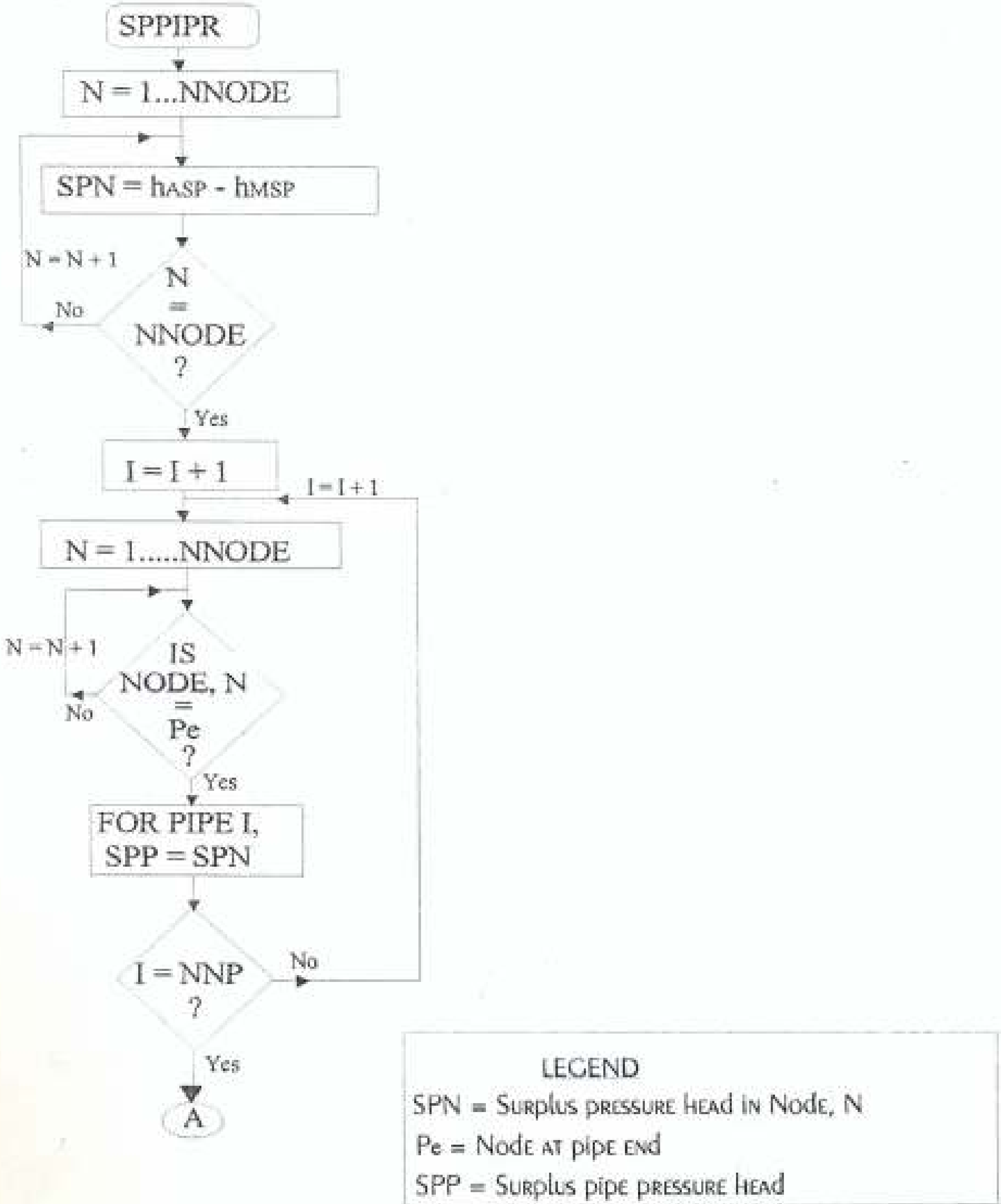
+ve: Flow away from the node to be computed

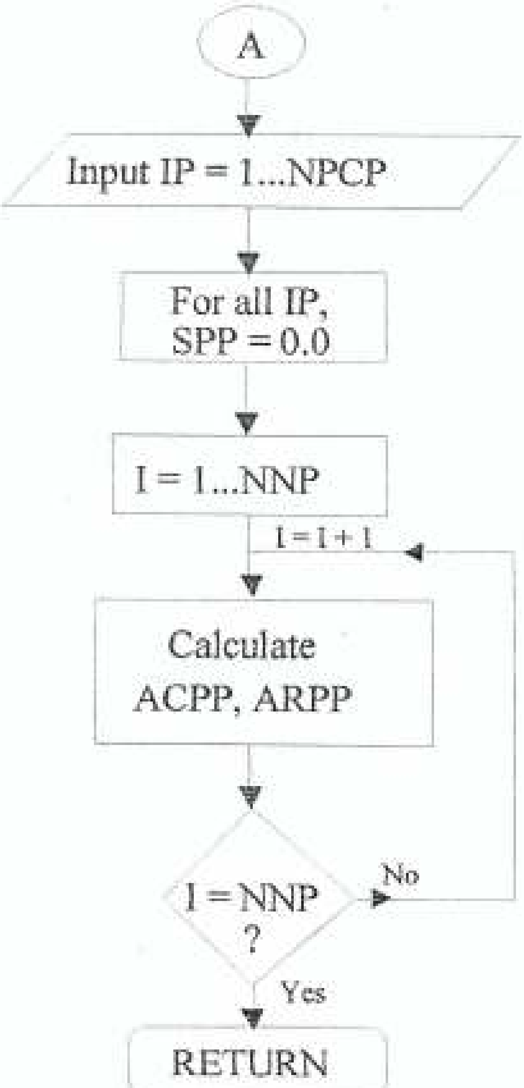
-ve: Flow towards node to be computed

*PPL = Pressure loss of pipe between the two nodes

Figure A7: Surplus Pipe Pressure Subroutine

Pipes on the critical link are assigned a value of zero surplus pipe pressure head, while surplus pipe pressure head for pipes on the non-critical links are calculated for diameters D, D-1, and D+1.





LEGEND

ACPP = Actual Increased pipe pressure head

ARPP = Actual Reduced pipe pressure head

IP = Critical path pipe counter

Figure A8: Pipe Modification Subroutine

Marginal energy and pipe costs for each pipe are calculated. The process then recommends and modifies pipe diameters towards an optimal solution.

legend

Mecx = Marginal Energy cost in x years

Tecx = Total Energy cost in x years

Pc = Pipe cost

RPCC = Reduced pipe cost

CPCC = Increased pipe cost

Sc = Standard cost of pipe of diameter, D

Sc-1 = Standard cost of pipe of diameter, D-1

Sc+1 = Standard cost of pipe of diameter, D+1

Tc = Total cost of Network

MECC = Increased Marginal Energy Cost

MECR = Reduced Marginal Energy cost

PCA = Additional Pipe costs

PCS = Pipe Cost Savings

TPCC = Total Pipe costs in the Network

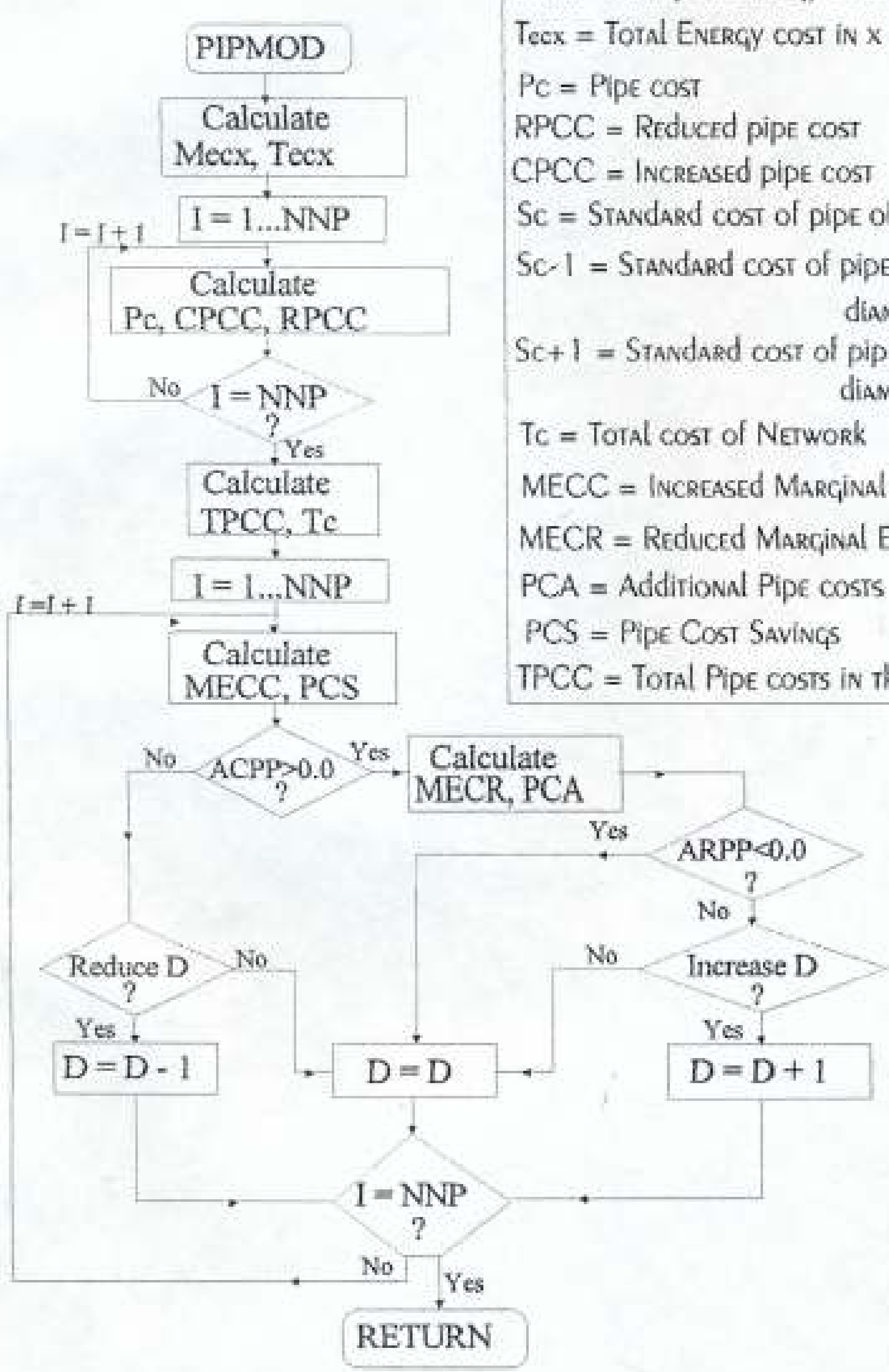
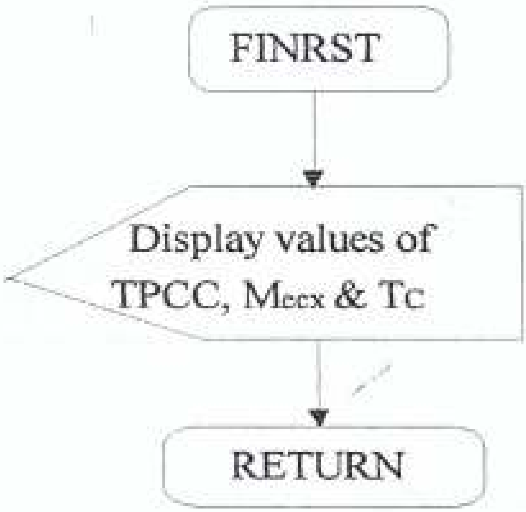


Figure A9: Final Result Subroutine

The Final Result subroutine displays the values of total pipe and energy costs for the distribution network.



```
**************************************************************************
*     SUMMARY OF MAIN PROGRAM THAT INCORPORATES BOTH THE               *
*          HARDY CROSS METHOD AND THE PIPE MODIFICATION                 *
*                          ALGORITHM                                      *
**************************************************************************
   PROGRAM        PIPE
    CHARACTER PC*2,LPC*2,LPS*2,PCP*2,CDEC*25,RDEC*25,IREP*1,
   +        ANS*1,REPLY*1,CDER*1,RDER*1,INFILE*10,OUFILE*10,
   +        PIPCOM*10
    INTEGER PE,PB,CN
    REAL LPF,LHF,LHFQ,LDQ,NC,NMSP,MECC,MECR
    DIMENSION PC(100),PL(100),PD(100),PQ(100),PCHW(100),LPC(100),
   + LPS(100),LPSS(100),LHF(100),LPF(100),SLHF(100),LHFQ(100),
   + SLHFQ(100),LDQ(100),PB(100),PE(100),SD(100),SC(100),NODE(100),
   + NC(100),NMSP(100),PPL(100),ASP(100),SPN(100),PCC(100),
   + PDM1(100),PDP1(100),PPLM1(100),PPLP1(100),CPP(100),RPP(100),
   + SPP(100),ACPP(100),ARPP(100),CPCC(100),RPCC(100),MECC(100),
   + PCS(100),MECR(100),PCA(100),PCPN(100),PCP(100),CDEC(100),
   + RDEC(100)
C CALL INFORMATION SUBROUTINE
     CALL INFORM(INFILE,PIPCOM,OUFILE,NNP,NL,NSP,E25,ISN,CN,NPCP,
   +   NNODE)
    OPEN(UNIT = 1, FILE = INFILE, ACCESS = 'SEQUENTIAL')
    OPEN(UNIT = 2, FILE = OUFILE, ACCESS = 'SEQUENTIAL')
    OPEN(UNIT = 3, FILE = PIPCOM, ACCESS = 'SEQUENTIAL')
C CALL THE READ DATA SUBROUTINE:
     CALL REDATA(NNP,NSP,NNODE,PC,PL,PD,PQ,PCHW,PB,PE,
   +   SD,SC,NODE,NC,NMSP)
C CALL THE HARDY CROSS SUBROUTINE:
  3 CALL HARDY(NL,NLP,NNP,LHF,PQ,PC,PL,PCHW,PD,SLHF)
DO 100 II = 1,NL
    WRITE(*,*)'SUM OF PIPE HEADLOSS FOR LOOP',II,'IS',SLHF(II)
100 CONTINUE
    WRITE(*,*)'IF THE SUMMATION OF PIPE HEADLOSSES IN ANY LOOP '
    WRITE(*,*)'IS GREATER THAN 0.5, THAT PARTICULAR LOOP IS NOT'
    WRITE(*,*)'       BALANCED. KINDLY RETURN TO LOOP 1 TO '
    WRITE(*,*)'             CORRECT ALL THE LOOPS AGAIN'
    WRITE(*,*)' WOULD YOU LIKE TO RETURN BACK FOR CORRECTION ?'
    WRITE(*,*)'             TYPE Y FOR YES AND N FOR NO'
    READ(*,'(A)') REPLY
    IF(REPLY.EQ.'Y') GO TO 3
C CALL THE PRESSURE LOSS SUBROUTINE:
    CALL PRLOSS(NNP,NSP,PD,SD,PPL,PQ,PL,PCHW,CPP,RPP,PC,NC,
   +          PDP1,PDM1)
```

```
C CALL THE ACTUAL SERVICE PRESSURE SUBROUTINE:
      CALL ACSRPR(NNODE,NODE,NNP,ASP,CN,NMSP,PE,PB,PPL,PC,NC)
C CALL THE SURPLUS PIPE PRESSURE HEAD SUBROUTINE:
      CALL SPPIPR(NNODE,NNP,NPCP,ASP,NMSP,PE,PC,SPP,
     +              SPN,CPP,RPP,ACPP,ARPP)
C CALL THE PIPE MODIFICATION SUBROUTINE:
      CALL PIPMOD(NNP,NSP,E25,ASP,PD,PL,PDP1,SD,SC,
     +       MECC,ACPP,ARPP,PC,NC,ISN,TEC25,TPCC,TC)
C CALL THE FINAL RESULT SUBROUTINE:
      CALL FINRST(TPCC,TEC25,TC)
      WRITE(*,*)'WOULD YOU LIKE TO OPTIMISE YOUR ENTIRE NETWORK'
      WRITE(*,*)'                                AGAIN ?'
      WRITE(*,*)'              PRESS Y FOR YES AND N FOR NO'
      READ(*,'(A)') ANS
      IF(ANS.EQ.'Y') GO TO 3
      WRITE(*,*)'  THIS IS THE END OF THE OPTIMIZATION PROCESS.'
      WRITE(*,*)'                                SHALOM     '
       STOP
      END
     *
*******************************************************************************
*            THIS ENDS THE SUMMARY OF THE MAIN PROGRAM                        *
*******************************************************************************
```

```
&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&
&                THIS IS THE INFORMATION SUBROUTINE                  &
&          INFORMATION REQUESTED ARE: NAMES OF INPUT AND             &
&          OUTPUT FILES; NUMBER OF PIPES, LOOPS, STANDARD            &
&     AVAILABLE COMMERCIAL PIPES, CRITICAL PIPES, & NODES;           &
&     SOURCE AND CRITICAL NODES AND ENERGY COST FOR 25YRS            &
&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&

      SUBROUTINE INFORM(INFILE,PIPCOM,OUFILE,NNP,NL,NSP,E25,
     + ISN,CN,NPCP,NNODE)
      INTEGER CN
      CHARACTER INFILE*10,OUFILE*10,PIPCOM*10

      WRITE(*,10)
      WRITE(*,10)
      WRITE(*,10)
      WRITE(*,10)
      WRITE(*,*) 'ENTER INPUT FILE WITH PIPE CHARACTERISTICS '
      READ(*,'(A)') INFILE
      WRITE(*,10)
      WRITE(*,10)
      WRITE(*,*) 'ENTER  INPUT FILE WITH COMMERCIAL PIPES '
      READ(*,'(A)') PIPCOM
      WRITE(*,10)
      WRITE(*,10)
      WRITE(*,*) 'ENTER OUTPUT FILE '
      READ(*,'(A)') OUFILE
      WRITE(*,10)
      WRITE(*,10)
      WRITE(*,*)'HOW MANY PIPES IN THE NETWORK'
      READ(*,*) NNP
      WRITE(*,10)
      WRITE(*,10)
      WRITE(*,*)'HOW MANY LOOPS IN THE NETWORK'
      READ(*,*)NL
      WRITE(*,10)
      WRITE(*,10)
      WRITE(*,*)' INPUT THE NUMBER OF NODES'
      READ(*,*) NNODE
      WRITE(*,10)
      WRITE(*,10)
      WRITE(*,*)'INPUT THE NUMBER OF STANDARD PIPES'
      READ(*,*) NSP
      WRITE(*,10)
      WRITE(*,10)
```

```
      WRITE(*,*)'INPUT ENERGY COST OVER 25 YEAR PERIOD'
      READ(*,*) E25
      WRITE(*,10)
      WRITE(*,10)
      WRITE(*,*)'WHAT IS THE SOURCE NODE ? '
      READ(*,*) ISN
      WRITE(*,10)
      WRITE(*,10)
      WRITE(*,*)'WHAT IS THE CRITICAL NODE ?'
      READ(*,*) CN
      WRITE(*,10)
      WRITE(*,10)
      WRITE(*,*) 'HOW MANY PIPES ARE ALONG THE CRITICAL PATH'
      READ(*,*) NPCP
   10 FORMAT(///,'  ',///)
      RETURN
      END
*


&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&& &&
&                   THIS IS THE READ DATA SUBROUTINE                  &
&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&

      SUBROUTINE REDATA(NNP,NSP,NNODE,PC,PL,PD,PQ,PCHW,PB,PE,
     +   SD,SC,NODE,NC,NMSP)

      CHARACTER PC*2
      REAL NC,NMSP
      INTEGER PB,PE
      DIMENSION PC(100),PL(100),PD(100),PQ(100),PCHW(100),
     +   PB(100),PE(100), SD(100),SC(100),NODE(100),NC(100),
     +   NMSP(100)
      DO 2 I = 1,NNP
      READ(1,'(A)')PC(I)
      READ(1,*) PL(I),PD(I),PQ(I),PCHW(I),PB(I),PE(I)
      WRITE(*,10)PC(I),PL(I),PD(I),PQ(I),PCHW(I),PB(I),PE(I)
C READ FROM INPUT FILE 1, PIPE LABELS, LENGHTS, DIAMETERS,
C FLOWS, HAZEN WILLIAM'S COEFFICIENT, NODES AT PIPE BEGINNING,
C AND ENDING.
    2 CONTINUE
      WRITE(2,*)'                    '
      WRITE(2,*)'         TABLE 1      '
      WRITE(2,*)'
      WRITE(2,*)'STAND. DIAMETER  | STAND. COST IN NAIRA'
```

```
   WRITE(2,*)'----------------------------------------'
    DO 8 K = 1,NSP
    READ(3,*) SD(K),SC(K)
    WRITE(2,40) SD(K),SC(K)
C READ FROM INPUT FILE 2, STANDARD PIPE DIAMETERS AND THIER
C COSTS AND COPY THEM TO THE OUTPUT FILE
  8 CONTINUE
    WRITE(2,*)'----------------------------------------'
    WRITE(2,*)'                         '
    WRITE(2,*)'            TABLE 2              '
    WRITE(2,*)'            =======              '
    WRITE(2,*)'   NODE    | CONSUMPTION | MIN.SERVICE.PRESS'
    WRITE(2,*)'--------------------------------------------'
    DO 9 N = 1,NNODE
    READ(1,*) NODE(N),NC(N),NMSP(N)
    WRITE(2,30) NODE(N),NC(N),NMSP(N)
C READ FROM INPUT FILE 1 NODE LABELS, CONSUMPTIONS & MINIMUM
C SERVICE PRESSURES AND COPY THEM TO THE OUPUT FILE.
  9 CONTINUE
    WRITE(2,*)'--------------------------------------------'
 10 FORMAT(1X,A2,3X,2('|',1X,F6.2,1X),'|',F13.8,'|',
   +                   1X,F6.2,2X,2('|',2X,I3,1X))
 30 FORMAT(3X,I2,5X,'|',2X,F10.6,1X,'|',6X,F6.3)
 40 FORMAT(3X,F7.3,7X,'|',8X,F12.3)
    RETURN
    END
   *
```

&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&

&   THIS IS THE HARDY CROSS SUBROUTINE   &

&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&

```
    SUBROUTINE HARDY(NL,NLP,NNP,LHF,PQ,PC,PL,PCHW,PD,SLHF)

    CHARACTER LPS*2,PC*2,LPC*2
    REAL SLHF,SLHFQ,LPF,LHFQ,LDQ
    REAL LHF
    DIMENSION LHF(100),PQ(100),LPS(100),SLHF(100),SLHFQ(100),
   +   LPC(100),LPF(100),LHFQ(100),LDQ(100),PC(100),
   +   PL(100),PCHW(100),PD(100)
 93 CONTINUE
C THE HARDY CROSS PROCESS FOR EACH LOOP WITHIN THE NETWORK
    WRITE(2,*)'            TABLE 3      '
    WRITE(2,*)'            =======      '
```

```
      DO 3 II = 1,NL
      WRITE(2,*)'                    '
      WRITE(2,*)'       RESULT OF LOOP',II
      WRITE(2,*)'       ========================'
      WRITE(*,11)
      WRITE(*,11)
      WRITE(*,*)'HOW MANY PIPES IN LOOP',II
      READ(*,*) NLP
      SLHF(II) = 0.0
      SLHFQ(II) = 0.0
      WRITE(2,*)' PIPE |      HF       |    HF/Q      '
      WRITE(2,*)'-----------------------------------------------'
      DO 4 J = 1,NLP
      WRITE(*,11)
      WRITE(*,11)
      WRITE(*,*)'WRITE LABEL OF PIPE ',J, 'IN LOOP',II
      READ(*,'(A)') LPC(J)
      WRITE(*,11)
      WRITE(*,11)
      WRITE(*,*)'INPUT THE DIRECTION OF PIPE',J,'IN LOOP',II
      WRITE(*,*)'       ( P for (+) and N for (-) )'
      READ(*,'(A)')LPS(J)
      IF( LPS(J).NE.'N')GO TO 21
      LPF(J) = -1.0
   21 IF( LPS(J).NE.'P')GO TO 22
      LPF(J) = 1.0
   22 DO 5 I = 1,NNP
      IF( LPC(J).NE.PC(I) ) GO TO 5
      WRITE(*,*) PQ(I)
      LHF(J) = (10.7*PL(I)*PQ(I)**1.852)/((PCHW(I)**1.852) *
     +(PD(I)**4.87))
      LHF(J) = LPF(J) * LHF(J)
      LHFQ(J) = ABS(LHF(J)/PQ(I))
      WRITE(2,20) LPC(J),LHF(J), LHFQ(J)
      SLHF(II) = SLHF(II) + LHF(J)
      SLHFQ(II) = SLHFQ(II) + LHFQ(J)
    5 CONTINUE
    4 CONTINUE
      WRITE(2,*)'-----------------------------------------------'
C CALCULATION OF THE LOOP CORRECTION FACTOR AND CORRECTION
C OF EACH PIPE FLOW
      WRITE(2,*)'  (a). SUM OF HF =',SLHF(II)
      WRITE(2,*)'  (b). SUM OF HF/Q =',SLHFQ(II)
      LDQ(II) = -1.0*SLHF(II)/(1.852*SLHFQ(II))
      WRITE(2,*)'  (c). CORRECTION, DQ =',LDQ(II)
```

```
      WRITE(2,*)'                                          '
      WRITE(2,*)'      CORRECTION TO PIPES AFTER ITERATION '
      WRITE(2,*)'      ==================================== '
      WRITE(2,*)'PIPE  |LENG.(m)|DIAM.(m)| FLOW(l/s)  | CHW   '
      WRITE(2,*)'-------------------------------------------------'
      DO 6 J = 1,NLP
      DO 7 I = 1,NNP
      IF( LPC(J).NE.PC(I) ) GO TO 7
      PQ(I)= LPF(J)*PQ(I) + LDQ(II)
      WRITE(2,10) PC(I),PL(I),PD(I),PQ(I),PCHW(I)
      PQ(I) = ABS(PQ(I))
   7 CONTINUE
   6 CONTINUE
      WRITE(2,*)'-------------------------------------------------'
   3 CONTINUE
  10 FORMAT(1X,A2,3X,2('|',1X,F6.2,1X),'|',F13.8,'|',
     +                1X,F6.2,2X,2('|',2X,I3,1X))
  20 FORMAT(3X,A2,2X,'|',1X,F15.8,1X,'|',1X,F15.8)
  11 FORMAT(///,'  ',///)
      RETURN
      END
*

&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&
&                THIS IS THE PRESSURE LOSS SUBROUTINE              &
&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&

      SUBROUTINE PRLOSS(NNP,NSP,PD,SD,PPL,PQ,PL,PCHW,CPP,RPP,PC,
     +         NC,PDP1,PDM1)

      CHARACTER PC*2
      REAL NC
      DIMENSION PD(100),SD(100),PDM1(100),PDP1(100),PPL(100),
     +     PQ(100),PL(100),PCHW(100),PPLM1(100),PPLP1(100),
     +     CPP(100),RPP(100),NC(100),PC(100)
C ASSIGN TO EACH PIPE THE NEXT AVAILABLE SMALLER(D-1) AND
C LARGER(D+1)DIAMETER
      WRITE(2,*)'                                  '
      WRITE(2,*)'                TABLE 4           '
      WRITE(2,*)'                =======           '
      WRITE(2,*)'REDUCED DIAM |ACTUAL DIAM.| INCREASED DIAM'
      WRITE(2,*)'---------------------------------------------'
      DO 35 I = 1,NNP
      WRITE(2,90) PDM1(I),PD(I),PDP1(I)
```

```
 35 CONTINUE
    WRITE(2,*)'---------------------------------------------'
C CALCULATE PRESSURE LOSSES FOR ALL PIPES, GIVEN DIAMETER D,
C D-1 & D+1
    WRITE(2,*)'                                          '
    WRITE(2,*)'                TABLE 5                   '
    WRITE(2,*)'                =======                   '
    WRITE(2,*)'   PIPE   |PRESSURE LOSS|PRESSURE LOSS|PRESSURE LOSS'
    WRITE(2,*)'          |    (D-1)    |     (D)     |    (D+1)     '
    WRITE(2,*)'--------------------------------------------------'
    DO 11 I = 1,NNP
    DO 31 IS = 1,NSP
    IF(PD(I).NE.SD(IS))GO TO 31
    PDM1(I) = SD(IS-1)
    PDP1(I) = SD(IS+1)
 31 CONTINUE
    PPL(I)=((PQ(I)*PL(I)**.54)/(.277*PCHW(I)*PD(I)**2.63))**1.8519
    PPLM1(I)=((PQ(I)*PL(I)**.54)/(.277*PCHW(I)*PDM1(I)**2.63))**1.8519
    PPLP1(I)=((PQ(I)*PL(I)**.54)/(.277*PCHW(I)*PDP1(I)**2.63))**1.8519
    CPP(I) = PPLM1(I) - PPL(I)
    RPP(I) = PPL(I) - PPLP1(I)
    WRITE(2,70) PC(I),PPLM1(I),PPL(I),PPLP1(I)
 11 CONTINUE
    WRITE(2,*)'--------------------------------------------------'
 70 FORMAT(4X,A2,4X,'|',2(1X,F10.4,1X,'|'),1X,F10.4)
 90 FORMAT(6X,F6.2,7X,'|',2X,F6.2,1X,'|',4X,F6.2)
    RETURN
    END
*
```

```
&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&
&     THIS IS THE ACTUAL SERVICE PRESSURE HEAD SUBROUTINE     &
&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&

    SUBROUTINE ACSRPR(NNODE,NODE,NNP,ASP,CN,NMSP,PE,PB,PPL,
   +                 PC, NC)
    REAL NMSP, NC
    INTEGER CN,PE,PB
    CHARACTER IREP*2,PC*2
    DIMENSION NODE(100),NMSP(100),PE(100),PB(100),PPL(100),
   +          ASP(100),PC(100),NC(100)
C CALCULATE THE ACTUAL SERVICE PRESSURE FOR EACH NODE
C STARTING FROM THE CRITICAL NODE. THE CRITICAL NODE ASSUMES
C AN ACTUAL SERVICE PRESSURE EQUAL TO IT'S MINIMUM SERVICE
```

```
C PRESSURE VALUE.
      IRN = CN
      ASP(IRN) = NMSP(CN)
      WRITE(2,*) 'ASP FOR CRITL. NODE',CN,' IS',ASP(CN)
      WRITE(*,11)
      WRITE(*,11)
 19 WRITE(*,*)'ACTUAL PESSURE OF WHICH ADJACENT NODE'
      WRITE(*,*)'     DO YOU WANT TO COMPUTE ? '
      DO 12 I =  1,NNP
      IF(IRN.NE.PE(I)) GO TO 13
      WRITE(*,*) PB(I)
 13 IF(IRN.NE.PB(I)) GO TO 12
      WRITE(*,*) PE(I)
 12 CONTINUE
      READ(*,*) ICN
      WRITE(*,11)
      WRITE(*,11)
      WRITE(*,*)'            INPUT DIRECTION OF FLOW         '
      WRITE(*,*)'               SIGN CONVENTION:             '
      WRITE(*,*)'     +VE; FLOW AWAY FROM NODE TO BE COMPUTED'
      WRITE(*,*)'      -VE; FLOW TOWARDS NODE TO BE COMPUTED  '
      WRITE(*,*)'      IF POSITIVE TYPE 1; IF NEGATIVE TYPE 2'
      READ(*,*) K
      IF(K.EQ.1)GO TO 15
      DK = -1.0
      GO TO 16
 15 DK = 1.0
 16 DO 14 N = 1,NNODE
      IF(ICN.NE.NODE(N)) GO TO 14
      DO 17 I =  1,NNP
      IF(IRN.EQ.PE(I).AND.ICN.EQ.PB(I)
     +         .OR.
     +   IRN.EQ.PB(I).AND.ICN.EQ.PE(I)) GO TO 180
       GO TO 17
 18 WRITE(*,*)'THIS IS PIPE:', PC(I),'EN = ',N
      ASP(ICN) = ASP(IRN) + DK*PPL(I)
      WRITE(*,*) 'ASP FOR NODE',ICN,'IS',ASP(ICN)
 17 CONTINUE
 14 CONTINUE
       IRN = ICN
      WRITE(*,11)
      WRITE(*,11)
      WRITE(*,*)'   MORE NODES TO BE COMPUTED ?'
      WRITE(*,*)' INPUT "Y" FOR YES AND "N" FOR NO'
```

```
      READ(*,'(A)') IREP
      IF(IREP.EQ.'Y') GO TO 19
      WRITE(2,*)'                                 '
      WRITE(2,*)'          TABLE 6                '
      WRITE(2,*)'          ========               '

      WRITE(2,*)'NODE |  MIN.S.P  |   ACTUAL.S.P '
      WRITE(2,*)'--------------------------------'
      DO 76 N = 1,NNODE
      WRITE(2,75) NODE(N),NMSP(N),ASP(N)
 76   CONTINUE
      WRITE(2,*)'------------------------------'
 75   FORMAT(1X,I2,2X,'|',1X,F9.5,1X,'|',2X,F14.5)
 11   FORMAT(///,'  ',///)
      RETURN
      END
*
```

```
&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&
&    THIS IS THE SURPLUS PIPE PRESSURE HEAD SUBROUTINE       &
&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&
```

```
      SUBROUTINE SPPIPR(NNODE,NNP,NPCP,ASP,NMSP,PE,PC,SPP,
     +           SPN,CPP,RPP,ACPP,ARPP)

      CHARACTER PC*2,PCP*2
      INTEGER PE
      REAL NMSP
      DIMENSION ASP(100),NMSP(100),PE(100),PC(100),SPP(100),
     +     SPN(100),CPP(100),RPP(100),PCP(100),ACPP(100),
     +     ARPP(100)
C CALCULATE SURPLUS PRESSURE OF ALL NODES AND PIPES WITHIN
C THE NETWORK
      DO 23 N = 1,NNODE
      SPN(N) = ASP(N) - NMSP(N)
 23   CONTINUE
      DO 71 I = 1,NNP
      DO 71 N = 1,NNODE
      IF(PE(I).NE.N) GO TO 71
      SPP(I) = SPN(N)
 71   CONTINUE
C INPUT ALL THE PIPES ON THE CRITICAL PATH FROM SOURCE TO
C CRITICAL NODE ASSIGN A ZERO VALUE FOR SURPLUS PRESSURE FOR
C ALL PIPES ON THE CRITICAL PATH
```

```
      DO 33 IP = 1, NPCP
       WRITE(*,11)
       WRITE(*,11)
       WRITE(*,*)'INPUT NO',IP,' PIPE ALONG C.P.'
       READ(*,'(A)') PCP(IP)
       WRITE(2,'(A)')'PIPE:',PCP(IP)
      DO 34 I = 1,NNP

      IF(PCP(IP).NE.PC(I)) GO TO 34
      SPP(I) = 0.0
34    CONTINUE
33    CONTINUE
      WRITE(2,*)'           TABLE 7                 '
      WRITE(2,*)'           =======                 '
      WRITE(2,*)'PIPE |   SURPLUS PRESSURE'
      WRITE(2,*)'--------------------------'
      DO 36 I = 1,NNP
      ACPP(I) = CPP(I) - SPP(I)
      ARPP(I) = RPP(I) - SPP(I)
      WRITE(2,95) PC(I),SPP(I)
36    CONTINUE
      WRITE(2,*)'------------------------'
11    FORMAT(///,'  ',///)
95    FORMAT(2X,A2,1X,'|',1X,F14.5)
      RETURN
      END
*
```

```
&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&
&           THIS IS THE PIPE MODIFICATION SUBROUTINE             &
&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&

      SUBROUTINE PIPMOD(NNP,NSP,E25,ASP,PD,PL,PDP1,SD,SC,
     +      MECC,ACPP,ARPP,PC,NC,ISN,TEC25,TPCC,TC)

      CHARACTER CDEC*25,RDEC*25,CDER*1,RDER*1,PC*2
      REAL NMSP,MECC,MECR, NC
      DIMENSION PL(100),PD(100),SD(100),SC(100),ASP(100),PCC(100),
     + PDP1(100),ACPP(100),ARPP(100),CPCC(100),RPCC(100),MECC(100),
     + PCS(100),MECR(100),PCA(100),PCPN(100),PCP(100),CDEC(100),
     + RDEC(100),PC(100),NC(100)
C CALCULATE MARGINAL AND TOTAL ENERGY COST FOR 25 YEARS
      MEC25 = E25 * ABS(NC(ISN))
      TEC25 = MEC25 * ASP(ISN)
```

```
      TPCC = 0.0
C CALCULATE TOTAL PIPE COST FOR NETWORK
      DO 24 I = 1,NNP
      DO 25 IS = 1,NSP
      IF(PD(I).NE.SD(IS))GO TO 25
      PCC(I) = SC(IS) * PL(I)
      CPCC(I) = SC(IS-1)*PL(I)
      RPCC(I) = SC(IS+1)*PL(I)
      TPCC = TPCC + PCC(I)

   25 CONTINUE
   24 CONTINUE
      TC = TEC25 + TPCC
C RECOMMENDATIONS SUGGESTED FOR THE REDUCTION OR INCREASE
C OF MOST OR ALL OF THE PIPES WITHIN THE NETWORK.
      WRITE(2,*)'                                    '
      WRITE(2,*)'            TABLE 9a                '
      WRITE(2,*)'            ========                '
      WRITE(2,*)'PP|A.P.LSS|MG.ENG.CTS| PPE.CST | DECISION.'
      WRITE(2,*)'-----------------------------------------'
      DO 38 I = 1,NNP
      MECC(I) = ACPP(I) * MEC25
      PCS(I) = PCC(I) - CPCC(I)
      IF(ACPP(I).GE.0.0) GO TO 41
      WRITE(*,11)
      WRITE(*,11)
      WRITE(*,*)'PROGRAM RECOMMENDS THAT YOU REDUCE THE DIAM'
      WRITE(*,*)'     [',PD(I), ']OF PIPE[',PC(I), ']. DO YOU REALLY'
      WRITE(*,*)'            WANT TO REDUCE THIS DIAMETER ?.'
      WRITE(*,*)'              TYPE Y FOR YES AND N FOR NO'
      READ(*,'(A)') CDER
      IF(CDER.NE.'Y') GO TO 101
      CDEC(I) = 'REDUCE PIPE DIAMETER'
      DO 84 IS = 1,NSP
      IF(PD(I).NE.SD(IS)) GO TO 84
      PD(I) = SD(IS-1)
      WRITE(2,*)'                                    '
      WRITE(2,*)'                                    '
      WRITE(*,*)PD(I)
   84 CONTINUE
      GO TO 37
  101 CDEC(I) = ' DIAMETER OKAY '
      WRITE(2,*)'                                    '
      WRITE(2,*)'                                    '
      WRITE(*,*)PD(I)
```

```
      GO TO 37
41 CDEC(I) = ' WAIT '
37 WRITE(2,80)PC(I), ACPP(I), MECC(I), PCS(I), CDEC(I)
38 CONTINUE
      WRITE(2,*)'-----------------------------------------'
      WRITE(2,*)'            TABLE 9b                      '
      WRITE(2,*)'            ========                      '
      WRITE(2,*)'PP|A.P.LSS|MG.ENG.SAV|A.PP.CST | DECISION.'
      WRITE(2,*)'-----------------------------------------'
      DO 82 I = 1,NNP
      IF(ACPP(I).LT.0) GO TO 82

      MECR(I) = ARPP(I) * MEC25
      PCA(I) = RPCC(I) - PCC(I)
      IF(ARPP(I).LT.0.0) GO TO 43
      WRITE(*,11)
      WRITE(*,11)
      WRITE(*,*)'PROGRAM RECOMMENDS INCREASE OF THE DIAM'
      WRITE(*,*)'   [',PD(I), '] OF PIPE[',PC(I), ']''. DO YOU'
      WRITE(*,*)'      REALLY WANT TO INCREASE THIS DIAMETER ?.'
      WRITE(*,*)'           TYPE Y FOR YES AND N FOR NO'
      READ(*,'(A)') RDER
      IF (RDER.NE.'Y') GO TO 43
      RDEC(I) = 'INCREASE PIPE DIAMETER'
      WRITE(2,80)PC(I), ARPP(I), MECR(I), PCA(I), RDEC(I)
      DO 83 K = 1,NSP
      WRITE(2,*)'                                          '
      WRITE(2,*)'                                          '
      PD(I) = PDP1(I)
      WRITE(*,*)PD(I)
 83 CONTINUE
      GO TO 82
 43 RDEC(I) = 'DIAMETER OKAY'
      WRITE(2,80)PC(I), ARPP(I), MECR(I), PCA(I), RDEC(I)
      WRITE(2,*)'                                          '
      WRITE(*,*)PD(I)
 82 CONTINUE
      WRITE(2,*)'-----------------------------------------'
 11 FORMAT(///,'  ',///)
 80 FORMAT(A2,'|',F6.2,1X,'|',F10.2,'|',F9.2,'|',A25)
      RETURN
      END
*
```

&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&
& THIS IS THE FINAL RESULT SUBROUTINE &
&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&

```
SUBROUTINE FINRST(TPCC,TEC25,TC)
WRITE(2,*)'                                    '
WRITE(2,*)'==================================================================='
WRITE(2,*)'TOTAL PIPE COST,TPCC IS:[',TPCC,']NAIRA'
WRITE(2,*)'TOTAL ENERGY COST,TEC25 IS:[',TEC25,']NAIRA'
WRITE(2,*)'           THEREFORE                   '
WRITE(2,*)'THE TOTAL COST OF DISTRIBUTION SYSTEM,TC'
WRITE(2,*)'    FOR 25 YEARS IS:[',TC,']NAIRA'
WRITE(2,*)'==================================================================='
WRITE(*,11)
WRITE(*,11)
WRITE(*,*)'TOTAL PIPE COST,TPCC IS:[',TPCC,']NAIRA'
WRITE(*,*)'TOTAL ENERGY COST,TEC25 IS:[',TEC25,']NAIRA'
WRITE(*,*)'           THEREFORE                   '
WRITE(*,*)'THE TOTAL COST OF DISTRIBUTION SYSTEM,TC'
WRITE(*,*)'    FOR 25 YEARS IS:[',TC,']NAIRA'
11 FORMAT(///,'  ',///)
RETURN
END
```