IMPROVED DESIGN OF WATER DISTRIBUTION SYSTEM FOR ONDO STATE SPECIALIST HOSPITAL AKURE

BY

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CERTIFICATION

This is to certify that this work on Improved Design of Water Distribution System For Ondo State Specialist Hospital Akure was carried out and submitted by

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DEDICATION

Dedicated to the almighty God, the solid rock on which I stand, my sufficiency, the creator and giver of wisdom, knowledge and understanding.

And

To my loving mother; late Mrs. (Evang.) D.M. Filani.

ACKNOWLEDGEMENT

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Finally, I am grateful to the God almighty for He did not allow my vision of attaining a master degree to fall into illusion. I love you father.

C.O. Filani.

ABSTRACT

Water is one of the most important natural providence for the continuity of life. The groundwater stored in aquifers of the earth crust within the Ondo State Specialist Hospital Akure was explored and exploited by drilling two Boreholes, at the two locations recommended in the geophysical investigation report previously carried out, to the depth of 50m and 45m respectively. After completion of the drilling the boreholes were tested to determine their production capacities (the yield). To ascertain the degree of compliance of the quality of the water from the boreholes with the requirement specified by World Health Organization (W.H.O), samples were taken to the laboratory for analysis. From the result of the analysis it was discovered that the water is colourless and neutral to acid and alkaline indicators, and also free from all causative agents of water hardness. Hence, they conform to the standard of good quality drinking water as prescribed by the World Health Organization. The water consumption within the hospital was determined to be 99,500 litres per day; hence, a water supply scheme was designed to meet the hospital demand.

The pipeline network was designed using Hazen-Williams formula, and the designed system has branches (parallel system) into which the water flows for easy distribution within the hospital. The flow rate in each of the three branches was calculated to be $0.00465m^3/s$; $0.00717m^3/s$, 0.00468m3/s, and the head loss in each branch were determined using Hardy Cross technique to be 1.60m, 1.64m and 1.59m respectively. In the analysis of the pipeline design, the three forms of energy head, (potential energy due to its elevation, kinetic energy due to the velocity of the fluid and flow energy which represent the amount of work necessary to move the element of fluid), are properly accounted for using the general energy equation and were found to be; 7.8m as potential head, 0.71m and 2.50m are kinetic and flow energy respectively in 75mm diameter pipe, while 0.23m and 3.25m are kinetic and flow energy respectively in 100mm diameter pipe.

The frictional losses in the system were analyzed and were found to 7.58m. The choice of pipe for the design was made base on condition of operations such as physical, climatic and environmental. For the main trunk 100mm Un-plasticized Polyvinyl Chloride (UPVC) pipes were used while 25mm and 32mm Galvanized Iron (G.I) pipes (depending on the demand) were used to service the buildings.

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CHAPTER ONE

1.1 INTRODUCTION

The adequacy of water supply is of vital importance not only to individuals and families but also to the public institutions and to the nations. A supply of clean water is vital for life, but in a crowded world it is a commodity we cannot take for granted, (Elirketh, 1999). It is said that a nation that cannot provide enough water for its inhabitants is not yet on the path of greatness, (Sonuga, 1990). This underlines the thrust of the policy in the water supply section, which is to ensure that Nigeria is self sufficient in safe drinking water.

Normally the easiest and most convenient way to meet the public demand for water is to utilize surface water resources. Unfortunately, fresh water, rivers and lakes are less plentiful than may at first be imagined and, as a matter of fact, account for less than 0.01% of the world's total water, and less than 2% of the world's fresh water, (Colten, 1998). The sources that are available have often been overdeveloped or polluted. Groundwater, on the other hand, account for about 98% of the world's fresh water and is fairly well distributed throughout the world, (Hamill and Bell, 1986).

Groundwater is trapped inside the earth aquifers. An aquifer is defined as the water-bearing soil or rock that can release its water in sufficient quantities to make it economically feasible to be developed for water supply, (Martins, 1990). The aquifer can provide much of the fresh water for public, industrial and agricultural use. According to Hamill and Bell (1986), groundwater provides a reasonable constant supply, which is not likely to dry up under natural conditions, as surface sources may do, and it's often of quite a high quality. Groundwater has always been one of the most important sources of water supply. Virtually all parts of the earth are underlain by water, and wells have been constructed throughout recorded history to provide a water supply when surface water was not readily available. In early times, the water wells were excavated by hand and were rarely more than a few meters deep. Today, modern water wells are drilled and in some localities are more than 30m deep. (Robertson et. al., 1998).

The purification is provided by layers of porous materials acting as filters, which in some locations, run to depth of 30m or more. It has been discovered that not all the water beneath the earth's surface can be recovered from the water bearing formation in which it is found.

Some lie in rock formations so deep that pumping cost alone rule out its recovery. Other water-bearing formations resist recovering to varying degree and may even defy pumping efforts. While comparative figures of the amount of water available from the surface and ground sources cannot be used as true indicators of actual water resources, they do point to the fact that the available supply of groundwater is many times greater than the available supply of surface water. The Ondo State Specialist Hospital Akure, which is a case study for this thesis, needs constant water supply for 24hours per day for an effective health delivery. Unfortunately however, Water Corporation could not cope with this demand. Investigation revealed that the hospital gets water supply from the corporation once or twice in a week. This is a serious threat to the health of the hospital patients. Hence, this project is basically designed to alleviate the problems of water shortage at the State General Hospital. Because of the mineral contents of the soil which are soluble in water, treatment plant was designed and constructed along with the borehole to get a pure water supply to the hospital.

This project is basically designed to alleviate the problem of water shortage at the General Hospital by designing;

(1) Boreholes with submersible pumps.
(2) A water treatment plant.
(3) Overhead reservoir, and
(4) A pipeline network within the hospital.

1.2 PROBLEM IDENTIFICATION

The only source of water supply to the hospital at inception was the State Water Corporation. As expected, development brings expansion at the hospital and consequently increases the infrastructure and demand for services such as water supply within the hospital community. As a result, the State Water Corporation could no longer meet up with the demand for water supply in the hospital and in the entire state. This thus poses a problem to health delivery services in the hospital.

It has been observed further that the water pipeline previously in place at the Hospital were laid without any professional planning, but on priority basis which resulted into poor water supply and distribution within the hospital, and some of the water stored in the tank escape to the town through wrong connections.

1.3 OBJECTIVES OF THE PROJECT

The general objective of this research work is:

The design of an improved water distribution system.

This research work is set out to achieve the following specific objectives:

(1) To determine the water requirements within the hospital;

(2) To design a water supply scheme to meet the hospital demand;

(3) To exploit the underground water, by drilling boreholes based on the geophysical investigation report of the hospital;

(4) To design a network of pipeline for water supply and distribution within the hospital.

1.4 METHODOLOGY

1.4.1 THE PROJECT

This research work is on improved design of water distribution system.

Boreholes were drilled with pumps installed; water is pumped from the boreholes into two overhead tanks. The water is thereafter distributed through a pipeline network to various buildings within the hospital. The network is equipped with facilities such as fire hydrants, air valves, wash out valves and other fittings such as stop corks, elbows, tees, ferrule and saddles.

1.4.2 THE METHOD OF APPROACH

The following methods of approach are employed to meet the objectives identified above.

(1) Literature review on location of the Specialist Hospital, study of Geophysical Investigation Report, previous source of water supply, and the distribution methods.

(2) Construction of boreholes, which includes drilling, casing, installation of pumps, pump testing and quality tests.

(3) Evaluation of energy losses as water flow through the system.

(4) Design and construction of pipeline network.

(5) Comparison of flow rate of water, (calculated and actual), through the system.

CHAPTER TWO

2.1 LITERATURE REVIEW

The Specialist Hospital Akure is located at the southern section of the main town (Fig. 2.1) within a terrain that gently slopes to the south. Beyond the minor valley south of the hospital, there exists a water supply facility consisting of a battery of shallow wells operated by Ondo State Water Corporation.

The surface of the earth has a total area of 500,000,00km^2 out of that, water, in the form of oceans and lakes 0ccupies 71% of this area, i.e., 360,000,000km^2, whereas only 29% or 140,000,000km2 is the land on which we work and live, (Denis, 1983). Surprisingly, groundwater is trapped inside the earth aquifer of the 29% land, this ground water accounts for 98% of the world's fresh water and is fairly well distributed throughout the world as mentioned in sec 1.1. Nature, obviously, did not intend us to be short of water, and yet such is the tragic case in many parts of the world, because it is not always in the right location or it is not of usable quality. Sea water is of course, salty and although it is possible to remove the salt and make it drinkable, the cost of processing even a small quantity and transporting it to those places which need it makes the whole task impossible in practical terms. The desperate need for and importance of engineers cannot, therefore, be overstressed.

The water supply scheme, which is previously in place at the hospital, was neither designed nor planned; it was a product of several attempts made in the past to boost the supply of water for distribution/diversion into the hospital on priority basis, without any provision for treatment of raw water. The hospital is underlain by rocks of Migmatic Complex, in other words it is within the basement complex terrain of Nigeria, (Egwebe and Chuckwuma, 1993). According to Egwebe and Chuckwuma (1993), the basement complex of Nigeria consists of a wide variety of ancient metamorphic and igneous crystalline rocks. Because of their crystalline nature, they are known to have poor to nil primary porosity and therefore have a far reduced ground water potential compared to classic sedimentary rocks.

Ground water in basement complex rocks occurs mainly within the weathered overburden or along zones of fracture. 'Water is life' is the succinct slogan used by the Djibouts National Authority; it is a message which is powerful in its every simplicity (Kenneth, 2000). This simple fact makes the government of Japan to go into Rehabilitation of Water Distribution Pipeline in Damascus city from 1998 to 2000 (Jurgen, 2002)

2.2 GEOGRAPHICAL REPORT

A geographical investigation comprising Vertical Electrical Sounding (VES) was conducted to make hydrological inferences on the subsurface geology for the purpose of borehole siting in State Hospital, Akure, Ondo State, carried out by Ondo State Health Rehabilitation Project, and Sponsored by African Development Bank (A.D.B). (Egwebe and Chuckwuma, 1993).

According to Egwebe and Chuckwuma (1993) the hospital is within the basement complex terrain of Nigeria (Fig. 2.2). The basement complex of Nigeria consists of wide variety of ancient metamorphic and igneous crystalline nature, they are known to have poor to nil primary porosity and therefore have a far reduced Groundwater in Basement Complex rocks occur mainly within the weathered overburden or long zones of fracture. Geophysical studies for groundwater exploration in basement area, therefore, have as their aim the determination of the thickness of the overburden or the existence of fracture zones within the bedrock. Furthermore, the result reveals that;

i. The hospital has weathered fractured bedrock.

ii. An overburden of less than 10m.

iii. Low resistive bedrock (this implies that it is far from dryness).

The results for VES 1 and VES 2 are shown in Table 2.1 and 2.2 respectively.

Table 2.1 Results for VES 1.

Layers	True Resistivity (Ohms)	Thickness (m)	Probable Lithologies
1	55.0	0 – 6	Top soil with clayey sand
2	23.0	6 – 22	Weathered basement
3	136.0	22 – 38	Fractured basement
4	680.0	38-inf.	Basement complex

VES 1: From Table 2.1 we observed that at 38m the geological formation is fresh basement complex, the formation has no water retention property, hence, it was recommended that the depth of the borehole at this point should not be more than 50m for maximum extraction of water.

Source: Egwebe and Chuckwuma (1993). Geophysical Investigation Report for Borehole siting in Ondo State. Sponsored by African Development Bank (A.D.B)

Table 2.2 Results for VES 2

Layers	True Resistivity (Ohms)	Thickness (m)	Probable Lithologies
1	70.0	0- 3	Topsoil with clayey
2	45.0	3- 18	Weathered basement
3	210.0	18-32	Fractured basement
4	783.0	32-inf.	Basement complex

VES 2: From Table 2.2 we observed that at 38m the geological formation is fresh basement complex, the formation has no water retention property, hence, it was recommended that the depth of the borehole at this point should not be more than 50m for maximum extraction of water.

Source: Egwebe and Chuckwuma (1993). Geophysical Investigation Report for Borehole sitting in Ondo State. Sponsored by African Development Bank (A.D.B)

Based on the above, the study has shown that most of the area within the hospital complex have either weathered or fractured bedrock, which means that productive boreholes could be sited. It is apparent that one form or the order of water supply effort has been attempted previously at the hospital. The lack of water in most cases is not due to lack of good aquifer condition but rather because of improper well development.

From the foregoing therefore, a systematic and professional approach to the water supply problem, consisting of drilling of boreholes to depth of at least 45m and not more than 60m is recommended. The total number of boreholes will be dictated by the aquifer characteristics as determined by the pump tests. These boreholes should be cased down throughout the whole overburden section and left uncased for the bedrock section so as to gain as much yield. With these, the present water supply problem in the hospital compound can be eliminated.

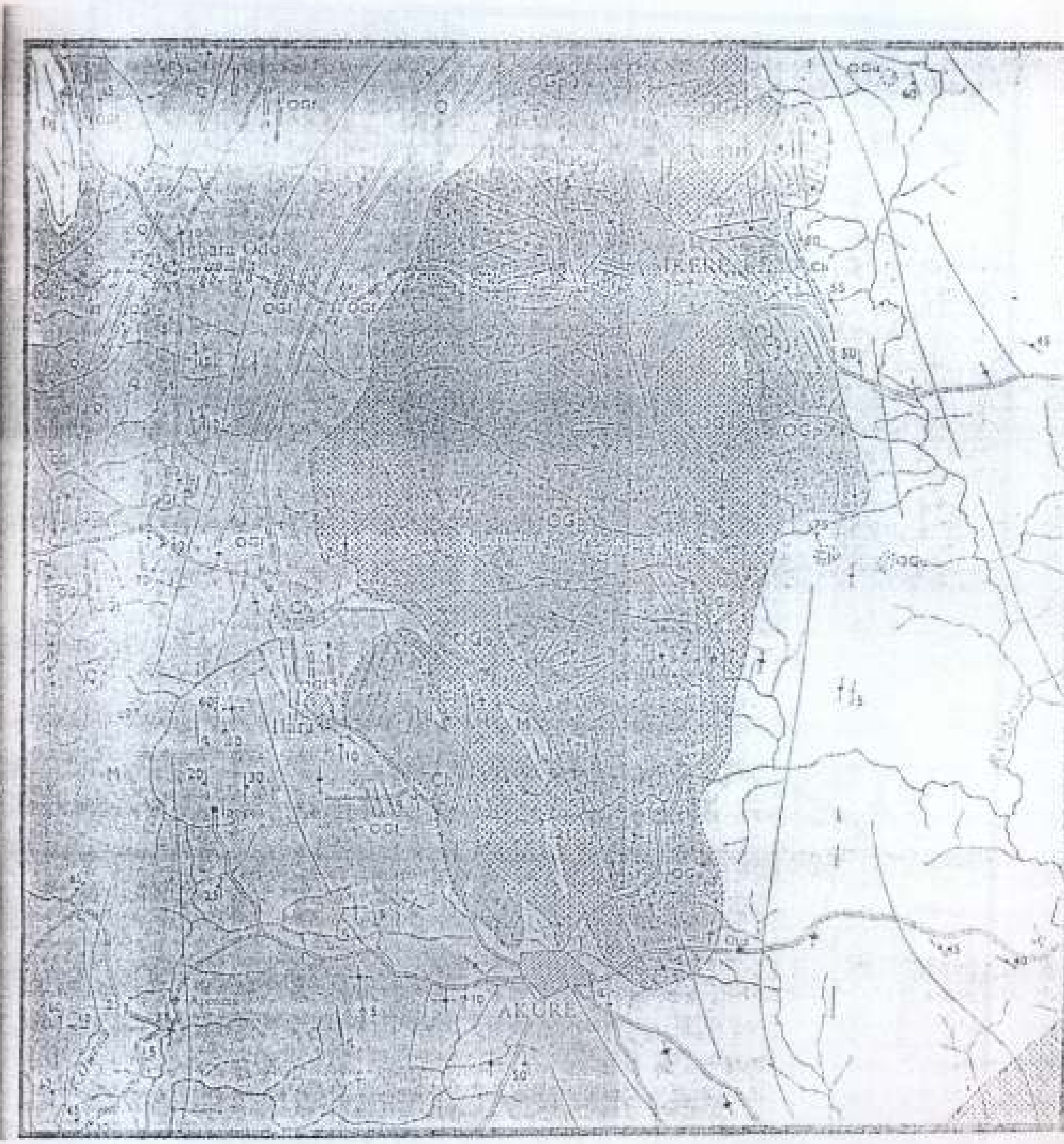


Figure 2.1 Geological map of the area around Akure, Ondo State (1:250,000 Scale)
Source: Egwebe and Chuckwuma, (1993). Geophysical Investigation Report for Borehole siting in Ondo State. Sponsored by African Development Bank (A.D.B)

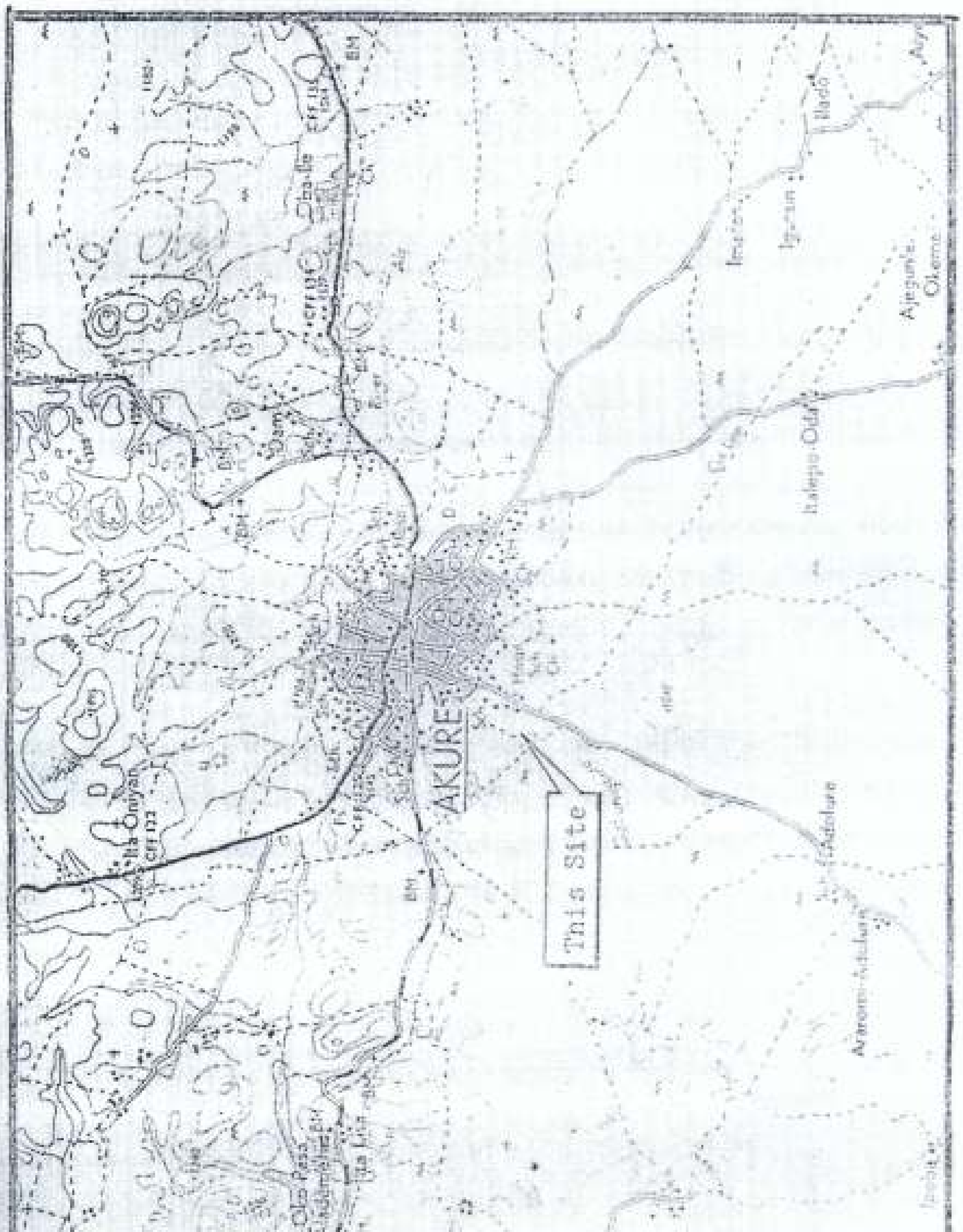


Figure 2.2 Topographical map showing the area around Akure, Ondo State (1:1000,000 Scale)
Source: Egwebe and Chuckwuma, (1993). Geophysical Investigation Report for Borehole siting in Ondo State. Sponsored by African Development Bank (A.D.B)

2.3 BACKGROUND THEORY

2.3.1 WATER CONSUMPTION AT THE HOSPITAL

Water consumption at the specialist hospital is a combination of about four different categories viz:

(i) Domestic

(ii) Trade

(iii) Public

(iv) Losses

(i) DOMESTIC: Water is been used domestically in the quarters within the hospital premises for in–house–drinking cooking, house cleaning and sanitation, car, clothes, washing and ablution. Domestic in–house demand varies widely from one house to another, even in identical housing. The "In–house'' demand is that used for domestic purposes, excluding garden watering and lawn sprinkling, but including washing cars and cleaning down outside yards.

(ii) Trade: As a health institution, the hospital has Wards, Theatre Labour rooms, Offices etc that require constant water supply. Allowances frequently adopted for water consumption in commercial and institutional establishment are as shown in the Table 2.3, (Twort et. al., 1984).

Table 2.3 Allowance frequently adopted for water consumption in commercial and industrial establishments

USAGE	CONSUMPTION
Wards/Theatre	190 1/d per bed
Offices	90 1/d per employee
Public	70 1/d per person
Residential	260 l/d per person

Source: www.searchyahoo.com/waterpipelinesanddistribution

(iii) PUBLIC: There is provision for fire-fighting, routine fire hydrant testing sewer flushing, which is normally so small when expressed as an average daily supply per capital during the year (Twort et. al., 1984). The main value of per capita consumption derived from a field test is the total consumption in all household surveyed (of a given class) divided by the total population counted in those households.

(iv) LOSSES: Losses can come in two different ways;

(a) Consumer wastage: - Leakages and wastages from consumer's premises, misuse or unnecessarily wastages from consumers.

(b) Distribution Losses:-leakages and overflow from service reservoirs, leakages from mains, service pipes and service pipes connections, leakages from valves, hydrants, and washouts.

Table 2.4 -WATER REQUIRED AT DIFERENT LOCATIONS IN THE HOSPITAL

LOCATION	POPULATION	REQUIREMENT IN LITTERS PER PRESON PER DAY	TOTAL REQUIREMENT IN LITTERS PER DAY
Male Surgical Ward	32 bed space	190	6,080
Male Medical Ward	32 bed space	190	6,080
Female Surgical Ward	32 bed space	190	6,080
Female Medical Ward	32 bed space	190	6,080
Children Ward	36 bed space	190	6,840
Maternity Ward	45 bed space	190	8,550
Theatre/Labour Room	20	190	3,800
Offices	550	90	49,500
Residential	24	260	6,240
TOTAL	**803**	**1,680**	**99,250**

From the table, it is shown that the total volume of water consumption at the hospital is approximately **99,250 litters per day**.
(Source: Physical data collected within the hospital in the year 2003).

2.4 NETWORK FEEDER AND TRUNK MAINS

The design of network feeder and trunk mains is mainly a planning exercise and largely empirical. Choices would have to be made concerning which source should supply each area, which routes are practicable for new mains and which are not, and how peak day demands are to be met.

To cope with this planning, we start by sketching possible mains layouts on a map of the area, on which have been marked the average and maximum day demands in each location, along which there are properties requiring a supply. Mains most frequently used for this are 100mm or 150mm diameter, smaller mains are sometimes used but this is not the best practice as anything less than 100mm diameter is unlikely to have adequate capacity to meet the minimum fire demand of $1m^3/m$ at any hydrant, (Twort et. al., 1984).

Considering the population density of the General Hospital involved in the project and the design of fire hydrant along with the pipeline, 100mm diameter polyethylene mains is specified. An important characteristic of distribution networks is that because of the interconnection of mains the pressure-drop versus flow characteristic does not follow the single pipeline law. For a single pipeline the head loss H is proportional to V^2 (or $V^{1.85}$ in the Hazen-Williams' formula), (Twort et. al., 1984), where V is the velocity of flow in the pipe. For a network H is proportional to Q where H is the head loss to a point in the network and Q is the total flow taken by the network.

Hence, if the demand in a network is doubled the pressure loss is doubled, and not quadrupled as implied by the v^2 law, which applies to single mains only.

2.5 FACTORS IN PIPELINE DESIGN

For many practical conditions the Hazen–Williams' formula, (Twort et. al., 1984),

$$H = 6.78L/d^{1.168}\,(V/C)^{1.85} \qquad (2.0)$$

For the flow of water in circular pipes is deservedly still popular for several reasons:

(i) It is well documented and easy to use.-

(ii) It has been in use for many years

(iii) There are numerous experimental results which indicate fairly reliable values of the coefficient C to take in most circumstances

(iv) It is reasonably accurate over a range of pipeline sizes and flows, which are widely experienced.

The formular, $H = [6.78L/d^{1.168}](V/C)^{1.85}$

can be rewritten as $V = 0.355Cd^{0.63}(H/L)^{0.5}$ (2.1)

Where L = Length of the pipeline (m)

V = Average velocity of flow (m/s)

d = Internal diameter of the pipe (m)

from equation (2.1), if C and H/L are constant then rate of flow

$$Q = \pi(d^2/4)V = Kd^{2.63} \quad (2.2)$$

Where K is a constant for any particular case.

Similarly, the Manning formula (Twort et. al., 1984), gives

$$Q = K d^{2.67}$$

Consequently, increasing the diameter of a pipeline by 30% doubles it flow capacity. For example, 32mm diameter pipes were used to supply water from the State Water Corporation into the entire hospital. Improving the flow capacity into the hospital will service the hospital better, increasing the diameter of the supply pipe can do this, and hence 50mm diameter pipe is specified, and for the trunk mains 100mm diameter pipe is specified.

Also, instead of the 19mm diameter pipes that were used to distribute the water within the hospital, 25mm and 32mm diameter pipes are specified depending on the daily demand of the building.

Furthermore, for any change in flow area, the principle of continuity applies. That is, the mass of fluid flowing past any section in a given amount of time is constant, if no fluid is added, stored or removed between the two sections, (say 1 and 2).

That is $m_1 = m_2$

Or $\rho_1 A_1 V_1 = \rho_2 A_2 V_2$ (2.3)

Where, m = mass of fluid

V = velocity of flow

ρ = density of fluid

A = cross sectional area of flow

This is valid for all fluids, whether gas or liquid.

Since the fluid in the pipe is a liquid, which is considered incompressible, then the term ρ_1 and ρ_2 in equation (2.3) are equal.

Then, $A_1 V_1 = A_2 V_2$ (2.4)

Equation (2.4) is the continuity equation as applied to liquids; and it states that for steady flow the volume flow rate is the same at any section.

But $A = \rho D^2/4$

Substitute for A, and rearrange equation (2.4) to give

$$V_2 = [(D_1^2)/(D_2^2)]\ V_1 \quad (2.5)$$

Where D = diameter of pipe, and V = velocity of flow

If we increase D_2, V_2 decreases. This implies that as the flow area increases, velocity decreases irrespective of pressure and elevation.

The design layout is simplified in Figure 2.3. This shows that there are three branches in the network. Each branch has its velocity and flow rate which is to be determined.

When three or more branches occur in a pipe flow system, it is called a network. Networks are indeterminate because there are more unknown factors, and then there are independent equations relating the factors. In Fig.2.3 there are three unknown velocities, one in each pipe.

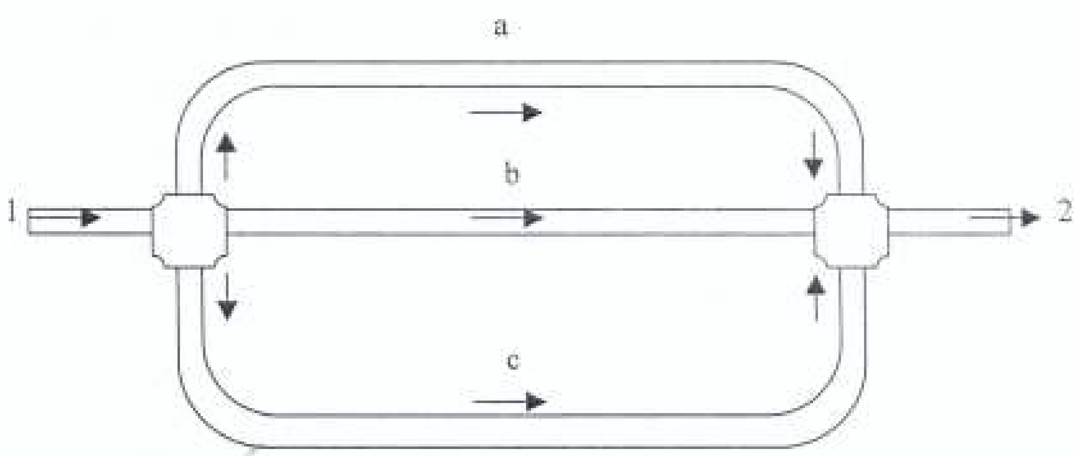


Figure 2.3 Network with three branches.

In this network design, the pressure required throughout the network for various operating conditions, are determined such that: -

1. Continuity is satisfied. That is the flow into a junction of the network is equal to the flow out of the junction. This is satisfied for all junctions.
2. The head loss between any two junctions is the same regardless of the part in the series of pipes taken to get to one junction point to the other. This requirement results because pressure must be continuous throughout the network (pressure can not have two values at a given point). This condition leads to the conclusion that the algebraic loss of head losses around a given loop must be equal to zero.
3. The flow and the head losses are consistent with the appropriate velocity-head-loss equation.

The method of solution given by Cross, (Roberson et. al., 1998), is presented as follows; The flow is first distributed throughout the network so that the continuity requirement (requirement 1) is satisfied for all junctions.

The equations available to describe the system in Fig.3.1 regarding head losses are

$$Q_1 = Q_2 = Q_a + Q_b + Q_c \qquad (2.6)$$

$$h_{L1\text{-}2} = h_a + h_b + h_c \qquad (2.7)$$

Equation (2.6) is the statement of continuity for steady flow in a parallel system. In equation (2.7), the term $h_{L1\text{-}2}$ is the energy loss per unit of fluid between points 1 and 2 in the main lines. The term h_a, h_b, and h_c are the energy losses per unit of fluid in each branch of the system. From the energy equation we can demonstrate that these quantities must all be equal;

$$P_1/\gamma + Z_1 + V^2_1/2g + h_A + h_R - h_L = P_2/\gamma + Z_2 + V^2_2/2g$$

Where, h_A = energy added by pump

h_R = energy removed by fluid motor

since there is no pump or motor in the pipeline network, h_A and h_R is zero

Recall that the expression

$$P_1/\gamma + Z + V^2/2g = E \text{ (total head)}$$

this represents the energy possessed by each unit of the fluid at a particular point in a system. Substituting E into the energy equation gives

$$E_1 - h_L = E_2, \qquad (h_A \ \& \ h_R = 0)$$

And $\quad h_L = E_1 - E_2$

The term h_L represents the loss of head between points 1 and 2 in Figure 2.3, each unit of fluid has the same total head at the point where the flow branches. As the fluid flows through the branches, some energy is lost. But at the point where the fluid rejoins, the total head of each unit of fluid is again the same. We can therefore draw the conclusion that the loss of head is the same, regardless of which part is taken between points 1 and 2. This conclusion is stated mathematically in equation (2.7)

The amount of fluid flowing in a particular branch of a parallel system is dependent on the resistance to flow in that branch relative to the resistance in other branches. The fluid will tend to follow the part of least resistance. A third independent equation is required to solve explicitly for the three velocities.

A more rational approach employing an iteration procedure has been developed by Hardy Cross, (Robert, 1990), to determine the third equation. This procedure converges on the correct flow rates quite rapidly. The Cross technique requires that the head loss terms for each pipe in the system be expressed in the form

$$h = KQ^n \tag{2.8}$$

where K = equivalent resistance to flow for the entire pipe and

Q = flow rate in the pipe.

Isolating loop 1a2b1 in figure 2.3 as shown in figure 2.4

In this loop, the loss of head in the clockwise sense will be given by

$$\Sigma h_{Lc} = h_{LAB} + h_{LBC} \tag{2.9}$$

$$= \Sigma KQ^n_c$$

The loss of head for the loop in the counter clockwise sense is

$$\Sigma h_{Lcc} = \Sigma KQ^n_{cc}$$

For a solution, the clockwise and counter clockwise head losses have to be equal

or $\Sigma h_{Lc} = \Sigma h_{Lcc}$

$$\Sigma KQ^n_c = \Sigma KQ^n_{cc}$$

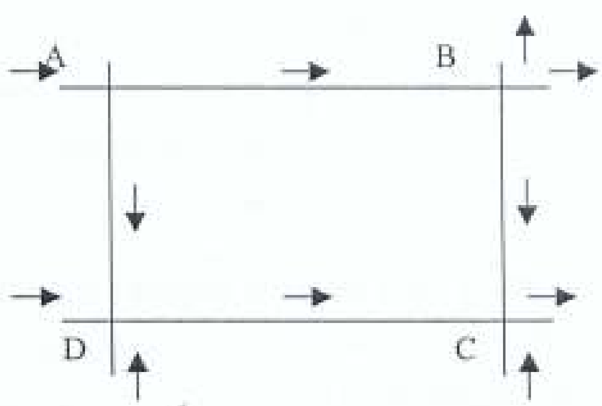


Figure 2.4; A typical loop of a pipeline network

The first guess for flow in the network is undoubtedly in error, as earlier stated; therefore, a correction in discharge, ΔQ, will have to be applied to satisfy the head loss requirement. If the clockwise head loss is greater than the counter clockwise head loss, ΔQ would have to be applied in the counter clockwise sense. That is, ΔQ is subtracted from the clockwise flows and added to the counter clockwise flows:

$$\Sigma K\,(Q_c - \Delta Q)^n = \Sigma K\,(Q_{cc} + \Delta Q)^n \tag{2.10}$$

Expanding the equation on either side of equation (2.10) and including only two terms of the expansion, we obtain

$$\Sigma K(Q^n_c - nQ^{n-1}_c\,\Delta Q) = \Sigma K(Q^n_{cc} + nQ^{n-1}_{cc}\,\Delta Q)^n$$

Then solving for ΔQ, we get

$$\Delta Q = \frac{\Sigma KQ^{n}_{c} - \Sigma KQ^{n}_{cc}}{\Sigma nKQ^{n-1}_{c} + \Sigma nKQ^{n-1}_{cc}} \quad (2.11)$$

Thus if ΔQ as computed from equation (2.11) is positive, the correction is applied in a counter clockwise sense (add ΔQ to counter clockwise flows and subtract it from clockwise flows).

A different ΔQ is computed for each loop of the network and applied to the pipes. Some pipes have two ΔQ applied because they will be common to two loops. The first of corrections usually will not yield the final desired result because the solution is approached only by successive approximations. Thus the corrections are applied successively until the corrections are negligible.

2.6 ENERGY LOSSES IN THE NETWORK

For a flow system such as that designed for Ondo State Specialist Hospital Akure, which has mechanical devices, valves and fittings, there are definitely some energy losses and addition between the sections of interest.

The two major energy losses in the network are: -

1. Energy loss due to friction
2. Energy loss due to valves and fittings (minor losses).

2.6.1 ENERGY LOSSES DUE TO FRICTION

As a fluid flow through the network of pipeline, energy loss occurs because of internal friction with the fluid. As indicated by the general energy equation, these energy loss result in the decrease in pressure between two points in the flow system.

In the general energy equation

$$P_1/\gamma + Z_1 + v^2_1/2g + h_A - h_R - h_L = P_2/\gamma + Z_2 + v^2_2/2g \quad (2.12)$$

Where h_L = Energy loss from the system due to friction in pipes or minor losses due to valves and fittings.

h_A = Energy added to the fluid with a mechanical device such as a pump.

h_R = Energy removed from the fluid by a mechanical device such as fluid motor.

If there is no mechanical device between the sections of interest, h_A and h_R will be zero and can be left out of the equation.

One component of the energy loss is due to friction in the flowing fluid.

Friction is proportional to the velocity head of the flow and to the ratio of the length to the diameter of the flow stream, for the case of flow in pipes this is expressed mathematically as.

$$h_L = f \times L/D \times v^2/2g \quad (2.13)$$

Equation (2.13) is referred to as Darcy's equation, where

h_L = energy loss due to friction (W.m/m)

L = length of flow stream (m)

D = diameter of pipe (m)

v = average velocity of flow (m/s)

f = friction factor.

The dimensionless number f is dependent on the Reynolds number and the relative roughness of the pipe, which are also dimensionless. The relative roughness is the ratio of the pipe diameter (D) to the average pipe wall roughness (ε), (Robert 1990). The behavior of a fluid, particularly with regard to energy losses, is quite dependent on whether the flow is laminar or turbulent. The character of a flow in a round pipe depends on four variables; namely, Fluid density ρ, fluid viscosity μ, pipe diameter D, and average velocity of flow v. Osborne Reynolds demonstrated that laminar or turbulent flow can be predicted if the magnitude of a dimensionless number now called Reynolds numbers (R_e) is known. (Robert, 1990). It is defined as

$$R_e = vD\rho/\mu \qquad (2.14)$$

Where v = average velocity of flow

μ = dynamic viscosity @ 32^0c

ρ = density of water

D = diameter of pipe

According to Robert, for practical applications we find that if the Reynolds number for the flow is less than 2000, the flow will be laminar. Also, if the Reynolds number is greater than 4000, the flow is said to be turbulent.

Table 2.5 Pipe Roughness – Design Value

Materials	Roughness ε (m)
Glass, Plastic	Smooth
Copper, brass, Lead (tubing)	$1.5\text{x}10^{-6}$
Cast – Iron-Uncoated	$2.4\text{x}10^{-4}$
Cast – Iron- asphalt coated	$1.2\text{x}10^{-4}$
Commercial steel or welded steel	$4.6\text{x}10^{-5}$
Wrought Iron	$4.6\text{x}10^{-5}$
Riveted steel	$1.8\text{x}10^{-3}$
Concrete	$1.2\text{x}10^{-3}$

(Source: Robert L. Mott 1990 Applied fluid mechanics 3rd edition Pg.273)

The friction factor f is evaluated using the moody diagram shown in Fig 2.5. The diagram shows friction factor f plotted against the Reynolds number R_e, with a series of parametric curves related to the relative roughness (D/ε).

Beyond $R_e = 4000$, the flow is turbulent. A procedure described by J.W. Murdock (1976) solves the problem. First an approximate value (f_a) for f can be calculated from:

$$f_a = 0.0055\ \{1 + [20{,}000/(D/\varepsilon)\ +\ (10^6 / N_R)]^{1/3}\} \qquad (2.15)$$

Then we can use equation (3.15) to find the actual f as follows:

$$1/\sqrt{f} = -2\log_{10}\{[1/3.7(D/\varepsilon)] + [2.51/(N_R\sqrt{f_a})]\} \qquad (2.16)$$

Having it in mind that we are interested in reducing the energy lose as much as possible, hence if we have large Reynolds number of flow as well as large relative roughness (D/ε) the friction factor f decreases on the Moody's diagram.

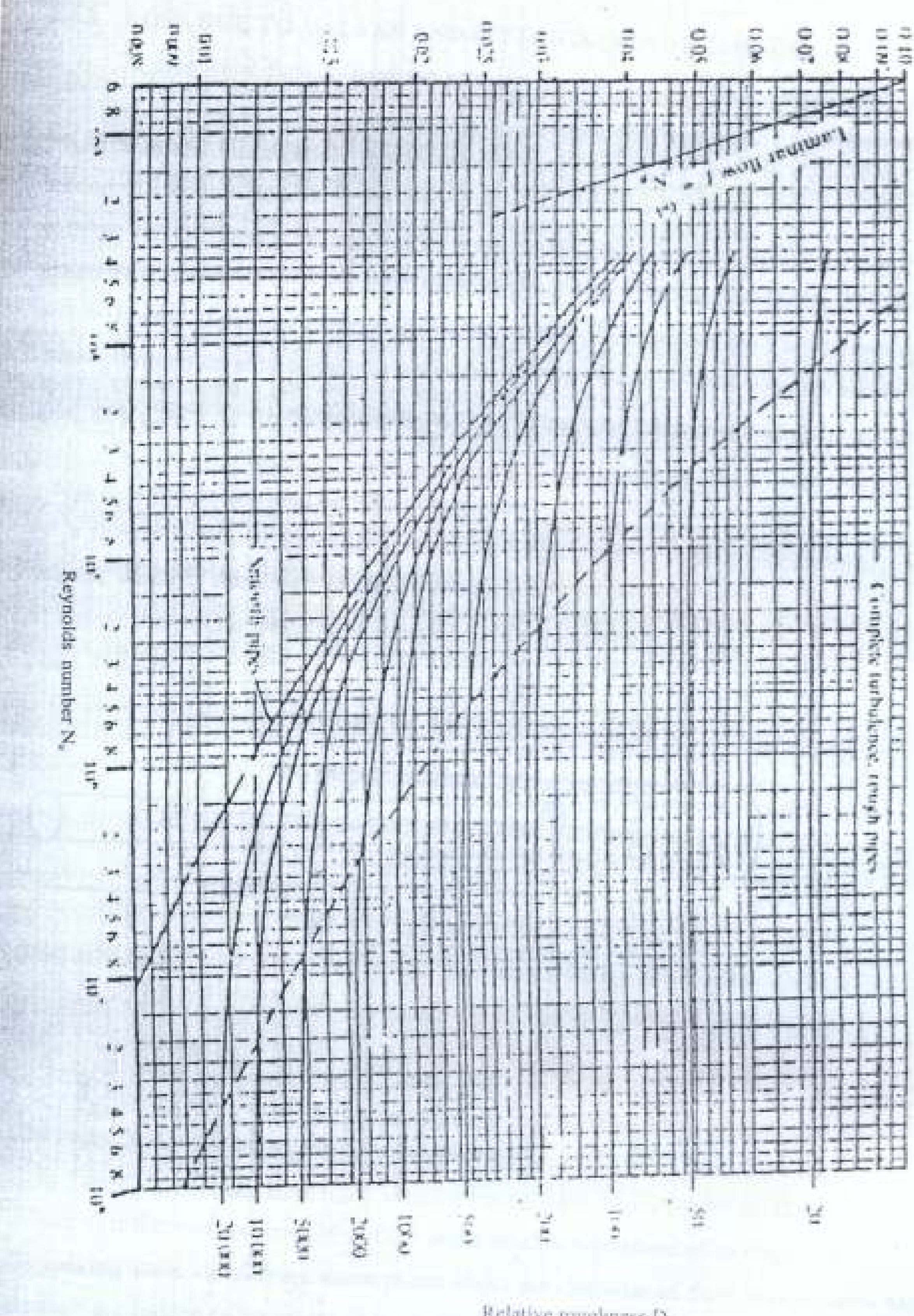


Figure 3.3 Moody's diagram

Source: Robert L. Mott (1990) Applied Fluid Mechanics 3rd Edition 274

2.6.2 LOSS DUE TO VALVES AND FITTINGS (MINOR LOSSES)

In most pipe flow system the primary energy loss is due to pipe friction, as earlier described in section 2.6.1. Other types of losses are usually small by comparison, and are therefore referred to as minor losses. Minor losses occur when there is a change in the cross section of the flow path or in the direction of flow or where the flow path is obstructed, as with a valve. Energy is lost under these conditions due to rather complex physical phenomena.

Energy losses are proportional to the velocity head of the fluid as it flows around an elbow, through an enlargement or contraction of the flow section or through a valve. Values for energy losses in the network were reported in terms of a loss coefficient C_L, as follows:

$$h_L = C_L\,(v^2/2g) \qquad (2.17)$$

where h_L = Minor loss, and C_L = loss coefficient.

As a fluid flows from a smaller pipe into a larger pipe through a sudden enlargement, its velocity abruptly decreases, causing turbulence which generates an energy loss. The amount of turbulence and the amount of energy loss is dependent on the ratio of the sizes of the two pipes. Fig.2.6

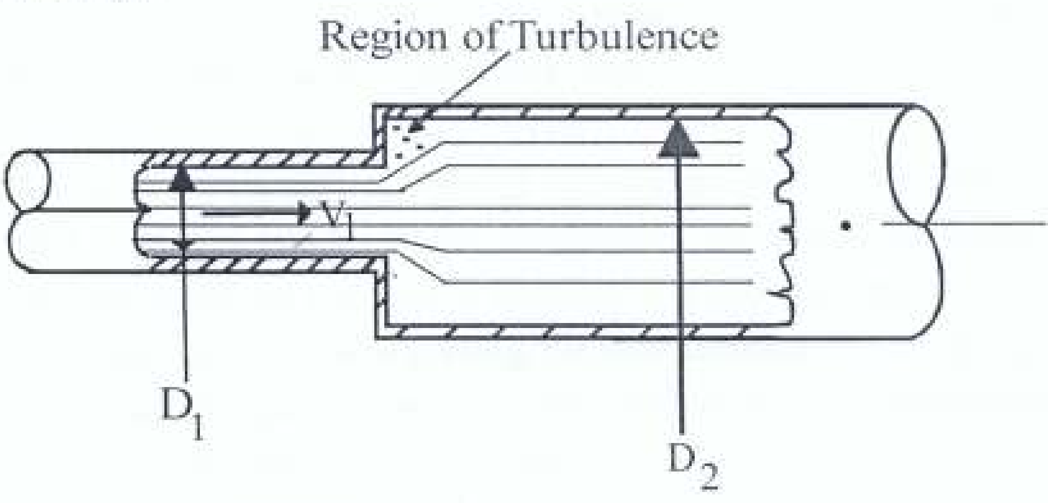


Fig. 2.6: Sudden enlargement.

The minor loss is calculated from the equation

$$h_L = C_L\,(v^2{}_1/2g) \qquad (2.18)$$

Where v_1 is the average velocity of flow in the smaller pipe ahead of an enlargement.

By making some simplifying assumptions about the character of flow in pipes as it expands through the sudden enlargement. It is possible to analytically predict the value of C_L from the following equation.

$$C_L = [1 - (A_1/A_2)\,]^2 = [1 - (D_1/D_2)^2]^{\,2} \qquad (2.19)$$

2.6.3 Entrance Loss

A special case of a contraction occurs when a fluid flow from a relatively large reservoir or tank into a pipe. The fluid must accelerate from a negligible velocity to the flow velocity in the pipe. The ease with which the acceleration is accomplished determines the amount of energy loss, and therefore, the value of the entrance loss coefficient is dependent on the geometry of the entrance. For the well rounded entrance, no vena contraction is formed and the energy loss is quite small due to surface friction only. We can calculate the energy loss at an entrance from

$$h_L = C_L (V^2_2/2g) \quad (2.20)$$

Where V^2 is the velocity of flow in the pipe.

$C_L = 0.04$ for well rounded inlet, (Robert 1990)

2.6.4 Valves and fittings

Energy losses are calculated in terms of the dimensionless loss coefficient C_L for changes in the flow across section as discussed in the preceding sections.

The valve of C_L is unaffected by the physical size of such devices. This is not true for valves and fittings such as elbows. For these types of devices, the energy loss can be calculated using the equivalent technique from the equation below.

$$h_L = f \times (L_e/D) \times (v^2/2g) \quad (2.21)$$

There is an obvious similarity between this equation and Darcy's equation, (equation 3.13), which is used to calculate friction loss in pipes. The term L_e in equation (2.21) is the equivalent length and is defined as the length of straight pipe, which would have the same total energy loss as the valve or fitting. For many commercially available valves and fittings the ratio L_e/D is approximately constant for a particular type of device regardless of size. The resistance to flow then varies with the valve of the friction factor f.

There is a similarity between equation (2.17) and equation (2.21), in which the loss coefficient C_L is used to calculate energy losses. It is sometimes convenient to convert an equivalent length to a loss coefficient and vice versa. This can be simply done.

The two equations for energy loss are

$$h_L = C_L (V^2/2g) \quad (2.22)$$

$$h_L = f (L_e/D) (v^2/2g) \quad (2.23)$$

In each of these equations, if v is the velocity in the pipe to which the valve or fittings is attached, then $C_L = f (L_e/D)$.

Manufacturers differ widely in the methods they use to report energy loss data for valves and fittings. Some report the equivalent length ratio L_e/D, where D is inside diameter of the standard pipe for which the value was designed. Table 2.6 lists respectively values of L_e/D for typical plumbing and process control valves and fittings. Equation (2.23) would be used to calculate the energy loss. Other manufacturers report the loss coefficient C_L (sometimes called k) for a particular type and size of device, equation (2.22) must then be used.

Table 2.6: Resistance in Valves and Fittings expressed as equivalent length in pipe diameters, L_e/D (m/m)

Type	Equivalent length in pipe diameters L_e/D
Globe valve - fully open	340
Angle valve - fully open	145
Gate valve - fully open	13
- ¾ open	35
- ½ open	160
- ¼ open	900
Check valve - swing type	135
- ball type	150
Butterfly valve – fully open	40
90^0 Standard elbow	30
90^0 Long radius elbow	20
90^0 Street elbow	50
45^0 Standard elbow	16
45^0Street elbow	26
Close return bend	50
Standard tee with flow through run	20
Standard tee with flow through branch	60

(Source: Robert L. Mott 1990. Applied Fluid Mechanics 3rd edition 274)

CHAPTER THREE

3..1 DESIGN AND INSTALLATION OF THE NETWORK

Now lets us take a section of the designed network mains as shown in Fig.3.1. The water is transferred at say 32°c from the overhead tank through 75mm diameter wrought iron pipe having a total length of 7.8m into 100mm diameter plastic pipe having a total length of 124.6m assuming the washout valve is open and the water runs into a ditch

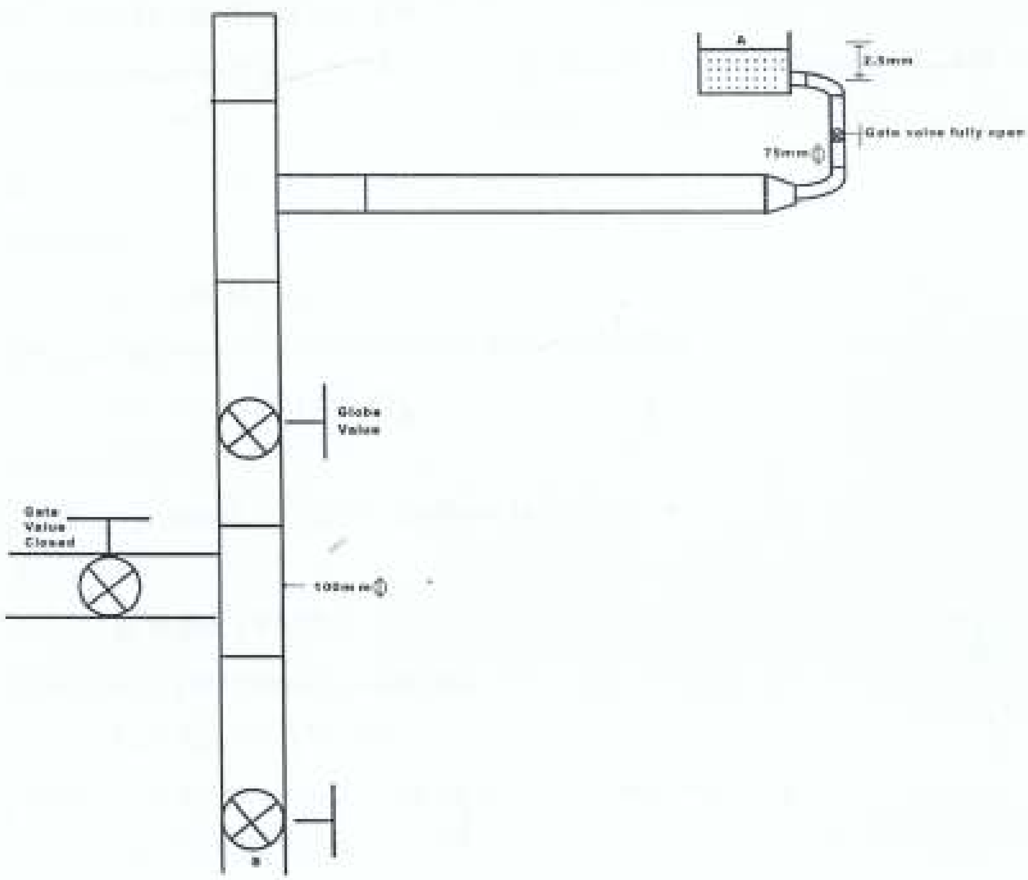


Figure 3.1 A section of the designed network

From equation (2.12), the energy equation is

$P_A/\gamma + Z_A + v^2{}_A/2g + h_A - h_R - h_L = P_B/\gamma + Z_B + v^2{}_A/2g$

Where h_A and $h_R = 0$

$P_A = P_B =$ (Atmospheric)

V_A is approximately zero. ($V_A = 0$)

Therefore the equation reduces to

$$Z_A - h_L = Z_B + V_B^2/2g$$

$$Z_A - Z_B = V_B^2/2g + h_L \text{ or}$$

$$h_L = (Z_A - Z_B) - V_B^2/2g$$

It is noted that stream of water at point B has the same velocity as that inside the pipe.

$Z_A - Z_B = 10.3$m (from Fig. 3.1)

The components of energy loss h_L are entrance loss h_1, from equation (2.22)

$h_1 = C_L (V^2{}_1/2g)$, where V_1 = Velocity in 75mm diameter pipe.

But $C_L = 1.0$ (inward projecting pipe)

Therefore,

$$h_1 = 1.0 (V^2{}_1/2g)$$

Loss due to two 90^0 standard elbow, equation (2.23)

$$h_2 = 2f_1 (Le/D) (V^2{}_1/2g)$$

From table 2.6

Le/D for standard 90^0 standard elbow is 30

Therefore,

$$h_2 = 60f_1 (V^2{}_1/2g)$$

Loss due to gate valve h_3, equation

$$h_3 = f_1(L_e/D) (V^2{}_1/2g)$$

where L_e/D for gate valve fully open is 13 (Table 2.6)

$$h_3 = 13f_1 (V^2{}_1/2g)$$

Friction loss in 75mm diameter pipe h_4 equation (2.13)

$$h_4 = f_1 (L/D) (V^2{}_1/2g)$$

$$= f_1 (7.8/0.075) (V^2{}_1/2g)$$

$$= 104f_1 (V^2{}_1/2g)$$

loss due to sudden enlargement h_5 from equation (2.22)

$$h_5 = C_L (V^2{}_1/2g)$$

from equation (2.19)

$$C_L = [1 - (D_1/D_2)^2]^2$$

$C_L = [1 - (0.075/0.1)^2]^2$

$= [1 - 0.5625]^2 \quad = \quad 0.1914$

Therefore,

$h_5 = 0.1914\,(V^2{}_1/2g)$

loss due to tee (flow through branch) h_6

from equation (2.23)

$h_6 = f_2\,(L_e/D)\,(v^2{}_2/2g)$

for flow through standard Tee branch $L_e/D = 60$ (Table 2.6)

$h_6 = 60f_2\,(v^2{}_2/2g)$

for flow through run $L_e/D = 20$

$h_7 = 20f_2\,(v^2{}_2/2g)$

loss due to friction in 100mm diameter pipe h_8

$h_8 = f_2(L/D)\,(v^2{}_2/2g)$

$= f_2\,(124.6/0.1)\,(v^2{}_2/2g)$

$= 1246f_2\,(v^2{}_2/2g)$

loss due to two gate valve h_9

$h_9 = 2f_2\,(L_e/D)\,(v^2{}_2/2g)$

L_e/D for gate valve is 13

$h_9 = 26f_2\,(v^2{}_2/2g)$

Therefore,

$h_L = h_1 + h_2 + h_3 + h_4 + h_5 + h_6 + h_7 + h_8 + h_9.$

i.e. $\quad h_L = (1.0 + 60f_1 + 13f_1 + 104f_1 + 0.1914)\,v^2{}_1/2g + (60 + 20 + 1246 + 26)\,f_2\,v^2{}_2/2g$

but, $\quad A_1v_1 = A_2v_2$

$v_1 = (A_2/A_1)\,v_2$

$= D^2{}_2/D^2{}_1v_2$

$= (0.1)^2/(0.075)^2v_2$

$= 1.78v_2$ m/s

Therefore,

$V^2{}_1 = 3.16\,v^2{}_2$

$h_L = (1.0 + 60f_1 + 13f_1 + 104f_1 + 0.1914)(3.16 \times v^2{}_2/2g) + (1352f_2)\,v^2{}_2/2g$

Recall that $h_L = (Z_A - Z_B) - v^2{}_2/2g$

Hence,

$10.3 - (v^2{}_2/2g) = (3.16 + 189.61f_1 + 41.08f_1 + 328.64f_1 + 0.60)\,(v^2{}_2/2g) + 1352f_2\,(v^2{}_2/2g)$

$10.3 = (3.16 + 189.61f_1 + 41.08f_1 + 328.64f_1 + 0.06 + 1352f_2 + 1)\,v^2{}_2/2g$

Therefore,

$$V^2_2 = (2g \times 10.3) / (4.76 + 189.61f_1 + 41.08f_1 + 328.64f_2 + 1352f_2)$$

$$V_2 = \sqrt{(2 \times 9.81 \times 10.3)} / \sqrt{(4.76 + 189.61f_1 + 41.08f_1 + 328.64f_1 + 1352f_2)}$$

The friction factor f_1 and f_2 can be determined thus:

$$R_{e1} = (v_1 \rho D_1)/\mu = (1000 \times 0.075\ v_1)/7.97 \times 10^{-4}$$

$$= 9.41 \times 10^4\ v_1$$

$$R_{e2} = (v_2 \rho D_2)/\mu = (1000 \times 0.1\ v_2)/\ 7.97 \times 10^{-4}$$

$$= 1.25 \times 10^5\ v_2$$

This shows that the flow is turbulent

Therefore, the relative roughness D/ε is

$$(D/\varepsilon)_1 = (0.075)/(4.6 \times 10^{-5}) \qquad \text{(from table 2.5)}$$

$$= 1630$$

$$(D/\varepsilon)_2 = \text{smooth}$$

we can now apply equation (2.15)

$$f_1 = 0.0055\ \{1 + [20{,}000/(D/\varepsilon_1) + (10^6/R_{e1})]^{1/3}\}$$

$$= 0.0055\ \{1 + [20{,}000/1630 + 10^6/9.41 \times 10^4]^{1/3}\}$$

$$= 0.0055\ \{1 + [12.269 + 10.626]^{1/3}\}$$

$$= 0.0055\ \{3.839\}$$

$$f_1 = 0.022$$

from figure 2.5 when $R_{e2} = 1.25 \times 10^5$

$$f_2 = 0.032$$

Hence,

$$V_2 = \sqrt{[2 \times 9.81 \times 10.3]} / \sqrt{[4.76 + 189.616(0.022) + 41.08(0.022) + 328.64(0.022) + 1352(0.032]}$$

$$= [\sqrt{202.086}]/[\sqrt{60.308}]$$

$$= 1.83\text{m/s}$$

Therefore,

$$V_1 = 1.78\ (1.83)$$

$$= 3.25\text{m/s}$$

$$R_{e1} = (9.41 \times 10^4) \times 3.25$$

$$= 3.06 \times 10^5$$

$$R_{e2} = (1.25 \times 10^5) \times 1.83$$

$$= 2.29 \times 10^5$$

Hence the actual f_1 and f_2 are

$$f_1 = 0.0055\ \{1 + [(20{,}000/1630) + (10^6/3.05 \times 10^5)]^{1/3}\}$$

$$= 0.0055\ \{1 + [12.269 + 3.278]^{1/3}\}$$

$$= 0.0055\ \{1 + [15.547]^{1/3}\}$$

$$= 0.0055\ \{3.495\}$$

$$f_1 = 0.019$$

and from figure 2.5

$$f_2 = 0.024$$

since these are different from the historical value then

$$V^2 = \sqrt{[2\times 9.81 \times 10.3]} / \sqrt{[4.76 + 189.61(0.019) + 41.08(0.019) + 328.14\ (0.019) + 1352(0.024)]}$$

$$= [\sqrt{202.086}]/[\sqrt{47.834}]$$

$$V_2 = 2.1\text{m/s}$$

$$V_1 = 1.78 \times 2.1$$

$$= 3.74\text{m/s}$$

The water consumption in the hospital is approximately 99,500 litters per day as shown in table 2.4

This can be expressed as

$$99.5\text{m}^3/24\text{h} \quad \text{or} \quad 1.15 \times 10^{-3}\ \text{m}^3/\text{s}$$

But the flow rate of the main is

$$Q = A_2 v_2$$

$$= (\pi D^2/4) \times v^2$$

$$= (\pi\ (0.1)^2/4) \times 2.1$$

$$= \underline{1.65 \times 10^{-2}\ \text{m}^3/\text{s}}$$

The flow rate within the system is a multiple of the demand per seconds within the hospital. Moreover, the yield/recharge rate of wells 1 and 2 were determined to be 2.5 litres per second and 2.1 litres per second respectively, which is more than the hospital demand of 1.15 litres per second.

This implies that the hospital demand can be met with the two (2) wells and pipeline network design.

PRESSURE (P_2) AT POINT B

From the energy equation, (see figure 3.1)

$$P_1/\gamma + Z_1 + v^2_1/2g + h_A - h_R - h_L = P_2/\gamma + Z_2 + v^2_2/2g$$

$$P_2 = P_1 + \gamma[(Z_1 - Z_2) + (v^2_1 - v^2_2)/2g - h_L]$$

Recall that

$$h_L = (Z_1 - Z_2) - (v^2_2/2g)$$

P_1 = Pressure at the inlet of 0.0075m Ø pipe = γh

$= 9.81 KN/m^3 \times 2.5m = 25Kpa$

V_1 = Velocity in 0.075m Ø pipe = 3.74 m/s

P_2 = Pressure at the outlet of 0.1m Ø pipe

V_2 = Velocity in 0.1m pipe = 2.1m/s

Hence,

$$(V^2_1 - V^2_2)/2g = (3.74^2 - 2.1^2)/ (2 \times 9.81)$$

$$= 0.488m$$

And,

$$h_L = (Z_1 - Z_2) - (v^2_2/2g)$$

$$= 7.8 - (2.1^2/2 \times 9.81)$$

$$= 7.8 - 0.22 \quad = 7.58m$$

Therefore,

$$P_2 = 25 + 9.81\ (7.8 + 0.488 - 7.58)$$

$$= 25 + 6.95$$

$$= 31.95$$

Approx. = 32Kpa

The Specialist Hospital pipeline network could be modified for the purpose of simplicity in calculation as shown in figure 3.2

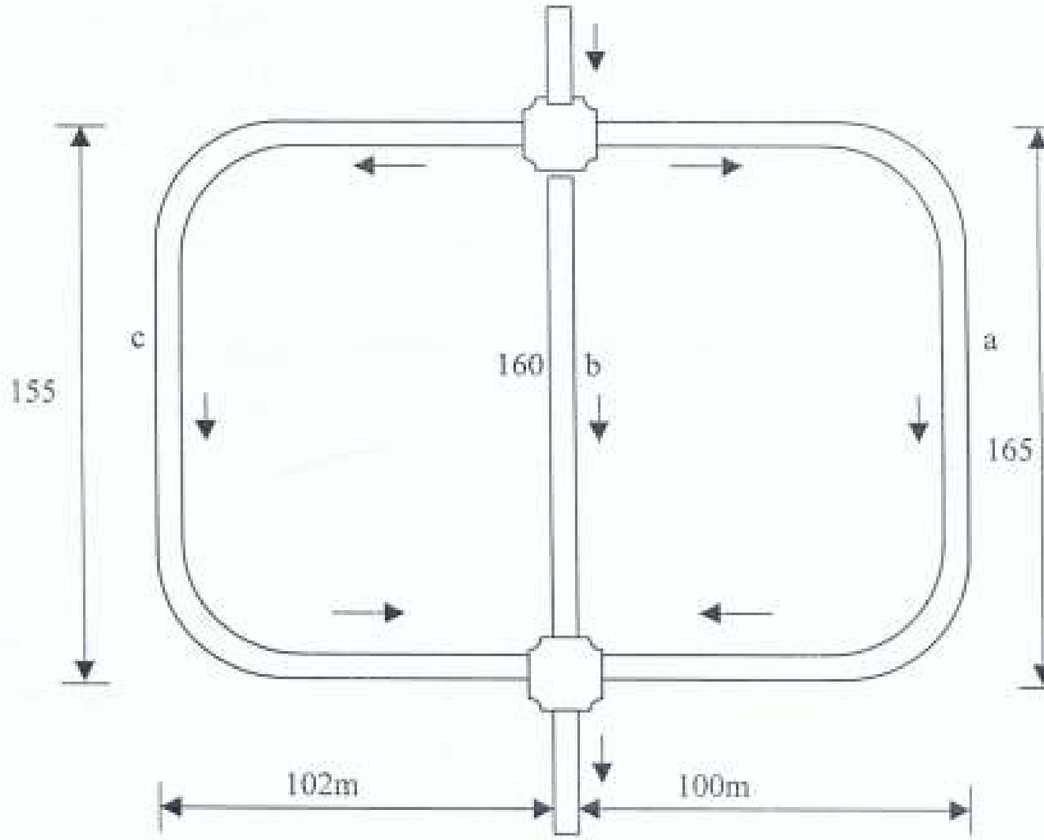


Figure 3.2. A pipeline network within the Hospital.

Branch pipes a, b, c = 0.1m

Inlet pipe = 0.075m, outlet pipe = 0.1m

Elbows are100mm 90^0 standards.

The flow rate (Q) in the system = $1.65 \times 10^{-2} m^3/s$

From equations (2.6), (2.7), and (2.8)

$$Q_1 = Q_2 = Q_a + Q_b + Q_c$$

$$h_{L1\text{-}2} = h_a = h_b = h_c \text{ and}$$

$$h = KQ^n$$

For branch a, two elbows, stop cork

$$h_a = 2f_a (L_e/D)\, v_a^2/2g + f_a (L_e/D)(v_a^2/2g) + f_a (L/D)\, v_a^2/2g$$

where equivalent lengths (Le/D) for elbow and stop cork are 30 and 13 respectively. Hence,

$$h_a = 60f_a (v_a^2/2g) + 13f_a (v_a^2/2g) + f_a(365/0.1)(v_a^2/2g)$$

friction factor f_a is calculated thus;

from table 2.5 pipe roughness, ε, for plastics is smooth.

$$R_e = \rho Dv/\mu$$

But $v^2 = Q^2/A^2 = (1.65 \times 10^{-2})^2/[\pi^2(0.1)^4/16] = 4.42$

Therefore, $v = 2.1$m/s

$$R_e = (2.1 \times 0.1 \times 1000)/(7.97 \times 10^{-4})$$

$$= 2.63 \times 10^5$$

Hence, from Moody diagram, Fig.2.5

$f = 0.024$ (same as f_2 calculated earlier)

Therefore,

$$h_a = [60(0.024) + 13(0.024) + 3650(0.024)]\ v_a^2/2g$$

$$= [1.44 + 0.31 + 89.35]\ v_a^2/2g$$

But $v_a^2 = Q^2/A^2$, and $A^2 = \pi^2(0.1)^4/16 = 6.17 \times 10^{-5}$

Therefore,

$$h_a = (89.35/2gA^2) \times Q_a^2$$

But $h_a = K_aQ_a^n$, $n = 2$

This implies that

$$K_a = (89.35)/(2 \times 9.81 \times 6.17 \times 10^{-5})$$

$\underline{K_a = 7.4 \times 10^4}$

For branch b with stop cork;

$$h_b = f_b\,(L_e/D)\ v_b^2/2g + f_b\,(L/D)\ v_b^2/2g$$

$$= 0.024\,(13 + 1600)\ v_b^2/2g$$

Hence,

$$K_b = (38.71)/2gA^2 = (38.71)/(2 \times 9.81 \times 6.17 \times 10^{-5})$$

$\underline{K_b = 3.2 \times 10^4}$

For branch c, two elbows, stop cork

$$h_c = 2f_c\,(L_e/D)\ v_c^2/2g + f_c\,(L_e/D)(v_c^2/2g) + f_c(L/D)\ v_c^2/2g$$

$$h_c = 60f_c\,(v_c^2/2g) + 13f_c\,(v_c^2/2g) + f_c(360/0.1)\,(v_c^2/2g)$$

$$= (60 + 13 + 360/0.1)\ f_c\,(v_c^2/2g)$$

$$= [3673(0.024)/(2gA^2)] \times Q_c^2$$

Hence,

$$K_c = (88.15)/(2gA^2) = 88.15/(2 \times 9.81 \times 6.17 \times 10^{-5}$$

$\underline{K_c = 7.3 \times 10^4}$

Branch b has the least value of K; therefore, it should carry the greatest flow. Conversely, branch a has the highest value of K, and then Q_a should be the smallest of the three.

We know that $Q_a + Q_b + Q_c = 1.65 \times 10^{-2} m^3/s$ or 0.0165

Let us assume

$Q_a = 0.00682$

$Q_b = 0.00295$

$Qc = 0.00673$

We now calculate the head loss using the assumed values.

From equation (2.8)

$h = KQ^n$

Therefore,

$h_a = (7.4 \times 10^4)(0.00682)^2 = 3.442$

$h_b = (3.2 \times 10^4)(0.00295)^2 = 0.278$

$h_c = (7.3 \times 10^4)(0.00673)^2 = 3.306$

For circuit 1

$\Sigma h_1 = h_a - h_b = 3.442 - 0.278 = 3.164$

For circuit 2

$\Sigma h_2 = h_b - h_c = 0.278 - 3.306 = -3.028$

The correct values for the pipes are

$2K_aQ_a = 2(7.4 \times 10^4) \times (0:00682) = 1009$

$2K_bQ_b = 2(3.2 \times 10^4) \times (0.00295) = 189$

$2K_cQ_c = 2(7.3 \times 10^4) \times (0.00673) = 983$

For circuit 1

$\Sigma(2KQ)_1 = 1009 + 189 = 1198$

For circuit 2

$\Sigma(2KQ)_2 = 189 + 983 = 1172$

For circuit 1 and 2 using equation (3.11)

$\Delta Q_1 = \Sigma h_1/\Sigma(2KQ)_1 = 3.164/1198$

$= 0.00264$

$\Delta Q_2 = \Sigma h_2/\Sigma(2KQ)_2 = -3.028/1172$

$= -0.00258$

The process is repeated until the magnitude of ΔQ is less than 1% of the assumed value of Q.

Step 2

Circuit 1 $Q'_a = Q_a - \Delta Q_1 = 0.00682 - 0.00264 = 0.00418\ m^3/s$

Since Q_b is common to both circuit 1 and 2, both ΔQ_1 and ΔQ_2 must be applied to Q_b

$$Q'_b = Q_b - \Delta Q_1 = -0.00295 - 0.00264$$

Circuit 2 $Q'_b = Q_b - \Delta Q_2 = 0.00295 - (-0.00258)$

This result is increasing Q_b. Q_b is actually increasing in absolute value by the sum of ΔQ_1 and ΔQ_2, that is;

$$Q'_b = 0.00295 + 0.00264 + 0.00258 = 0.00817 m^3/s$$

$$Q'_c = Q_c - \Delta Q_2 = -0.00673 - (-0.00258) = -0.00415\ m^3/s$$

Using $\mathbf{Q'_a = 0.00418 m^3/s}$, $\mathbf{Q'_b = 0.00817 m^3/s}$, **and** $\mathbf{Q'_c = -0.00415 m^3/s}$

Therefore,

$$h'_a = (7.4 \times 10^4) \times (0.00418)^2 = 1.292$$

$$h'_b = (3.2 \times 10^4) \times (0.00817)^2 = 2.135$$

$$h'_c = (7.3 \times 10^4) \times (0.00415)^2 = 1.257$$

For circuit 1

$$\Sigma h'_1 = h'_a - h'_b = 1.292 - 2.135 = -0.843$$

For circuit 2

$$\Sigma h'_2 = h'_b - h'_c = 2.135 - 1.257 = 0.878$$

Values for the pipes are

$$2K_aQ'_a = 2\ (7.4 \times 10^4) \times (0.00418) = 619$$

$$2K_bQ'_b = 2(3.2 \times 10^4) \times (0.00817) = 523$$

$$2K_cQ'_c = 2(7.3 \times 10^4) \times (0.00415) = 606$$

For circuit 1

$$\Sigma(2KQ')_1 = 619 + 523 = 1142$$

For circuit 2

$$\Sigma(2KQ')_2 = 523 + 606 = 1129$$

For circuit 1

$$\Delta Q'_1 = \Sigma h'_1/\Sigma(2KQ')_1 = -0.843/1142$$

$$\mathbf{= -0.000738}$$

For circuit 2

$$\Delta Q'_2 = \Sigma h'_2/\Sigma(2KQ')_2 = 2.50/1222$$

$$\mathbf{= -0.002046}$$

Step 3

Circuit 1 $\quad Q^{I}_{a} = Q^{I}_{a} - \Delta Q^{I}_{1} = 0.00418 - (-0.000738) = 0.00492 m^3/s$

Since Q_b is common to both circuit 1 and 2 ΔQ_1 and ΔQ_2 must be applied to Q_b

$$Q^{I}_{b} = Q^{I}_{b} - \Delta Q^{I}_{1} = -0.00817 - (-0.000738)$$

Circuit 2 $\quad Q^{I}_{b} = Q^{I}_{b} - \Delta Q^{I}_{2} = 0.00817 - 0.000778$

This results is reducing Q_b. Q_b is actually reducing in absolute value by the sum of ΔQ_1 and ΔQ_2, that is

$$Q^{I}_{b} = 0.00817 - 0.000738 - 0.000778 = 0.00665 m^3/s$$

$$Q^{I}_{c} = Q^{I}_{c} - \Delta Q^{I}_{2} = -0.00415 - 0.000778 = -0.00493 m^3/s$$

Using $\mathbf{Q^{II}_{a} = 0.00492 m^3/s}$, $\mathbf{Q^{II}_{b} = 0.00665 m^3/s}$, **and** $\mathbf{Q^{II}_{c} = -0.00493 m^3/s}$

Therefore,

$$h^{I}_{a} = (7.4 \times 10^4) \times (0.00492)^2 = 1.791$$

$$h^{I}_{b} = (3.2 \times 10^4) \times (0.00665)^2 = 1.415$$

$$h^{I}_{c} = (7.3 \times 10^4) \times (0.00493)^2 = 1.774$$

For circuit 1

$$\Sigma h^{I}_{1} = h^{I}_{a} - h^{I}_{b} = 1.791 - 1.415 = 0.378$$

For circuit 2

$$\Sigma h^{I}_{2} = h^{I}_{b} - h^{I}_{c} = 1.415 - 1.774 = -0.359$$

Values for the pipes are

$$2K_aQ^{I}_{a} = 2(7.4 \times 10^4) \times (0.00492) = 728$$

$$2K_bQ^{I}_{b} = 2(3.2 \times 10^4) \times (0.00665) = 426$$

$$2K_cQ^{I}_{c} = 2(7.3 \times 10^4) \times (0.00493) = 720$$

For circuit 1

$$\Sigma(2KQ^{I})_1 = 728 + 426 = 1154$$

For circuit 2

$$\Sigma(2KQ^{I})_2 = 426 + 720 = 1146$$

For circuit 1

$$\Delta Q^{I}_{1} = \Sigma h^{I}_{1}/\Sigma(2KQ^{I})_1 = 0.378/1154$$

$$= \mathbf{0.000327}$$

For circuit 2

$$\Delta Q^{I}_{2} = \Sigma h^{I}_{2}/\Sigma(2KQ^{I})_2 = -0.359/1146$$

$$= \mathbf{-0.000313}$$

Step 4

Circuit 1 $\quad Q^{II}_a = Q^{I}_a - \Delta Q^{I}_1 = 0.00492 - 0.000327 = 0.00459 m^3/s$

$Q^{II}_b = Q^{I}_b - \Delta Q^{I}_1 = -0.00665 - 0.000327$

Circuit 2 $\quad Q^{II}_b = Q^{I}_b - \Delta Q^{I}_2 = 0.00665 - (-0.000313)$

This results is increasing Q_b. Q_b is actually increasing in absolute value by the sum of ΔQ_1 and ΔQ_2, that is

$$Q^{II}_b = 0.00665 + 0.000327 + 0.000313 = 0.00729 m^3/s$$

$$Q^{II}_c = Q^{I}_c - \Delta Q^{I}_2 = -0.00493 - (-0.000313) = -0.00462 m^3/s$$

Using $\mathbf{Q^{II}_a = 0.00459 m^3/s}$, $\mathbf{Q^{II}_b = 0.00729 m^3/s}$ **and** $\mathbf{Q^{II}_c = -0.00462 m^3/s}$

Therefore,

$$h^{II}_a = (7.4 \times 10^4) \times (0.00459)^2 = 1.559$$

$$h^{II}_b = (3.2 \times 10^4) \times (0.00729)^2 = 1.701$$

$$h^{II}_c = (7.3 \times 10^4) \times (0.00462)^2 = 1.558$$

For circuit 1

$$\Sigma h^{II}_1 = h^{II}_a - h^{II}_b = 1.559 - 1.701 = -0.142$$

For circuit 2

$$\Sigma h^{II}_2 = h^{II}_b - h^{II}_c = 1.701 - 1.558 = 0.143$$

Values for the pipes are

$$2K_aQ^{II}_a = 2(7.4 \times 10^4) \times (0.00459) = 679$$

$$2k_bQ^{II}_b = 2(3.2 \times 10^4) \times (0.00729) = 467$$

$$2kcQ^{II}_c = 2(7.3 \times 10^4) \times (0.00462) = 675$$

For circuit 1

$$\Sigma(2kQ^{II})_1 = 679 + 467 = 946$$

For circuit 2

$$\Sigma(2KQ^{II})_2 = 467 + 675 = 1142$$

For circuit 1

$$\Delta Q^{II}_1 = \Sigma h^{II}_1/\Sigma(2KQ^{II})_1 = -0.142/946$$

$$= -0.000150$$

For circuit 2

$$\Delta Q^{II}_2 = \Sigma h^{II}_2/\Sigma(2KQ^{II})_2 = 0.143/1142$$

$$= 0.000125$$

Step 5

Circuit 1 $\quad Q^{II}_a = Q^{I}_a - \Delta Q^{I}_1 = 0.00459 - (-0.000150) = 0.00474 m^3/s$

$Q^{II}_b = Q^{I}_b - \Delta Q^{I}_1 = -0.00729 - (-0.000150)$

Circuit 2 $\quad Q^{II}_b = Q^{I}_b - \Delta Q^{I}_2 = 0.00729 - 0.000125$

This results is increasing Q_b. Q_b is actually reducing in absolute value by the sum of ΔQ_1 and ΔQ_2, that is

$$Q^{II}_b = 0.00729 - 0.000150 - 0.000125 = 0.00702 m^3/s$$

$$Q^{II}_c = Q^{I}_c - \Delta Q^{I}_2 = -0.00462 - 0.000125 = -0.00474 m^3/s$$

Using **$Q^{II}_a = 0.00474 m^3/s$, $Q^{II}_b = 0.00702 m^3/s$ and $Q^{II}_c = -0.00474 m^3/s$**

Therefore,

$$h^{II}_a = (7.4 \times 10^4) \times (0.00474)^2 = 1.663$$

$$h^{II}_b = (3.2 \times 10^4) \times (0.00702)^2 = 1.577$$

$$h^{II}_c = (7.3 \times 10^4) \times (0.00474)^2 = 1.640$$

For circuit 1

$$\Sigma h^{II}_1 = h^{II}_a - h^{II}_b = 1.663 - 1.577 = 0.106$$

For circuit 2

$$\Sigma h^{II}_2 = h^{II}_b - h^{II}_c = 1.577 - 1.640 = -0.063$$

Values for the pipes are

$$2K_aQ^{II}_a = 2(7.4 \times 10^4) \times (0.00474) = 702$$

$$2k_bQ^{II}_b = 2(3.2 \times 10^4) \times (0.00702) = 449$$

$$2kcQ^{II}_c = 2(7.3 \times 10^4) \times (0.00474) = 692$$

For circuit 1

$$\Sigma(2kQ^{II})_1 = 702 + 449 = 1151$$

For circuit 2

$$\Sigma(2KQ^{II})_2 = 449 + 692 = 1141$$

For circuit 1

$$\Delta Q^{II}_1 = \Sigma h^{II}_1/\Sigma(2KQ^{II})_1 = 0.106/1151$$

$$= 0.000092$$

For circuit 2

$$\Delta Q^{II}_2 = \Sigma h^{II}_2/\Sigma(2KQ^{II})_2 = -0.063/1141$$

$$= -0.000055$$

Step 6

Circuit 1 $\quad Q^{III}_a = Q^{II}_a - \Delta Q^{II}_1 = 0.00474 - 0.000092 = 0.00465 m^3/s$

$Q^{III}_b = Q^{II}_b - \Delta Q^{II}_1 = -0.00702 - 0.000092$

Circuit 2 $\quad Q^{III}_b = Q^{II}_b - \Delta Q^{II}_2 = 0.00702 - (-0.000055)$

This results is increasing Q_b. Q_b is actually increasing in absolute value by the sum of ΔQ_1 and ΔQ_2, that is

$$Q^{III}_b = 0.00702 + 0.000092 + 0.000055 = 0.00717 m^3/s$$

$$Q^{III}_c = Q^{II}_c - \Delta Q^{II}_2 = -0.00474 - (-0.000055) = -0.00468 m^3/s$$

Using **$Q^{III}_a = 0.00465 m^3/s$, $Q^{III}_b = 0.00717 m^3/s$ and $Q^{III}_c = -0.00468 m^3/s$**

Therefore,

$$h^{III}_a = (7.4 \times 10^4) \times (0.00465)^2 = 1.600$$

$$h^{III}_b = (3.2 \times 10^4) \times (0.00717)^2 = 1.645$$

$$h^{III}_c = (7.3 \times 10^4) \times (0.00468)^2 = 1.598$$

For circuit 1

$$\Sigma h^{III}_1 = h^{III}_a - h^{III}_b = 1.600 - 1.645 = -0.045$$

For circuit 2

$$\Sigma h^{III}_2 = h^{III}_b - h^{III}_c = 1.645 - 1.598 = 0.047$$

Values for the pipes are

$$2K_a Q^{III}_a = 2(7.4 \times 10^4) \times (0.00465) = 688$$

$$2k_b Q^{III}_b = 2(3.2 \times 10^4) \times (0.00717) = 459$$

$$2kc Q^{III}_c = 2(7.3 \times 10^4) \times (0.00468) = 683$$

For circuit 1

$$\Sigma(2kQ^{III})_1 = 688 + 459 = 1147$$

For circuit 2

$$\Sigma(2KQ^{III})_2 = 459 + 683 = 1142$$

For circuit 1

$$\Delta Q^{III}_1 = \Sigma h^{III}_1 / \Sigma(2KQ^{III})_1 = -0.045/1147$$

$$= -0.000039$$

For circuit 2

$$\Delta Q^{III}_2 = \Sigma h^{III}_2 / \Sigma(2KQ^{III})_2 = 0.047/1142$$

$$= 0.000041$$

The value of ΔQ^{III}_1 and ΔQ^{III}_2 each is less than 1% of the assumed values of Q, hence the actual value of Q for branches a, b and c are:

$Q_a = 0.00465 m^3/s$, $\quad Q_b = 0.00717 m^3/s$ and $\quad Q_c = 0.00468 m^3/s$

The actual values of h are; **$h_a = 1.60m$ $\quad h_b = 1.64m$ and $\quad h_c = 1.59m$**

From equation (2.6): $Q_1 = Q_2 = Q_a + Q_b + Q_c$

Substituting for Q_a, Q_b, and Q_c gives

$$0.00465 + 0.00717 + 0.00468 = 0.0165 = Q_1$$

The flow rate Q_1 before the water enters the branches is 0.0165; hence, the result of step 6 above gives the correct flow through each branch.

The results for the four iteration cycles are summarized in table 3.1

Table 3.1: Summary of iteration for parallel pipeline network for Specialist Hospital Akure using the Hardy Cross technique

TRIAL	CIRCUIT	PIPE	Q	$h = KQ^2$	2KQ	ΔQ
1	i	a b	0.00682 -0.00295	3.442 -0.278 3.164	1009 189 1198	+0.00264
	ii	b c	0.00295 -0.00673	0.278 -3.306 -3.028	189 983 1172	- 0.00258
2	i	a b	0.00418 -0.00817	1.292 -2.135 -0.843	619 523 1142	- 0.000738
	ii	b c	0.00817 -0.00415	2.135 -1.257 0.878	523 606 1129	+0.000778
3	i	a b	0.00492 -0.00415	1.791 -1.415 0.376	728 426 1154	+0.000327
	ii	b c	0.00415 -0.00493	1.415 -1.774 -0.359	426 720 1146	-0.000313
4	i	a b	0.00459 -0.00729	1.559 1.701 -0.142	679 467 946	-0.000150
	ii	b c	0.00729 -0.00462	1.701 -1.558 0.143	467 675 1142	+0.000125
5	i	a b	0.00474 -0.00702	1.663 -1.577 0.106	702 449 1151	+0.000092
	ii	b c	0.00702 -0.00474	1.577 -1.640 -0.063	449 692 1141	-0.000055
6	i	a b	0.00465 -0.00717	1.600 -1.645 -0.045	688 459 1147	-0.000039
	ii	b c	0.00717 -0.00468	1.645 -1.598 0.047	459 683 1142	+0.000041

3.2 BOREHOLE CONSTRUCTION

3.2.1 LOCATION AND DRILLING OF WELLS

The two wells are located at the VES points 1 and 2 within the Hospital premises, Figure 3.1. The drilling was done for five consecutive days. The drilling was carried out using Chicago Pneumatic (T650) rotary type of drilling Rig of 90kw (120hp) capacity deck engine, and drilling depth capacity of 650m. The boring of the hole was done, by rotary method, to 200mm (8ins) diameter down the hole to a depth of 10.5m, (this is referred to as overburden thickness), using Tri-con Roller bit. The bit is equipped with three toothed cutters of hard steel or diamond, which rotates and breaks up the formation. Water is fed to the cutters down the drill rods and, rising upward through the boring, brings the rock chippings with it. At the depth of 10.5m the roller bit hits the weathered rock formation; the bit was therefore changed to Tri-con Hammer bit. The hole was therefore hammered, using 150mm diameter hammer bit, from this depth to 50m depth through the weathered rock formation. A compressor with a capacity of 250kpa was used to deliver the pressure needed by the hammer bit to produce the impact needed to break up the rock formation. The pressure from the compressor equally forced the cuttings through the boring to the surface. This is being referred to as Percussion drilling.

3.2.2 LINING OF THE BOREHOLE

The upper part of the boring was lined so as to prevent surface pollution entering. The sides of the boring was too soft to stand up without support and in that case, slotted linings was inserted, this is technically referred to as casing, which support the formation but permit water to enter the boring. The boring was made slightly larger than the casing to be inserted and the outer annular space was sealed with concrete. The depth of the lining was 9m. It was done carefully so that the casing does not seal off good water-bearing strata.

3.2.3 GRAVEL PACKING

Gravel packing is necessary so that when pumping water from a borehole it does not bring sand out of the formation. The borehole was drilled to 200mm diameter, a slotted tube (casing) of 150mm diameter was hung in the hole, and specially graded gravel of 12.5mm size mix was poured into the annular space. The borehole was gravel packed such that the area exposed to the formation permits sufficient water to be drawn into the boring to give the yield required.

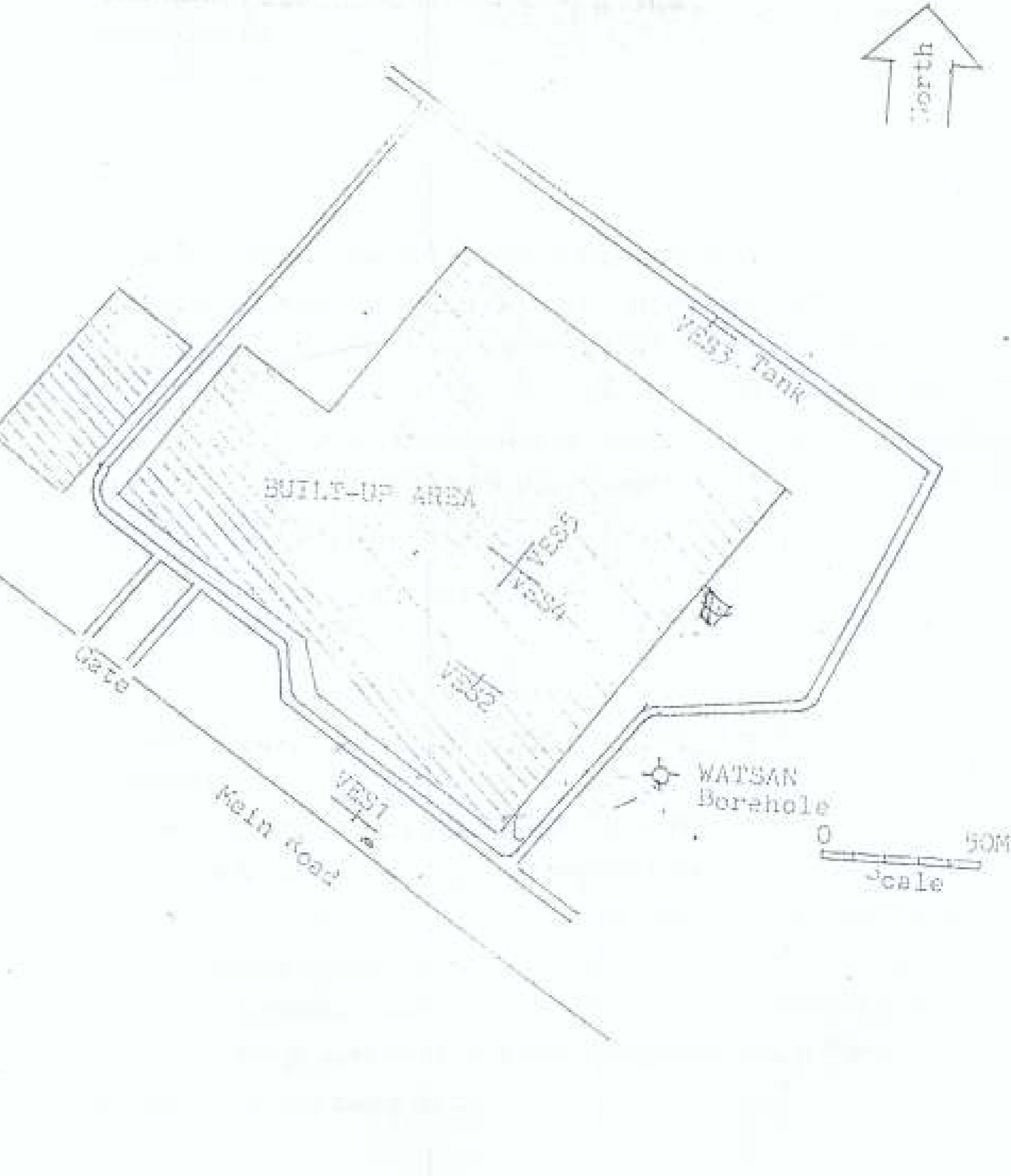


Figure 3.3: Sketch map of General Hospital, Akure showing VES locations. (Scale: 1: 2000)
Source: Egwebe and Chuckwuma, 1993. Geophysical Investigation Report for Borehole sitting in Ondo State. Sponsored by African Development Bank (A.D.B)

3.3 PIPELINE NETWORK DESIGN AND CONSTRUCTION FOR SPECIALIST HOSPITAL AKURE

The Specialist Hospital Akure is located within a terrain that gently slopes to the South as earlier stated in Sec.2.2. The water supply and distribution to the hospital was designed such that the water from the borehole will pass through the treatment plant into the constructed underground tank, from where the water will be lifted into the overhead tank by booster pump. By gravity the water will be distributed to the hospital for consumption. In this regard, the location of the overhead tank is very vital as well as important, so that the whole project will not be a complete failure.

Since the hospital slopes gently to the South, the overhead tank is better constructed on the Northern side of the hospital. The fact is that there are many buildings, including story buildings, scattered at different locations within the hospital. These buildings must receive water supply constantly without any difficulties, hence the pipeline network was professionally designed and constructed round the hospital. From it the water is distributed to each of the buildings through pipes with lesser diameter, according to the demand of the buildings, by means of saddle and ferrule.

3.4 OVERHEAD TANK

An overhead tank has two main functions.

(i) To balance the fluctuating demand from the distribution system against the output from the source.

(ii) To act as a safeguard for the continuance of the supply should there be any breakdown at the source or on the main trunk pipelines.

The aim must be to keep the source work and the main trunk time working as mush as possible at maximum capacity, and to balance the changes in demand by an overhead tank because of the continuous round-the-clock working of the hospital and because many departments has storage tanks which fill at night the night-time rate of flow can be quite high, but the peak period occur during the day.

3.5 PUMP SELECTION

When selecting pumps for these boreholes the following factors are considered.

(1) The nature of the liquid to be pumped which characterized its temperature and the Pumped condition, its specific gravity, its viscosity, its tendency to corrode or erode the pump parts, and its vapor pressure at the pumping temperature.

(2) The required capacity (volume flow rate Q)

(3) The condition on the suction (inlet) side of the pump.

(4) The condition on the discharge (outlet) side of the pump

(5) The total head on the pump (the term h_A from the energy equation)

(6) Space, weight, and position limitations.

The water to be pumped is from the two boreholes with the following characteristics:

Temperature	Specific Weight (γ)	Density (ρ)	Dynamic Viscosity (μ)	Kinematic Viscosity (v)
32^0C	$9.81\ KN/m^3$	$1{,}000\ kg/m^3$	$7.97 x 10^{-4}$ Pa.s	$1.15 x 10^{-6} m^2/s$

(Source: Robert Mott, 1990. Applied Fluid Mechanics 3rd Edition)

From the result of the test carried out on the water sample from each borehole, It is discovered that the waters are colorless and neutral to acidic and alkaline indicators. They are free from all causative agents of water hardness. Hence they conform to the standard of good quality drinking water and they are not harmful to the pump parts. The required capacity: the pump test carried out on boreholes 1 and 2 with 50m and 46m depth respectively reveals that the recharging rate is 2.50 litres per second ($2.5x10^{-3}m^3/s$) and 2.10 litres per second ($2.1x10^{-3}m^3/s$) respectively. But the installed pumps must have a flow rate not greater that 50% of the yield of the borehole, so as to get a better productivity and services. Hence, a submersible pump with maximum capacity of 1.13kw (1.5Hp) with a maximum flow rate of 1.5litres per second ($1.5x10^{-3}m^3/s$) is specified. Table 4.2 gives some examples of reasonable pipe size for the given flow rate with the resulting velocity of flow. From the table the flow rate of approximately 1.6l/s ($1.6x10^{-3}m^3/s$) requires $1^1/_4$" (32mm) pipe size at the discharge.

Table 3.2 Discharge pipe versus flow rate and velocity

Pipe size (in)	Flow rate (l/s)	Velocity (m/s)
1	0.75	1.35
1¼	1.58	1.63
1½	2.17	1.68
1½	3.17	2.40
2	4.75	2.18
2½	7.92	2.55
3	11.00	2.31
4	15.83	1.92
5	31.50	2.44
6	63.33	3.38
8	63.33	1.95
8	78.83	2.44
8	110.42	3.42
10	110.42	2.17
10	157.50	3.10
12	157.50	2.18
12	220.83	3.06

(Source: Robert Mott, 1990. Applied Fluid Mechanics 3rd Edition Page 490)

3.6 TREATMENT PLANTS

Treatment objectives

Records of peoples desire to improve water quality rate back to as early as 2000BC (A.S.C.E and A.W.W.A 1990). Since that time, advance in water treatment have continuously been made.

(1) Production of water that is safe for human consumption.

(2) Production of water that is appealing to the consumer

(3) Production of water using facilities, which can be constructed and operated at a reasonable cost.

The production of biologically and chemically safe water is the primary goal in the design of water treatment plants; anything less is unacceptable. A properly designed plant is not a guarantee of safely. However, skillful and alert Plant operation and attention to the sanitary requirements of the source of supply and the distribution system are equally important (A.S.C.E and A.W.W.A 1990). Figure 3.4 shows the treatment plant layout for the General Hospital.

The second basic objective of water treatment is the production of water that is appealing to the consumer. Ideally, appealing water is one that is clear and colorless, pleasant to the taste, odorless and cool. It is non-staining, neither corrosive nor scale forming and reasonable soft. Water utility operations should be such that quality is not impaired as water flows from treatment plant through the distribution system to the consumer.

The control point for the determination of water quality should be the consumer's tap because the consumer is principally interested in the quality of water delivered to the tap, not the quality at the treatment plant.

The Third basic objective of water treatment is that it is accomplished using facilities with reasonable capital and operating cost.

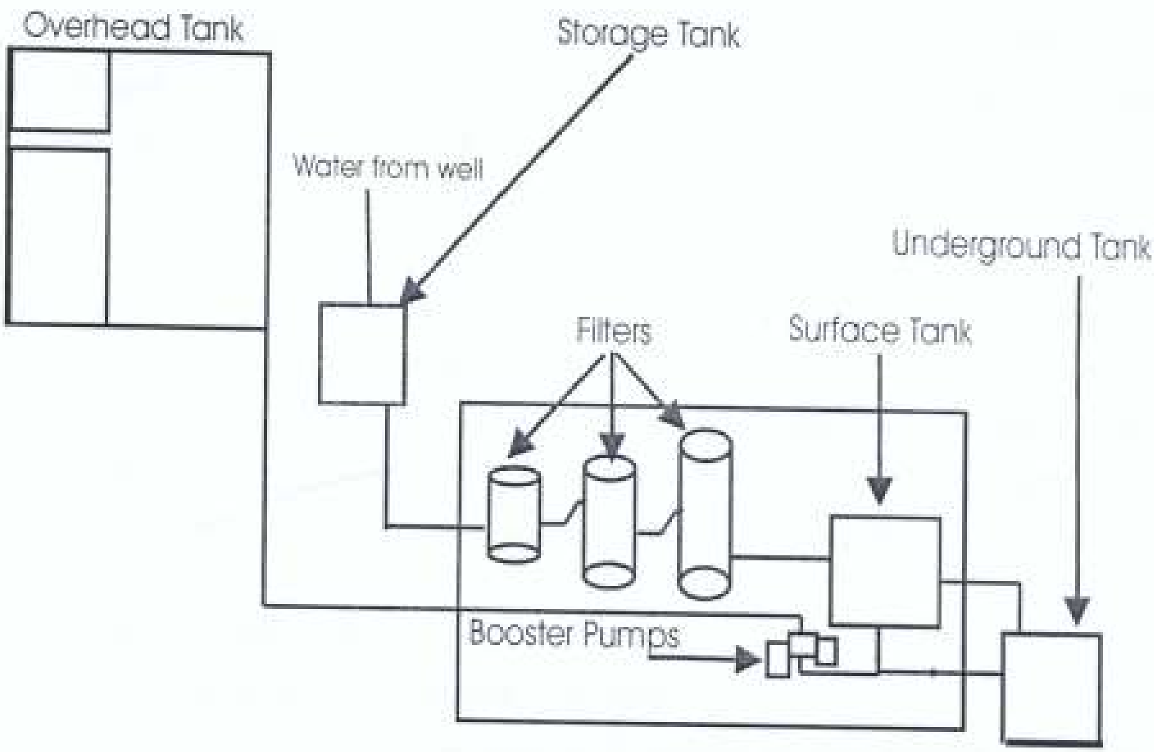


Figure 3.4 Treatment Plant Layout

3.7 PLANT SITTING ANG ARRANGEMENT

The treatment plant was designed such that the water coming from the sources (well 1 and well 2) goes into a 2,000 litre tank at an elevation reasonable enough to build up pressure head for gravity flow into the three specified filter arranged in series, i.e. they are placed next to each other along one side of (the supplying) a pipe gallery. This approach provides the most compact arranged and also simplifies filters operation and maintenance.

The filtered water goes into a 1000 liter tank before it finally gets into the underground tank from where is pumped by booster pump into the over head tank for distribution, For the purpose of maintenance or breakdown of the plant, the pipeline was designed with a diversion system, such that the plant is bye passed and the water get into the underground tank and from there it will be pumped into the overhead tank. Also the water in the 1000-litre tank can be pumped directly into the overhead tank without getting into the underground tank to allow for maintenance of the underground tank.

3.8 TREATMENT PROCESSES

The treatment process employed is filtration.

Filtration as it applied to water treatment, is the passage of water through a porous medium for the removal of suspended solids (A.S.C.E and A.W.W.A). According to A.S.C.E and A.W.W.A, the earliest written records of water treatment, dating from about 400 B.C, mentioned filtration of water through charcoal or sand and gravel. Filtration is generally required for groundwater for softening and removal of iron manganese that may be present.

The type of filter used is "Pressure Filter", it is called pressure filter because it allow pumping through the unit under relatively high pressure which may eliminate the need for additional pumping facilities. This is more economical. Figure 4.5 shows a typical vertical pressure filters. The media of filtration is granular media, and the media are natural Silica Sand and crushed Anthracite Coal. The arrangement is Coarse-to-Fine.

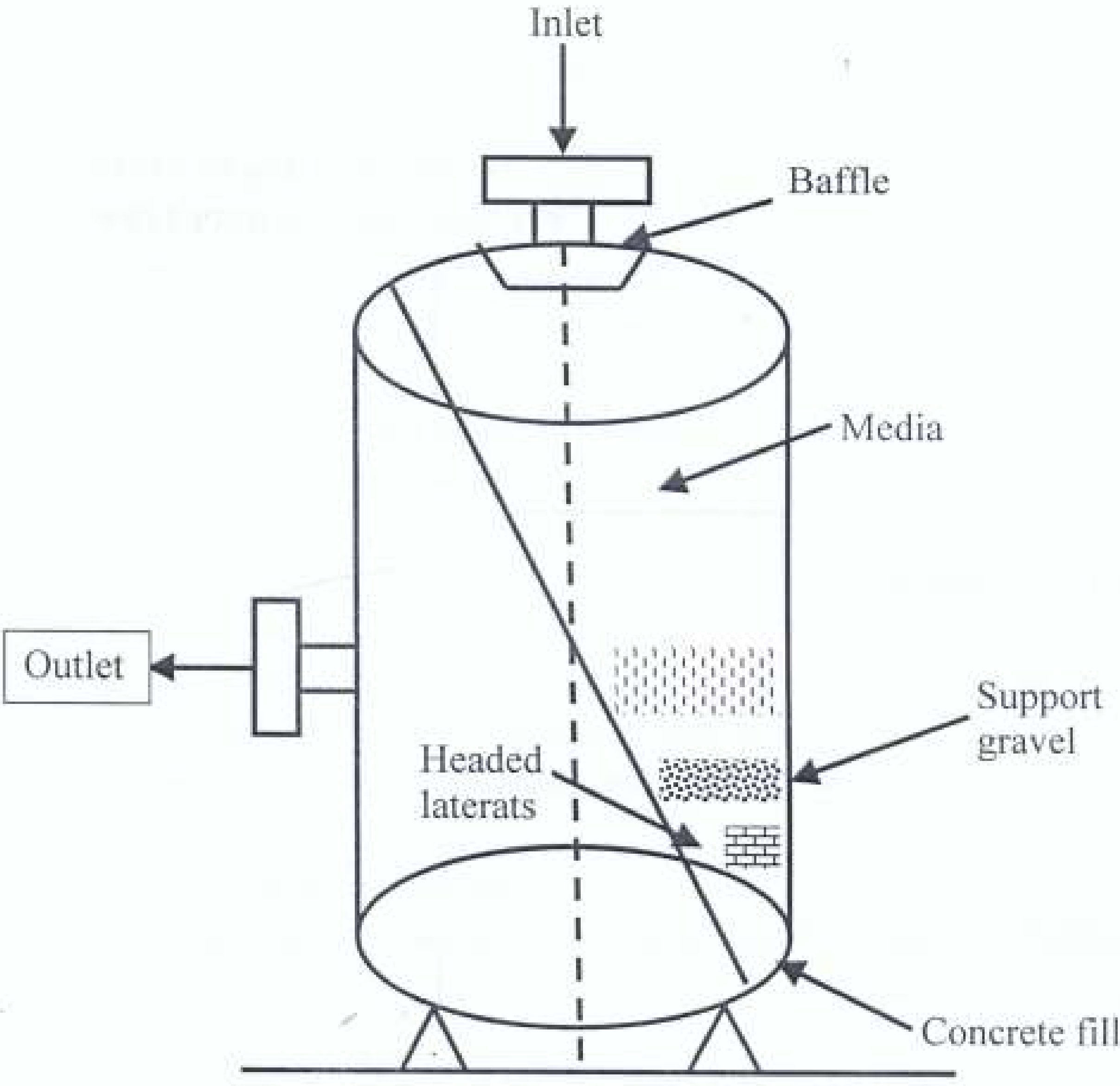


Fig.3.5 Typical Vertical Pressure Filter

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

4.1 WELL PRODUCTION CAPACITY

The well were properly flushed and cleaned during drilling operation Process.

After flushing and installation, a thorough pumping test was carried out on the wells to determine their production capacity parameters.

The depth of well 1 is 50m while that of well 2 is 46m.

The production parameters were determined. The static water level of well 1 is 5.5m while for well 2 it is 3.4m. The pump test lasted for 50minutes for each of the wells, and the Dynamic water levels were found to be 9.5m for well 1 and 8.4m for well 2. The water drawdown for each of them is 4.0m and 5.0m respectively, Table 4.1.

The recharge rate for each of the wells was determined to be 2.5litres per second ($2.5x10^{-3}m^3/s$) and 2.1litres per second ($2.1x10^{-3}m^3/s$) respectively.

The summary of their production capacity is as follows:

Table 4.1. Borehole Production Capacity.

	Water Level (m)	Static Water Level (m)	Dynamic Water Level (m)	Water Drawdown (m)	Yield/Recharge (l/s)
Well 1	50.0	5.50	9.50	4.0	2.50
Well 2	46.0	3.40	8.40	5.0	2.10

The recharging rates are 2.50litters per seconds ($2.5x10^{-3}m^3/s$) and 2.10 litters per second ($2.1x10^{-3}m^3/s$) for wells 1 and 2 respectively. A submersible pump of 1.5Hp (1.13kw) with minimum flow rate of 1.5 litters per second ($1.5x10^{-3}m^3/s$) was installed in each of the borehole. The water from the two boreholes was pumped simultaneously, to fill each of the 50,000litre ($50m^3$) tank in less than four hours forty-five minutes.

4.2 COMPARISON OF VES INTERPRITATION RESULTS WITH BOREHOLE LITHOLOGIC SECTION

From Figure 4.1, it is discovered that the VES interpretation prediction show appreciable correlation. The VES for well 1 predicts a clayey topsoil of 6m while this layer was actually 8m while drilling. The weathered and fractured basements were respectively predicted for between 22m and 38m. However during drilling these layers were intersected at 26m and 46m respectively. For well 2, there is appreciable correlation between the VES results and drilling sections encountered, Figure 5.2. Hence the geophysical reports of sec.2.2 for the purpose of the pre-drilling investigation had proved highly successful and helpful to drillers during drilling.

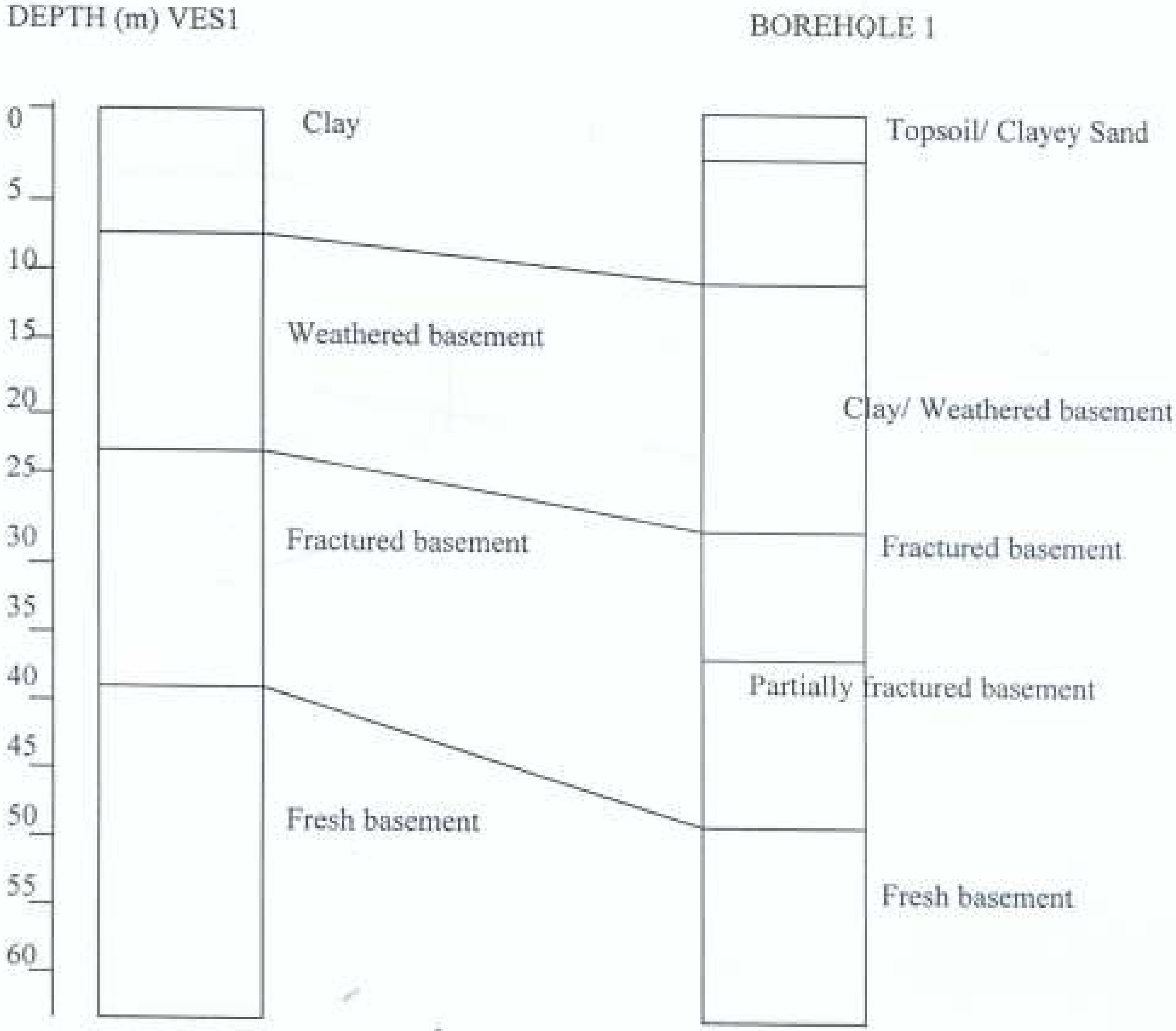


Fig. 4.1. Comparison of VES 1 Interpretation Results with Borehole 1 Lithological Section.

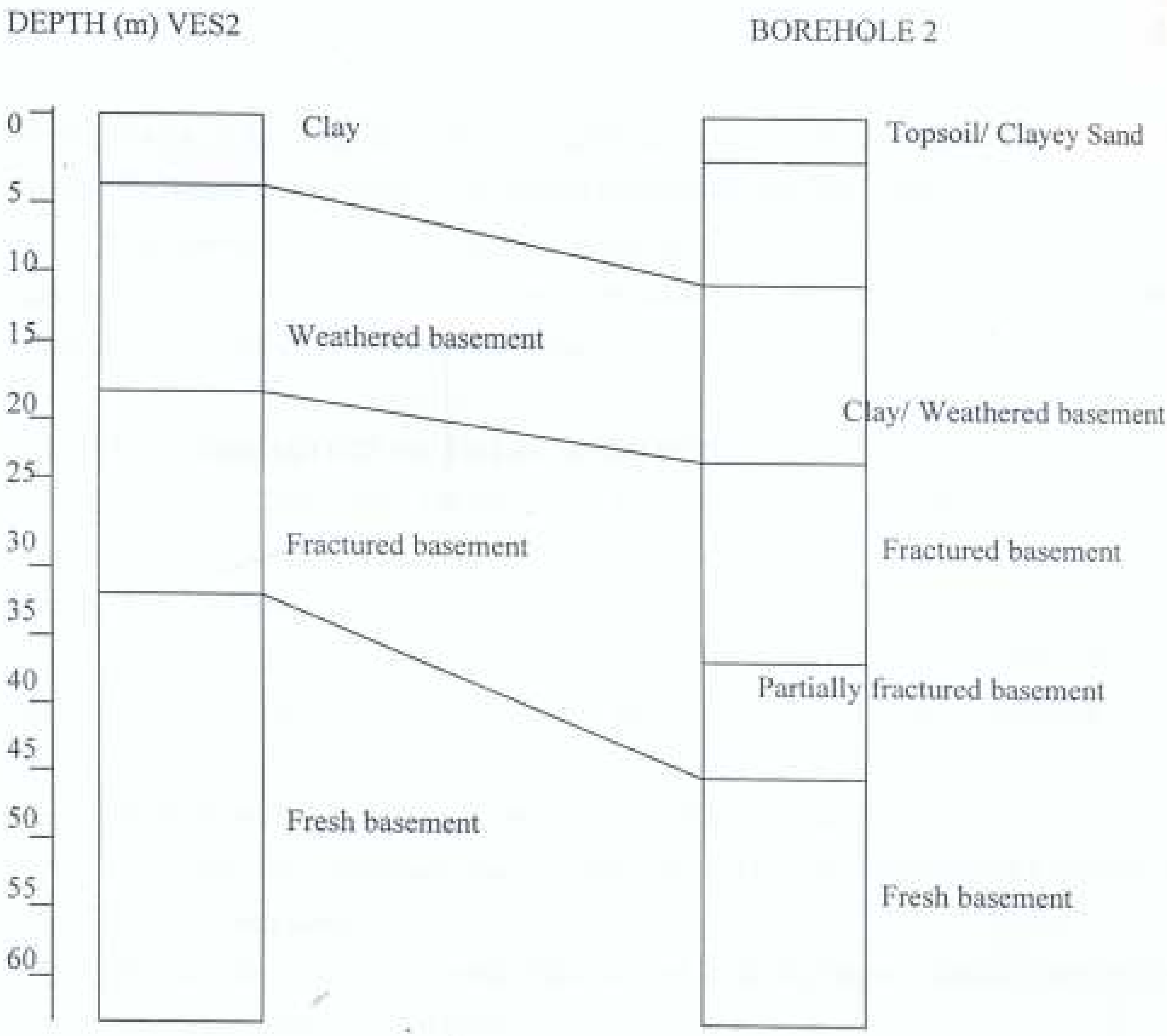


Fig. 4.2. Comparison of VES 2 Interpretation Results with Borehole 2 Lithological Section.

4.3 QUALITY CRITERIA

In any evaluation of groundwater resources the quality of the water is of almost equal importance to the quantity available. In other words the physical, chemical and biological characteristics of the water is of major importance in determining whether or not water is suitable for domestic, industrial or agricultural use (Hamill and Bell, 1986).

In conformity with World Health Organization (W.H.O) standards of water source, quality and suitable drinking water, water from each well was taken to the laboratory for analysis; the experiment is as described below.

4.3.1 DETERMINATION OF TOTAL HARDNESS

METHOD: - TITREMETRIC METHOD, USING DISODIUM SLAT OF ETHYLENE DIAMINE TETRA ACETIC ACID (EDTA)

PRINCIPLE: - Calcium and Magnesium in water form chelate complexes with EDTA. In the presence of Eriochrome Black T there is a colour change from pink to blue.

REAGENTS: - 0.01mol of Ethylene Diamine Tetra Acetic Acid and Eriochrome Black T

PROCEDURE: -

(1) 50ml of the Water sample was pipette into a 250ml conical flask.

(2) 2 to 3 drops of Eriochrome Black T (indicator) and two (2) drops of buffer solution for hardness was added.

(3) Ethylene Diamine Tetra Acetic Acid was put inside the burette, making sure that the jet of the burette was also filled.

(4) The readings of the burette was carefully taken and the content was run gradually to the sample mixture in the conical flask, shaking the flask in such a way as to give a rotary motion to the content until the colour changes from pink to blue.

(5) The burette reading was noted and the difference between the two burette readings gives the Titre value.

The total hardness is therefore calculated from the equation below:

$$\text{Total hardness} = (\text{Titre value} \times 1000)/\text{Sample volume}$$
$$= 1.70 \times 1000/\ 50$$
$$= 34\text{mg/l}$$

National Agency for Food and Drug Administration Control (NAFDAC) of Nigeria recommend that total hardness in water sample for human consumption should not be more than 100mg/l. It therefore follows that the water samples from the wells meets the recommended standard for human consumption.

4.3.2 DETERMINATION OF TOTAL RESIDUAL CHLORINE

APPARATUS: - The Standard Lovibond Disc (Range 0.1 – 3), Comparator.

PROCEDURE: -

(1) A test tube containing the water sample only was placed in the left hand compartment behind the colour standards of the disc.

(2) A similar test tube was rinsed with the water sample; 10ml of the sample was put in the tube.

(3) Outlidine indicator was added and mixed rapidly.

(4) The test tube was then placed at the right hand compartment of the comparator.

(5) It was matched after two minutes by holding the comparator facing the Lovibond white light.

(6) The disc was revolved until the correct standard was found.

The figure shown in the indicator window (0.2) represents part per million (PPM) of Total Residual Chlorine present in the sample. The World Health Organization (W.H.O) recommends 0.50PPM as the maximum allowable residual chlorine for any water sample meant for human consumption. Hence the water from the two wells is safe for human consumption.

4.3.3 DETERMINATION OF COLOUR

This is carried out using Lovibond colour disc with permanent colour ranging from 0–50 formazin haze with platinum-cobalt scale.

PROCEDURE: -

(i) A test tube was filled to the mark with distilled water and placed in the left sample position one on the comparator.

(ii) A second tube was also filled to the mark with the water sample and was placed in the right sample position.

(iii) The colour disc was then put rotated to match the colour.

(iv) The number of the disc was recorded which is 5.

4.3.4 DETERMINATION OF PH

This is carried out using Lovibond comparator ranging from 6.0 – 7.6

PROCEDURE: -

(i) A test tube was filled to the mark with the water sample and placed in the left sample position one on the comparator.

(ii) A second tube was also filled to the mark with the water sample, Bromothymol Blue indicator was added, the tube was then placed in the right sample position.

(iii) The colour disc was then rotated to match the colour.

(iv) The number of the disc was recorded which is 7.

The procedure was repeated using Lovibond comparator ranging from 7.6 – 8.4, and Phenol Red as indicator, the number of the disc was also found to be 7. This shows that the water from the two wells is neutral, that is, it is neither acidic nor alkaline. Hence it meets the World Health Organization standard for human consumption.

4.3.5 DETERMINATION OF COLIFORM ORGANISM

METHOD: - The method used for the determination and enumeration of coliform organisms is the Multiple-tube method.

PROCEDURE: -

(i) 50ml of the water sample was measured into a test tube; 10ml of the same water sample was measured into each of another set of five (5) test tubes, and 5ml into another set of five (5) test tubes.

(ii) 10ml, 5ml and 1ml of Macconkey Broth (food for the coliform organisms) was added to 50ml, 10ml and 5ml of the water sample respectively. The presence of any organism in the water sample produces bubbles, and the colour changes.

OBSERVATION: - No bubbles were produced and no colour changes. This is an indication that the water sample was free from any coliform organism.

The result of the analysis is as presented in table 4.2 below

TABLE 4.2 Quality Test Results

S/N		Well 1	Well 2	W.H.O Requirement	Remark
1	Hardness	Nil	Nil	Nil	O.K
2	Total Residual Chlorine (PPM)	0.2	0.2	0.5	OK
3	Colour on Lovibond colour disc	5	5	5	Colourless
4	PH On Bromo Thymol Blue	7	7	7 (Neutral)	OK
5	PH On Phenol Red	7	7	7 (Neutral)	OK
6	Coliform Organism in c/100mm sample	Nil	Nil	3	OK

The World Health Organization has prescribed the following minimum standards which any drinking water must satisfy for it to be recommendable for general public consumption.

(1) It must contain less than 0.5ppm of free residual chlorine.

(2) It must not contain exceeding 3 coliforms organism in c/100ml of sample of water.

(3) It must be free from acidic and alkaline indicators.

(4) It must be colorless.

From the results, it was discovered that the water samples are colorless, and neutral to acid and alkaline indicators. They are also free from all causative agents of water hardness. Hence they conform to the standard of good quality drinking water as prescribed by the World Health Organization.

4.4 VERTICAL SECTION OF THE BOREHOLES

Borehole 1

The vertical section of the borehole is as shown in Figure 4.3. From the figure it would be discovered that the total depth of the borehole is 50m, the submersible pump is installed at a depth of 45m inside the 50m long 150mm diameter Un-plasticized UPVC casing. The static water level is 5.5m while the dynamic water level is 9.5m with a drawdown of 4m, the length of the riser pipe is 36m. The borehole is proved with a securing rope while it is gravel packed to provide secondary filtration for the aquifer and also afford mechanical support for the casing and its ancillary structures.

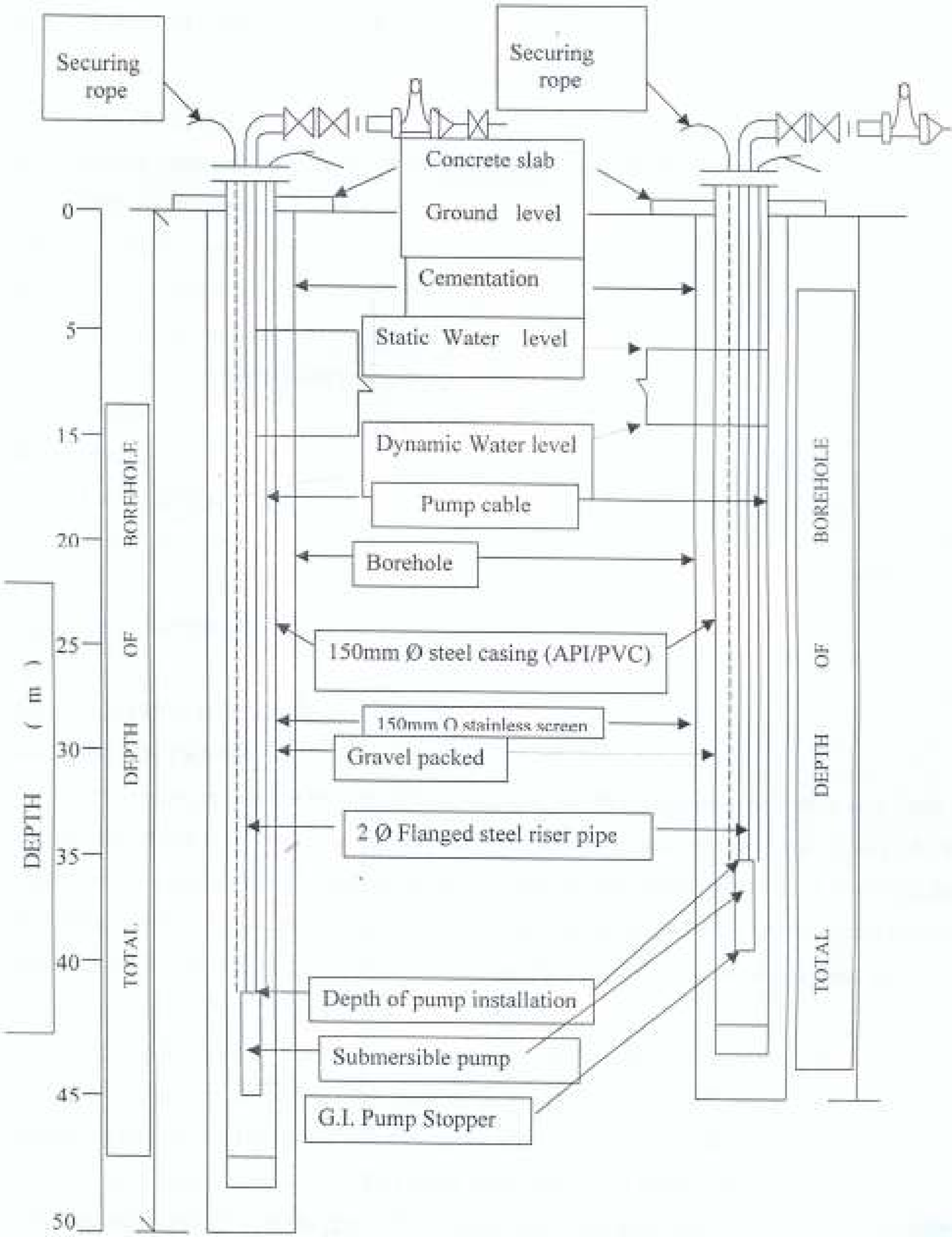
The summary of the borehole data is as follows

1	Drilling point	-	VES point 1
2	Depth of well	-	50m
3	Position of submersible pump	-	4.5m
4	Length of riser pipes	-	45m
5	Size of riser pipes	-	32mm ($1^1/_4$") Ø
6	Casing	-	150mm Ø UPVC
7	Screen length	-	9m × 115mm (3 unit of 3m × 150mm)
8	Gravel packing	-	Yes
9	Pump type	-	SEAR. 1.5H.p (1.13kw)
10	Pump head	-	60m

Borehole 2

The vertical section of the well is as shown in Fig.4.3. From the figure, the following data could be revealed.

1	Drilling point	-	VES point 2
2	Depth of well	-	45m
3	Position of submersible pump	-	40m
4	Length of riser pipes	-	40m
5	Size of riser pipes	-	32mm ($1^1/_4$") Ø
6	Casing	-	150mm Ø UPVC
7	Screen length	-	6m × 115mm (3 unit of 3m × 150mm)
8	Gravel packing	-	Yes
9	Pump type	-	SEAR. 1.5H.p (1.13kw)
10	Pump head	-	60m



WELL 1

Figure 4.3. Vertical sections of the drilled holes

WELL 2

4.5 LAYING AND INSTALLATION OF ALL COMPONENTS OF THE WATER SCHEME

4.5.1 MATERIALS

(1) 100mm diameter Un-plasticized Polyvinyl Chloride (UPVC) pipes.

(2) 75mm diameter ductile iron pipes.

(3) 50mm diameter ductile iron pipes.

(4) 32mm diameter ductile iron pipes.

(5) 28mm diameter ductile iron pipes.

(6) 1.5Hp Submersible pumps.

(7) Tanks.

(8) Filters.

(9) Fire hydrants.

(10) Washout valves

(11) Stop corks (Gate valve type).

(12) Tees and elbows.

4.6 LAYING OF PIPES

4.6.1 MAIN TRUNK

The pipes and all the fittings were brought to site, the pipes were provided with timber support on which they were laid carefully one above the other to preserve the quality of the pipes. An even bed of trench were made for the pipes on the previously marked out areas, all large stones were removed from the bed, so that they do not straddle the trench. Strict control was exercised over the laying of pipes because of the high capital cost of pipelines and their swift deterioration if not properly laid.

When pipes were delivered, and again just before they were lowered into the trench, they were inspected for flaws. Any coating or lining flaws detected were made good. The interior of the pipes were inspected as the pipes were lifted and any debris was brushed out.

The 100mm diameter (UPVC) pipes were laid in the trench, and all the fittings were kept secured together with the help of PVC gum (e.g. Oaty gum) and eliminate the possibility of leakages. Moreover, a principle requirement for satisfactory pipe laying is care in making the joints, other mechanical devices on the main trunk are Stop corks, Washout valve (all are of gate valve type), Fire hydrants and Air valves to the pipeline with Oaty gum.

Chambers were constructed for the washout valves and the fire hydrants. The washout valves were located at the lower point on the pipeline and discharge by gravity into the nearest watercourse. The fire hydrants were located very close to the road for easy accessibility for fire fighting equipment, (See Fig.4.1).

Length of pipeline that were laid to even grades CP2010, suggests not less than 1:500 on a rising grade (in the direction of flow) and not less than 1:300 on a falling grade. For distribution pipes, which have service connections, a flat grade does not particularly matter, as air will be drawn off via the services connections. On trunk pipeline however, it is important, hence, even rises to air valves were arranged.

Backfilling to pipes were placed in even layers either side of the pipe up to surface level and in addition, for non-rigid pipes the backfill was very carefully compacted to keep the pipe in a true cylindrical form. The backfill material adjacent to the pipe and for 150mm above its crown was free of large sharp stones that could puncture the sheathing of the pipe.

Achieving cleanliness in a muddy trench is far from easy, therefore, the men were provided with the facilities required, such as clean water and buckets, plenty of wiping rays and enough room to work and time to make the joints properly. Cover to pipe is normally not less than 0.9m with 1.2 to 1.3m is necessary to check that they are strong enough to withstand the soil loading.

4.6.2 SERVICE LINE

The Maning formula gives

$$Q = K\, d^{2.67}$$

Previously, 32mm diameter pipes were used to supply water from the state water cooperation into the hospital, giving the flow rate to be

$$Q = K\,(32)^{2.67} = 1.02 \times 10^{-4} m^3$$

Where K = Constant

Increasing the pipe diameter to 100mm gives the flow rate to be

$$Q = (100)^{2.67} = 2.14 \times 10^{-3} m^3$$

This implies that increase in the diameter of a pipe by 30% double its flow capacity. Hence, 25mm and 32mm diameter pipes were used to service the buildings, depending on the daily demand of each building, to replace the existing 19mm diameter pipes.

The service lines to the buildings were connected to the mains using saddles and ferrules. A little hole, about 5mm diameter was punched on the 100mm diameter pipe with a punch or nail. The hole must not be too wide because of the high pressure in the main trunk.

The ferrules are connected to the saddles and the saddles are connected to mains with bolts and nuts through the saddle straps having rubber seal in- between the saddle and the mains. The ferrules are then connected to the services pipes with the help of union connector.

4.6.3 SUBMERSIBLE PUMP INSTALLATION AND WATER SUPPLY

A 1.13kw submersible pump was installed at the depth of 45m in each of the borehole with the help of a securing rope and 38mm diameter ductile iron riser pipes. The first riser pipe was screwed into the outlet of the submersible pump with tread tape in order to make the joint leak proof. The pump together with the first riser pipe was then lowered into the well while the securing rope is tied to a pole or stud; this prevents the pump from falling freely into the well and thereby damaging the pump and above all condemning the hole. The securing rope was being released gradually as the pump was lowered into the well until the riser pipe was about 30mm above the mouth of the borehole. A vice was then used to suspend the pipe: subsequent pipes were then connected to each other using 38mm threaded ductile iron sockets. This process continues till the required installation depth was reached. It is always advisable not to install pumps at the same depth of your hole so as to give room for likely settlement, which can not be completely ruled out. This prolongs the life of the pump, and the hole serves you better.

The last riser pipe was connected to the socket located at the center of the borehole cover made of 5mm thick steel flat plat. On the outer part of the cover, connection of pipes continues with other fittings like nipples, elbows, stop corks, union connectors. The pipes were thereafter laid through to the treatment plant.

4.7 THE TREATMENT PLANT

At the treatment plant the water flows into 2,000litre tank located at an elevation of 3.5m to cause flow into the three pressure filters, arranged in series, by gravity. (See fig 5.1 treatment plant layout). The filtered water then goes into a 1,000litre tank before it finally get into the underground tank. From the underground tank, it is pumped by booster pump into the overhead tank for distribution. For the purpose of maintenance or breakdown of the treatment plant, the pipeline is designed with a diversion system, such that the plant is by-passed and the water gets into the underground and from there, it is pumped directly into the overhead tank. Also the water in the 100litre tank can be pumped directly into the overhead tank without flowing into the underground tank to allow for maintenance of the underground tank.

Water supply from the water corporation was also directed into the underground tank and from there it is pumped into the overhead tank for distribution into the hospital.

4.8 FLOW RATE AND ENERGY LOSSES

The energy addition and losses in the pipeline system are:

hA = Energy added to the fluid with a mechanical devices such as a pump.

hR= Energy removed from the fluid by a mechanical device such as motor.

HL = Energy loss from the system due to friction in pipes or minor losses due to valves and fittings.

4.9 EFFICIENCY OF THE DESIGN

The real flow rate of the designed water supply scheme was determined practically in order to determine the efficiency of the scheme

Methods

A 100mm diameter hose was fixed with clips into the exit of the washout valve located at the lowest level point.

Four different tanks of 2000litre were positioned such that the hose could be directed into each of the tank when the washout valve is opened.

The water was allowed to run into the tanks for a period of 60 seconds each, monitored with a stopwatch. At the end, the volume in each of the tank was determined thus:

Volume (v) of the tank = $\pi r^2 h$

Where,

r = radius of the tank = 0.6m

h = height of the tank, (variable).

This implies that the volume of water inside the tank is a function of the level of the water inside the tank. Hence, the volume of water in each tank is as given in table 4.3 below.

TABLE 4.3 Volume of Water Collected at 60 Seconds Interval.

Tank	Level of Water (h)	Volume of water after 60s ($\pi r^2 h$) m^3/min
1	0.80	0.90
2	0.75	0.85
3	0.70	0.79
4	0.75	0.85

The mean valve is 0.85 m^3/min.or 1.41×10^{-2} m^3/s

Hence,

the flow rate $Q = 1.41 \times 10^{-2}$ m^3/s

The efficiency of the design is given by

eff. = (Actual flow rate/ Designed flow rate) × 100%

The designed flow rate is 1.65×10^{-2} (Page 32)

eff = (1.41×10^{-2}/1.65×10^{-2}) × 100%

= 85.5%

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION

Two boreholes were drilled at points specified by geophysical survey report. The recharging rates are 2.50littres per second and 2.10littres per second for wells 1 and 2 respectively. A submersible pump of 1.13kw (1.5Hp) with minimum flow rate of 1.5littres per second was installed in each of the borehole. The water from the two boreholes was pumped with the 1.3kw pump, simultaneously, to fill each of the 50,000littre tank in less than four hours forty-five minutes.

The parallel pipeline system designed and constructed has flow rate of 1.65×10^{-2} m^3/s. Moreover, the recharge rate, 2.50littres per second and 2.10littres per second for boreholes 1 and 2 respectively is more than enough to support the hospital demand.

The Total energy loss h_L in the network = 4.70m

Pressure head P/γ_1 in 75mm diameter pipe = 2.5m

Pressure head P/γ_2 in 100mm diameter pipe = 3.25m

Velocity head $V^2{}_1/2g$ in 75mm diameter pipe = 0.71m

Velocity head $V^2{}_2/2g$ in 100mm diameter pipe = 0.23m

Elevation head Z = 7.80m

5.2 SUGGESTION FOR FURTHER WORK

Further work could be done in future on the following:

(1) Vary pressure head, elevation head, and velocity head, and determine its effect(s) on the flow rate and energy losses.

(2) Design of a thermal station for production of hot water for hospital consumption.

(3) Improve on the design to serve a larger community.

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NOMENCLATURES

g	=	Acceleration due to gravity
A	=	Cross sectional area
ρ	=	Density
D	=	Diameter
μ	=	Dynamic Viscousity
Z	=	Elevation
h_L	=	Energy Loss due to Friction
L_e	=	Equivalent Length
Q	=	Flow Rate
F	=	Force
f	=	Frictional Force
ν	=	Kinematic Viscousity
L	=	Length
C_L	=	Loss Coefficient
m	=	Mass
h_A	=	Mechanical Energy Added
h_R	=	Mechanical Energy Removed
π	=	Pi
ε	=	Pipe Roughness
P	=	Pressure
R_e	=	Reynolds Number
ω	=	Specific Weight
V	=	Velocity
w	=	Weight
$\Delta\dot{Q}$	=	Correction in Discharge
Σ	=	Summation
K	=	Equivalent Resistance to flow