THE UPGRADING OF PRIVY METHODS OF SEWAGE DISPOSAL BY THE BIODISC PROCESS

by

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TO

MY WIFE BISI,
FOR HER PATIENCE, UNDERSTANDING
AND ENCOURAGEMENT

A B S T R A C T

This thesis describes studies carried out on the septic tank and the nightsoil conservancy system, the two most widely used methods of sewage disposal in Lagos and other parts of Nigeria.

A background survey of the problems of waterborne sanitation in Nigeria is given. Even though these problems will make municipal waterborne sanitation remain for a long time an unaccomplished ideal in most towns in Nigeria and other developing countries, this work holds that where the right conditions exist, certain sections of such towns could be provided with limited waterbone sanitation facilities. A survey is made of a number of such facilities in the Lagos Area in educational institutions, hospitals, army barracks, housing and industrial estates in which these conditions exist.

Results are reported of laboratory scale model tests to show that only a little strip at the bottom of the standard circular design of the soakaway that has been used in the Nigerian building practice for some four decades is effective in leaching away tank effluents; suggestions for an alternative design are made.

A critical appraisal is made of the empirical formula for converting the percolation rate of water in a soil to the rate of sewage loading on the soil, together with a suggested modification of the original American method for percolation tests in the sandy-laterite soils of Lagos.

The results of a six-year study on the water table in parts of Lagos are reported. They indicate that the water table is too near ground level to make the septic tank system an effective method of sewage disposal in these low lying areas of Lagos.

The biodisc process is discussed together with the results with the laboratory scale model in the treatment of milk, domestic sewage, nightsoil and industrial wastes obtained from different parts of the Lagos Metropolitan Area. The results show that the process is efficient in the treatment of all of them in terms of reduction in Chemical Oxygen Demand (COD), Biochemical Oxygen Demand (BOD) and Suspended Solids (SS).

The thesis ends with a design for converting a septic tank into a biodisc plant, and discussion on the possibilities of the biodisc process as an inexpensive alternative to the septic tank in the low-lying areas where the tank is ineffective, and to the relatively expensive package plants based on the Activated Sludge Process now existing in a number of institutions in the Lagos Area. It is hoped that the findings in these studies will be a contribution towards the solution of the serious problem of sanitation in Nigeria and other countries with similar problems.

P R E F A C E

This Thesis is divided into three sections.

The first section (Chapters I and II) is introductory, dealing with the problems of water borne sanitation in Nigeria and a survey of the limited facilities in waterborne sanitation in a number of public institutions in the Metropolitan Lagos Area where the conditions are favourable for the installation of these facilities.

The second section (Chapters III - VII) deals with the nightsoil conservancy system and the septic tank, the two methods of sewage disposal mostly used in Lagos. This section deals in some detail with the ineffectiveness of the septic tank - cum - soakaway in the low-lying areas of Lagos and the suitability of some soils in the Metropolitan Lagos Area for leaching away septic tank effluents.

The last section (Chapters (VIII - X) deals with the Biodisc Process and experiments with the process in the treatment of wastes in Lagos. It includes a chapter (Chapter X) on a design for converting an existing septic tank into a biodisc treatment plant and another (Chapter XI) on Discussion and Conclusions.

The Thesis ends with a Bibliography.

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CHAPTER 1

INTRODUCTION: PROSPECTS FOR WATERBORNE SANITATION IN NIGERIA

1.1 THE PROBLEM OF INCREASING POPULATION

Food experts have for a long time expressed concern over the fact that the overall world population appears to be growing at a faster rate than the world's food resources. What has now become another cause of concern in the alarming rate of world population growth, this time to the international health community, is man's inability to keep pace with this growth in the provision of an extension of facilities for removing from his immediate environment the wastes from his body, his dwellings and his industries, and for their safe disposal after suitable treatment.

This latter problem is serious the world over. Even in the more developed countries of Europe and North America where most towns have the central sewerage system the World Health Organisation reckons that the rate at which extensions are being made to these existing facilities and the rate at which sanitary engineers, sewage plant operators and technicians are being trained in these countries are not keeping pace with the rate at which the communities are growing (LOGAN, J.A. 1967).

The situation is grievous in the developing countries. While in 1967 the overall world population growth rate was 1.9% per annum, the population in West Africa was increasing at 2.5% per annum, and that in Nigeria at 2.7% per annum (U.N.O., 1967). Further in most communities in the developed world, only extensions are required to existing facilities but in most developing countries existing facilities are not adequate to meet immediate demands. Arrangements therefore need to be made from scratch for these communities that are growing at a faster rate than their counterparts in the developed countries. Co-existing with these disadvantages in the developing countries are the other serious consequences of under-development: a low gross national product, low technological development, inadequate education at all levels and the lack of appreciation of the need to invest so much of scarce natural resources in schemes that have little political appeal.

If the overall population growth is fast, the cities of the world are growing faster still. The World Health Organisation reckons that urban growth rates are more than twice the national rates (HANSON, H.G. 1967). This means that more and more people crowd into the cities. These in consequence grow either vertically upwards, leading to great population densities, or horizontally, leading to urban sprawl. In either case, increasing quantities of wastes from a rapidly increasing urban community now have to be collected and transported over long distances before treatment and subsequent disposal. Authorities in countries where city growth is properly planned and controlled are barely able to keep pace with the needed extensions to the existing sewerage and sewage disposal facilities. Ajegunle, Shomolu and Ketu in the suburbs of Lagos in Nigeria are examples of the chaos that occurs where city growth is little controlled, and sewerage and sewage disposal facilities are practically non-existent.

1.2 PRESENT DAY SEWERAGE TECHNOLOGY

The most satisfactory method known to technology today is the central sewerage system in which a network of sewers collect and carry the raw sewage from plumbered buildings to treatment works and thence to disposal points. Three essential processes are involved in the system: collection and transportation of sewage, its treatment, and finally its disposal. At the treatment works the sewage undergoes processes essentially designed either to remove the harmful suspended, colloidal or dissolved solids in the sewage fluid, or to convert these into harmless and less complex substances which can be more easily disposed of. Disposal of the harmless effluent is usually in a large body of water like he sea, a lake, the lagoon or a river.

It is not all cities in the developed countries, and there are very few indeed in the developing countries, that have the central sewerage system. These other cities employ other less complex but less efficient methods for their municipal sanitation. Of these the nightsoil conservancy system, also known as the bucket latrine, is widely used in many towns in Nigeria, though it is not as inexpensive as many think. It is very unpopular with all concerned:

(a) with health authorities as it fails the essential requirements of a good excreta disposal method,

(b) with the community which it serves as it is the cause of much irritation and annoyance to see nightsoil workers handle and transport on their heads buckets of foul smelling and half septic sewage, and

(c) with the workers themselves as they suffer a psychological and social stigma which makes them wear masks when they go on their rounds (ALUKO, T.M. 1971).

1.3 CENTRAL SEWERAGE AS AN ENGINEERING SERVICE

In modern city planning practice, the central sewerage system is considered an engineering service just like water supply, electricity and street lighting. These engineering services are installed as part of the initial operation of land development and in advance of building development. The capital cost of these services is therefore part of the cost of land development, and is reflected in the sale prices or lease rates of the developed land. The maintenance of the central sewerage system is usually the responsibility of the municipal authority or the responsibility of a semi-public body specially set up for the purpose.

In Nigeria there is as yet not a single town with the central sewerage system. It is hoped that new towns to be built in future will have this modern system as part of the engineering services being installed from the beginning. In particular the proposed new Federal Capital must have a central sewerage system. There should also be a programme of introducing this system to existing towns.

Water supply is in many ways akin to sewerage and sewage disposal. It is, like sewerage and sewage disposal, divisible practically into the same processes though in the reverse order. A number of the treatment processes are common, and the approach is practically the same for the design of the conduits that carry the water to the consumer and those that carry the sewage away from the householder. Indeed water supply and sewage disposal are complementary in sanitary engineering.

No cost data or construction experience in the central sewerage system exists in Nigeria. Some ten years ago the Federal Government had before it proposals for the waterborne sanitation of Lagos. The project included the network of sewers, pumping stations and treatment plant for sewage. It also included facilities for solid waste collection, treatment and disposal. The proposals were estimated to cost £178m (₦356m)* and were to serve an ultimate population of 4.35 million which was the consultants's forecast population of Metropolitan Lagos in 2000 A.D., the end of the plan period (SIMPSON, R.W. 1966). This works out at £41 (₦82) per capita for both liquid and solid waste disposal. The latter, however, accounts for less than (₦2) per capita.

∠capital

A reasonable amount of information exists on water supply∠costs in Western Nigeria. A study of 10 supplies completed in the 5-year period 1961 - 66 shows that the per capita cost varies from £3.23 in Oyo (pop. 271,500) to £11.65 in Shaki (pop. 50,500). The average per capita cost was £4.62 for an average population of 169,400 (Table 1.1). It is to be noted that the design standard of 10 gallons (45.5 litres) per head per day used in Western Nigeria water supply practice at that time is lower than the quantity of water allowed per head in modern communities. SALVATO, J.A. (1972) Says that the design figure in U.S.A. was 585 litres and that the corresponding figure for developing countries was 113 litres per capita per day but that a figure as low as 38 litres per day per capita is usual for these countries. Upgrading the per capita consumption to 20 gallons per day (91 litres per day) will probably raise the average per capita cost by 50%.

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*The estimates were given in £ sterling as they were made before the country adopted a new system of currency (in which the naira is the new unit) in 1973. The original rate of exchange between the £ sterling and the Nigerian naira was £1 = ₦2.00 (January 1973). The exchange rate in July 1976 was £1 = ₦1.18.

TABLE 1.1

WATER SUPPLY CAPITAL COSTS IN WESTERN NIGERIA

1961 - 1966

PROJECT	year COMMISSIONED	CAPITAL COST AT TIME OF COMMISSION (£)	CALCULATED CAPITAL COST AS FOR 1966 (£)	DESIGN POPULATION	COST PER CAPITA	
					AT TIME COMMISSIONED (£)	CALCULATED FOR 1966 (£)
1. Shaki	1966	589,800	589,800	50,500	11.65	11.65
2. Owo	1961	346,700	442,600	60,200	5.76	7.35
3. Ado-Ekiti	1961	308,700	394,000	56,800	5.44	6.94
4. Badagry	1965	84,500	88,740	14,150	5.98	6.23
5. Abeokuta	1962	862,000	1,043,000	170,000	5.07	6.16
6. Owena	1965	1,483,900	1,559,000	322,000	4.61	4.84
7. Ogbomosho	1964	1,105,600	1,219,000	293,000	3.77	4.16
8. Ijebu-Ode	1962	318,000	386,600	96,500	3.30	4.01
9. Ife-Gbongan	1965	1,164,000	1,223,000	359,000	3.25	3.61
10. Oyo	1961	687,100	877,000	271,500	2.53	3.23

NOTES:

1. Each Scheme provided for both public stand-pipes and individual house connections.
2. Average cost per capita as at 1966 = £4.62
3. Figures are given in £ sterling as the projects were constructed before the change to the new currency in January 1973, when £1 = ₦2.00
4. 1966 costs were computed from actual costs as at date of commission at an assumed growth rate of 5%.
5. Owena scheme comprises the Ondo-Akure-Idanre-Ikere complex.

From the little information available on electricity capital costs in Western Nigeria the average per capita cost appears to be under £4.00 for supplies constructed about the same period (HOWEIDY, A. 1966).

This analysis shows that sewerage is by far the most expensive of the three services per capita. This is the first and perhaps the most serious problem of the central sewerage system in Nigeria and other developing countries, that is, the relatively high capital cost of a service the need for which is little appreciated by the people and their political leaders, in comparison with other engineering services.

1.4 SEWERAGE, TOWN PLANNING AND SLUM CLEARANCE

£126m of the £178m estimated capital cost of the scheme for the waterborne sanitation of Metropolitan Lagos mentioned earlier was for the construction of the network of sewers for collecting and transporting the sewage; this is 71% of the total cost. All the other components of the scheme consisting of pumping and treatment equipment for the sewage, buildings and composting plant were estimated to cost the remaining 29%. The construction of the sewerage or the network of sewers is therefore the most expensive item in the central sewerage system. This is a very important point in this thesis.

Expensiveness of installation apart, the construction of a network of sewers in an existing town will be accompanied by dislocation of traffic and general disorganisation of business in the community. In parts of Metropolitan Lagos where the land is very flat and the soil sandy relatively deep excavations will be inevitable to obtain the required gradients in the sewers. Apart from the disorganising effect of the excavation and pipe laying activities, there is the time consuming procedure of public acquisition of land now under different and private ownership.

In most of the older towns growth has been little controlled. In large areas around the hubs of these towns the street pattern is hardly discernible and there are chunks of building development where individual buildings do not face on to any streets. Such streets as exist are narrow and building development on them does not follow an identifiable building line. The older parts of Lagos Island and most of the older parts of the city of Ibadan fall under this category. It is in these areas that the difficulties of excavating for and laying sewers to the inaccessible properties are greatest.

The disorganizing effects and high cost of sewerage in existing towns will be less serious in cases where town growth has followed accepted town planning principles. This will be the case in the newer colonial type towns, like Kaduna and Port Harcourt, the Sabongaris outside the native cities of the Northern States, and the later extensions to existing towns like Ebute-Metta in Metropolitan Lagos. The adverse effects of uncontrolled town growth are reduced considerably in view of the grid-iron type layout in these towns.

It is considered that the only effective way of tackling the problem in these cases is to combine sewerage with slum clearance and rehousing. There is little financial wisdom in spending so much money on the construction of a public sewer along an existing street which in a few years will cease to be a street, or through private property to a mud house of little value which will in a few years cease to exist. Unfortunately it is in these central areas of the older towns that population densities are greatest and the rehousing problems that must follow slum clearance are most serious.

1.5 SEWERAGE AND URBANISM

The prospect for the central sewerage system in Nigeria is made further difficut by the relatively high degree of urbanism in parts of the country. The 1963 census indicates that there were 23 towns with populations of 100,000 and above, sharing a total population of 5,043,386 which is 9.1% of the total population of 55,670,046 for the whole country (Table 1.2). 14 of these 23 towns with a total population of 3,579,160 or 71% are situated in the Yoruba-inhabited south-west area of the country, within a radius of some 160 km of Ibadan (Fig. 1.2). It is noted for comparison that there was not a single city of up to 100,000 population in Liberia, Senegal, Mali or Gabon in 1963. There was only one such city in each of the following countries: Sierra Leone, Uganda, and Tanzania. There were two in Ethiopia, two in Kenya, two in Sudan and three in Ghana (Fig. 1.1).

The main problem posed by a large town is the magnitude of the capital construction of a central sewerage system relative to other undertakings like schools, hospitals, and electricity supply. The £178m estimated expenditure on the sewerage scheme for Lagos stared the Lagos State Government in the face for a long time. Proposals for the sewerage of Lagos submitted by consultants at fairly regular intervals to successive Governments never got off the mark due to the high costs relative to the government budget and the competing needs of other and cheaper areas of development. If Lagos had been the size of Freetown (148,000) or Blantyre (109,795) or Libreville (57,000) the estimate for a central sewerage scheme would have gone down to a size more acceptable to the authorities. Since in the relatively small but densely populated south-western area of the country alone there are 14 towns with populations of 100,000 and above, the problem of embarking on the central sewerage system for them and the many smaller towns assumes serious dimensions.

TABLE 1.2

TOWNS OF 100,000 POPULATION AND ABOVE IN NIGERIA

(1963)

No.	TOWN	POPULATION
1.	Aba	131,003
2.	Abeokuta	187,292
3.	Ado-Ekiti	157,519
4.	Benin	100,694
5.	Ede	134,550
6.	Enugu	138,457
7.	Ibadan	627,379
8.	Ife	130,050
9.	Ikerre	107,216
10.	Ilesha	165,822
11.	Ilorin	208,546
12.	Iwo	158,583
13.	Kaduna	149,910
14.	Kano	295,432
15.	Lagos	665,546
16.	Maiduguri	139,965
17.	Mushin	312,063
18.	Oṣogbo	268,966
19.	Ogbomosho	343,279
20.	Onitsha	163,032
21.	Oyo	112,349
22.	Port Harcourt	179,563
23.	Zaria	166,170
	Total	5,043,386

The population of 5,043,386 for the 23 towns represents 9.1% of the total population of 55,670,046 for the whole country in 1963.

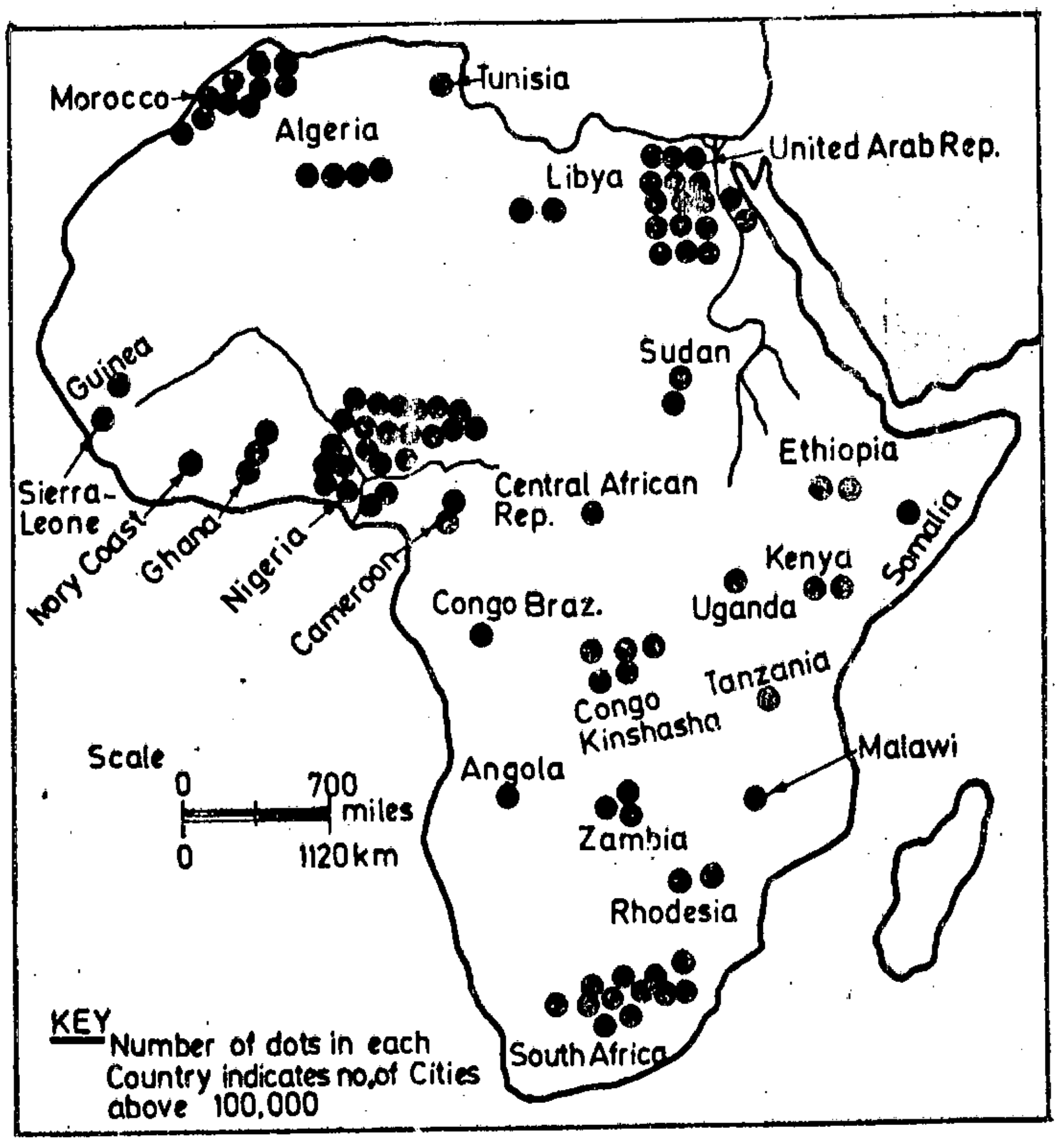
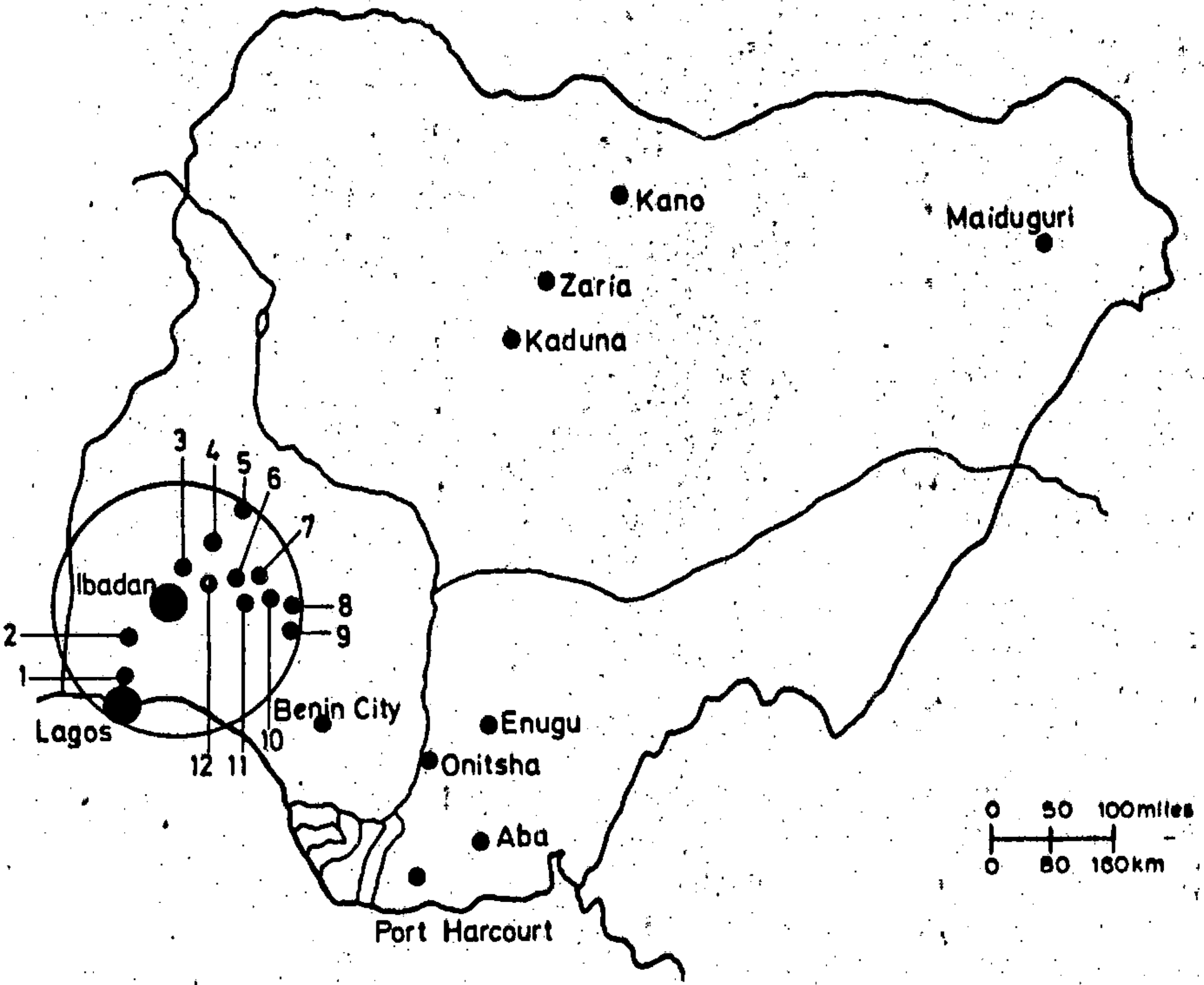


FIG. 1·1 AFRICA SHOWING CITIES OF 100,000 POPULATION AND OVER (1963)



Key to Cities in Circle

1	Mushin	7	Oshogbo
2	Abeokuta	8	Ado Ekiti
3	Oyo	9	Ikere
4	Ogbomosho	10	Ilesha
5	Ilorin	11	Ife
6	Ede	12	Iwo

Note:

14 out of the 23 Cities are within 100 miles (160 km) of Ibadan in the south-west of Nigeria

FIG. 1·2 CITIES OF 100,000 POPULATION AND OVER IN NIGERIA (1963)

The pattern of urbanism in Nigeria is the urban sprawl in which the town extensions are mostly in one or two-storey developments. These extensions are seldom characterised by the high densities associated with the older parts of the towns. But while sewerage works in them may be free from slum complications, they involve longer lines of sewers and higher pumping costs per acre of land served compared with the other parts.

Waterborne sanitation for little towns also has problems. Small plants employing conventional methods for treating sewage from small population cost more per capita than large plants designed for large towns and cities. But since in most countries more than half of the population lives in small communities of a few hundred to a few thousand inhabitants (PASVEER, A. 1966), there is a need for the development of sewage treatment plants employing methods that are less expensive than those employed in conventional plants. Of these so-called low-cost methods the septic tank has been used extensively in Nigeria and other developing countries, the oxidation pond has been used in some institutions in Nigeria, while the aerated lagoon has now been introduced in Nigeria.

1.6 CASE FOR AN INTERIM PROGRAMME OF WATERBORNE SANITATION FACILITIES IN SUITABLE AREAS IN TOWNS

Municipal waterborne sanitation will for the reasons already explained in this chapter remain an unrealisable ideal for many towns in Nigeria for some time to come. Research into new methods and new materials will, it is hoped, bring down the per capita cost of this modern and efficient method of sewage disposal. The Nigerian economy will hopefully continue to rise. At a point of time in future the two things, possibly catalysed by international aid, will make it a reality in the Nigerian town. It is hoped that this time will not be very distant.

There are however certain sections of these towns which even now can have the benefit of some form of limited waterborne sanitation facilities, provided certain criteria are met, without having to wait till the coming of the central sewerage system. The analysis of the central sewerage capital costs has shown that most of the cost goes into the actual sewerage, that is, the network of sewers. In the Lagos proposals earlier mentioned the sewerage accounts for 71% of the total estimate. This leads to the obvious conclusion that maximum economy in the sewerage system will lead to savings which might bring within realisation now a scheme that would otherwise have had to be shelved when sewerage costs bear a disproportionate fraction of the total cost. This is the first criterion. Again there are in the community building and estate developers who can afford and are willing to stand the expenses of waterborne sanitation in their individual developments where building development is under single control and better still, under single ownership. This is the second criterion. It is met in the building of the ordinary dwelling house where the owner-developer decides to build a septic tank within his own premises. It is met in the University where the authorities build a small sewage treatment plant on the campus. It is also met in the Government Residential Area where Government can decide to install a sewage plant on land which it owns to serve buildings which it also owns. A variation of this criterion occurs where even though the land is under single ownership actual building development on it is under different ownerships but single control. The Lagos State Development and Property Corporation estates in Ikoyi, Victoria Island, Apapa and parts of Surulere and the Western Nigeria Housing Corporation estates in Ibadan and Ikeja are in this category.

In the next chapter a number of sewage treatment plants in parts of the Lagos area which meet the criteria mentioned in the last paragraph are discussed. It is shown in Chapter III that the majority of premises in Lagos are on the septic tank and in Chapter VII, that the ground water is too high in the low-lying areas to make the septic tank soakaway effective. In Chapter VIII the biodisc process is discussed and the encouraging results obtained in the tests on this process reported in Chapter IX suggest that the process can be used in upgrading existing septic tanks particularly those in low-lying areas as high water table has no adverse effect on the process. Finally in Chapter X calculations are made for upgrading the size V tank which is the largest size in the standard P.W.D. design discussed in Chapter III.

Fig. 1.3 shows a map of the Metropolitan Area for reference purpose.

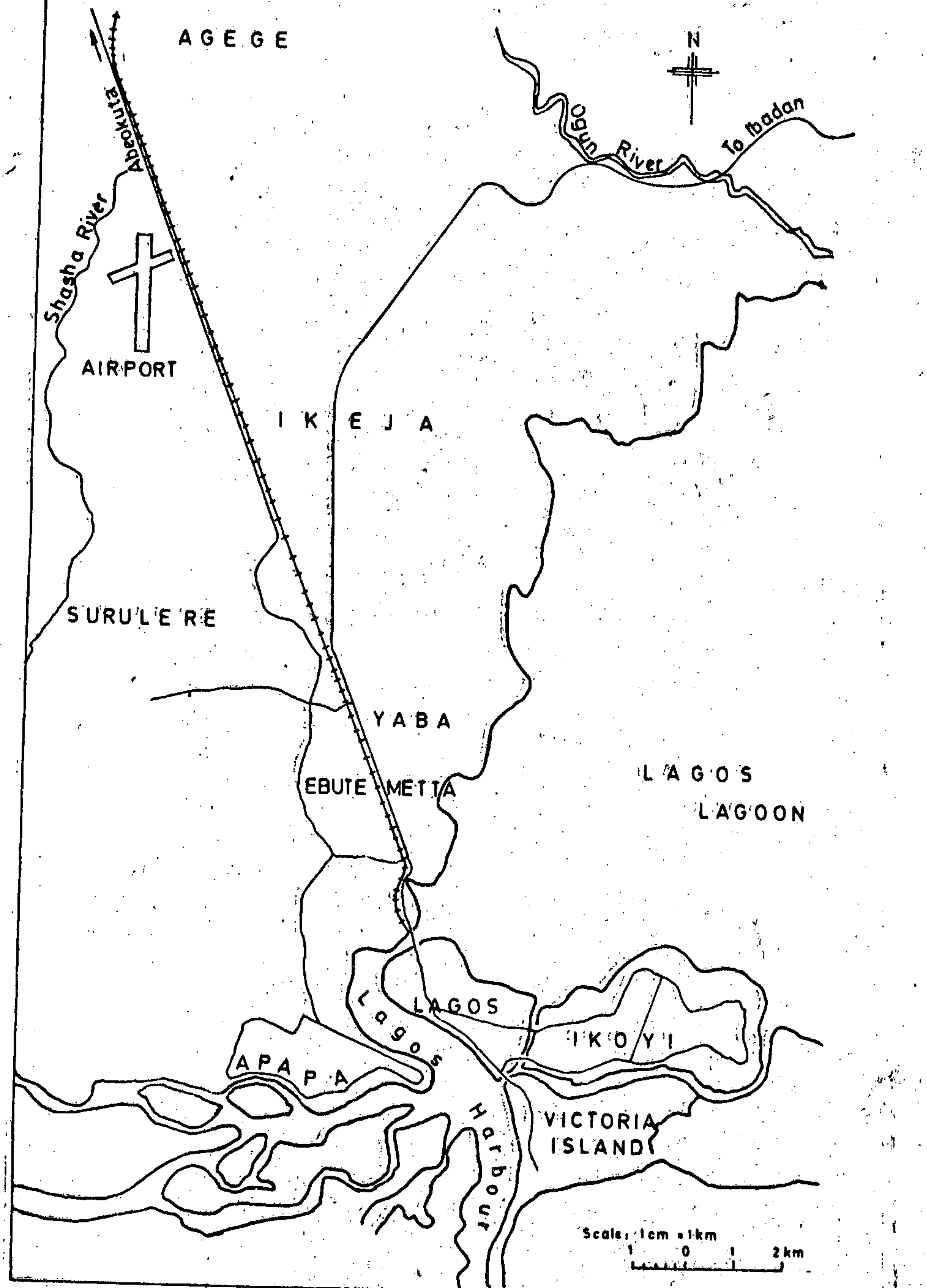


FIG. 1·3 METROPOLITAN LAGOS AREA

CHAPTER II

WATERBORNE SANITATION IN LAGOS

2.1 It was shown in Chapter I that while the central sewerage system is the most satisfactory method of waterborne sanitation in cities, it is very expensive, and is likely to remain an unrealised ideal in many towns in Nigeria for some time yet. It was however argued in that Chapter that given certain conditions there are certain sections in some of these towns that can have the benefit of some form of limited waterborne sanitation facilities now, without having to wait for the installation of the central sewerage system in the whole town.

There are in such towns estate developers and public authorities who own institutions and can afford and are willing to pay the expenses of waterborne sanitation within these estates and institutions. Such authorities could be a housing corporation with respect to its Estate, a University Council with respect to the University Campus, or a Government with respect to a residential area where its civil servants live. In this Chapter a number of schemes in the Metropolitan Lagos Area are described to illustrate how the various authorities concerned have taken advantage of the existence of the right conditions to provide waterborne sanitation facilites in their respective areas. The locations of these schemes are shown in Fig. 2.1.

2.2 UNIVERSITY OF LAGOS SEWAGE TREATMENT PLANT

The first of these treatment plants is the "Oxigest" Model 31R 80 which was built at the University of Lagos in 1965. It was designed to handle the canteen and domestic waste from a population of 1,000 from buildings covering a substantial area of the campus, and has a design capacity of 50,000 gallons per day (227.5m^3/d). It is a package plant in which the preliminary, primary and secondary processes all take place in a single unit but in different compartments as shown in Fig. 2.2.

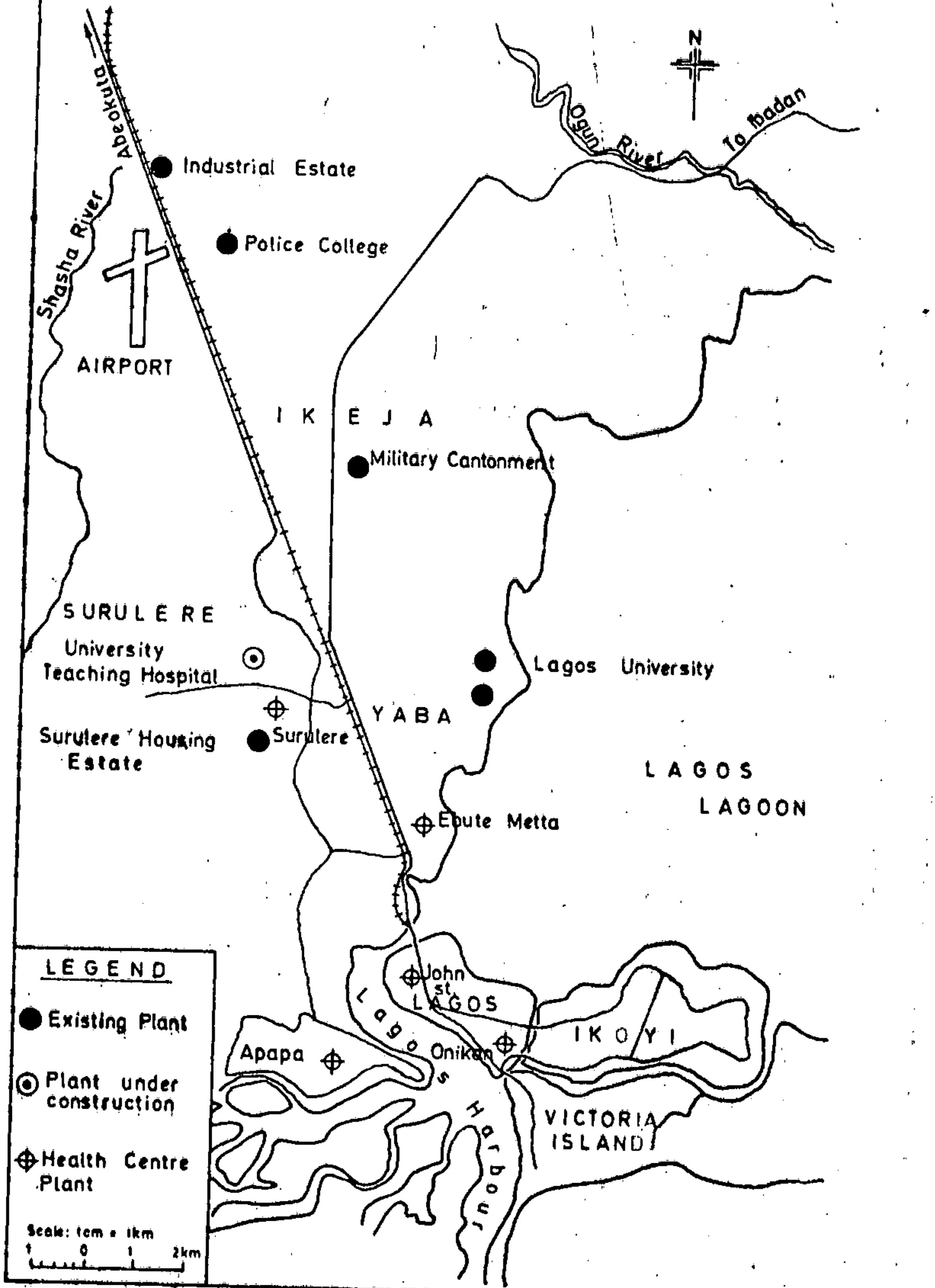
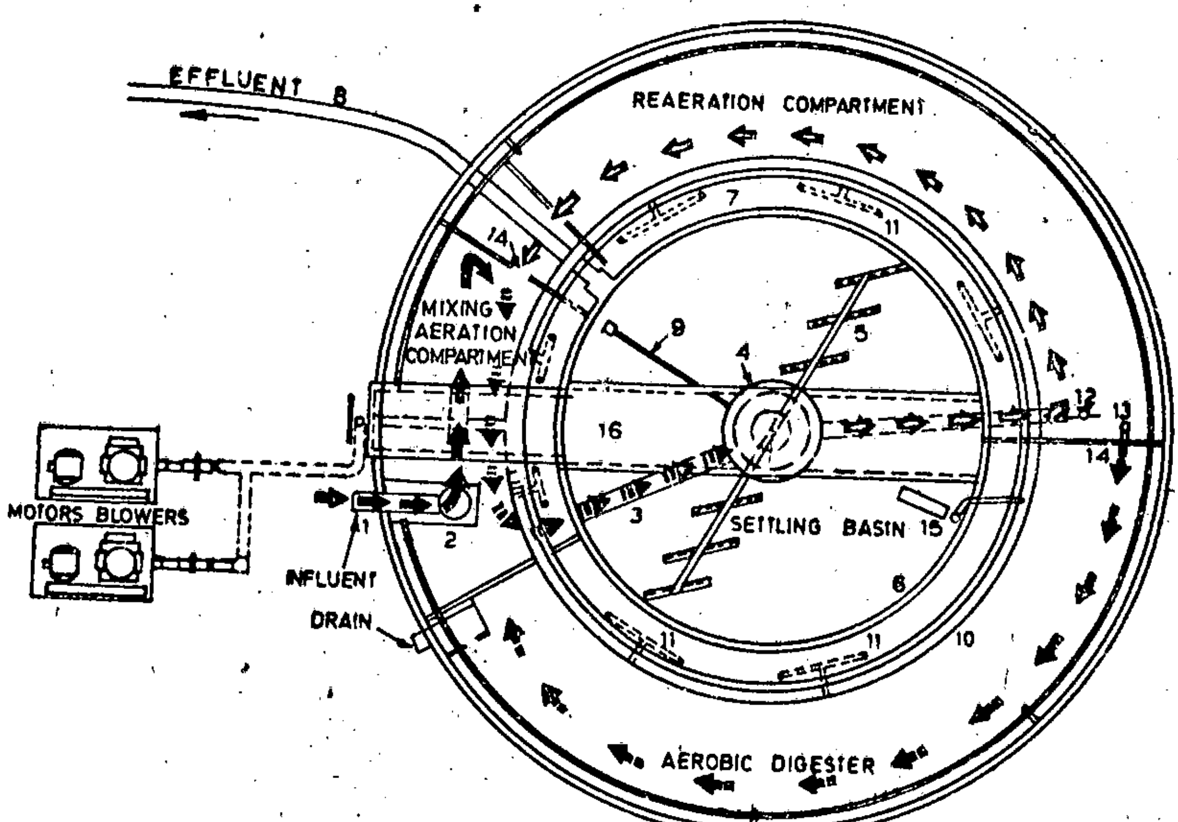


FIG. 2·1 EXISTING SEWAGE TREATMENT PLANTS IN METROPOLITAN LAGOS



INCOMING SEWAGE

WASTED ACTIVATED SLUDGE

MIXURE OF INCOMING SEWAGE & ACTIVATED SLUDGE

ACTIVATED SLUDGE

1 INFLUENT PIPE
2 COMMUNITOR
3 INLET PIPE TO SETTLING BASIN
4 INLET STILLING WELL
5 SLUDGE SCRAPER
6 WEIR
7 EFFLUENT TROUGH
8 EFFLUENT PIPE
9 SURFACE SKIMMER
10 AIR HEADER
11 DIFFUSER
12 AIR LIFT FOR SLUDGE FOR REAERATION
13 AIR LIFT FOR SURPLUS SLUDGE
14 GATE
15 SCUM TROUGH
16 STEEL BRIDGE

FIG. 2.2 PLAN OF MODEL R OXIGEST SEWAGE TREATMENT PLANT

The Oxigest Model 31R 80 is a contact stabilisation modification of an activated sludge process plant in which the organic material in a waste is removed by activated sludge, after a short contact period in a contact chamber, for aeration in a separate chamber. The modification possesses the advantage of confining aeration to the mixture of activated sludge and the organics in it as compared with the conventional type of activated sludge plant in which the whole of the sewage is aerated at a correspondingly higher cost, both capital and operational.

The unit consists of two cylindrical sheet steel tanks arranged concentrically on a concrete base and welded to steel channels embedded in the concrete which forms the bottom of the tank. Steel partitions welded between the two tanks divide the outer tank into three compartments: a contact zone or mixing aeration compartment, an aerobic digesting compartment, and a reaeration compartment. A structural steel bridge (16)* supports the inlet stilling well (4), sludge scraper (5) and a surface skimmer (9). A circular air inlet header (10) supported from the settling basin serves to lead air to diffusers (11) at the bottom of the outer tank as well as acting as a handrail for the walkway. Roots type positive displacement blowers Type RBST 6 are housed in a separate building which also contains the offices and stores of the operator and is situated about 30m away from the plant.

Raw sewage enters the contact zone through the inlet pipe (1). Here it is mixed with re-aerated sludge from the re-aeration zone. The organic material in the waste is rapidly removed by flocculation and adsorption onto the activated sludge. Fresh sewage entering the plant displaces an equal amount of mixed liquor from the contact zone into the inlet stilling well (4) of the settling basin via pipe (3). Here solids heavy enough to fall to the bottom do so to form sludge which is scraped to the central sludge well by a mechanically driven scraper (5).

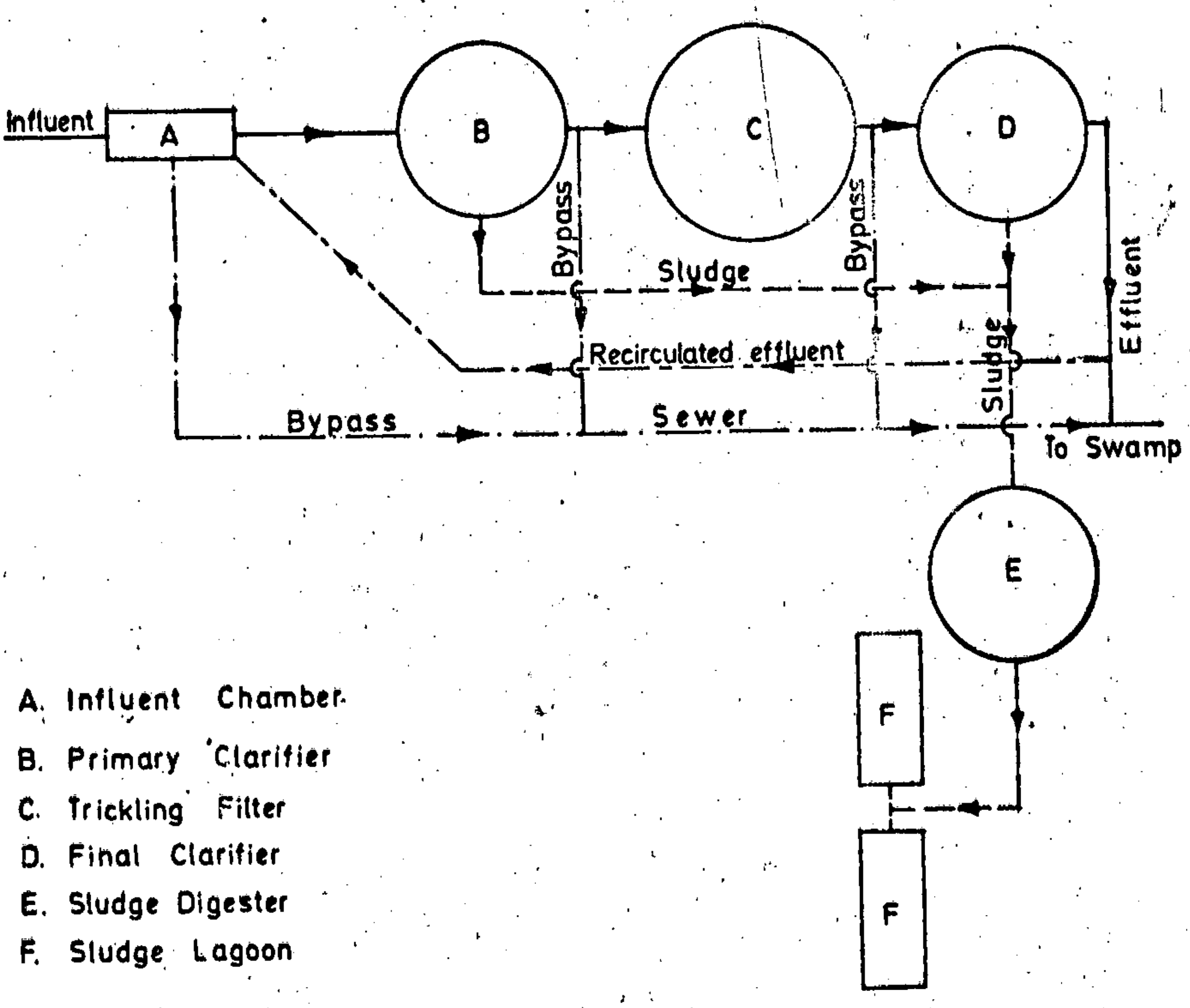
*Figures in bracket refer to Fig. 2.2.

Supernatant liquid from the settling tank flows over the notched weir (6) to the effluent trough (7) from which it leaves the plant through the effluent pipe (8) and is discharged into the swamp not far away. Grease and other floating matter in the settling basin are diverted by the mechanically driven surface skimmer (9) into a scum trough from which it is automatically led into the aerobic digesting compartment.

Sludge is drawn from the sludge well in the settling basin by air lift (12) to enter the reaeration zone. Here it flows round slowly to enable it to be thoroughly aerated so that on re-entering the contact zone it is already fully conditioned to treat sewage.

Sludge surplus to requirement is transferred by air lift (13) to the aerobic digester where it is digested. Surplus digested sludge is removed from the plant via the waste sludge sluice valve for disposal on two sludge drying beds located some 10 metres away from the plant.

Air Products Ltd., who market the equipment, claim in their brochure that provided that the organic and/or hydraulic loading does not exceed the design figures, and the plant is maintained regularly in accordance with their instructions, it will produce at all times a final effluent that will conform to the Royal Commission Standard of 20mg/l BOD and 30mg/l suspended solids. The design loading was exceeded a few years ago and a second Oxigest plant, this time a rectangular model 20B27 of 22,500 gallons (102.4m^3) per day capacity, was built in 1968 to cater for new housing units for the staff. The new plant is some 400m away from the older plant. Neither plant has been producing effluent of this quality. This is probably due more to inadequate maintenance than defects in the design.



A. Influent Chamber
B. Primary Clarifier
C. Trickling Filter
D. Final Clarifier
E. Sludge Digester
F. Sludge Lagoon

FIG. 2·3 FLOW DIAGRAM OF IKEJA MILITARY CANTONMENT TREATMENT PLANT.

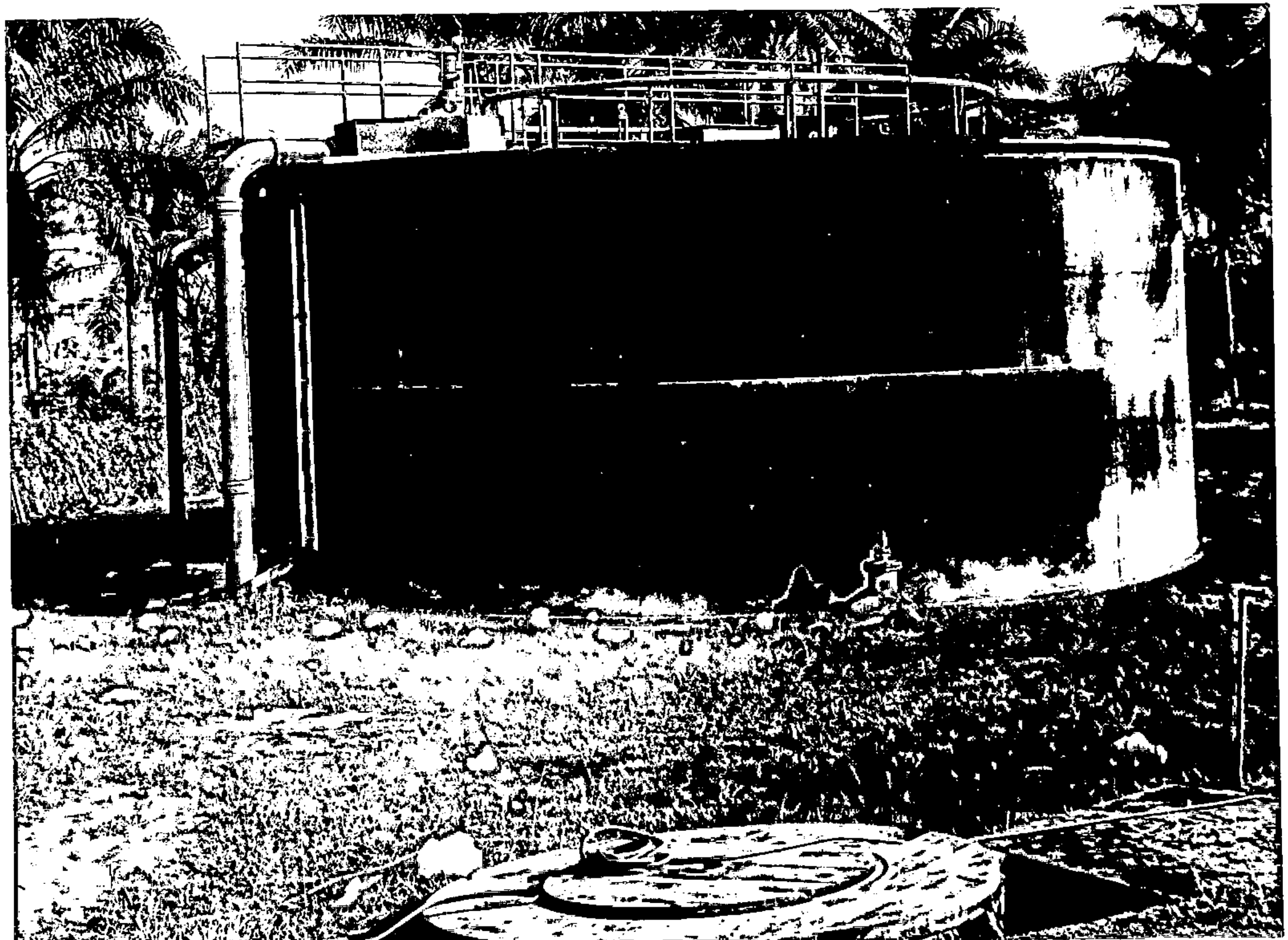
Records show that the installation cost of the bigger and older plant was £47,500 in 1966. This works out at £47.5 per capita of the design population of 1,000. This per capita cost appears high. Relatively long sewers and a lift station are probably contributory factors to this high cost. Cost details of the smaller plant are not available.

2.3 THE IKEJA MILITARY CANTONMENT SEWAGE TREATMENT PLANT

The second example of the provision of sewage facilities in an institution in the Metropolitan Lagos Area is the treatment plant serving the Military Cantonment of the Nigerian Army at Ikeja. The Cantonment covers some 480 acres (1.92 km^2) and consists of the usual accommodation for officers and other ranks, offices, a hospital and other facilities. The raw sewage from these buildings is carried in an 18 inch (.46m) dia concrete gravity sewer, designed to flow half full at maximum flow at ultimate development, to the treatment plant some two and a half kilometers away at the edge of the swamp on the other side of the heavily trafficked Trunk A road from Lagos to Shagamu. It is a conventional type plant in which the different treatment processes take place in separate units as compared with the Oxigest plant in which these processes take place in the same unit though in different compartments.

2.3.1 Design Data

Fig. 2.3 is a schematic layout of the plant. The plant was designed to treat the sewage fron an initial population of 6,000 at a per capita contribution of 37.5 gallons (171 litres) per day or a total of 225,000 gallons (1,023m^3) per day. The ultimate design population was 18,000 persons which at the same per capita contribution will give an ultimate flow of 675,000 gallons (3070m^3) per day. The first phase development of the treatment plant which was commissioned in 1965 covers only about one-sixth of the total area of 4½ acres (18,200m^2) available for ultimate development.



MODEL 31-R-80 OXIGEST TREATMENT PLANT, LAGOS UNIVERSITY
(Capacity: 227.5m³/day, Commissioned 1966).

2.3.2 Sewage Flow

Sewage first passes through a comminutor in the influent chamber A which has a capacity of half the ultimate flow. Provision has been made for a second comminutor to be installed in future. A by-pass through a hand cleaned screen is provided for use when the comminutor is out of use either due to breakdown or maintenance operation. The sewage from the comminutor passes into the Parshal Flume in the influent chamber which measures the flow. The sewage then goes into a division box from which it normally goes into the primary clarifier B. From the division box the flow can be made to by-pass the clarifier through an 18" (.46m) dia pipe to the outfall sewer.

Before leaving the influent chamber the sewage is mixed with recirculated effluent from the final clarifier. The sewage now enters the primary clarifier in which according to the designers' claim, approximately 35% of the BOD and 65% of the suspended solids are settled. The effluent passes over the weir from where it can be made to by-pass the rest of the plant into the outfall sewer.

From the primary clarifier the sewage flows into the trickling filter C where an additional 50% of the BOD is claimed to be removed by biological action. The sewage is distributed by gravity through the rotating distributor arms and passes through the filter bed. It is then collected in an under-drain system of precast concrete blocks. From the under-drain it can be made to by-pass, but normally goes into the final clarifier D.

Recirculated effluent is drawn from the final clarifier D and pumped back into the end of the influent chamber where it is mixed with the raw sewage, which improves the efficiency of the trickling filter. The design specified a recirculation ratio of 1:1 as near as possible. There is a flow gauge in the recirculation section of the influent chamber.

2.3.3 Sludge System

Sludge is normally drawn from the final clarifier and pumped into the sludge digester E. Sludge is drawn every morning and afternoon and the operation is stopped only when the sludge has thinned to the consistency of sewage. Digester overflow flows automatically into the lagoon and is taken from a level 2.5 feet (.76m) below the top, which allows a layer of scum to form at the top. The two lagoons are filled alternately one being used for 3 months before changing to the other.

A special feature of this plant is the by-pass arrangement whereby the sewage can by-pass certain units or indeed the entire plant and go straight into the outfall sewer and thence into the drainage swamp. In particular primary sludge can be made to by-pass the digester and pumped straight into the sludge lagoon F. By-passing would be occasioned by damage or breakdown in certain units, or by power failure which unfortunately occurs much too often in this and other areas of Lagos. It would also be necessary during periods when units were under routine maintenance operations.

The design provided for the addition in future of a primary clarifier, a trickling filter, a final clarifier and another sludge digester of twice the size of the present one which would operate in parallel with it.

Information received from the consultants indicated that the construction cost in 1965 was £28,400 and the equipment cost £14,600. This works out at only £7.15 per capita of the first phase population of 6,000. This, however, does not include the cost of the construction of sewers at the Cantonment itself as well as the cost of the 18 inch (.46m) dia sewer from the Cantonment to the treatment plant. Sewerage items have been shown in Chapter I to be relatively expensive.

2.3.4 General Dimensions of Plant Units

Influent Chamber:

Overall length = 33' - 4½" (10.2m)

Width at Comminutor = 12' - 4" (3.8m) external

Width at Flume = 4' - 4" (1.3m) external

Clarifiers:

Tank Diameter = 25' - 0" (7.6m) internal

Depth = 7' - 8" (2.3m) at circumference

Trickling Filter:

Diameter = 50' - 0" (15.2m) internal

Depth of Bed = 5' - 8" (1.7m)

Digester:

Diameter = 28' - 0" (8.6m) internal

Depth at circumference = 15' - 6" (4.7m)

Depth at centre = 23' - 0" (7.0m)

Sludge Drying Lagoons (Measurements at bottom):

Length = 50' - 0" (15.2m)

Width = 15' - 0" (4.6m)

2.4 THE IKEJA INDUSTRIAL ESTATE TRADE EFFLUENT TREATMENT PLANT

This plant was constructed at the Ikeja Industrial Estate by Reliance Engineering and Construction Company for the Western Nigeria Housing Corporation and commissioned in April 1965. It has a design capacity of 640,000 gallons (2912m^3) per day and is meant to serve all factories in the 188 acre (.75 km^2) Industrial Estate.

A feature of the plant is its treatment of sewage of different characteristics from different factories. On entering the plant these different components are first mixed in a mixing tank. The mixture goes through a disintegrator before flowing into an aerator clarifier. This consists of two concentric tanks in which sedimentation takes place in the outer tank and aeration in the inner tank. The aerated sewage then flows into two high-rate trickling filters from which it goes into a secondary clarifier before the effluent is finally discharged after chlorination into the Shasha River 1.2km away. There are two sludge digesters for treating sludge from both the aerator-clarifier and the secondary clarifier. There is also provision for sludge filtration in the design.

For some reason only 4 of the many factories in the Estate were effectively connected to the plant in 1971 even though it was said that arrangements were on hand then to connect more. Three of these connected are however the largest factories in the Estate. They are Nigerian Textile Mill, Dunlop Factory and Guiness Brewery.

The capital cost was £500,000 including the outfall sewer to the Shasha. This was a package deal project and it was known to the author in his former capacity as Controller of Works Services in the Western Nigeria Public Service that the authorities did not escape the problems usually attendant upon this type of contract.

The plant, has had a continuous history of unsatisfactory performance, most probably due to defects in design as well as poor maintenance. Following aeration with high rate filtration as done in this design is considered unusual. It is known that the authorities are now considering the construction of an entirely new plant.

2.5 THE SURULERE HOUSING ESTATE TREATMENT PLANT

This plant treats the raw sewage from a group of 45 buildings covering an area of 29 acres (.12km²) in one of the housing estates of the Lagos State Development and Property Corporation in Surulere. The buildings vary from 4-storey blocks of flats to the higher class 2-storey detached houses commonly found in these estates. It is another 'Oxigest' plant designed to treat 200,000 gallons (910m³) of sewage a day at an average flow of 8,330 gallons (38m³) or peak flow of 25,000 gallons (114m³) an hour.

The capital cost (1966) was £61,000 which works out at £12.2 per capita of the design population of 5,000. Again this appears low and must be accounted for by the absence of expensive construction of sewers in the scheme.

The Surulere Treatment Plant has been included in this survey to prove the point that where estate development and, as sometimes happens, building ownership are under a single control, it becomes relatively easy for the estate authorities to instal sewage treatment facilities in the estate while the rest of the town suffers from the absence of the central sewerage system.

2.6 OTHER SEWAGE TREATMENT PLANTS IN THE METROPOLITAN LAGOS AREA

There are a number of other treatment plants in the area under consideration. About 1965 the Federal Ministry of Works and Housing installed in a number of Lagos Primary Schools 'Oxigest' Models A and B (rectangular type) plants in place of the septic tanks which had had a long history of unsatisfactory service particularly during the wet season. The schools vary in population from 740 to 5,920 pupils. Assuming a 10-hour attendance at school these figures become 310 and 2,480 respectively for a full 24 hours use of sewage disposal facilities.

Between 1964& 1966, the Chicago Pump type plant was installed for the Federal Ministry of Health at the Health Centres at Randle Road (Surulere), John's Street, (Lagos), Randle Road (Apapa), Cemetry Street (Ebute-Metta) and King George V Road (Onikan). The design capacity of the John's Street plant is 2,500 gallons (11.88m^3) per day while the capacity of the others is 4,150 gallons (18.88m^3) per day each.

In 1967 a plant similar to the plant for the Ikeja Military Cantonment already described in 2.3 above was built for the Nigeria Police at the Southern Police College, Ikeja. It has a design capacity of 45,000 gallons (205m^3) per day to serve a population of 1,500. The budget cost in 1966 was £28,350 which works out at £18.9 per capita.

Finally building work is nearing completion on the construction of a new treatment plant at the Lagos University Teaching Hospital, Idi-Araba. This is a CLOW MODEL CS-660-F7-100 plant manufactured by the Clow Corporation of Florence, Kentucky, U.S.A. Like the 'Oxigest', it is a contact stabilisation modification of the activated sludge process.

It was designed for an average daily flow of 660,000 gallons ($3003m^3$) per day with a total 5-day BOD loading of 1650 lb. (750kg) per day. The peak hydraulic flow capacity is 3 times the design average daily flow.

Again as in the 'Oxigest', primary and secondary treatment take place in two concentric circular steel tanks. The inner tank is the clarifier where the raw sewage, mixed with activated sludge, is settled. The outer tank is divided into three compartments: a mixing chamber, a sludge activating chamber and an aerobic digester. The steel tanks are encased in concrete and are open at the top. There is provision for chlorination of the final effluent in a separate building.

The concentric tanks are 40 ft. 6 ins. (12.3m) and 86 ft. 3 ins. (26.2m) internal dia, and the depth at the wall of the outer tank is 16 ft. (4.9m). The structure is on piled foundations and was originally estimated to cost ₦473,000 together with the office building, chlorine room and laboratory. It is however apparent that it will cost much more than this in view of the present upward trend in building costs.

At an assumed per capita 5-day BOD contribution of 63gm, the population equivalent of the organic loading of 750kg. per day for which the plant was designed is 11,000. The original estimate of ₦473,000 for the capita cost works out at ₦43 per capita. This is £29.3 at the present rate of exchange (1975). This is only two-thirds the cost in 1966 of the Lagos University plant which was £47.5 per capita. The lower per capita cost in the Teaching Hospital Plant is due to the fact that the bigger the plant the lower the per capita cost becomes. The Teaching Hospital Plant is ten times the capacity of the University plant in terms of population equivalents.

In this account of the small waterborne sanitation schemes in Lagos sewerage costs and treatment plant costs have not been treated separately because the sewerage component in nearly all the cases was little thus satisfying the first of the criteria set in Chapter I for embarking on such schemes for sections of a town. This probably accounted for the fact that information on costs usually did not separate the two components.

Nearly all the examples discussed in this Chapter are modifications of the activated sludge process to which air must be supplied at a very high rate. This is because large quantities of oxygen are required by the aerobic bacteria for the decomposition of the organic material in a waste. Unfortunately apart from the oxygen produced by algae in their own synthesis, the atmosphere is the source of all oxygen used in the process of aerobic oxidation of organic compounds. Oxygen however is only about one-fifth by volume of air and its density is only 1.43gm per litre at $0^{o}C$ and standard pressure. Moreover it is only slightly soluble in water, the solubility in fresh water ranging from 14.6 mg/ℓ at $0^{o}C$ to about 7.0 mg/ℓ at $35^{o}C$ under 1 atmosphere of pressure (SAWYER, C.N. and McCARTY, P.L. 1967).

It is apparent from the foregoing that only little quantities of oxygen are available to the bacteria in a waste from large volumes of air. The cost of the mechanical installations for forcing large volumes of air through the aeration tanks of activated sludge plants makes the system unsuitable for sewage works where flows are under 0.5mgd (2,275m^3). (BABBIT, H.E. and BAUMANN, E.R. 1965). All the plants under consideration are below this capacity and must therefore be considered unsuitable in the circumstances.

CHAPTER III

PRIVY METHODS OF SEWAGE DISPOSAL IN LAGOS

In the last Chapter a survey was made of the few water-borne sanitation facilities existing in institutions in Lagos where the right conditions exist for such installations. The combined capacity of these installations amounts to only 5175m^3/day catering for 24250 people. This represents only 1.2% of the total population of Lagos estimated at 2.1 million in 1975. This means that the vast majority of the people in Lagos depend on either the nightsoil conservancy system or the septic tank for sewage disposal. These are the two most extensively used methods in Lagos. In this chapter a short description and the operation of each of these systems is made.

3.1 NIGHTSOIL CONSERVANCY SYSTEM:

In nearly all the older parts of Lagos as well as in most towns in Nigeria and the other countries of the developing world, sewage disposal is by the nightsoil conservancy system. In this system the householder provides a bucket usually of galvanised iron about 0.38m dia at the top and 0.30m deep. This is placed in a little closet usually 1.83m by 0.91m internal dimensions. There is a timber seat with a covered hole over the bucket and the closet is provided with a small door in the external wall for access for the nightsoilman.

The collection system may vary from place to place according to local circumstances. Generally a nightsoilman is allocated a number of houses where he calls one after the other at regular intervals, emptying the contents of each pail into his own jar. When this is nearly full he either takes it to a final disposal point if this is not distant, or to an intermediate depot where he empties his jar into one of several drums carried on a waiting lorry, or into a specially built tanker that is later on driven by a tractor to the disposal point. The buckets are required to be thoroughly washed and disinfected after each emptying.

In general, the nightsoilman and the men working at the disposal site are required to observe the maximum possible rules of hygiene in the rather dangerous circumstances of their work.

3.1.2 DISPOSAL METHODS:

Methods usually employed for nightsoil disposal include: tipping in the sea or in the Lagoon or in a big river; burial in earth trenches; incineration along with municipal refuse; detention in specially designed tanks; application on land/as manure; disposal directly into sewers where there is sewerage in part of an urban area; anaerobic digestion in closed tanks; and heating for sterilisation (WAGNER, E.G. and LANOIX, J.N. 1958).

SHAW, V.A. (1962) is of the opinion that while these methods if properly carried out may be effective and hygienic, they are often most unhygienic and they constitute a nuisance due to fly-breeding, smells and unsightliness and, in some cases, may lead to water pollution arising from surface run-off from the disposal sites.

Trenching is a common method in Nigeria because it is relatively simple. It however, requires much labour, and hardly satisfies the generally accepted criteria of a good sewage disposal method. Land must be reasonably cheap and located at a reasonable distance from the collection point. The trench is usually dug about .61m deep. The bottom .30m to .46m is filled with nightsoil which is then covered with .30m to .46m of earth. The earth fill stands some .15m proud of ground level to allow for the inevitable settlement both in the digesting night-soil and in the earth cover. The bottom of the trench should be above the water table and in areas of heavy rainfall deeper trenches should be provided. The same plot of land is usually used again and again in a cycle of a few months to a couple of years.

3.1.3 NIGHTSOIL CONSERVANCY IN LAGOS:

The nightsoil conservancy method of sewage disposal was introduced on Lagos Island in the latter third of the nineteenth century (SIMPSON, R.W. 1966). Organised collection of pails later spread into the newer settlements in Ebute-Metta, Mushin and parts of Yaba and Surulere. In the late fifties nightsoil collection was done in a sub-department under the City Engineer. The Municipality was divided into three nightsoil collection zones, two on Lagos Island and the third on the Mainland. Clearance was done by contract labour who emtpied the contents of their pails into a number of 500-gallon tankers placed at intermediate collecting depots. From here the tankers, first introduced in 1958, were driven by tractor to the Council's Nightsoil jetty at Ebute Ero on the north side of Lagos Island just east of Carter Bridge (Fig. 7.1). Here the nightsoil was tipped into the Lagoon. It was believed that the current at this point was sufficiently strong to ensure rapid mixing of the nightsoil with sufficient volumes of water to reduce the concentration of pollution of the harbour waters to acceptable levels. In a 1963 re-organisation the zone III collection area was expanded and divided into two separate areas. In 1965, 172,693 pails were cleared from 135,552 premises in all the four nightsoil clearing zones. These pails were emptied at 57 intermediate depots into 72 tankers pulled by 37 tractors to the Jetty (SIMPSON, R.W. 1966).

Table 3.1 shows statistics of the operation of nightsoil clearance for the five years from 1960-61 to 1964-65. These figures show that while there was an overall increase of only 2% in the number of premises where pails were cleared between 1960 and 1965, there was in fact an overall increase of 34% in the number of pails cleared, and an increase of 47% in the volume (m^3) of nightsoil discharged at Ebute-Ero. This can only indicate further crowding of the population into already overcrowded premises. Indeed while there was a decrease of 31% in the number of premises

TABLE 3.1

NIGHTSOIL AND SEPTIC TANK SLUDGE CLEARED IN LAGOS 1960–1965

Zone	1960 – 1961		1961 – 1962		1962 – 1963		1963 – 1964		1964 – 1965	
	No. of Premises with Pails	No. of Pails Cleared	No. of Premises with Pails	No. of Pails Cleared	No. of Premises with Pails	No. of Pails Cleared	No. of Premises with Pails	No. of Pails Cleared	No. of Premises with Pails	No. of Pails Cleared
I	56,687	44,052	45,560	44,362	37,633	44,910	37,540	42 825	37,970	43,177
II	55,148	52,643	46,319	52,079	42,135	50,978	41,891	50,723	41,699	50,652
III	20,930	31,358	25,071	35,004	*23,872	33,961	31,385	51,156	32,787	49,108
IV							3,828	4,919	23,098	29,756
Total	132,765	128,053	116,950	131,445	103,640	129,849	114,644	149,623	135,554	172,693

	1960 – 1961		1961 – 1962		1962 – 1963		1963 – 1964		1964 – 1965	
VOLUME OF NIGHTSOIL DISCHARGED AT JETTY	m^3 48,992		m^3 49,324		m^3 53,974		m^3 66,430		m^3 72,332	
SEPTIC TANKS EMPTIED	No. 732	m^3 4,684	No. 816	m^3 5,221	No. 824	m^3 5,272	No. 843	m^3 5,393	No. 950	m^3 6,079

* (Quantities were estimated from No. of tankers discharged)

serviced between 1960 and 1964 there was in fact an increase of 36% in the volume of nightsoil discharged at the Jetty, giving an increase of 57% in the annual volume of nightsoil cleared from each house in that same period, which further confirms the overcrowding.

The figure of 72,322m^3 discharged at Ebute-Ero in 1964.65 (Table 3.1) works out at a daily discharge of 198m^3 in that year. WARD (1970) reported that a physical check on 14th March 1970 revealed that 90m^3 were discharged that day, which therefore was only 45% of the average daily discharge in 1964-65. A repeat of the exercise on 16th March 1970 shows a discharge of 95m^3 which was close enough to the figure obtained two days earlier.

SIMPSON, R.W. (1966) reckons that on the basis of the figures in Table 3.1 the average per capita contribution of nightsoil per day in 1964-65 was approximately 0.09 gal. (0.41 litres) and that this was lower than the average figures for other municipalities using nightsoil disposal methods, which varied from 0.22 gal. (1.00 litre) to 0.30 gal. (1.37 litres).

Apart from failing to satisfy most of the requirements of a good sewage disposal method the nightsoil conservancy system in Lagos has its own peculiar problems. One of these is the stigma attached to the occupation of a nightsoilman which makes it increasingly difficult to recruit labour for the service. The author was the City Engineer in 1958 when there was a strike of nightsoilmen in Lagos during which there was a rapid build-up of nightsoil in premises. Prison labour was used to run a skeleton service but the danger of an epidemic was very near indeed.

The other problem of the nightsoil conservancy system in Lagos is the objection and hostility of the Lagos populace to its operation. People object to the siting of the intermediate collecting depots within their areas. After numerous complaints to the Lagos City Council the authorities of a church directly opposite the nightsoil jetty at Ebute-Ero got a High Court injunction against the Council ordering it to cease using the Ebute-Ero site for its nightsoil operation by 31st May, 1971. Whichever new site the Council selected for its operation could not satisfy the Nigerian Ports Authority and the Health authorities if the Council continued the original method of just dumping the nightsoil into the harbour channel without any form of treatment whatsoever.

In 1970, a firm of consultants submitted proposals to the Lagos City Council for partially treating the nightsoil before discharging the effluent into the Lagoon water at a different site. Treatment would be effected by aerobic digestion in aerated lagoons built on a reclaimed island in the Lagos Lagoon at Ebute Elefun on the north of Lagos Island some 1.3 km south-east of the Ebute-Ero Jetty (Fig. 7.1). On arrival at the plant the nightsoil would be screened to remove trash, water being pumped in from the Lagoon to aid this process and to dilute the nightsoil to acceptable concentration levels for treatment. The diluted nightsoil after passing through screens would be macerated and pumped into treatment lagoons the contents of which would be aerated and agitated by floating cone aerators. The lagoons would provide a retention period of 10 days for the diluted nightsoil of suspended solids concentration of the order of 15,000mg/l. There is unfortunately no indication in the preliminary report from the consultants of the final quality of the effluent.

The efficacy of treating nightsoil by aerobic method is open to doubt. Nightsoil is a highly concentrated waste which has to be diluted with water to bring the concentration down to a level in which it will be amenable to treatment in an aerobic device. This is not desirable as it increases the volume of waste handled and, correspondingly, plant capacity and cost. For this reason it is more usual to treat nightsoil anaerobically. This subject is discussed further in Chapter IX.

The capital cost at 1970 prices for civil engineering works and machinery was estimated at £371,200. This worked out at only £2.05 per capita of the estimated 180,000 population to be covered by the service. This was revised to £670,000 or £3.7 per capita in 1972, and the final cost would most certainly be higher still.

The plant was constructed but broke down shortly after commissioning. Failure is understood to have occurred in the structure of the aerated lagoons and not in the biological treatment process. Eventually the Council evolved a better method for keeping the walls of the lagoons watertight, and the plant started operating again. Unfortunately the hydraulic sand reclamation connected with the construction of the new highway round Lagos Island has again stopped the operation of the plant. Each time the plant stopped operating dumping of nightsoil has had to continue at the Ebute-Ero Jetty, much to the annoyance of the church authorities, and the disappointment of all interested in the sanitation of Lagos.

The installation of a nightsoil treatment plant is only a partial solution to the enormous sewerage problem of Lagos. Successful treatment of nightsoil at Ebute Elefun removes only one third of the problem. It only partially solves the problem of disposal as the effluent is still discharged into the enclosed waters of the Harbour. The new installation does not at all touch the very unsatisfactory collection and transportation method. This last aspect has been shown in Chapter I to be the most serious part of this complex problem. Indeed collection and transportation both constitute a much greater problem than treatment of nightsoil in Lagos.

The authorities have nevertheless accepted the installation of the nightsoil treatment plant as a step forward pending the coming of a complete municipal sewerage system. Sewage treatment by the forced absorption of oxygen which is the system used in the new plant was shown in Chapter I to be expensive. If the biodisc process described later in Chapter VIII can treat nightsoil successfully then its many advantages listed in that chapter would recommend it for consideration in nightsoil treatment in Lagos and elsewhere. Results of experiments on the treatment of nightsoil with the laboratory scale model of the biodisc are reported in Chapter IX.

3.2 THE SEPTIC TANK SYSTEM

3.2.1 DESCRIPTION AND OPERATION OF TANK

The Septic tank is a covered settling tank in which both primary and secondary treatment take place concurrently (Fig. 3.1). The effluent from the tank undergoes further treatment in a soakaway before leaching through the soil under the soakaway. The final effluent is a source of contamination of both subsoil and ground water and therefore constitutes a positive danger where water supply is from wells or springs near a dwelling served by a septic tank. Water pollution caused by septic tank effluent moves in the direction of the ground water which follows the gradient of the water table. As the latter generally follows the general contours of the ground surface septic tanks are usually located downhill of a well or spring. Even then it is recommended that septic tanks should never be closer than 15m to any source of water supply (HOLLIS, M.D. 1963).

The primary treatment consists in the settling out of the solids in the sewage from the household water closet and sometimes from the kitchen, while the secondary treatment consists in the breaking down of these putrescible solids in the process of anaerobic digestion. The three main functions of the tank are to provide a place:

(a) for sewage solids to settle out of the liquid;

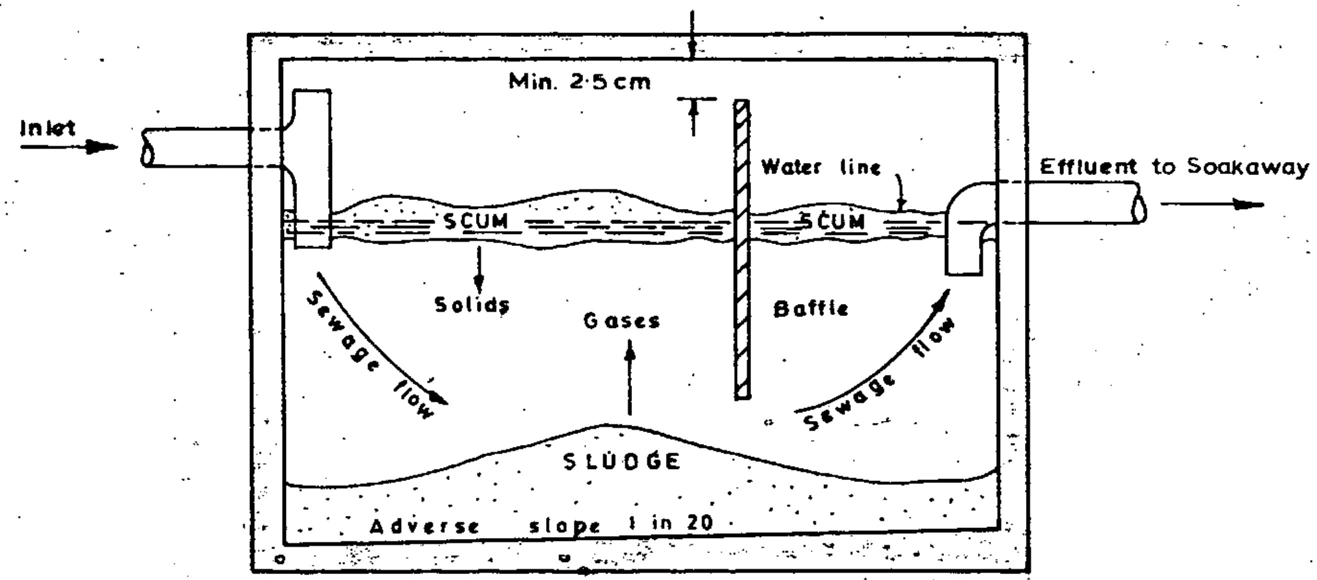


FIG. 3·1 THE DOMESTIC SEPTIC TANK.

(b) for bacterial action to digest a major portion of the settled solids;

(c) for storage of residual solids (WRIGHT, F.B. 1966).

The tank neither purifies the sewage, eliminates odours, nor destroys all solid matter. It however conditions the sewage so that it can be further treated and disposed of in a sub-surface soil leaching system which is an integral part of the system. The pre-treatment in the tank prevents premature clogging of the leaching system (SALVATO JR. J.A. 1972).

At each pull of the chain of the water closet some 9 litres quanity of raw sewage enters the influent end of the tank which pushes out a similar quantity of partially treated sewage from the effluent end into the sewer leading into the leaching system (Fig. 3.3) As the sewage flows through the tank the heavier solids settle to the bottom. The lighter solids including fats and grease rise to the surface and form a layer of scum (HOLLIS, M.D. 1963). In the process of anaerobic digestion a considerable portion of both the sludge at the bottom and the scum at the top are liquefied. Gases (CO_2, CH_4, NH_3 etc.) are liberated from the digesting sludge. These rise to join the lighter solids which accummulate with the scum where they again undergo further digestion, with scum fragments heavy enough again settling to join the sludge at the bottom. The sifnificant result of anaerobic digestion in the tank is a considerable reduction in the volume of the sludge which explains why a tank can operate for 1 to 4 years without the need for desludging (WAGNER, E.G., and LANOIX, J.N. 1958).

The important features of the tank which aid its functions as a settling tank are the baffle and the adverse slope at the bottom along the length of the tank. The baffle in Nigerian building practice is a 5cm thick precast concrete wall built across the width of the tank and penetrating 60cm into the liquid depth. The baffle prevents shortcircuiting in the tank as all flow from the influent end passes under it before rising to the effluent pipe invert level. The 1 in 20 adverse slope on the floor of the tank retards flow and aids the deposition of solids.

As both the scum layer and the sludge grow in size they encroach more and more on the space available for the flow of sewage through the tank and in particular reduce the capacity of the tank as a settling basin. A situation could be reached when the flow-through section has become so narrow that adequate sedimentation of the suspended matter is no longer possible. The presence of large amounts of floating matter in the effluent is indication that this condition has been reached and that desludging is due (WAGNER, E.G. and LANOIX, J.N. 1958).

The production of gases in the anaerobic digestion of the sludge blanket results in these gases bubbling through the sewage across the main stream flow. This fouls the quiescent conditions essential for the efficient functioning of the tank as a settling basin.

The Imhoff Tank is a development of the septic tank which eliminates this disadvantage. It is a two-storey structure in which settling of solids through the flowing sewage takes place undisturbed in the upper storey while the anaerobic digestion of the sludge and the production of gases take place in the lower storey.

The capacity of the septic tank is decided from a consideration of the number of users, the per capita production of sewage, the detention period and the length of time the tank is desired to operate between one desluding operation and the next.

The average per capita flow of sewage is related to the per capita water consumption. WAGNER and LANOIX (1958) reckon that per capita consumption is likely to be lower than 100 litres per day in most rural areas of the world. It is however in the urban areas in Nigeria that the septic tank is most extensively used and it is only a little used in the rural areas. In the former Western State of Nigeria the Water Corporation used a design figure of 45 litres to 90 litres (10-20 gal.) per capita per day for all new water supplies. The pumping and treatment facilities in the Lagos Water Supply were recently expanded to 205,000m^3 (45MG) per day even though the actual daily consumption in January 1974 was only 135,000m^3 (30MG) which works out at 75 litres (16 gal.) per head of an estimated population of 1.8m (1973).

In Nigerian building practice, kitchen wastes and bathwater are generally excluded from the septic tank. What is admitted into the tank therefore is limited to what results from the flushing of the w.c. some 3 to

6 times a day by each occupant of a house. For a 9 litre cistern the per capita sewage flow is therefore 27 to 54 litres per day.

Again WAGNER and LANOIX (1958) state that the detention period is from 1 to 3 days, usually 24 hours. The detention period in the primary settling tank in a conventional waste treatment plant is usually taken to be 2 hours for 3DWF or 6 hours for 1DWF (dry weather flow). The equivalent figures for the secondary settling tank are 1½ and 4½ hours respectively. The septic tank is designed to be water-tight and therefore carries only 1DWF. Allowance should however be made for possible infiltration of some ground water and for the impairing of the settling efficiency of the tank due to the gasification effect of anaerobic digestion earlier mentioned. Adopting a detention period of 24 hours which is 4 times the detention period in a 1DWF carrying primary settling tank is considered adequate allowance for these two contigencies. The same authors however recommend a design figure of 190 litre tank volume per person which is 3½ - 7 times the figures just argued above. This additional capacity is required for the storage for some 2 to 4 years of the constantly growing volume of solids residue consequent upon digestion. It is usual to measure tank capacity in net liquid volume which does not include the space for gases above the liquid line.

Septic tanks are usually rectangular in section but tanks with circular sections are known and are reported to function as efficiently as rectangular ones.

The tank depth must be adequate to leave enough flow-through space for the main sewage after allowance for the accumulation of sludge at the bottom and the penetration of some thickness of scum from the top. WAGNER and LANOIX recommend that the depth should be between 1.27m and 1.70m. The length to breadth ratio must recognise the fact that a settling tank should be long and narrow. Again WAGNER and LANOIX recommend tl the length should be about 2 to 3 times the breadth which is the usual proportion for rectangular settling tanks (WHITE, J.B. 1970). Clearance above the liquid line is recommended to be 30cm. (WAGNER and LANOIX) but the volume of space so provided should never be below 0.20 litre (HOLLIS, M.D. 1963).

While septic tanks are usually built in single compartments big tanks of two or three or even four compartments are not unknown. These compartments, separated by water-tight partitions, are linked together through elbow pipes and function as a single tank in series. In a two compartment tank the first compartment is usually a half to two-thirds of the total volume and this type of tank is reported to provide an extra degree of suspended solids removal (HOLLIS, M.D. 1963). It is not usual to have tanks of more than four compartments. There should be a 2.5cm space between the tank cover and the top of each partition to allow free passage of gases between compartments.

3.2.2 LEACHING SYSTEM

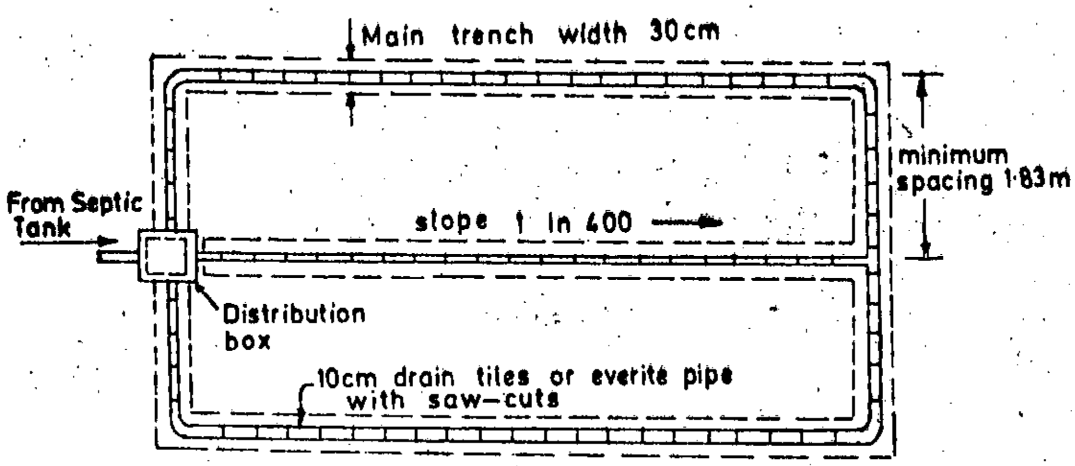
The effluent sewer leads, usually by gravity, the partially treated sewage from the tank to the absorption area where it is discharged into the soil at a shallow depth. In its passage through the soil it is attacked by aerobic bacteria in the process of biological oxidation of organic wastes. These bacteria abound in the top 1m or so of soil, and effluent discharged deeper than this does not receive the benefit of their activities (ROCKEY, J.W. 1963).

Effluent disposal is the second function of the leaching system which can take place within or below the top 1m of soil. External factors known to affect the passage of the sewage effluent through the soil are the flow gradient and the position of the water table. Authorities (WAGNER and LANOIX, SALVATO, WRIGHT etc.) agree that the bottom of the leaching system should be at least 1m above the water table. In Chapter VII is described a 6-year investigation to establish how far the soil in certain parts of Lagos satisfies this criterion.

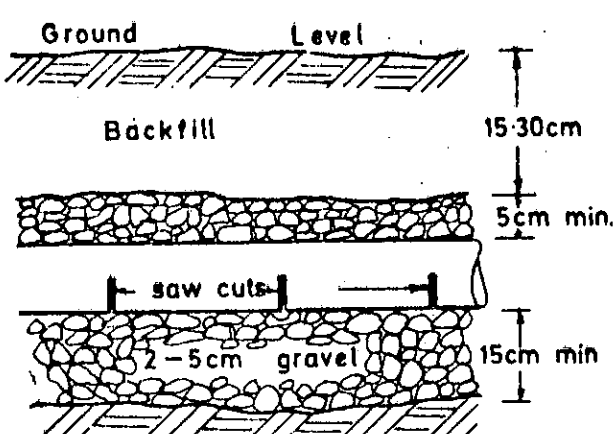
The suitability of a soil as an absorption area for septic tank effluent depends on the rate at which the effluent will percolate through it. This is determined indirectly from percolation tests which determine the rate of percolation of water through the soil. The theory and practice of percolation tests and the results of some tests in Lagos are described in Chapter V & VI.

The leaching system most described in literature on the septic tank consists of 10m piped drains spaced from 1.8m to 2.3m apart, surrounded by gravel, and buried at a depth of .45m to .90m. Design practice is for trench widths to vary from .45m to .60m with the pipe laid in .15m of gravel. To allow uniform distribution of effluent over the entire drain length each

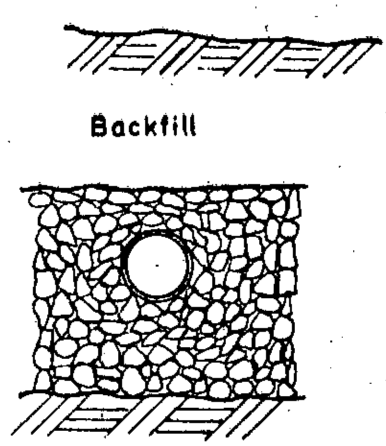
FIG. 3·2 EFFLUENT ABSORPTION TILE FIELD ARRANGEMENT



LAYOUT OF FIELD



LONGITUDINAL SECTION



CROSS SECTION

line is constructed of short pieces of pipe .60m to .90m lengths, open jointed, or of longer lengths with saw cuts spaced at .15m apart, each cut constituting the lower third of the pipe circumference. Fig. 3.3 shows a typical tile field arrangement and some details.

The total Area A of trenches required to dispose the daily sewage flow Q from a building is given by:

$$A = \frac{Q}{q} \text{ sq. meters} \qquad \ldots\ldots\ldots \quad (3\text{-}1)$$

where q = allowable rate of sewage application in litres/m^2. For trenches of width b, the total length L of trenches required in the tile fields is given by:

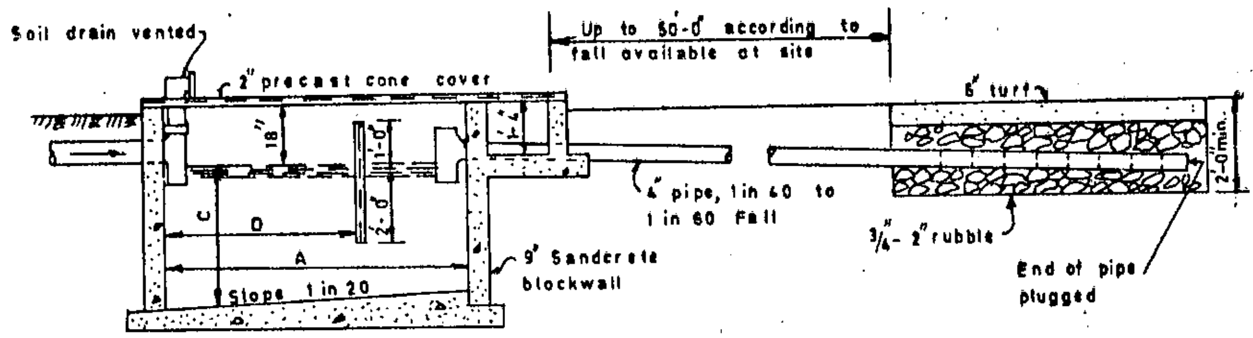
$$L = \frac{Q}{q.b} \text{ meters} \qquad \ldots\ldots\ldots \quad (3\text{-}2)$$

Soakage pits and soakaways are not considered as effective as soakage trenches and are recommended only as an alternative where porous sand or gravel exists below the top 1 to 1.2m of the soil. They are sometimes used in combination with absorption fields. The pit consists of a hole in the ground, walled up with either porous materials or with bricks and blocks placed without mortar in the joints. The effective area of the pit is the vertical area, which is the product of the circumference and depth. The area through impervious strata should be disregarded in the calculation. HOLLIS, M.D. (1963) recommends that the bottom area of the pit should also be ignored in the calculation of the effective area. This would appear valid only in cases where the whole or most of the pit bottom is covered by the concrete slab foundation for the pit walls. It could also be justified in the case of a deep pit of small diameter where the bottom area is small relative to the vertical wall area. It would be difficult to justify however in case of pits of relatively large diameter-depth ratio where a substantial area at the pit bottom is available for effluent egress. The only valid justification for ignoring the bottom area in such circumstances would be the case where the water table is so near the bottom of the pit that there can be no effective leaching of the effluent in the pit into the soil through the pit bottom.

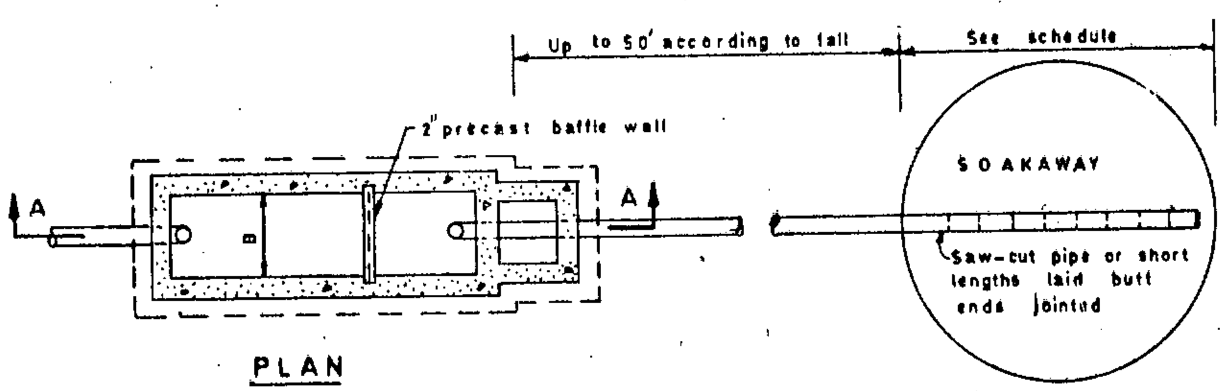
3.2.4 THE SEPTIC TANK IN NIGERIAN BUILDING PRACTICE

Fig. 3.3 and Tables 3.2 and 3.3 are adapted from INFORMATION BOOK SANITARY STRUCTURES (P.W.D. NIGERIA, 1943) and show the details of the septic tank in Nigeria building practice. These details have not changed for over 40 years. Whereever township building bye-laws are enforced building plans submitted for the approval of Local Authorities always include one sheet of P.W.D. Drawing No. 21040 showing the standard septic tank. Table 3.2 shows the dimensions and capacities of five different tank sizes applicable to the various numbers of users shown in columns 8 and 9 of the Table. The tank in each case provides for 24 hours detention of a per capita production of about 114 litres (25 gallons) of all wastes per day.

The reason for keeping the tank depth C constant at 4'.0 (1.2m) in Table 3.2 is obscure. It is probably the minimum depth which would satisfy the requirements for effective settling of suspended solids in the main sewage flow without the need for frequent desludging.



SECTION A-A



PLAN

Note

Details are taken from information Book : Sanitary Structure (Nigeria Public Works Department 1940)

FIG.33 THE DOMESTIC SEPTIC TANK AND SOAKAWAY (PWD STANDARD)

TABLE 3.2 – SCHEDULE OF SEPTIC TANK SIZES AND DIMENSIONS

Tank Size	Dimensions (m)				Capacity (m^3)	No. of Users		Effluent per capita (litres)	
	A	B	C	D		All Wastes	Soil Wastes Only	All Wastes	Soil Wastes Only
1	2	3	4	5	6	7	8	9	10
I	2.032	0.457	1.220	1.295	1.134	10	10	113.4	113.4
II	2.286	0.534	1.220	1.448	1.488	13	15	114.5	99.2
III	2.286	0.610	1.220	1.448	1.700	15	20	113.3	85.0
IV	2.540	0.686	1.220	1.677	2.125	18	30	118.1	70.3
V	3.048	0.762	1.220	1.982	2.834	25	40	113.4	70.9

NOTE: This table is adapted from schedule in Information Book: Sanitary Structures (Nigeria Public Works Department, 1940)

TABLE 3.3: SCHEDULE OF SOAKAGE PIT/TRENCH SIZES (NIGERIA P. W. D. STANDARD)

Tank Size	No. of Users		Effluent (m^3)	DIMENSIONS					
	All Wastes	Soil Wastes Only		98 litres/m^2		196 litres/m^2		294 litres/m^2	
				Size (m)	Area (m^2)	Size (m)	Area (m^2)	Size (m)	Area (m^2)
1	2	3	4	5	6	7	8	9	10
I	10	10	1.134	3.81 dia.	11.57	2.74 dia.	5.90	2.21 dia.	3.85
II	13	15	1.488	4.42 dia.	15.29	3.13 dia.	7.62	2.59 dia.	5.25
III	15	20	1.700	4.73 dia.	17.47	3.35 dia.	8.78	2.74 dia.	5.86
IV	18	30	2.125	38.11 run	23.24	18.28 run	11.15	12.20 run	7.44
V	25	40	2.834	45.73 run	27.89	22.87 run	13.94	15.24 run	9.30

NOTE: This table is adapted from Schedule in Information Book: Sanitary Structures (Nigeria Public Works Department, 1940)

There is, however, no reason why in the larger tanks liquid depths up to 2m cannot be used where the position of the water table permits.

Columns 5-10 in Table 3.3 show soakage area dimensions for three different hydraulic loadings: 2gal/ft^2 (98 litres/m^2), 4gal/ft^2 (196 litres/m^2) and 6gal/ft^2 (294 litres/m^2). These correspond to loadings applicable to soils with percolation rates of 4.39, 1.10 and 0.49 minutes per inch (2.54cm) as shown in Fig. 5.2. In Chapter V it is shown that soils with percolation rates above 60 minutes per inch are considered not suitable for seepage pits and those with percolation rates greater than 30 minutes per inch are considered not suitable for any type of leaching system whatsoever. The soakage system shown in these specifications would appear therefore to apply only to to the excellent soils covered in the shaded portion of Fig. 5.2 which represents a mere 7% of the wide range of percolation rates for which leaching systems can be designed. Such excellent soils do exist in the sandy coastal areas or in the vicinity of rivers and streams. It is certain however that many thousands of soakaways constructed in the country are in soils which do not meet the percolation rate requirements applicable to their sizes.

In the design in Fig. 3.3 the effluent sewer from the septic tank enters and traverses the circular soakaway along a diameter near the bottom particularly in a shallow soakaway. The effluent therefore percolates through only a short depth of the gravel media before reaching the bottom. Tests to check the validity of the assumption that the whole of the circular area at the bottom is available for leaching away the effluent in these circumstances are described in Chapter IV. As a table adapted from the original work leading to equation 5.1 in Chapter V and to Fig. 5.2 derived from it appears to have been recommended for use by W.H.O. in the rural areas of the world (WAGNER, E.G. and LANOIX, J.N. 1958), the applicability of this equation to conditions outside the Campus of the Senator Robert A Taft Sanitary Engineering Center is examined critically in that Chapter.

The soakaway in Fig. 3.4 is an unlined hole, circular in section and with an open bottom. Presumably because of the small depth-diameter ratio and because the effluent pipe discharges near the bottom of the soakaway the P.W.D. design ignored the sides in calculating the area available for leaching away the effluent. The design of the soakaway sizes shown in columns 5, 7, and 9 of Table 3.3 therefore assumes that the leaching of effluent takes place through the bottom only.

While an unlined hole can be dug in stiff laterite soil it cannot be dug to any appreciable depth in sandy soils without the walls caving in. In practice therefore the soakaway in granular soils is built with side walls of honeycomb sandcrete construction standing either on a concrete slab or on strip foundation. The area available for leaching away effluent is only the sides where the walls are on a concrete foundation or both the side and a fraction of the bottom where the walls are on strip foundation. This is clearly a disadvantage in shallow soakaways.

The septic tank method of sewage disposal is very important in Lagos and the other towns of Nigeria. For many years municipal authorities have made the provision of a septic tank a bye-law requirement in all new building development. It is for this reason that the greater part of this Chapter and the whole of the next four chapters of this Thesis are devoted to a consideration of the leaching system in order to justify the existing practice and make suggestions for improvement.

CHAPTER IV

FLOW THROUGH SOAKAWAY AGGREGATE

4.1 INTRODUCTION

The design of the soakaway in Fig. 3.4 assumes that all the bottom area is available for leaching away the effluent and that the side area is not effective in the exercise. This is apparent from columns 4,and 8 in Table 3.3 in the last chapter where the bottom area of the soakaway or trench in each tank size multiplied by the daily rate of sewage application to the soakaway gives the daily quantity of sewage to be treated. This assumption is examined in this chapter.

The tank effluent discharges into the soakaway aggregate either through saw-cuts in the lower one-third of the periphery of the sewer or through the loose butt joints between the short pipe pieces. The path of the discharging effluent from a point in the invert of the pipe through the aggregate will depend on the velocity of discharge and the porosity of the aggregate. The porosity in turn - depends on the particle size and the packing of the aggregate.

At each pull of the chain of the cistern of a water closet a quantity of raw sewage approximately equal to the volume of the water in the cistern enters the influent end of the septic tank. It pushes out approximately a similar quantity of partially treated sewage from the effluent end of the tank into the sewer leading into the soakaway. The rate at which the sewage flows through the sewer depends on the influent velocity, the material as well as the gradient of the pipe.

The influent velocity will depend on the head of flow at entry to the sewer. The head in turn depends on the rate at which the raw sewage enters the tank and the horizontal area of the tank.

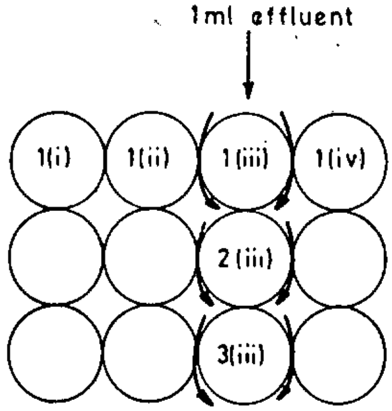
The surface area is $2.32m^2$,* which is large relative to the 9 litre quantity of sewage entering the tank at each pull of the chain. The rise in the level of the sewage in the tank due to this is only 4mm if even it is assumed that all 9 litres enters the tank instantaneously. The head available for flow through the sewer is therefore very small for the range of tank sizes shown in Table 3.2. In these circumstances the rate of flow through the sewer is small and the flow itself would be laminar. It is also non-uniform, varying theoretically from zero to a maximum, and back to zero over the period of discharging the effluent. Observations at a functioning domestic septic tank however revealed that there was always a 'trickle' flow from the sewer into the soakaway even when the WC was at rest and that the WC had to be flushed 3 times before an appreciable increase was noticed in the trickle flow, which continued after the cessation of discharge from the WC.

4.2 A THEORETICAL CONSIDERATION

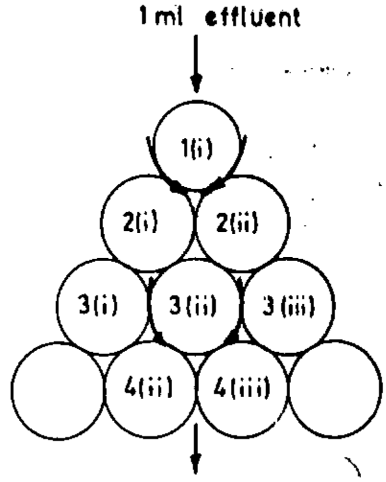
The aggregate specified in Fig. 3.4 for the soakaway medium is 3/4" - 2" (1.9cm - 5cm) rubble. What is used in practice is 5cm - 10cm rubble from broken sandcrete blocks and concrete. This aggregate is usually of different and irregular shapes. For simplicity however only aggregate of two regular shapes and two types of packing will be considered in this theoretical exercise. These are shown in Fig. 4.1.

The packing shown in Fig. 4.1(b) is more compact and results in less porosity than the other shown in (a) and could be expected to impede the percolation of sewage through it relative to (a). An elemental volume of effluent issuing from a saw-cut in the sewer and dropping symmetrically on particle 1(iii) in the top row of Fig. 4.1(a) could in the first instance be conceived as flowing round the particle in one plane in which half the quantity flows to the left and half to the right. The two halves again meet under 1(iii) but immediately begin to flow round particle 2(iii) and subsequently particles 3(iii) and 4(iii) in a similar manner. Should the velocity with which this elemental effluent strike the top of 1(iii) be high and above a certain value, the drop of effluent could splash and a fraction of it go to particle 1(ii) and a similar fraction to particle 1(iv). This would result in these other particles conducting part of the sewage element

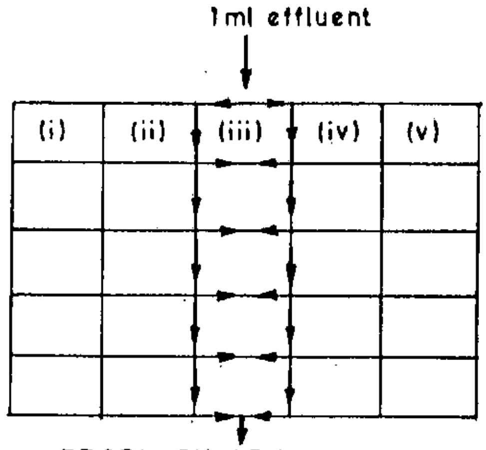
*Figure given is for the biggest tank size (V).



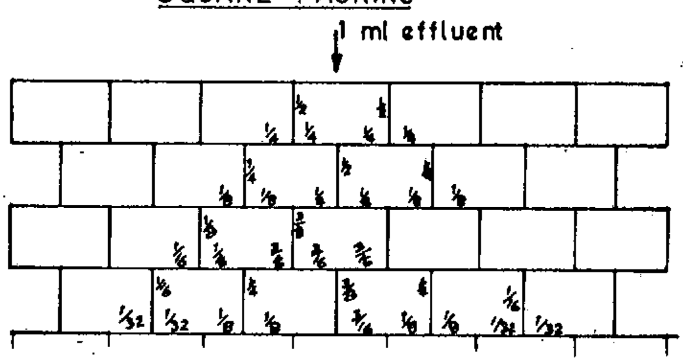
(a) SPHERICAL: SQUARE PACKING



(b) SPHERICAL: HEXAGONAL PACKING



(c) RECTANGULAR: SQUARE PACKING



(d) RECTANGULAR: BONDED PACKING

FIG. 4.1 TWO REGULAR SHAPES AND TWO TYPES OF PACKING OF AGGREGATE IN IDEALISED SOAKAWAY MEDIUM vertically downwards.

By the same reasoning an elemental effluent volume dropping symmetrically on top of particle 1(i) in Fig. 4.1(b) below this velocity will flow round 1(i) and the other particles vertically below it in a similar manner. In both (a) and (b) in Fig. 4.1 therefore the effluent goes through the soakaway medium vertically downwards in a column one particle wide. The one-plane concept can now be extended to include the other plane at right angles to get a true three dimensional picture of the flow. The result is the same, namely that the effluent flows through the soakaway medium vertically downwards in a column one particle thick.

In the case of the rectangular packing in which the blocks are not bonded but are loosely butt jointed, 4(c) the percolating effluent will also go through the soakaway aggregate in a vertical column one particle thick, as in the previous cases (Fig. 4.1.c). The bonded rectangular packing shown in Fig. 4.1(d) however presents a different situation. Here after each half of the original elemental volume flows round the rectangular block to the upper surface of the next course of blocks, it does not join the other half as in the previous cases. Rather on striking the horizontal face of the block below, each half sub-divides into two, the two halves of the original half now flowing round that block as shown in (d). In this case therefore there is a spread of the percolating effluent sideways down the medium.. The spread is symmetrical, in three dimensions, about the vertical axis through the point where the effluent strikes the first block.

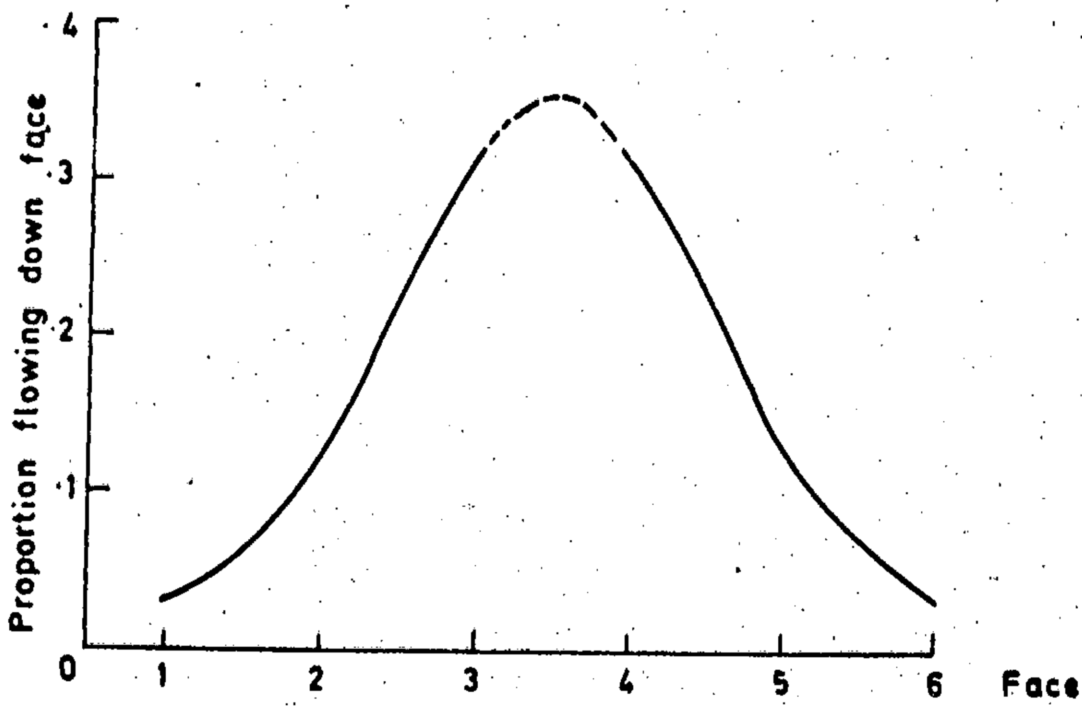
A re-arrangement of the numerators of the fractions of the quantities flowing along the vertical faces of the blocks is shown in Fig. 4.2(a). The numerators of these fractions are seen to form a Pascal's Triangle. This re-arrangement makes it easier to see how in this idealised set-up most of the flow goes through the centre and diminishes towards the periphery.

Where as in a sewer traversing the whole diameter of a circular soakaway the tank effluent discharges into the aggregate not through a single point but through a number of points spaced along the sewer, the spread of the effluent through the aggregate can be determined by considering the spread due to the discharge from each hole separately,

Course											
						$\frac{1}{2^0}$					
1					$\frac{1}{2^1}$		$\frac{1}{2^1}$				
2				$\frac{1}{2^2}$		$\frac{2}{2^2}$		$\frac{1}{2^2}$			
3			$\frac{1}{2^3}$		$\frac{3}{2^3}$		$\frac{3}{2^3}$		$\frac{1}{2^3}$		
4		$\frac{1}{2^4}$		$\frac{4}{2^4}$		$\frac{6}{2^4}$		$\frac{4}{2^4}$		$\frac{1}{2^4}$	
5	$\frac{1}{2^5}$		$\frac{5}{2^5}$		$\frac{10}{2^5}$		$\frac{10}{2^5}$		$\frac{5}{2^5}$		$\frac{1}{2^5}$

Numerators form a Pascal's Triangle

(a) Rearrangement of fractions of water flowing down vertical faces of blocks.



(b) Proportional flow down vertical faces in course 5

FIG.4·2 PROPORTIONAL FLOW OF WATER DOWN VERTICAL FACES IN RECTANGULAR BONDED PACKING super-imposing the various spreads on one another and finally adding up the various flow fractions at the appropriate points along the sewer length. It is emphasised that these results apply to idealised cases only.

4.3 TESTS ON THE PERCOLATION OF WATER THROUGH THE AGGREGATE OF A MODEL SOAKAWAY

The object of these tests is to verify the applicability of the theory developed above to the flow of water through a model soakaway.

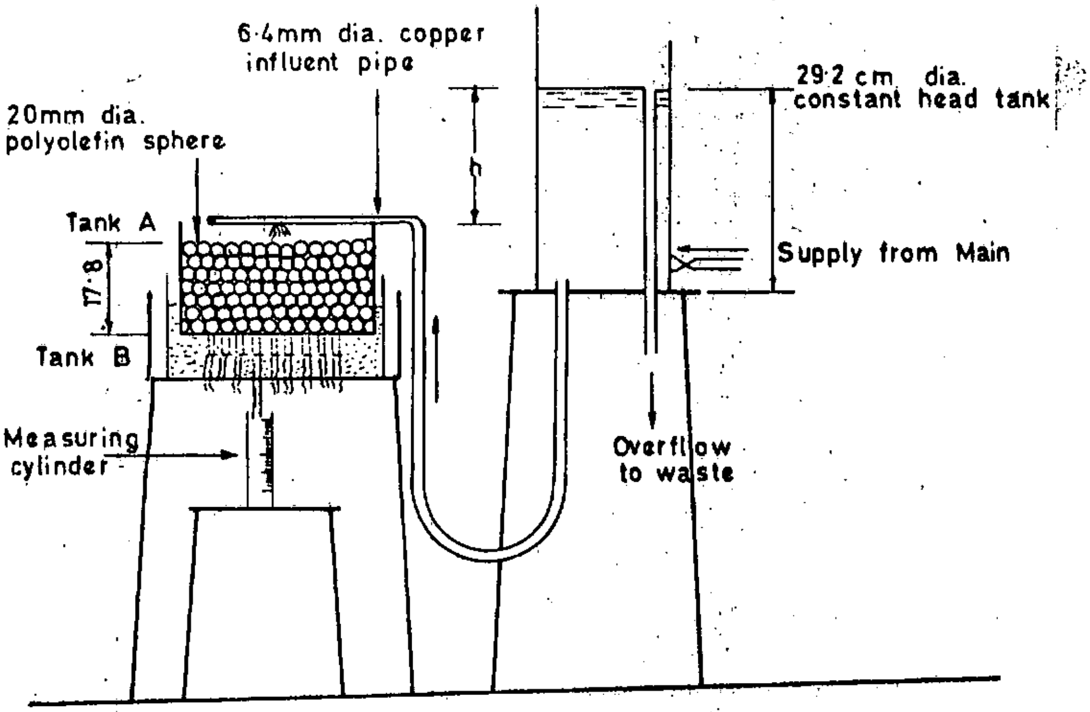
4.3.1 Materials and Method

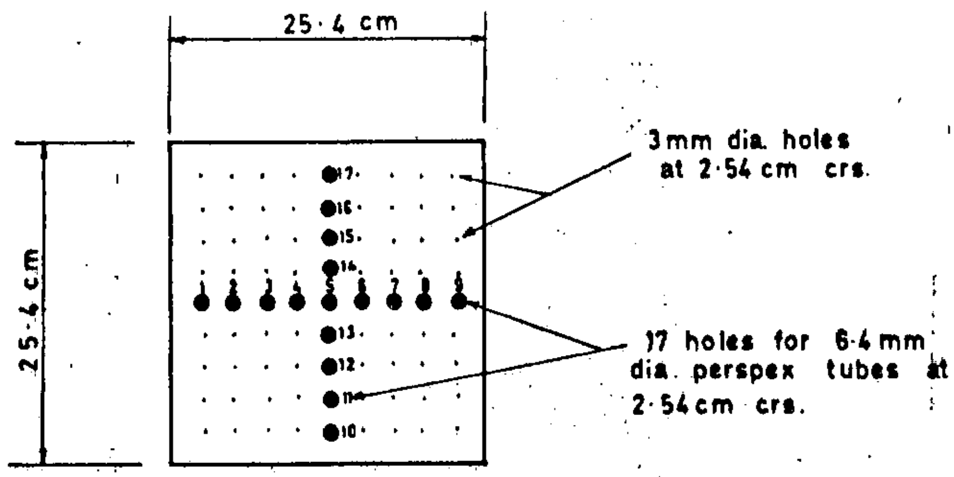
/made

The model was/from .64cm thick perspex sheets into a tank 20.3cm high and 25.4cm by 25.4cm square section. 13 rows of 3mm dia holes spaced 2.54cm apart were drilled at the bottom of the tank in a grid formation as shown in Fig. 4.3. 17 of the holes symmetrically arranged in the 2 middle rows at right angles were enlarged to take 6.4mm dia perspex tubes, as shown in Fig. 4.3. These tubes were connected to 4mm bore plastic tubes, and numbered 1 to 17.

There was an outer tank of 6.4mm thick galvanised iron, 15.2cm high and 45.7cm square section. 17 holes of 15mm dia were drilled at 2.54cm centres along the two central axes of the bottom of this second and lower tank. Through these holes the 17 tubes from the enlarged holes at the bottom of the upper tank were passed as shown in Fig. 4.3. In the first series of tests the upper tank was filled with 20mm dia polyolefin spheres in a hexagonal packing arrangement as in Fig. 4.1(b). In the second series the spheres were replaced with 19.1mm size gravel. In both series the bottom tank was filled with lagoon sand which represented the soil into which water would enter after percolating through the aggregate in the upper tank.

A 12mm dia plastic tube was connected to the exit pipe of a 29.2cm dia constant head perspex tank at one end, and a 6.4mm dia 18cm long copper tube at the other end. The copper tube was plugged at one end and had a 6.4mm dia hole drilled in it 10.5cm from that end.





PLAN OF BOTTOM OF TANK A
SHOWING GRID OF DRAINAGE HOLES

FIG. 4·3 MODEL SOAKAWAY FOR INVESTIGATING FLOW THROUGH AGGREGATE MEDIUM.

The two tanks, one on top of the other were carried on a platform in a 65mm x 50mm x 3mm thick No. 260 Dexion iron angle frame. By this arrangement the height of the platform from the floor could be adjusted to give the desired variation in the difference between the level of the water in the constant head tank and the exit hole in the copper tube, and so vary the flow. This head is designated h in Fig. 4.3.

At the start of a test h was adjusted to the desired value. The copper tube was placed horizontally just 2mm above the top layer of aggregate in Tank A. Care was taken to see that the hole in the tube was directly at the centre of area as represented by the intersection of the diagonals of the square plan of the tank. After this the tap at the supply end of the constant head tank was opened. Water then began to come out of a number of the 17 tubes connected to the bottom of tank A.

After the test had been running for 15 minutes when it could be assumed that a steady state had been reached in the flow through the aggregate into the tubes, the flow in each tube was measured by collecting the quantity coming out of the tube in 3 minutes and measuring this in a graduated cylinder. The samples were collected and measured from tube to tube. As expected there was no flow from a number of tubes while the flow was plenty in a few others. There were 3 and sometimes 4 readings each day at each setting of the platform in the Dexion Iron angle frame.

Some of the results are shown in Tables 4.1 and 4.2.

4.3.2 <u>Results and Discussion</u>

Table 4.1 shows that in the tests on the flow of water through the gravel medium all the flow collected through the tubes came from 7 out of the 17 holes at the bottom of the inner tank. These are holes 4, 5, 6, 7, 12, 13 and 14. Fig. 4.3 shows that these comprise the centre hole 5, the four holes immediately next to it on both axes, as well as the 2 holes (7 and 12), each one step still further away from the centre hole. The highest flow came from hole 13, just one step from the centre hole.

TABLE 4.1

COMPARISON OF SPREAD OF WATER FLOWING VERTICALLY THROUGH (a) SPHERICAL BALLS, HEXAGONAL PACKING AND (b) IRREGULAR SHAPED, RANDOM PACKING AGGREGATE MEDIUM IN SOAKAWAY MODEL

Hole No.	Gravel Medium, 27.4.76					Spherical balls medium 24.6.76				
	Run 1	2 (ml)	3	Av. flow	Duration (min)	Run 1	2 (ml)	3	Av. flow	Duration (min)
1	2	3	4	5	6	7	8	9	10	11
1	0	0	0	0	3	0	0	0	0	3
2	0	0	0	0	3	0	0	0	0	3
3	0	0	0	0	3	120	125	85	110	3
4	90	90	80	87	3	80	870	715	555	3
5	320	370	280	323	3	775	885	630	763	3
6	420	488	500	469	3	615	470	150	412	3
7	55	31	34	40	3	135	85	120	113	3
8	0	0	0	0	3	0	0	0	0	3
9	0	0	0	0	3	0	0	0	0	3
10	0	0	0	0	3	0	0	0	0	3
11	0	0	0	0	3	0	0	0	0	3
12	255	310	300	288	3	0	0	0	0	3
13	805	690	703	733	3	380	90	0	157	3
14	0	29	7	12	3	540	510	410	487	3
15	0	0	0	0	3	0	10	30	13	3
16	0	0	0	0	3	165	130	180	158	3
17	0	0	0	0	3	0	0	0	0	3
TOTAL	1945	2008	1904	1952	-	2810	3175	2320	2768	-

Total flow rate = 1600ml/min
Total flow collected in 17 holes
= 2768 mls in 3 mins.
Total flow in 5 middle holes
4, 5, 6, 13, 14 = 2374mls.
% of flow through middle holes
$= \frac{2374}{2768} = 85.8\%$

Total flow rate = 1524ml/min.
Total flow collected in 17 holes
= 1952mls in 3 mins.
Total flow through middle holes
4, 5, 6, 13, 14 = 1624mls
% of flow through middle holes
$= \frac{1624}{1952} = 83.2\%$

TABLE 4.2

VARIATION OF SPREAD OF WATER FLOWING VERTICALLY THROUGH SPHERICAL BALLS HEXAGONAL PACKING MEDIUM WITH RATE OF FLOW IN 'SEWER'

Date	Average flow through 17 holes (ml/min)	Flow through 5 centre holes	% flow through centre (ml/min)	Total flow in 'sewer'	Average influent velocity in medium (cm/sec)
1	2	3	4	5	6
24.6.76	483.8	427.7	88	820	43.16
29.6.76	123.3	108.9	88	610	32.10
30.6.76	70.2	68.7	98	308	16.21
1.7.76	46.3	46.0	99	200	10.53
5.7.76	23.3	23.3	100	118	6.21

The proportion of flow through the 5 centre holes increases with decreasing flow in the model sewer.

The highest flow came from hole 13, just one step from the centre hole. The calculations at the bottom of Table 4.1 show that 85.8% of all the flow collected through all 17 holes in fact came through the 5 middle holes.

Table 4.1 shows that in case of the tests with spherical balls medium the flow is a little more spread out as flows were recorded in holes 15 and 16 (2 and 3 positions away from the centre hole). Furthermore the flow in hole 16 which was 3 positions distant from the centre was much greater than the flow in hole 15 one position nearer the centre. The proportion of the flow through the five middle holes to the flow through all 17 holes is however 83.2%, which agrees reasonably with the figure of 85.8% for the medium with gravel. These tests show quite convincingly that most of the flow came through the centre area of both the gravel medium and the sperical balls medium.

Table 4.2 shows that in the tests with the spherical balls medium in which the rate of flow was progressively <u>decreased</u> from an average of 820ml/min. on 24/6/76 to 118 ml/min. on 5/7/76 the proportion of the flow through the five middle holes <u>increased</u> progressively. It is seen in this Table that at flows of 200 ml/min. and under practically all the flow came through these middle holes.

Column 6 in Table 4.2 shows the average influent velocity of the water entering the medium. The velocity was quite small on the last day of the tests. This velocity (6.21cm/sec) was however still greater than the velocity with which the tank effluent discharges into the medium of an operational soakaway. As mentioned earlier observation in such an operational tank and soakaway showed that there was always trickle flow into the soakaway when the WC has not been flushed and that even after the WC has been flushed there was little increase in the flow into the soakaway. This is due to the large surface area of the tank compared with the volume of the WC cistern, which creates only a small head (under 1cm) available for increasing the flow in the sewer into the soakaway.

For the flow in the tests on the last 2 days in Table 4.2 the effective cover of the water at the bottom of the model soakaway is approximately a circle with centre at the centre of hole 5 and radius 2.54cm, which is the distance from the centre of hole 5 to the centre of hole 4 (Fig. 4.3). This radius happens to be 2.54cm which is the spacing of the holes at the bottom of the tank in these tests. Obviously it would change with a different spacing of the holes.

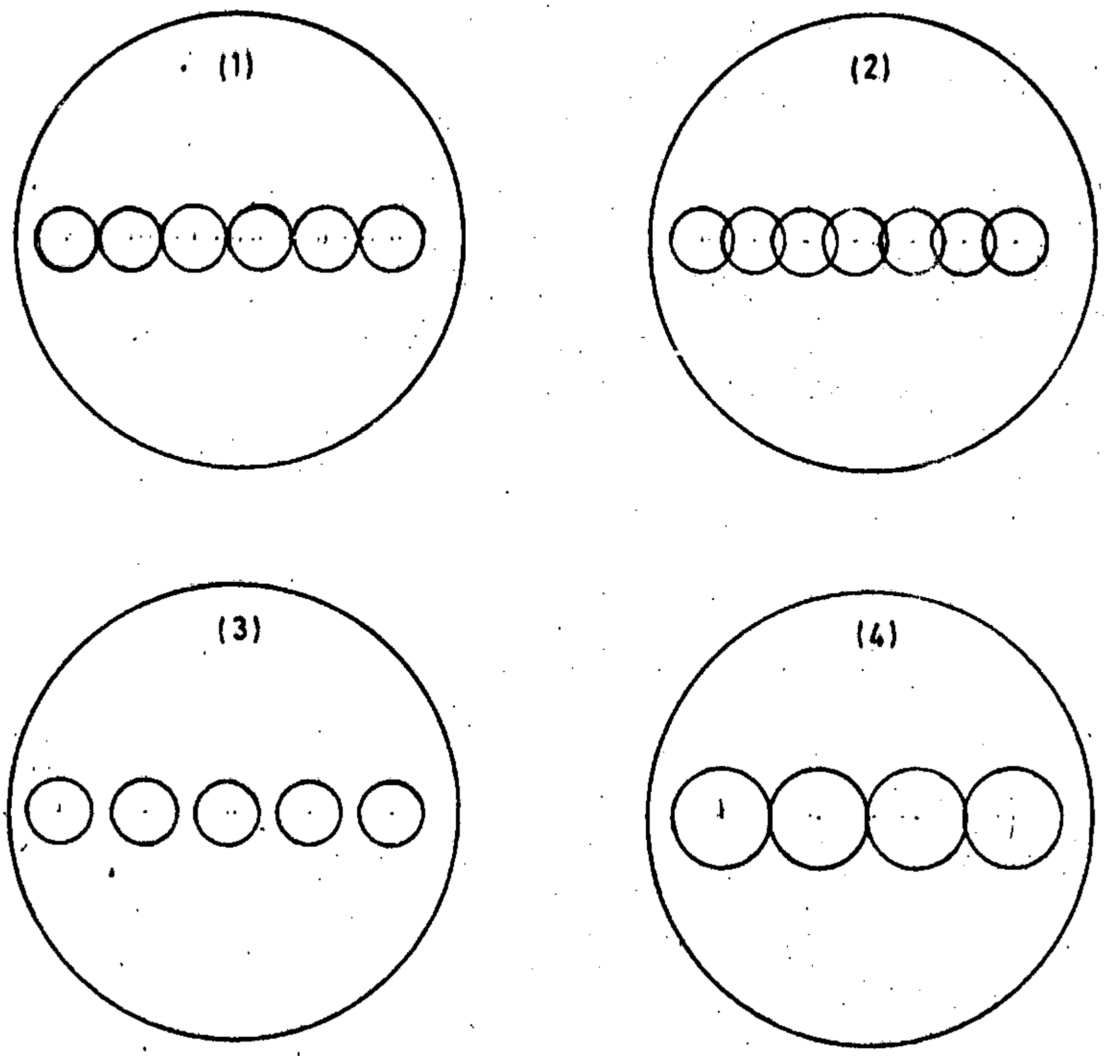
The spread of the water downwards through the medium could be assumed to be a cone of height 17.8cm, which is the depth of the medium, and subtending an angle of 2 $\tan^{-1}$ 0.1427

$$(\tan \frac{\Theta}{2} = \frac{2.54}{17.8} = 0.1427) .$$

Assuming this conical formation obtains and extends through the medium of an operational soakaway the circle covered at the bottom of a 1m. deep soakaway would have a diameter of 28.54cm while that at the bottom of a 1.2m deep soakaway would have a diameter of 34.25cm. The sizes of the circles at the bottom of soakaways of different depths for this particular conical formation are shown below.

Depth of Soakaway (m)	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0
Diameter of wetted circle at bottom (m)	0.171	0.228	0.285	0.343	0.400	0.457	0.514	0.571

Since in the 1.2m deep soakaway the diameter of the circle irrigated at the bottom of the soakaway by the effluent discharging from a particular hole from the invert of the sewer is .434m. the circles formed at the bottom of the soakaway by holes spaced this distance apart along the invert of the sewer will just touch one another. Holes spaced nearer each other than this distance will overlap each other at the edges while holes spaced longer distances apart will create gaps of un-irrigated areas between the circles at the bottom (Fig. 4.4).



(1) At soakaway depth 1·2m and hole spacing 0·343m crs. along sewer 6 circles of 0·343m dia. form at bottom.

(2) At same depth but closer spacing of holes along sewer, same size circles form but overlap

(3) At same depth but wider spacing of holes along sewer, same size circles form with gaps.

(4) At soakaway depth 1·8m and hole spacing 0·514m dia form at bottom.

FIG.4·4 CIRCLES OF WETTED AREA AT BOTTOM OF 2·21m DIA. SOAKAWAY (TANK SIZE I)

In actual practice the effluent discharges from the invert of the sewer not through point holes but through short cuts in the lower third of the periphery of the sewer section or through the butt joints between adjacent pipes where the sewer is formed of short pipe lengths. In this case the area at the bottom of each hole irrigated by the effluent from it forms not a circle but an oval, with a long axis at right angles to the length of the sewer (Fig. 4.4). Again holes spaced .343m apart in the invert of such a sewer in a 1.2m deep soakaway will create irrigated areas at the bottom which again just touch each other.

4.4 THE SOAKAWAY IN NIGERIAN BUILDING PRACTICE

The results of the tests discussed above would appear to indicate that only a little strip at the bottom of the soakaway vertically below the sewer actually receives the effluent, and that the larger portion on either side of this strip receives no effluent. The effectiveness of this larger portion could therefore be queried.

From the table above it could be assumed that at a depth of 1.2m where the diameter of the wetted circle is .343m if the saw-cuts along the sewer are spaced at .343m or under, the wetted portion would form a strip, approximately rectangular and .343m wide. The area of such a strip in a 4.73m dia soakaway (size III tank, Table 3.3) is $1.61m^2$. The total bottom area of the soakaway is $17.47m^2$, giving a ratio of $\frac{1.61}{17.47} = .0919$ of wetted area to total area. This would mean that only 9.19% of the area provided at the bottom of the soakaway is effective. Table 4.3 shows the percentages of the effective area to the total area in three of the soakaway sizes in Table 3.3 for depths varying from 0.6m to 2.0m. calculated on this basis. The soakaway sizes considered are of 2.21m, 3.13m and 4.73m (Table 3.3).

Table 4.3 shows that only mere fractions of the bottom areas of the three soakaway sizes are being effectively used at the range of depths considered. The logical inference from this would be to come out with new soakaway designs of smalller dimensions and more economic shape.

TABLE 4.3

PERCENTAGE AREA OF BOTTOM WETTED BY EFFLUENT IN 3 STANDARD CIRCULAR SECTION SOAKAWAY SIZES AT VARIOUS DEPTHS

Depth (m)	Spacing of saw-cuts in sewer (m)	Percentage of bottom area wetted		
		2.21m dia soak	3.13m dia away	4.73m dia
1	2	3	4	5
0.6	0.171	9.84	6.96	4.61
0.8	0.228	13.10	9.28	6.92
1.0	0.285	16.42	11.63	7.71
1.2	0.343	19.60	13.92	9.19
1.4	0.400	22.92	16.25	10.76
1.6	0.457	26.15	18.55	12.29
1.8	0.514	29.35	20.85	13.82
2.0	0.571	32.53	23.13	15.35

It was shown in the last chapter that the P.W.D. design figures of 98, 196 and 294 litres / m² are very high on the sewage loading rate - percolation rate curve developed by the Senator Robert A. Taft's Sanitary Engineering Center's researchers (3.2.3). If this curve is applicable to Nigerian soil conditions then only soils with percolation rates of 4.39 minutes* and under should be used for septic tank soakaways at the loading rates in the P.W.D. specifications. The vast majority of the soils in which these standard size soakaways have been constructed are shown to have percolation rate figures far in excess of 4.39 minutes. If this curve is applicable to Nigerian soil conditions then the use of the P.W.D. design figures should lead to overloading and subsequent failure of the soakaways constructed to these specifications.

As no mass failure of soakaways has been reported one or the other of two things may be happening: either the curve is not applicable to conditions in Nigeria which would mean that for Nigerian conditions the allowable sewage loading rates would be higher than those derived from equation 5.3 in the next chapter; or the P.W.D. design rates in fact lead to overloading of the central strip of the soakaway bottom in the first instance but that the excess effluent that the central strip can not absorb flows sideways into the area now considered ineffectively used on either side of the central strip from where it leaches away into the soil. Further work is required to establish which of these two alternative possibilites is actually operative. It may well be a bit of both of them.

The observations made so far apply only to the actual area of the soakaway and not to its shape. There is no doubt that having decided on the actual area required, providing this in a long and narrow rectangular shape would be more satisfactory than providing it in a circular section. A rectangular shape has the advantage of ease of construction. Also where the sides participate in the leaching away of the effluent the rectangular shape offers a larger side area than the circular shape for a given area.

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*definition of percolation rate at page 79.

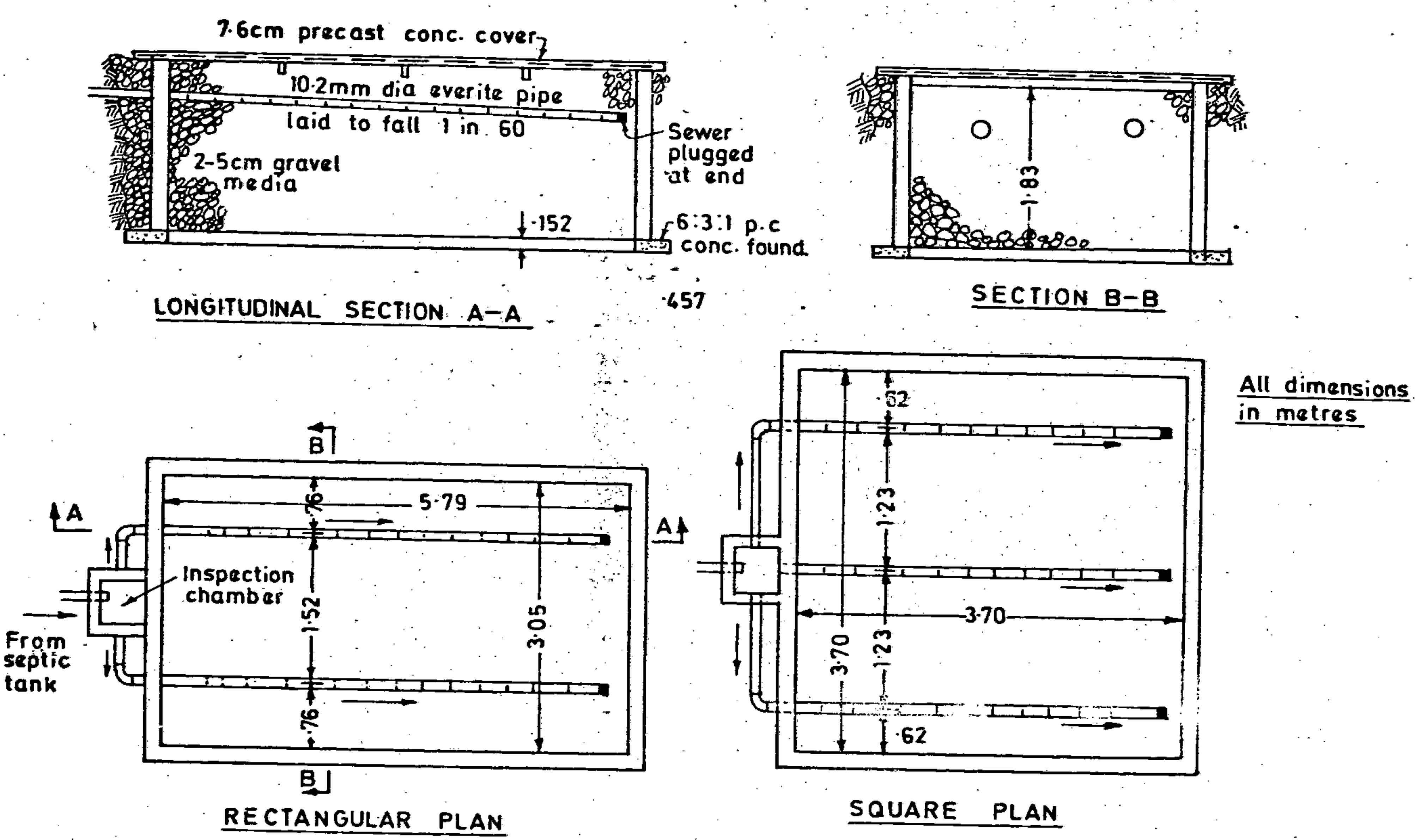


FIG. 4·5 SUGGESTED ALTERNATIVE DESIGN FOR P. W. D. STANDARD SOAKAWAY FOR SEPTIC TANK SIZE III (4·73m DIA.)

Fig. 4.5 is a suggestion for a rectangular design for a soakaway in granular soil to take effluent from a size III Septic Tank at an effluent loading rate of 98 litres/m². This loading rate and therefore the original P.W.D. design area of 188m² have been used in this exercise. The innovation here is the rectangular shape and the better distribution of the effluent through the soakaway aggregate medium to bring a larger area at the bottom of the soakaway into immediate effectiveness in the leaching away of the effluent. The square section possesses the advantage of economy in the use of land and wall material. It however suffers the same disadvantage as the circle in the smallness of the percentage area of the strip immediately effective at the bottom of the soakaway.

CHAPTER V

THE PERCOLATION OF WATER AND SEWAGE EFFLUENT THROUGH SOILS

5.1 The water content in the average raw domestic sewage is of the order of 99.94% (Griffin, G.E.). The water content in a septic tank effluent, which is the original raw sewage minus some 60% of the suspended solids, is still higher than this figure. The effluent therefore is very largely water, and would be expected to behave to a large extent like water in its passage through the soil under a soakaway or soakage trench. Super-imposed on this normal percolation of water through soils however is the effect of biological activity in which bacteria present both in the sewage effluent and in the soil break down the soluble and insoluble organic compounds in the sewage effluent in addition to forming new biological solids. The study of the percolation of septic tank effluent through the soil of the absorption area is therefore essentially a study of the combined effect of these two processes.

5.2 PERCOLATION OF WATER THROUGH SOILS

5.2.1 In a review of the literature of filtration theories Sakthivadivel, R. and Irmay S. (1966) noted that previous workers had variously assumed, observed or concluded that:

(a) the movement of fines through a filter is the result of two antagonistic processes:

(i) deposition of fines in the filter caused by the adherence of fines to the filter sand or to the sediment particles settled before, and

(ii) removal of sediment from the deposit into suspension caused by hydrodynamic forces (Mints and Krishtal, 1951 and 1960);

(b) the clogging of filters is due to the adhesion of coagulated fines to the filter sand and that during filtration the interstitial velocity increased until it reached a value where the hydrodynamic forces overcame the forces of adhesion when no deposition occurred (Maskrle, 1960),

(c) the rate of deposition of fines depends mainly on turbidity, concentration of fines in suspension and the amount already deposited: a particle once deposited was not further removed (Shekhtamn, 1961);

(d) any suspended particle brought in contact with a rough filter particle adhered to it by Van der Waals' forces (Ives, 1960, 1962).

5.2.2 Sakthivadivel, R. (1966) himself contributed the theory of deposition by bridging in which a fine particle arriving at a pore less than twice the diameter of the fine could not pass and formed a bridge which is strengthened by drag and fall velocity and on which other fines arriving at the reduced pore are deposited.

5.2.3 Taylor, D.W. (1962) listed the following factors in the soil itself which affect the passage of water through it:-

(i) equivalent grain size, Ds,
(ii) the density w and coefficient of viscosity μ of the pore fluid,
(iii) the void ratio e,
(iv) the shape and arrangement of the pores,
and (v) the quantity of the undissolved gases in the pore water.

The first three of these factors are connected in the following relationship:-

$$k = D_s^2 \cdot \frac{w}{\mu} \cdot \frac{e^3}{1+e} \cdot C \quad \ldots \ldots (5.1)$$

where k = coefficient of permeability (cm/sec),
C = constant, reflecting shape effects.

5.3 PERCOLATION OF EFFLUENT THROUGH SOILS

5.3.1 Bendixen, T.W. et al (1950) separated the causes of soil clogging due to the addition of septic tank effluents into three factors:-

(i) physical action of the solids added,
(ii) biological action and the effect of the products formed from it,
(iii) deterioration of the soil structure due to chemical changes in the soil.

They concluded that clogging during the first several days of dosing is directly proportional to the amount of suspended solids added to the soil after which biological activity in the soil reduced the rate of clogging.

5.3.2 McGauhey, P.H. and Winneberger, J.H. (1964) observed that failure in subsurface trenches and seepage beds used in septic tank installations is caused by a clogging mat that is deposited as a thin sheet at the surface of the sand and not by clogging within the filter.

It is seen from this short review that in the percolation of water through a porous bed of sand or soil fine particles in the water are deposited in certain pores, which reduces the effective size of these pores and consequently the void ratio e. This would have the effect of reducing the permeability coefficient k in equation (5.1) as well as the rate at which the water passes through. The clogging of the pores in the passage of a sewage effluent through a soil is nearly entirely in the surface layer, which prevents the effluent from penetrating through to the lower layers.

5.4 SOIL PERCOLATION TESTS

This difference in the nature of clogging of soil pores in the passage of water through a soil on the one hand and the clogging consequent upon the passage of sewage effluent on the other leads to a corresponding difference in the rate at which water percolates through a soil and the rate at which sewage will percolate through the same soil. Previous investigations into the sewage effluent absorption capacity of soils have attempted to establish a link between the two rates. Such correlation would mean that it would be necessary to know only the percolation rate of water through a soil to predict the percolation rate of sewage effluent through the soil. This would be advantageous as water is more conveneint to handle than sewage effluent.

In the percolation test developed at the Robert A Taft Sanitary Engineering Center, Cincinnati, the rate at which the level of water dropped in a vertical hole 10 to 30 cm dia dug in the soil to the depth of the proposed absorption tiles is measured at intervals of 30 minutes in a 4-hour test. The drop in level over the final 30 minutes is used in calculating the percolation rate which the Cincinnati investigators observed to be independent of the shape and size of the hole. In sandy soils in which the water seeps away fast the test is run for 1 hour, observations being taken at 10 minutes intervals and the percolation rate calculated from the drop in level over the last 10 minutes of the test. The percolation rate is defined as the time t required for water in the hole to drop 2.54cm (1 inch.)

5.4.1 Development of the Empirical Formula for Percolation Tests:

The Cincinnati studies covered 97 septic tank effluent absorption fields in 10 States in U.S.A. over a period of 17 months. The percolation rate was determined for each field. The per capita daily water consumption for each household was determined from water meter records over a period of six months. From this the total daily load of sewage was calculated. The area of the tile line trench was measured or estimated in the field. The daily load divided by the trench area gave the daily rate of application of sewage effluent per unt area of the tile field.

Information was obtained from each household about the history of each absorption field which together with the observation of the field at the time of inspection, was used in evaluating the efficiency of each field. There were three categories of efficiency employed as follows:-

(i) good - tile field has never given any trouble;
(ii) fair - seepage to the ground surface has been observed occasionally;
and (iii) poor - tile field has given trouble continuously.

The applied load in gallons per sq. ft. was then plotted against the percolation rate in minutes per inch for each field, three different symbols being used, one for each of the categories of efficiency (Fig. 5.1). Finally a curve was drawn separating points denoting fields of 'good' performance from those of 'fair' and 'poor' performance. The researchers found that the curve approximated closely to the equation:

$$q = 5\ t^{-\frac{1}{2}} \quad \ldots \quad (5.2)$$

where q = allowable rate of sewage application in U.S. gallons per sq. ft. per day.

t = percolation rate in minutes per inch.

This converts to the following equation in metric units:

$$q = 204\ t^{-\frac{1}{2}} \quad \ldots \quad (5.3)$$

where q = allowable rate of sewage application in litres/m² /day,

and t = percolation rate in minutes per 2.54cm (Fig.5.2).

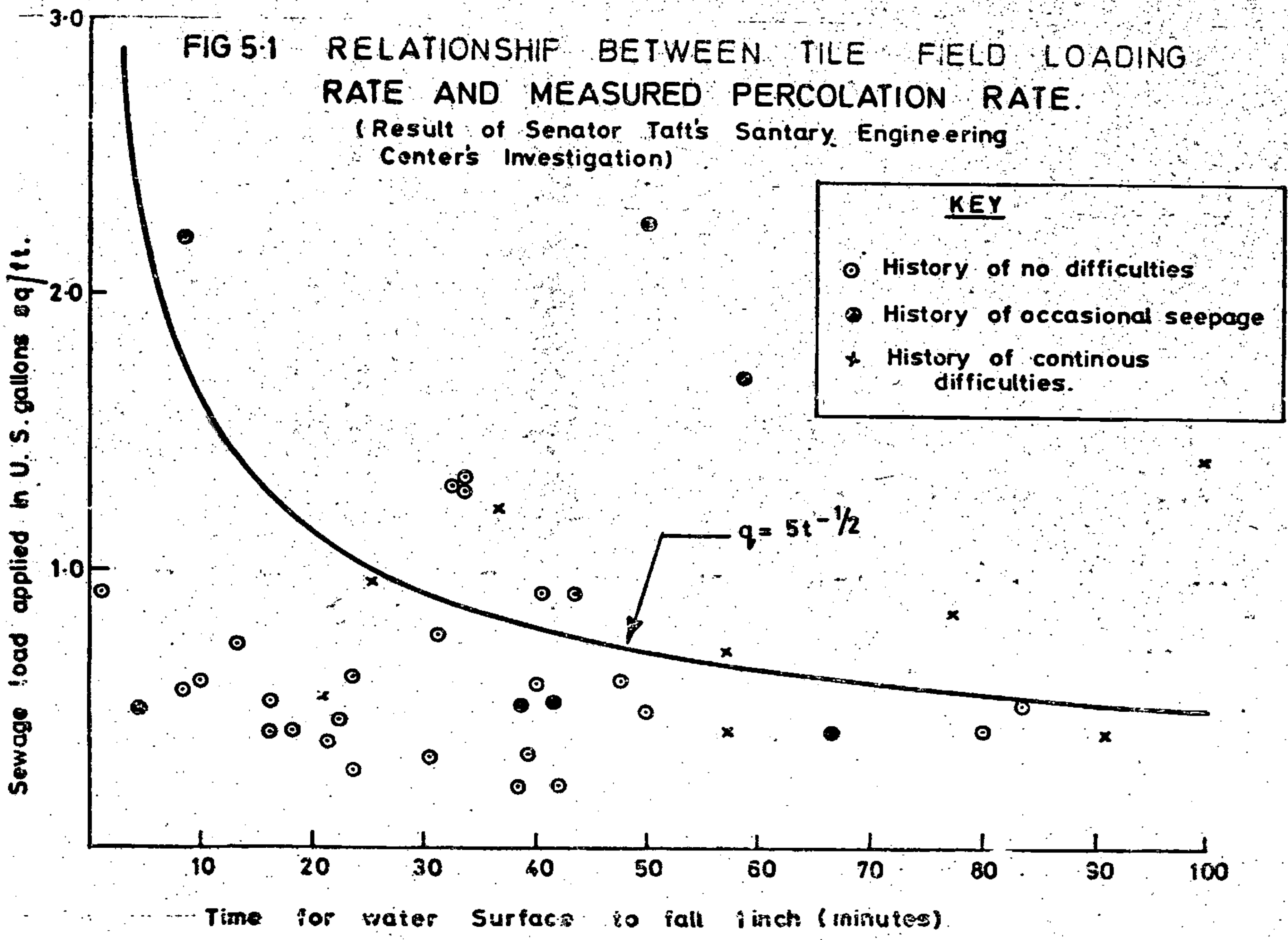
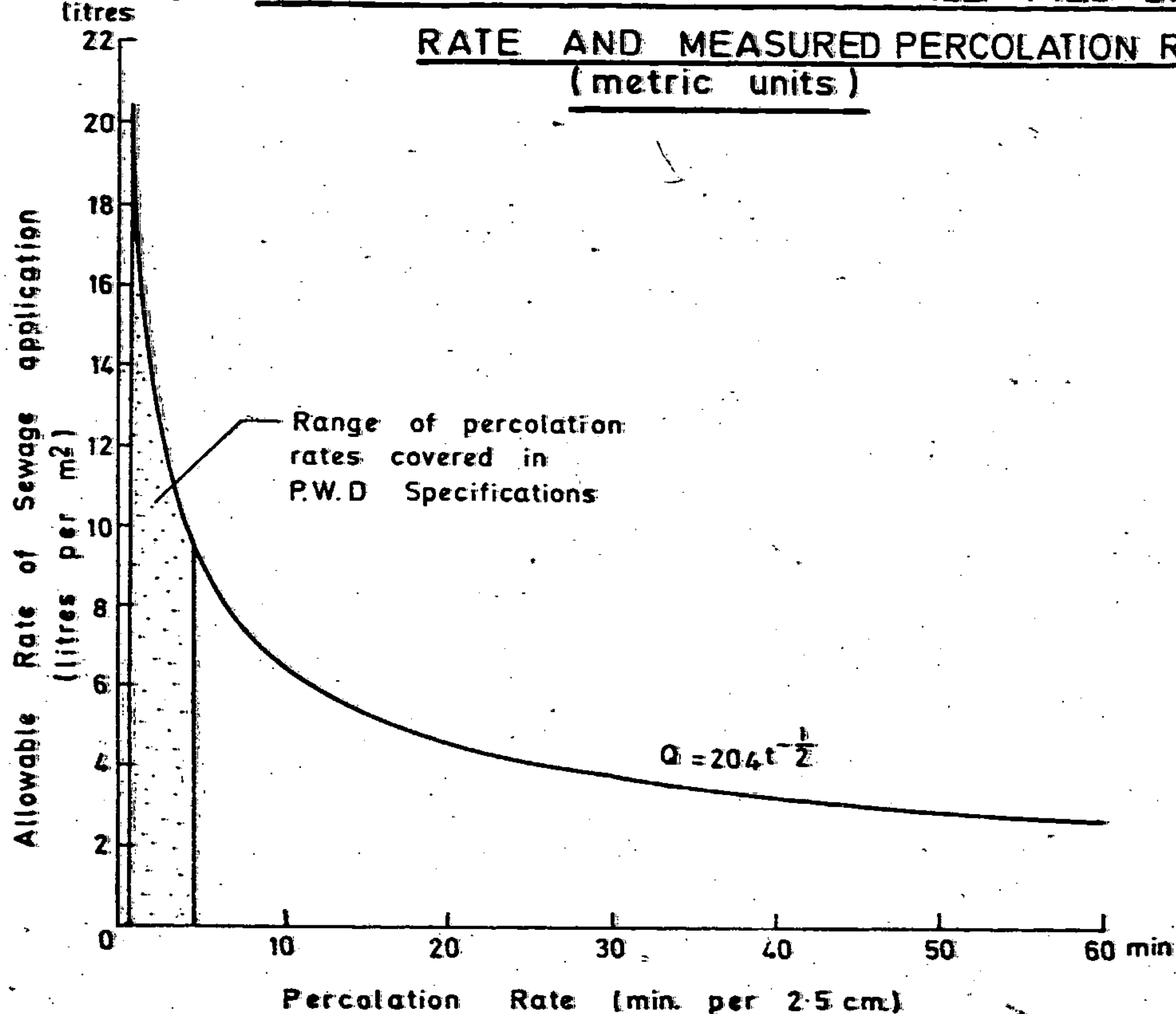


FIG 5·1 RELATIONSHIF BETWEEN TILE FIELD LOADING RATE AND MEASURED PERCOLATION RATE.
(Result of Senator Taft's Santary Engineering Center's Investigation)

FIG. 5·2 RELATIONSHIP BETWEEN TILE FIED LOADING RATE AND MEASURED PERCOLATION RATE (metric units)



5.4.2 Criticism of the Empirical Formula for Percolation Tests:

The recommendation of the Cincinnati equation for universal application can be critised on three different grounds. Firstly soil is not a homogeneous material, and is known to show considerable variation in property in short distances both horizontally and vertically. In fact variation in property has been observed in the same hole from day to day. A percolation rate value determined for a particular site can only be approximate in view of these limitations. The value will be more reliable if it is the average of values obtained from several holes closely spaced on the site, and spread over a period of time. Even though the Cincinnati researchers were aware of these limitations they made only two percolation tests on each tile field. It is considered that any equation developed in such circumstances will suffer the adverse effect of inadequacy of date. Secondly the researchers relied on evidence from house-holders using the individual tile fields. This is subject to human error of fact on the part of house-holders and of evaluation on the part of the investigators. This is not considered acceptable in the development of a formula of importance. Thirdly, and finally, even if the research had overcome the first two criticisms it would be difficult to support the universal application of this equation developed in one part of a country to other countries thousands of miles away and located in entirely different climatic regions.

It is considered that an experiment under laboratory conditions in which both water and sewage effluent are made to percolate side by side in the same soil would yield more convincing results about the comparative rates of percolation of both. Both would be subject to the same external factors during the experiment. Also internal factors like the grain size and void ratio could be varied from test to test to see how the percolation rate of both water and sewage effluent vary with these factors.

However, in view of what appears to be universal acceptance of the empirical formula, in spite of the adverse observations made above on its development, the next section is devoted to a mathematical formulation for percolation tests in both lined and unlined holes.

5.4.3 A Mathematical Formulation for Percolation Tests:

Let r = radius of circular hole (Fig. 5.3)

H = original height of water in hole at time $t = 0$,

h = height of water in hole at time t,

s = progressive fall in water level at time t,

Q = quantity of water discharged in time t.

Water escapes through both sides and bottom.

Assume $v = k\,(2gh)^{\frac{1}{2}}$

Water escaping through bottom:

$$\frac{dQ}{dt} = av$$

$$= \pi r^2\, k_b\, (2gh)^{\frac{1}{2}} \quad .. \quad .. \quad .. \quad (5.4)$$

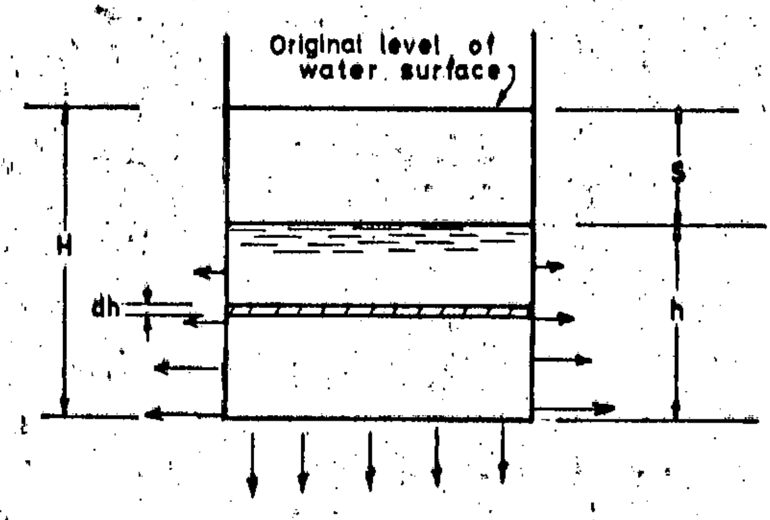
Water escaping through sides:

$$\frac{dQ}{dt} = a \int_0^h v\, dh$$

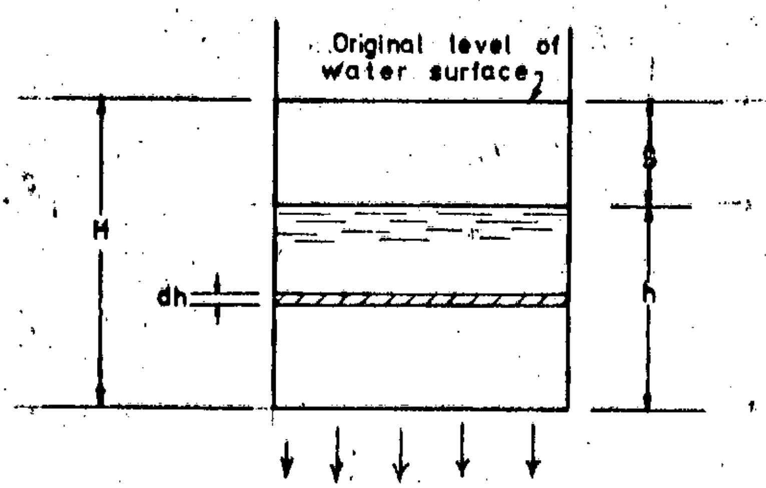
$$= 2\pi r\, k_s \int_0^h (2gh)^{\frac{1}{2}}\, dh$$

$$= 2\pi r\, k_s\, (2g)^{\frac{1}{2}} \int_0^h h^{\frac{1}{2}}\, dh$$

$$= \frac{4}{3}\pi r\, k_s\, (2g)^{\frac{1}{2}} \Big[h^{3/2}\Big]_H^h \quad .. \quad .. \quad (5.5)$$



(a) DISCHARGE THROUGH UNLINED HOLE



(b) DISCHARGE THROUGH LINED HOLE

FIG. 5·3 DISCHARGE OF WATER FROM (a) UNLINED AND (b) LINED HOLES.

Water escaping through both sides and bottom:

$$\frac{dQ}{dt} = \pi r^2 k_b (2g)^{\frac{1}{2}} \left[h^{\frac{1}{2}}\right]_H^h + \frac{4}{3}\pi r k_s (2g)^{\frac{1}{2}} \left[h^{\frac{3}{2}}\right]_H^h$$

$$= \pi r (2g)^{\frac{1}{2}} \left[k_b r h^{\frac{1}{2}} + \frac{4}{3} k_s h^{3/2}\right]_H^h$$

$$\frac{ds}{dt} = \frac{1}{a}\frac{dQ}{dt} = \frac{dQ}{dt} \cdot \frac{1}{\pi r^2}$$

$$= \frac{(2g)^{\frac{1}{2}}}{r} \left[k_b r h^{\frac{1}{2}} + \frac{4}{3} k_s h^{3/2}\right]_H^h \quad \ldots \quad \ldots \quad (5.6)$$

$$\frac{ds}{dt} = -\frac{dh}{dt} \quad \text{(Fig. 5.3)}$$

$$-\frac{dh}{dt} = \frac{(2g)^{\frac{1}{2}}}{r} \left[k_b r h^{\frac{1}{2}} + \frac{4}{3} k_s h^{3/2}\right]_H^h$$

$$-\frac{dh}{k_b r h^{\frac{1}{2}} + \frac{4}{3} k_s h^{3/2}} = \frac{(2g)^{\frac{1}{2}}}{r} \cdot dt$$

Putting $\frac{4}{3}k_s = m$, $k_b r = n$,

$$-\frac{dh}{mh^{3/2} + nh^{\frac{1}{2}}} = \frac{(2g)^{\frac{1}{2}}}{r} dt$$

Now $$\int \frac{dx}{x^{\frac{1}{2}}(a + bx)} = \frac{2}{(ab)^{\frac{1}{2}}} \tan^{-1}\left(\frac{bx}{a}\right)^{\frac{1}{2}}$$

$$\therefore \int_H^h \frac{dh}{h^{\frac{1}{2}}(mh + n)} = \frac{2}{(mn)^{\frac{1}{2}}} \tan^{-1}\left(\frac{mh}{n}\right)^{\frac{1}{2}}$$

$$\int_o^t \frac{(2g)^{\frac{1}{2}}}{r} \cdot dt = -\frac{2}{(mn)^{\frac{1}{2}}} \tan^{-1}\left[\left(\frac{mh}{n}\right)^{\frac{1}{2}}\right]_h^H$$

$$\frac{(2g)^{\frac{1}{2}}}{r} \cdot t = \frac{2^{\frac{1}{2}}}{(mn)^{\frac{1}{2}}} \tan^{-1}\left(\frac{mH}{n}\right) - \tan^{-1}\left(\frac{mh}{n}\right) \quad \ldots \quad \ldots \quad (5.7)$$

Proceeding to the limit m, n → o,

$$k_b = \frac{1}{(2g)^{\frac{1}{2}} t} (H^{\frac{1}{2}} - h^{\frac{1}{2}})$$

$$= \frac{1}{(2g)^{\frac{1}{2}} t} \left[H^{\frac{1}{2}} - (H - s)^{\frac{1}{2}} \right] \quad \ldots \quad \ldots \quad (5.8)$$

$$k_s = \frac{3r}{2(2g)^{\frac{1}{2}} t} \left(\frac{1}{h^{\frac{1}{2}}} - \frac{1}{H^{\frac{1}{2}}} \right)$$

$$= \frac{3r}{2(2g)^{\frac{1}{2}} t} \left[\frac{1}{(H - s)^{\frac{1}{2}}} - \frac{1}{H^{\frac{1}{2}}} \right] \quad \ldots \quad (5.9)$$

Rearranging terms in both 5.8 and 5.9,

$$s = H - \left[H^{\frac{1}{2}} - \tfrac{1}{2} (2g)^{\frac{1}{2}} k_b t \right]^2 \quad \ldots \quad \ldots \quad \ldots \quad (5.10)$$

$$s = H - \frac{1}{\left[\frac{2}{3} \frac{(2g)^{\frac{1}{2}}}{r} k_s t + \frac{1}{H^{\frac{1}{2}}} \right]^2} \quad \ldots \quad \ldots \quad (5.11)$$

Equation (5.10) can be used when the escape of water is limited to the bottom while equation (5.11) can be used when the escape of water is limited to the sides.

During the first few of the tests reported in the next Chapter the following shortcomings in the method of doing percolation tests in unlined holes in the sandy-laterite soils of Lagos were observed:

(a) it was difficult to dig holes to true circular dimensions;

(b) the sides tended to cave in;

(c) erosion occurred at the sides, altering the cross-sectional area of the hole, and adversely affecting the results;

(d) a certain amount of fines were deposited at the bottom.

To overcome or minimise these difficulties the Senator Taft's Sanitary Engineering Center's method was modified in a number of the tests reported in the next Chapter. This modification consisted in lining test holes with everite pipes in some tests, galvanised iron pipes in some others and expanded metal in some others yet as described in that chapter. This arrangement eliminated or reduced the erosion as well as the caving in of the sides. Lining would however interfere with the discharge through the sides, which could be eliminated completely if the sides are completely sealed off.

This modification however introduces a new problem. The relationship between the allowable rate of sewage application to soils and the percolation rate shown in equations 5.2 and 5.3 as well as in Figs. 5.1 and 5.2 was established in respect of unlined holes in which water discharged from both the bottom and the sides. This established relationship cannot be applied to a lined hole without first establishing another relationship between the discharge through the bottom only and the discharge through both the bottom and the sides. Two of the tests reported in the next chapter attempt to establish this relationship for a laterite soil in Lagos, and two others for a sandy laterite soil.

Both k_b and k_s generally decrease with time t. The need to compute the appropriate value of k_b or k_s at each value of t before the value of s can be computed makes the application of equations

5.10 and 5.11 difficult. An inspection of the curves of s against t in Figs. 6.2 and 6.3 indicate that s and t are related in the mathematical model:

$$s = pt^q \quad \ldots \quad \ldots \quad \ldots \qquad (5.12)$$

where p and q are constants.

$$\log s = \log p + q \log t \quad \ldots \quad \ldots \quad \ldots \qquad (5.13)$$

Equation (5.13 is a straight line with gradient q and an intercept of $\log p$ on the y axis. Both p and q can be determined graphically, more accurately by the method of linear regression. An electronic calculator made the method reasonably quick in the present work.

An additional advantage of the model in equation 5.12, it was discovered, was its applicability not only to lined holes but to unlined holes as well. Results of the application of this equation gave values of s invariably within 5% of the actual measured values.

CHAPTER VI

PERCOLATION TESTS IN SOME LAGOS SOILS

6.1 The object of these tests was to study the rate at which water percolated through some Lagos soils, and to obtain some indication of modifications that might be necessary in the method for determining the percolation rate developed at the Senator Taft's Sanitary Engineering Center mentioned in the last Chapter. The tests were performed in holes in natural soil as well as in made soil.

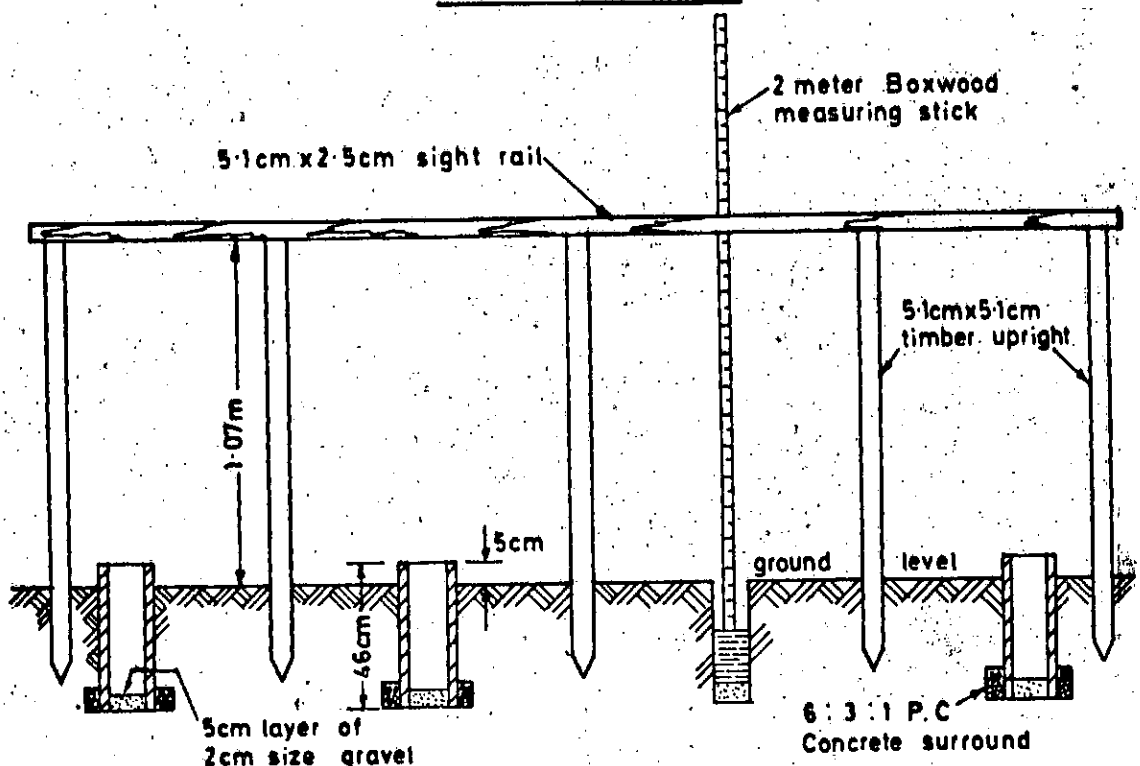
6.2 TESTS IN NATURAL SOIL

6.2.1 MATERIALS AND METHOD:

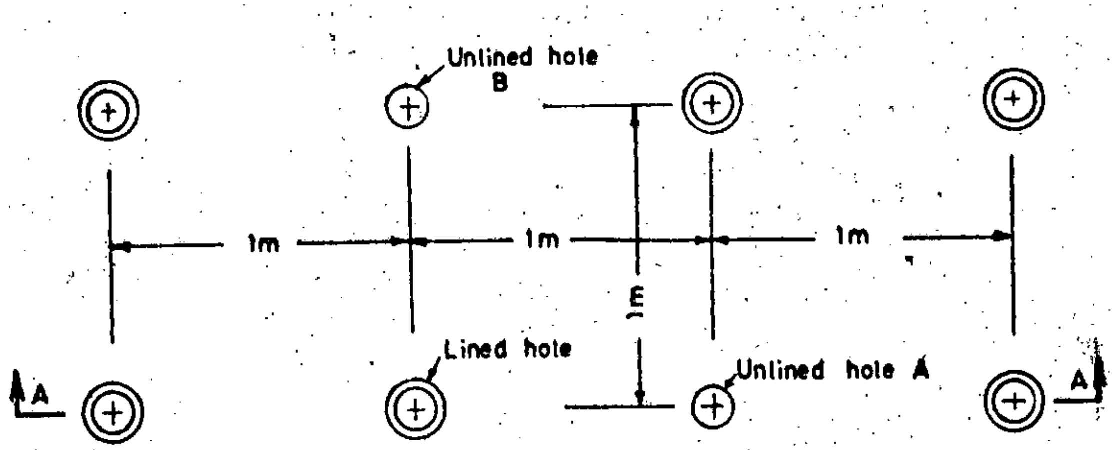
Three, and sometimes four holes were dug in a row at 1m centres at each location. Each hole was dug with a 10.2cm (4 ins) auger to a depth of 46cm (18 ins). Samples of the soil were taken from this depth for subsequent sieve analysis for grain size determination. A light sight rail consisting of a 5.1cm x 2.5cm (2"x1") timber horizontal nailed across a pair of timber uprights 7.6cm x 5.1cm (3"x2") was erected at a height of 1.07m above ground level as shown in Fig. 6.1. A 7.6cm layer of 2cm dia. size clean gravel was placed carefully in each hole to prevent erosion at the bottom of the soil while the hole was being filled with water.

A day before the tests described in both

FIG.6·1 FIELD ARRANGEMENT FOR PERCOLATION TESTS IN NATURAL SOIL (UNILAG SITE 'A'



ELEVATION A–A



LAYOUT OF HOLES

6.2.2 and 6.2.3 each hole was kept continuously wet for a period of 4 hours by refilling each time that the level of the water had dropped to the level of the layer of gravel at the bottom. This ensured that the swelling characteristic of soils with clay occurred before the test was started and that the soil approached the state it would attain under a functioning soakaway.

The test after the preliminary wetting consisted in measuring the distance of the water level in each hole below the upper edge of the horizontal rail at intervals of 5 minutes in Tests 6.2.2 and 6.2.3, and 3 minutes in Tests 6.2.5.

6.2.2 TESTS AT A BUILDING CONSTRUCTION SITE AT IGBOBI:

The first set of tests were performed in 3 holes of dimensions and spacing as described above and as shown in Fig. 6.1 at the site of the proposed soakaway for a pair of duplex houses then under construction at Igbobi on the Lagos Mainland. The soil was sandy-clay and appeared suitable for a soakaway. In addition to the preliminary wetting of the holes a day before the tests on February 26th and 27th 1976, there was heavy rain on the night of 25th - 26th February so that the soil was thoroughly wet for the first day of testing.

It was observed in these tests on both days that while the layer of gravel at the bottom of each hole prevented erosion at the bottom there was erosion at the sides as shown by a layer of fine material deposited on top of the gravel layer. The results are shown in Table 6.1 and Fig. 6.2.

6.2.3 FIRST TESTS AT UNIVERSITY OF LAGOS SITE A:

In the first of the set of tests at the University of Lagos site 8 holes, 4 in each of 2 rows spaced 1m apart were dug, prepared and tested in the manner already described. The layout of the holes and arrangement for measuring are as shown in Fig. 6.1.

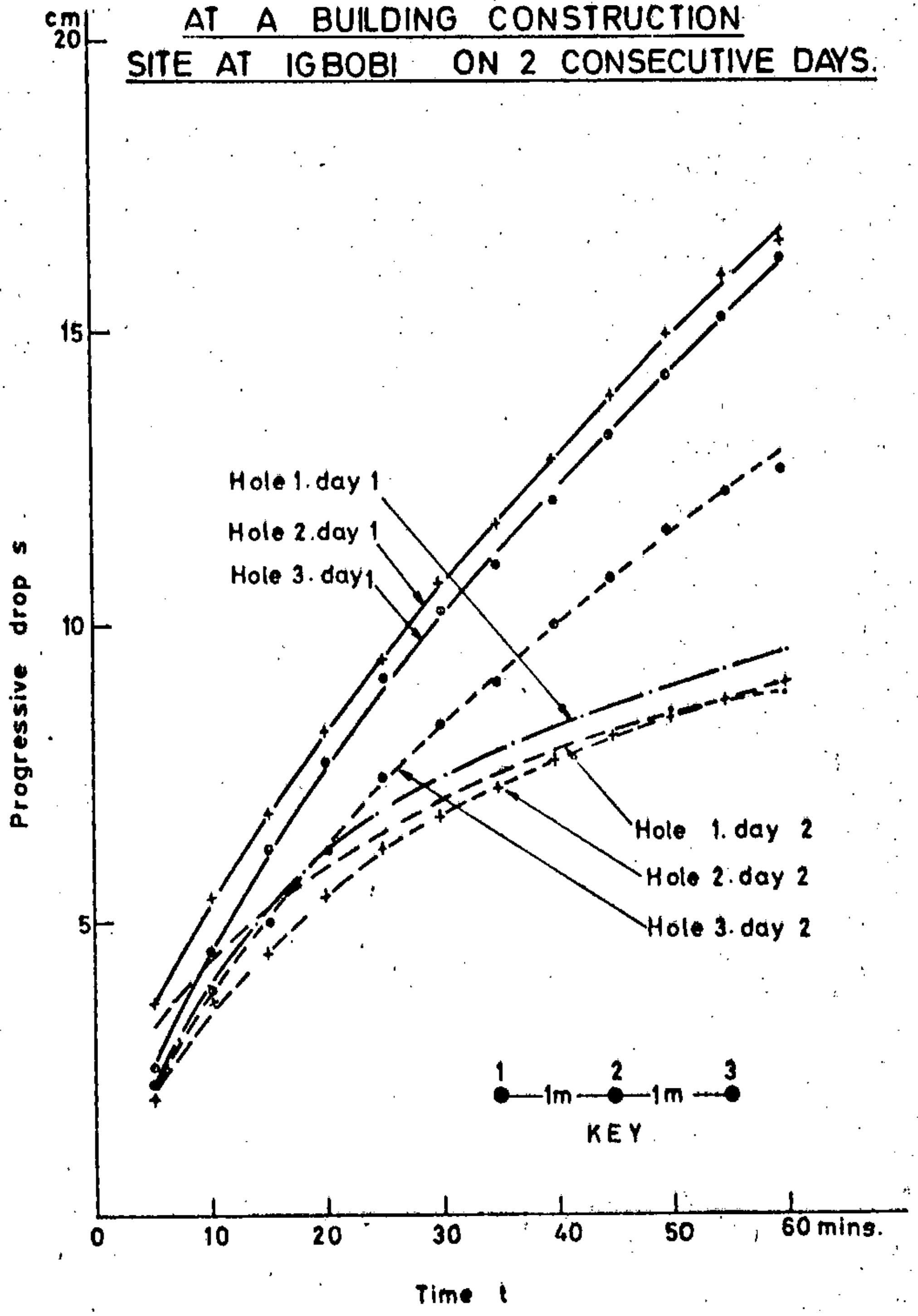
It was observed in these tests that the water level in all eight holes dropped faster than had been anticipated, all the water disappearing in a question of minutes in three of the holes. It was also observed that as in the tests at Igbobi there was erosion of the sides as seen from a layer of fine material on top of the gravel layer after each test. It was therefore decided to line the sides of the holes which would prevent erosion. This might also reduce the rate at which the water escaped from the hole since the sides would be sealed off and water escape

TABLE 6.1: RESULTS OF PERCOLATION TESTS IN UNLINED HOLES AT BUILDING CONSTRUCTION SITE, IGBOBI ON 26.2.76 AND 27.2.76

DAY	26-2.76						27.2.76					
Hole No.	1		2		3		1		2		3	
t	Δs	s	Δs	s	Δs	s	Δs	s	Δs	s	Δs	s
Min.	cm						cm					
5	2.3	2.3	3.6	2.6	2.5	2.5	3.1	3.1	2.0	2.0	2.2	2.2
10	1.6	3.9	1.8	5.4	2.0	4.5	1.3	4.4	1.6	3.6	1.6	3.8
15	1.4	5.3	1.4	6.8	1.7	6.2	.8	5.2	.8	4.4	1.2	5.0
20	.9	6.2	1.4	8.2	1.5	7.7	.7	5.9	1.0	5.4	1.2	6.2
25	.7	6.9	1.2	9.4	1.4	9.1	.5	6.4	.8	6.2	1.2	7.4
30	.5	7.4	1.3	10.7	1.1	10.2	.6	7.0	.5	6.7	.9	8.3
35	.5	7.9	1.0	11.7	.8	11.0	.5	7.5	.5	7.2	.7	9.0
40	.3	8.2	1.1	12.8	1.1	12.1	.3	7.8	.5	7.7	1.0	10.0
45	.3	8.5	1.1	13.9	1.1	13.2	.4	8.2	.4	8.1	.8	10.8
50	.5	9.0	1.0	14.9	1.0	14.2	.3	8.5	.3	8.4	.8	11.6
55	.3	9.3	1.0	15.9	1.0	15.2	.2	8.7	.3	8.7	.6	12.2
60	.2	9.5	.6	16.5	1.0	16.2	.1	8.8	.3	9.0	.4	12.6

Day	26.2.76			27.2.76		
Hole No.	1	2	3	1	2	3
Drop-in final 10 mins (cm)	0.5	1.6	2.0	0.3	0.6	1.0
Percolation Rate (mins/2.54cm)	50.80	15.88	12.70	84.67	42.33	25.4

FIG. 6·2 PROFILE OF PROGRESSIVE DROP S IN 3 ADJACENT UNLINED HOLES AT A BUILDING CONSTRUCTION SITE AT IGBOBI ON 2 CONSECUTIVE DAYS.



limited to the bottom.

For this purpose a 51cm length of 10.2 cm dia everite pipe was buried vertically in each of 3 out of the 4 holes in each row. The pipe was held in position with a 6:3:1 Portland Cement concrete sorround. Special care was taken to see that no concrete got inside the area enclosed by the pipe as such concrete on hardening would seal off part of the bottom soil area available for seeping away the water. The fourth hole that was left unlined in each case was the one in which the seeping away of the water had been slowest during the first two tests.

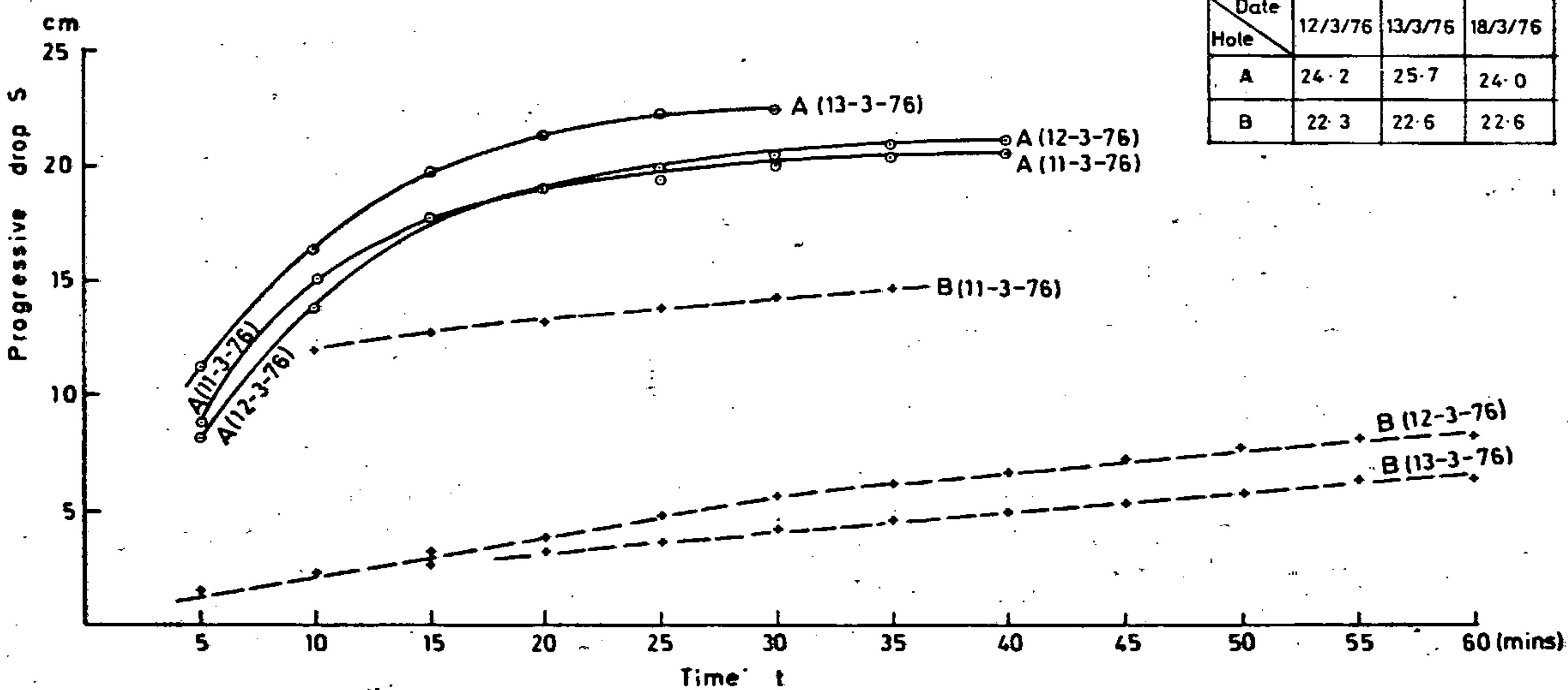
It was observed that lining appeared to increase the rate of escape of water, which seeped away within minutes in all the lined holes. This observation was confirmed all the three times the tests were done, when the water in the lined holes disappeared too fast for measurements to be taken in them. It was therefore decided to confine subsequent tests to the two unlined holes; one in each row, which were subsequently designated A and B.

The results are shown in Fig. 6.3.

6.2.4 MORE TESTS AT LAGOS UNIVERSITY SITE A:

These tests were performed to study the effect of repetition of tests in a hole and to compare the rate of water

FIG. 6·3 PROFILE OF PROGRESSIVE DROP S IN 2 ADJACENT UNLINED HOLES IN NATURAL SOIL AT UNILAG SITE A ON 3 CONSECUTIVE DAYS



Original head H (cm)			
Hole \ Date	12/3/76	13/3/76	18/3/76
A	24·2	25·7	24·0
B	22·3	22·6	22·6

percolation through the sides with the rate through the bottom.

The tests were performed in another three holes again spaced 1m apart in a row 1m from the row containing hole B in Fig. 6.1. All three holes were dug to the same dimensions and prepared in the same manner as the holes in Test 6.2.3. The two end holes 1 and 3 were lined with 7.6cm dia. everite pipes while the middle hole 2 was left unlined. After the usual preliminary wetting of the holes 2 sixty minutes runs were done in each hole on 4 consecutive days 20th, 21st, 22nd and 23rd April 1976, the depth of the water below the reference level being measured at 5 minutes intervals in each case. It is noted that the water in these tests lasted the full 60 minutes of each test.

At the beginning of each test the level of the water was adjusted to an original fixed distance below the reference point at the upper edge of the horizontal rail. This arrangement ensured that in all the 6 tests in a particular hole the original head was the same. This offered a good basis for comparison of the percolation rate from test to test in each hole.

On the 4th day of the test four and not two runs were made in the unlined hole 2.

While the first and the third of the tests were done in the normal way, a 7.6cm dia. everite pipe 30cm long was inserted in the hole for the second and the fourth tests, thus converting this into a lined hole for those two tests.

The results in respect of the unlined hole 2 are shown in Table 6.2.

6.2.5 <u>TESTS AT LAGOS UNVERSITY SITE B</u>:

The results of the tests in 6.2.4 shown in Table 6.2 and discussed later in this Chapter appeared to indicate that the lined hole was slower than the unlined hole in natural soil, contrary to the observation already reported in 6.2.3. It was therefore decided to do more tests in natural soil at another site in the University to resolve these conflicting observations..

The new site was in the foreground of the University Power House, opposite the Bookshop. Here two sets of three holes were prepared exactly as described in 6.2.1. The preliminary wetting of the holes was not done in these tests to prevent premature clogging at the bottom of the holes.

Eight tests each of 12 minutes duration were done in each of the three holes in the first series of tests. The

TABLE 6.2: RESULTS OF PERCOLATION TESTS IN UNLINED HOLE 2 AT UNILAG SITE ON 4 CONSECUTIVE DAYS

	20.4.76				21.4.76				22.4.76				23.4.76							
Test No.	1		2		1		2		1		2		1		2*		3		4*	
Time (mins)	Δs	s	Δs	s	Δs	s	Δs	s	Δs	s	Δs	s	Δs	s	Δs	s	Δs	s	Δs	s
	cm																			
5	2.5	2.5	3.3	3.3	3.8	1.8	1.8	1.2	1.7	1.7	1.6	1.6	1.4	1.4	.1	.1	1.8	1.8	-	-
10	1.6	4.1	1.7	5.0	1.3	3.1	1.1	2.3	1.5	3.2	1.2	2.8	1.1	2.5	-	.1	1.1	2.9	-	-
15	1.1	5.2	1.4	6.4	1.1	4.2	1.1	3.4	.9	4.1	1.1	3.9	.9	3.4	-	.1	.8	3.7	-	-
20	.9	6.1	1.2	7.6	1.1	5.3	.6	4.0	.8	4.9	.8	4.7	.7	4.1	-	.1	.5	4.2	-	-
25	.8	6.9	.9	8.5	1.0	6.3	1.2	5.2	.7	5.6	.7	5.4	.8	4.9	-	.1	.6	4.8	-	-
30	.7	7.6	.9	9.4	1.0	7.3	.4	5.6	.8	6.4	.7	6.1	.6	5.5	-	.1	.5	5.3	-	-
35	.6	8.2	.7	10.1	1.0	8.3	.6	6.2	.9	7.3	.5	6.6	.6	6.1	-	.1	.3	5.6	-	-
40	.6	8.8	.7	10.8	.7	9.0	.6	6.8	.7	8.0	.7	7.3	.7	6.8	.1	.2	.5	6.1	.1	.1
45	.6	9.4	.6	11.4	.6	9.6	.6	7.4	.6	8.6	.7	8.0	.5	7.3	-	.2	.4	6.5	-	.1
50	.6	10.0	.6	12.0	.5	10.1	.6	8.0	.6	9.2	.7	8.7	.5	7.8	-	.2	.5	7.0	-	.1
55	.6	10.6	.6	12.6	.5	10.6	.6	8.6	.5	9.7	.6	9.3	.5	8.3	-	.2	.5	7.5	-	.1
60	.5	11.1	.6	13.2	.5	11.1	.6	9.2	.2	9.9	.4	9.7	.5	8.8	-	.2	.3	7.8	-	.1
Percolation Rate (mins)	23.09		21.17		20.05		21.17		21.77		21.17		23.09		381.00		30.48		762.00	

NOTES: (i) –Δs = drop in water level in 5 minutes.
(ii) s = progressive drop in water level
(iii) Original head H = 22.7cm
* In runs 2 and 4 on 23/4/76 hole was lined.

measurements were taken at 3 minutes intervals in each test, which thus gave four measurements of the progressive drop in each test. Immediately after the fourth measurement in each test more water was added to the hole to bring the level to the starting point for the next test which was begun immediately after. This ensured that throughout the approximately 2 hours occupied by the eight tests the percolation of water in the hole was continuous. The starting head of water was kept constant for all eight tests in each hole.

Another modification introduced here was the lining of each hole in two out of the eight tests. This was done by carefully placing in the hole a cylindrical shell made from a 22 gauge galvanised iron sheet. It was 25.10cm long. Its external diameter of 9.86cm just fitted into the 10.16cm diameter hole without the shell damaging the sides of the hole during placing and withdrawal. It was provided with handles rivetted to a 7.6cm wide flange of the same material, which facilitated such placing and withdrawal before and after each test.

The first hole was lined in the third and sixth tests, the second hole in the fourth and seventh tests, and the third hole in the fifth and eighth tests as indicated in Tables 6.3 and 6.5.

Experience in preliminary tests with this method indicated that the best results were obtained when the lower edge of the lining shell did not penetrate into the soil at the bottom of the hole, but remained about 5mm above the bottom.

The eight tests in the second series were done 3 days after and lasted only 9 minutes each instead of the 12 minutes in the first series. The holes were located some 10m from the holes in the first series. The results are shown in Tables 6.3 - 6.6.

6.2.6 RESULTS AND DISCUSSION OF TESTS IN NATURAL SOIL:

Fig. 6.2 shows significant differences in the rate at which water disappeared in holes that were spaced only 1m apart. While the performance of hole 1 in Fig. 6.2 was practically the same on both occasions of the test, hole 2 was about twice as 'fast' on the first day as on the second.

Fig. 6.3 shows the results of tests in holes A and B on 12/3/76, 13/3/76 and 18/3/76 at University site A. It also shows as inset the original head H of the water in each hole on each of the three occasions.

TABLE 6.3: RESULTS OF PERCOLATION TESTS IN NATURAL SOIL AT UNIVERSITY SITE B-1

Hole No.	Test No.	Progressive falls 3min	6min	9min	12min	Average time to fall 1cm in 12-min test	Remarks
		CM				min	
1	1	4.8	8.3	10.	12.8	0.94	* indicates tests in which hole was lined
	2	2.4	4.6	6.5	7.9	1.52	
	3*	2.7	4.8	6.7	8.1	1.48	
	4	1.5	3.1	4.5	5.7	2.11	
	5	1.6	2.9	4.3	5.6	2.14	
	6*	2.0	3.5	4.8	6.0	2.00	
	7	1.5	2.8	3.9	4.8	2.50	
	8	1.4	2.5	3.7	4.8	2.50	
2	1	7.6	12.1	15.0	16.9	0.71	
	2	4.3	7.4	9.9	11.8	1.02	
	3	3.1	5.3	7.3	8.9	1.35	
	4*	4.4	7.0	8.4	9.7	1.24	
	5	2.7	4.8	6.4	7.3	1.65	
	6	2.4	4.1	5.5	6.6	1.82	
	7*	2.2	4.1	5.7	7.7	1.56	
	8	2.0	3.8	5.2	6.4	1.88	
3	1	?	-	-	-	-	Hole 3 was too fast for measurements to be taken after 2 mins in most of the tests.
	2	14.1	-	-	-	-	
	3	13.7	-	-	-	-	
	4	14.6	-	-	-	-	
	5*	13.2	-	-	-	-	
	6	8.1	12.2	16.7	-	-	
	7	6.3	11.1	13.8	15.1	0.80	
	8*	4.8	7.9	10.2	12.1	0.99	

TABLE 6.4: CALCULATIONS FOR RATIO OF PERCOLATION RATES IN LINED AND UNLINED HOLES AT UNIVERSITY SITE B-1

Hole No.	Test No. n	Average time t to fall 1cm Measured t	Calculated t^1 (min)	% difference	Ratio $\frac{t}{t^1}$	Remarks
1	2	3	4	5	6	
1	1	0.94	1.000	-6.38		
	2	1.52	1.400	7.90		* indicates test in which hole was lined.
	3*	1.48	1.704	-	.87	
	4	2.11	1.959	7.16		
	5	2.14	2.183	-2.01		
	6*	2.00	2.385	-	.84	
	7	2.50	2.570	-2.80		
	8	2.50	2.742	-9.68		
2	1	0.71	0.711	-0.14		
	2	1.02	1.011	0.88		
	3	1.35	1.243	7.93		
	4*	1.24	1.438	-	.86	
	5	1.65	1.611	2.36		
	6	1.82	1.767	2.91		
	7*	1.56	1.911	-	.82	
	8	1.88	2.045	-8.78		

NOTES:

(1) t^1 in column 4 is calculated from the model $t^1 = pn^m$ developed from regression of log t on log n.
For hole 1, $t^1 = n^{.485}$
For hole 2, $t^1 = 0.711n^{.508}$

(2) Ratio t/t^1 in col. 6 is obtained by dividing measured time t in lined hole by calculated t^1 in unlined hole, e.g. test 4 in which hole 2 was lined t = 1.24 min, while t^1 for unlined hole is calculated to be 1.438 min. Ratio = 1.24 ÷ 1.438 = .86.

TABLE 6.5: RESULTS OF PERCOLATION TESTS IN NATURAL SOIL AT UNIVERSITY SITE B-2

Hole No.	Test No.	Progressive fall s 3 min	6 min cm	9 min	Average time to fall 1cm in 9-min test min	Remarks
1	1	8.2	14.5	19.3	0.47	* indicates tests in which hole was lined.
	2	7.5	12.7	17.0	0.53	
	3*	6.3				
	4	6.1	10.4	14.3	0.63	
	5	6.0	9.9	13.7	0.66	
	6*	5.2	9.5	13.2	0.68	
	7	4.7	8.8	12.1	0.74	
	8	5.0	8.9	12.2	0.74	
2	1	7.3	11.7	14.8	0.61	
	2	5.7	10.0	12.9	0.70	
	3	6.7	9.2	12.3	0.73	
	4*	5.2	9.0	12.1	0.74	
	5	4.7	8.4	11.2	0.80	
	6	4.7	8.4	11.1	0.81	
	7*	5.1	8.5	11.4	0.79	
	8	4.6	7.9	10.7	0.84	
3	1	5.5	9.0	11.8	0.76	
	2	4.7	7.8	10.7	0.84	
	3	3.9	7.1	9.6	0.94	
	4	3.7	6.6	9.1	0.99	
	5*	3.9	6.6	9.1	0.99	
	6	3.8	6.4	8.5	1.06	
	7	3.3	6.2	8.2	1.10	
	8*	3.3	5.8	8.0	1.13	

1.41m apart (Fig. 6.1) there are significant differences between the profiles of the progressive drops in the holes on the three occasions. The water in A did not lsst the 60 minutes test on any of the three occasions of the test while the water in B lasted 60 minutes twice. While A started falling on each occasion with a steep gradient which decreased rapidly till it became quite small at the point where the hole ran dry, B fell with a uniformly small gradient throughout each run. Whereas the overall rate of drop of level of water increased slightly with each test in A it decreased with each test in B. The slight differences in H shown in the inset table in Fig. 6.3 cannot account for these substantial differences in the behaviour of the water in the two holes.

Table 6.2 which shows the results of test 6.2.4 in unlined hole 2 also shows that while s varied appreciably from test to test and from day to day at the beginning of each test, the drop in the last 30 minutes of each test varied little in the first three days, giving percolation rate values between 20.1 and 21.8 minutes per 2.54cm.

The more significant revelation in Table 6.2 however was that the bottom of hole 2 was nearly completely impervious on 23/4/76 as shown in the results of the two runs 2 and 4 performed when the hole was lined. The results of the tests on that day appear to show therefore that the water escaped very much faster in the unlined hole than in the lined hole in the natural soil. This is different from the observation made in the unlined holes in Test 6.2.3 which became faster after being converted into lined holes. It is also contrary to the observation made in Tests 6.3.2 and 6.3.4 in made soil which are described later in this Chapter. The repetition of tests in the three holes in 6.2.4 in the previous three days however most probably resulted in clogging at the bottom of the holes due to the erosion of fine material from both the sides and the upper layer at the bottom of the hole, and the subsequent depositing of these in the pores in the soil at the bottom of the hole. This could reduce substantially the percolation capacity of the soil at the bottom of the hole.

Tables 6.3 - 6.6 however show that the water escaped faster in the lined hole than in the unlined hole, as already established

TABLE 6.6: CALCULATIONS FOR RATIO OF PERCOLATION RATES IN LINED AND UNLINED HOLES AT UNIVERSITY SITE B-2

Hole No.	Test No. n	Average time t to fall 1cm			Ratio $\frac{t^1}{t}$	Remarks
		Measured t (min)	Calculated t^1	%difference		
1	2	3	4	5	6	
1	1	0.47	0.466	0.96		* indicates tests in which was lined.
	2	0.53	0.543	-2.52		
	3*	0.58	0.595	-	.97	
	4	0.63	0.634	-0.67		
	5	0.66	0.667	-0.99		For hole 1
	6*	0.68	0.694	-	.98	$t^1 = .628n^{.145}$
	7	0.74	0.719	2.91		
	8	0.74	0.740	0.00		
2	1	0.61	0.628	-2.30		
	2	0.70	0.695	0.87		
	3	0.73	0.737	-0.91		For hole 2
	4*	0.74	0.768	-	.96	$t^1 = .628n^{.145}$
	5	0.80	0.793	0.84		
	6	0.81	0.815	-0.56		
	7*	0.79	0.833	-	.95	
	8	0.84	0.849	-1.09		
3	1	0.76	0.756	0.62		
	2	0.84	0.864	-2.86		For hole 3
	3	0.94	0.935	0.56		$t^1 = .755n^{.194}$
	4	0.99	0.988	0.17		
	5*	0.99	1.032	-	.96	
	6	1.06	1.069	-0.87		
	7	1.10	1.102	-0.15		
	8*	1.13	1.131	-	1.00	

See footnote under Table 6.4 for explanation of column 6.

in test 6.2.3. They also strengthen the assumption that the contrary observation in Table 6.2 was due to clogging at the bottom of the hole due to the several repetitions of the test in the same hole.

In both Tables 6.4 and 6.6 the ratio t/t^1 was calculated to find how much faster the lined hole was than the unlined hole. In Table 6.3 the values of t in column 3 are found to be related to the values of n in column 2 in the equation.

$$t = pn^m \quad \ldots\ldots\ (6.1)$$

Both the constant p and the slope m have been claculated by the regression of log t on log n. The equations for the two holes that functioned at Site B-1 and the three holes at Site B-2 are shown in Tables 6.4 and 6.6.

The value 1.48 for t in column 3 of Table 6.4 in Test 3 was excluded from the values of t used in calculating equation 6.1 for hole 1 in the tests at University Site B-1 as this value was known to be smaller than what it would have been if the hole had not been lined. Rather, the value 1.704 in column 4 shows the calculated value of the time t^1 if the hole had not been lined. The ratio 1.48 to 1.704 = 0.87 given in column 6 shows how much the lined hole 1 is faster than the unlined hole in Test 3.

In Table 6.4 the ratio t/t^1 varies from 0.82 to 0.87, with an average of 0.85. Similarly in Table 6.6 the ratio varies from 0.95 to 1.00 with an average of 0.97. The range is small in each case, which indicates a reasonable degree of consistency in the work. The difference of 14% (.97-.85) in the average of this ratio within a short distance of 10 metres is consistent with the known variation in soil properties within short distances, and from day to day.

It has been noted earlier that the best results for lined holes in Test 6.2.5 were obtained when the lower edge of the lining shell did not penetrate the soil at the bottom of the hole, but remained about 5mm above. In this position it cannot be said that the shell truly seals off the sides and that the escape of water was limited to the bottom only since some water under pressure could go round the lower edge of theshell and flow upward to fill the little space between the outer surface of the shell and the sides of the hole. Such water would then escape into the soil through the sides. It is considered therefore that results obtained from tests in holes lined this way cannot be used for calculating k_b in equation 5.8. The main advantages of lining should be seen as the prevention of erosion of material from the sides and the provision of support for the sides.

6.2.7 CALCULATIONS FOR PERCOLATION RATES IN NATURAL SOIL:

In the tests at Igbobi and at the University of Lagos Site A, most of the water seeped away in the holes in just 60 minutes. Therefore calculations for percolation rates should be made in respect of the drop in level in the last 10 minutes of each test according to the definition in Section 5.4.

IGBOBI SITE, HOLE 1 (26.2.76, LINED):

Drop in water level in last 10 mins. = 0.5 cm (Table 6.1).

Percolation Rate = time to drop 2.54cm

$$= 10 \times \frac{2.54}{.5} = \underline{50.80 \text{ mins.}}$$

Percolation Rates similarly obtained for all 3 holes on both days are as in table below:

Date	Hole No.	Drop in s in last 10 mins. (cm)	Percolation Rate (min/2.54cm)
26.2.76	1 2 3	0.5 1.6 2.0	50.80 15.88 12.70
27.2.76	1 2 3	0.3 0.6 1.0	84.67 42.33 25.4

UNIVERSITY SITE A, (20.4.76, UNLINED, HOLE 2):

Drop in water level in last 10 mins in Run 1

= 1.1 cm.

$$\text{Percolation Rate} = \frac{10 \times 2.54}{1.1} = 23.09 \text{ mins.}$$

Percolation Rates in this hole on the 4 days of Test 6.2.3 are as follows:

Day	Run	Drop in Last 10 Minutes (cm)	Percolation Rate (Min/2.54cm)	
1	1 2	1.1 1.2	23.09 21.17	
2	1 2	1.0 1.2	25.40 21.17	
3	1 2	0.7 1.0	36.29 25.40	
4	1 3	1.0 0.8	25.40 31.75	Results of runs 2, 4 ignored.

6.2.8 CALCULATIONS FOR FORMULA $s = pt^q$ IN RESPECT OF TEST 6.2.4:

From equation 5.13 (Chapter V),

$$\log s = \log p + q \log t.$$

The constants p and q can be found by the linear regression of log s upon log t, using the values of s against the appropriate values of t in a test. The equation for the test in hole 1 day 2 at the Igbobi building construction site was found to be:

$$s = 1.64t^{.42} \quad \ldots\ldots\ldots \quad (6.2)$$

using the values of s in Table 6.1. The equation for the test in unlined hole 2 at the University site A on 20.4.76 is:

$$s = 1.14t^{.56} \quad \ldots\ldots\ldots \quad (6.3)$$

using the values of s in Table 6.2. Table 6.7 shows the values of s calculated from the formula:

$$s = pt^q$$

compared with the measured values of s in both the Igbobi site hole and the University site hole under consideration. The % differences in the measured values and the calculated values are very small, in all cases under 5% showing that the mathematical model is good. To calculate the percolation rate from the formula $s = pt^q$ for the Igbobi site hole 1 on day 2:

$$s = 1.64t^{.42}$$

$$s_{60} = 1.64 \times 60^{.42} = 9.16$$

$$s_{50} = 1.64 \times 50^{.42} = 8.48$$

$$s_{60} - s_{50} = 9.16 - 8.48 = .68\text{cm}$$

$$\text{Percolation Rate} = \frac{10 \times 2.54}{.68} =$$

$$\underline{37.35 \text{ minutes}}$$

Similarly for the University site A hole,

$$s = 1.14t^{.56}$$

$$s_{60} = 1.14 \times 60^{.56} = 11.29$$

$$s_{50} = 1.14 \times 50^{.56} = 10.19$$

$$s_{60} - s_{50} = 11.29 - 10.19 = 1.10$$

$$\text{Percolation Rate} = \frac{10 \times 2.54}{1.10} =$$

$$\underline{23.09 \text{ minutes}}$$

The figure of 37.35 minutes obtained for the percolation rate in the Igbobi hole 1 day 2 by calculating $s_{60} - s_{50}$ from the relationship $s = pt^q$ is smaller than the figure of 84.67 minutes calculated from the drop in the water level actually observed in the last 10 minutes of the

TABLE 6.7: COMPARISON OF VALUES OF S ACTUALLY MEASURED with VALUES CALCULATED FROM FORMULA $s = pt^q$ IN TESTS 6.2.2 & 6.2.3

t	TEST 6.2.2 (IGBOBI)			TEST 6.2.3 (UNIVERSITY A)		
	s Measured (m)	s Calculated (m)	% Differences	s Measured (m)	s Calculated (m)	% Differences
5	3.10	3.23	-4.19	2.80	2.81	-0.36
10	4.40	4.33	1.82	4.10	4.14	-0.98
15	5.20	5.13	1.35	5.20	5.19	0.19
20	5.90	5.79	1.86	6.10	6.10	0.00
25	6.40	6.36	0.63	6.90	6.91	-0.14
30	7.00	6.87	1.86	7.60	7.66	-0.79
35	7.50	7.33	2.27	8.20	8.35	-1.83
40	7.80	7.50	3.85	8.80	9.00	-2.27
45	8.20	8.15	1.83	9.40	9.61	-2.23
50	8.50	8.52	-0.24	10.00	10.19	-1.90
55	8.70	8.87	-1.95	10.60	10.75	-1.42
60	8.80	9.20	-4.55	11.10	11.29	-1.71

Results fit well into mathematical models

(1) $s = 1.64t^{.42}$ .. for the Igbobi hole

(2) $s = 1.14t^{.56}$ .. for the University site A hole

test. The latter drop was only 0.3cm whereas the drop as calculated from the formula was 0.68 cm. It is considered that the figure obtained from the formula is likely to be more reliable than the other one as probable errors in the last two readings of the test have been distributed through all the 12 readings in developing the formula.

The calculations in section 6.2.7 for the percolation rates in the three holes at Igbobi show that on the first day the rates varied between 12.70 minutes (hole 3) and 50.80 minutes (hole 1), with an average of 26.46 minutes for the three. The average for the second day was 50.83 minutes. ~~holes~~. According to HOLLIS, M.D. (1963) soils with percolating rates above 30 minutes are not suitable for seepage pits and those with percolation rates above 60 minutes not suitable for any kind of leaching system. According to this criterion, the soil under consideration is not suitable for a seepage pit since the average percolation rate for the two days of the test was 38.65 minutes which is greater than 30 minutes.

Applying equation 5.3 (Chapter V) to these figures, the allowalbe rate of application of sewage effluent to the soil is obtained thus:

$$q = 204t^{-\frac{1}{2}} \text{ litres/m}^2/\text{d} \quad \ldots\ldots\ldots (5.3)$$

As average t for the first day is 26.46 minutes,

$$q = 204 \times (26.46)^{-\frac{1}{2}}$$

$$= 39.66 \text{ litres/m}^2/\text{d}.$$

On the second day when the three holes were slower still the average figure for t was 50.80

minutes, yielding an allowable sewage effluent rate q of 28.62 litres/m^2/d.

The building under construction is a pair of duplex houses in which two families of 8 persons each, producing 140 litres of sewage per capita could be expected to live.

Total sewage = 16 x 140 = 2240 litres/d.

Area of soakaway required = $\frac{2240}{34.14}$ = 65.61m^2.

The actual area provided by the developer is known to be less than this.

The average percolation rate calculated from the results of the six runs in unlined hole 2 at the University site A from 20.4.76 to 22.4.76 was 25.42 minutes, for which q was calculated to be 41 litres/m^2/d.

The smallest of the three rates of sewage application q provided for in Table 3.3 is 98 litres/m^2/d (2 gal/ft^2/d). This is 2.9 times and 2.4 times the values of q calculated above for the Igbobi site and the University of Lagos site A respectively. This shows that by the long used P.W.D. Standard Specifications these two sites are not suitable for soakaway installations.

6.3 TESTS IN MADE SOIL:

The results of tests in natural soil could be subject to the adverse effects of external factors like rain, evaporation and the movement of the water table. It was therefore decided to perform the next series of tests in made soil in the laboratory to eliminate these adverse effects.

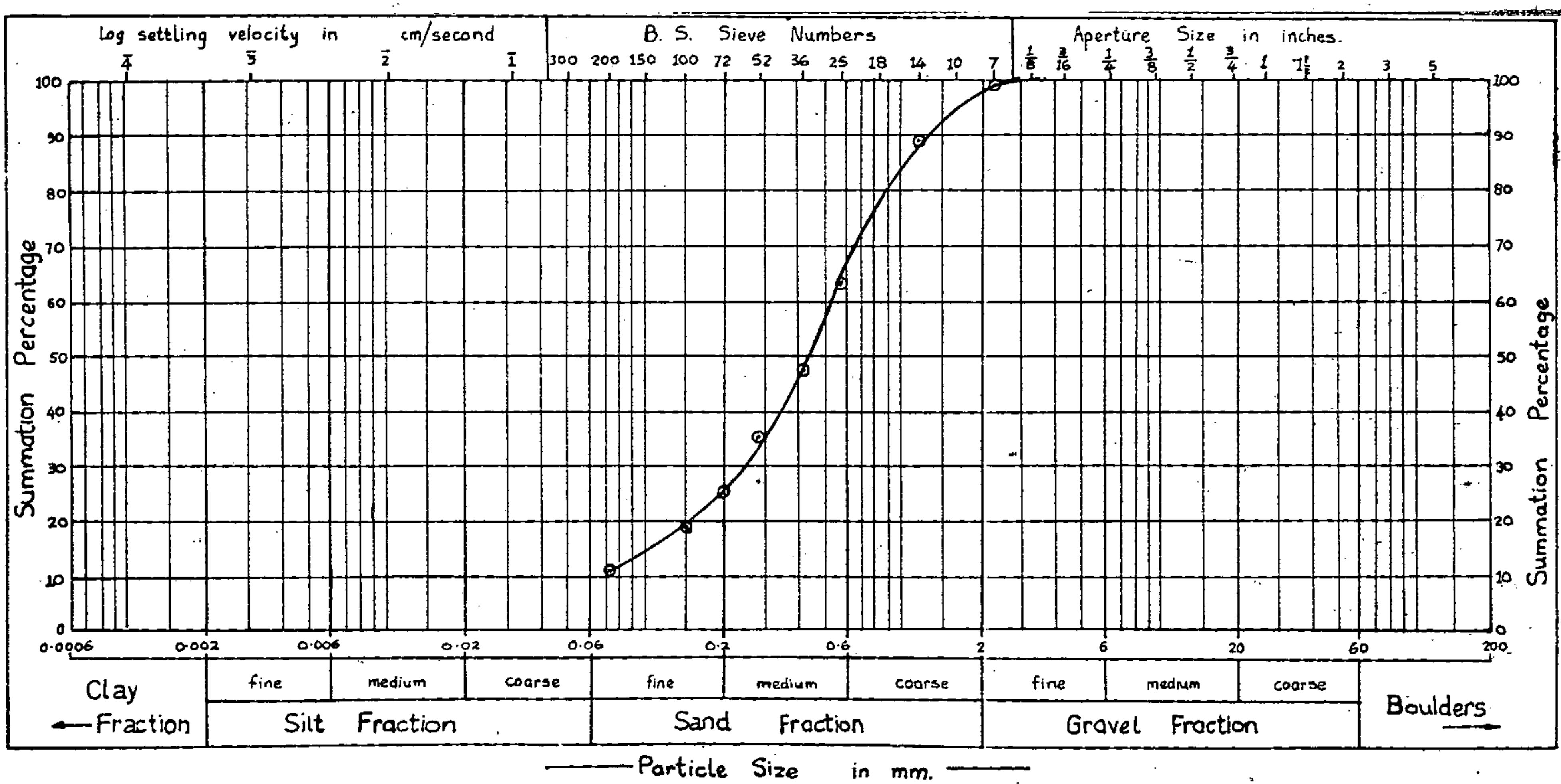
6.3.1 VARIATION OF THE TIME FOR WATER LEVEL TO FALL THROUGH FIXED DISTANCE WITH NUMBER OF TESTS:

The tests in natural soil show that the rate of increase of s (Fig.5.3) in equal time intervals (5 minutes in tests 6.2.2 - 6.2.4) decreases with time. It appears reasonable to assume from this that the time taken for all the water, starting each time from the same level, to completely disappear in a hole will increase with the number of times the test is performed in that hole. This assumption was verified in this test in the belief that it could be useful in establishing a relationship between the rates of escape of water in a lined hole and the rate in an unlined hole, as was done in the tests in natural soil.

The soil was selected from a 5 ton lorry load obtained from a bridge construction site at Mile 9, Ikorodu Road. It was a laterite soil with a grain size distribution shown in Fig. 6.4. It was first dried in the sun, then loaded in three layers into a timber box 46cm square section, 61cm deep and open at both ends. Each layer was consolidated with a poker vibrator before the addition of the next layer.

A hole 6.5cm dia. and 30cm deep was dug centrally in the consolidated soil. Into this hole was placed a cylindrical shell 6.35cm dia. and 30cm deep made of light expanded metal 12mm x 8mm x 22gauge. The purpose of this shell was to reduce the

FIG. 6·4 PARTICLE SIZE DISTRIBUTION OF SOIL AT BRIDGE CONSTRUCTION SITE, MILE 9 LAGOS-IKORODU ROAD



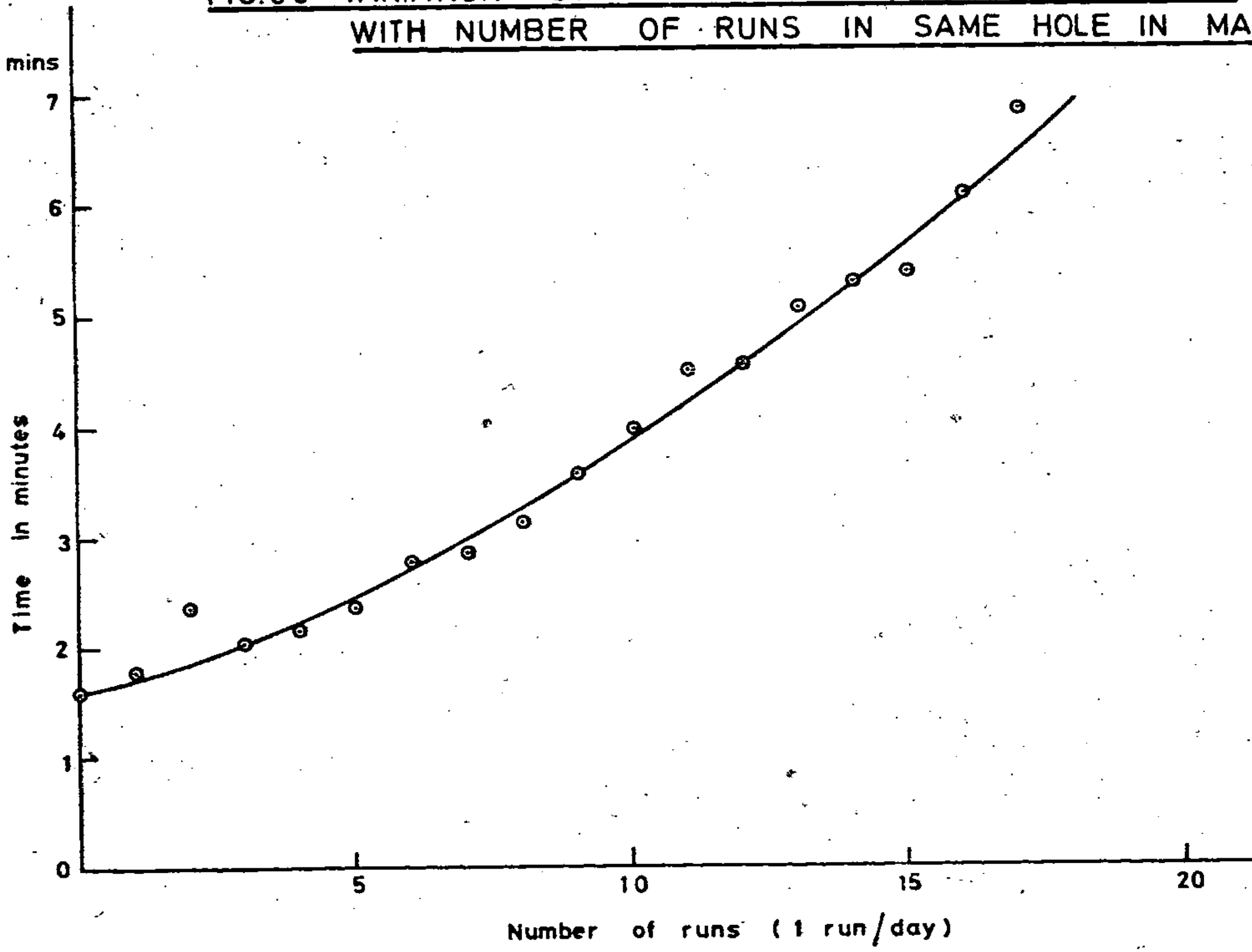
amount of erosion as well as support the sides of the hole during each test. To ensure that the soil was properly packed round the shell further consolidating of the last layer of the soil was done, this time with a short piece of 1.25cm square timber. Care was taken to see that little soil got into the hole through the meshes of the shell. As in the tests in the field a 5.1cm thick layer of gravel was placed at the bottom of the hole.

As some fine material was eroded from the sides and deposited at the bottom of the hole during each test there was a slight decrease in H (Fig. 5.3) each time the test was repeated even though the water level was adjusted to start from the same point at the beginning of the test. To take account of this therefore it was necessary to measure the depth of the bottom below the water level before and after each test.

The hole was carefully filled with water at the beginning of a test. A stop-watch was started at the instant the water level reached the top of the hole. The time it took the water to completely disappear from the hole was noted.

The test was performed once a day for a period of 17 days. The results are shown in Fig. 6.5.

FIG. 6·5 VARIATION OF TIME TO SEEP THROUGH 1 CM WITH NUMBER OF RUNS IN SAME HOLE IN MADE SOIL.



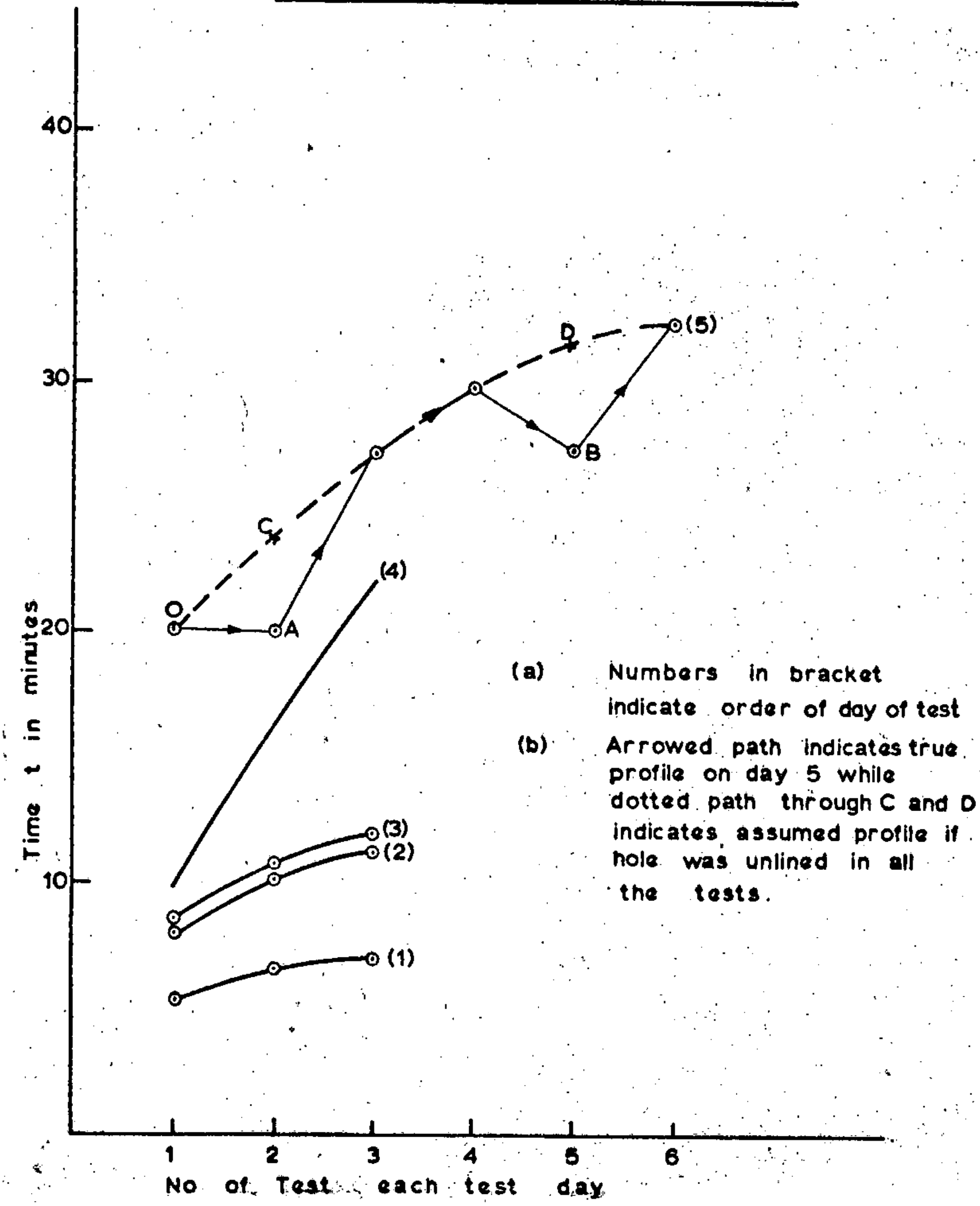
6.3.2 TESTS FOR COMPARING PERCOLATION RATES IN LINED AND UNLINED HOLES:

This is a variation of the last series of tests in another batch of the same laterite soil sample prepared, loaded and consolidated into the timber box as before. The expanded metal cylindrical shell was also placed in position as before.

The tests were run for five consecutive days. There were three tests on each of the first four days and six on the fifth day. Thus there were eighteen tests in all. The tests on each day were run immediately after each other. A preliminary observation of the results obtained in the first three days tests showed that the time taken for the water to completely disappear in the hole increased at a decreasing rate during the three tests each day (Fig. 6.6). This observation was again confirmed in the three tests on the fourth day. It was observed also that the time recorded at the first test each day was lower than the time recorded in the last test the day before, indicating some recovery in the percolation capacity of the soil during the rest hours.

Before the second of the six tests on the fifth day an inner cylindrical shell of galvanised iron, 6.25cm dia. and 30cm deep was slipped into the existing hole with XPM lining. This just fitted into the hole and

FIG.6.6 VARIATION OF TIME FOR WATER LEVEL TO FALL THROUGH FIXED DISTANCE WITH NUMBER OF TESTS IN SAME HOLE IN MADE SOIL EACH DAY (1)



thus formed a solid lining for this test. The hole was filled with water as usual and the time for the water to completely disappear noted.

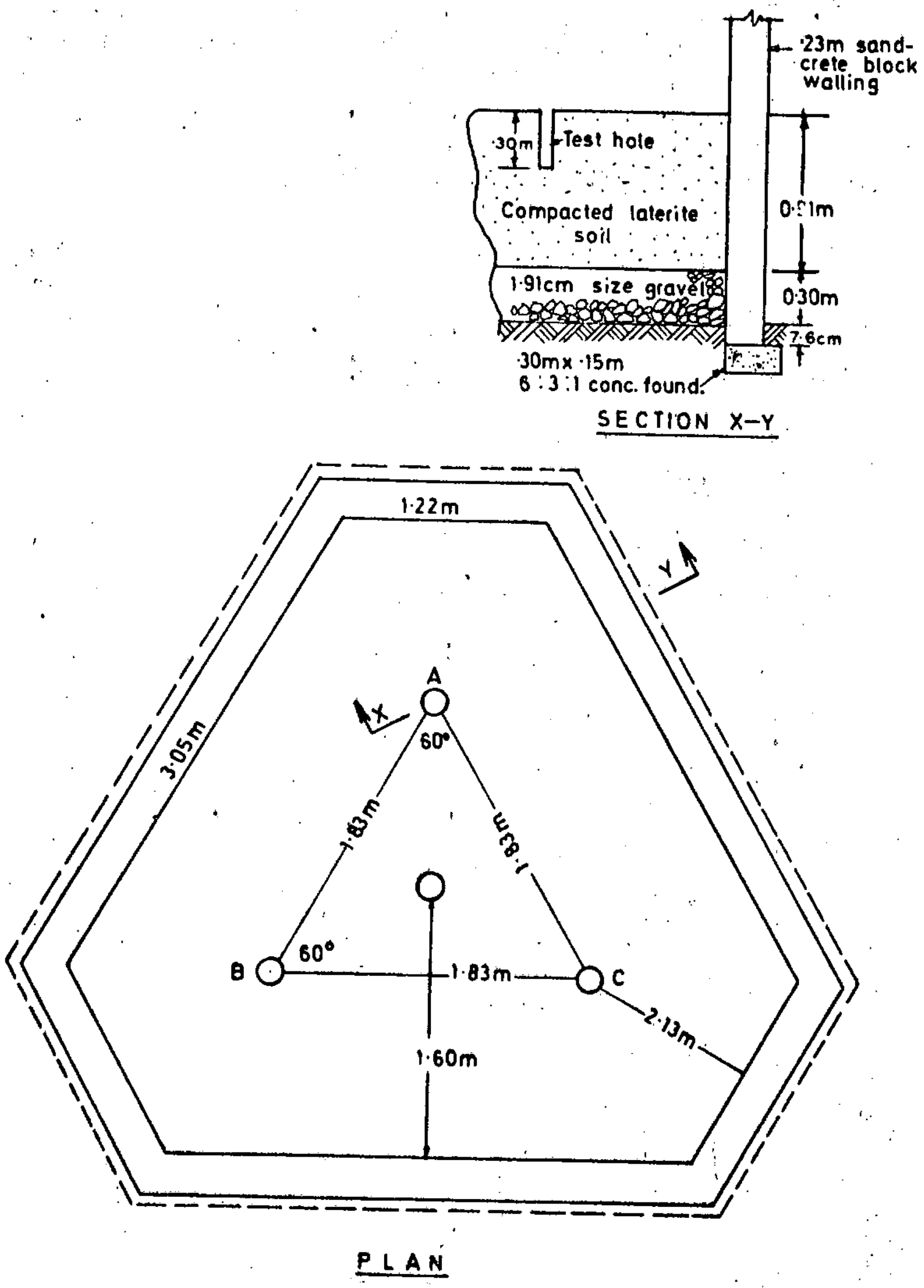
For the third test the inner shell was removed. This and the fourth tests were performed with only the outer shell of expanded metal in position. For the fifth test however the inner shell was again inserted, but for the sixth and last test it was removed. The hole therefore was lined for the second and fifth tests but unlined for the first, third, fourth and sixth tests on the last day of the tests. The results are shown in Fig. 6.6.

6.3.3 TESTS TO STUDY PERCOLATION RATES IN 3 EQUIDISTANT HOLES IN A TEST PIT:

This is a variation of each of the last two sets of tests. The soil used this time was from another batch from the bridge construction site at Mile 9 on Lagos Ikorodu Road.

The tests here were performed in a relatively large expanse of soil contained in a test pit with sides 22.9cm thick sand-crete block walls arranged to form an equilateral triangle in section as shown in Fig. 6.7. The natural ground below the pit was prepared by removing the top 7.6cm

FIG. 6·7 DETAILS OF PERCOLATION TEST-PIT IN MADE SOIL



of surface soil and levelling off. A 0.30m layer of 2cm size gravel was then spread over this to aid drainage into the natural soil. After this the laterite was loaded into the pit and consolidated manually with 45.7cm x 22.9cm x 22.9cm (18" x 9" x 9") hollow sandcrete blocks in four layers to a finished thickness of 0.91m above the gravel layer. The side walls were built up to an average height of 2.1m above the test soil surface and completed with a lean-to roof. There were two louvre windows, a door, and a sink, together with electricity and water. This was thus a little laboratory in which the test soil was completely protected from rain and sun, and the movement of ground water.

Three holes, each of the same dimensions and prepared in the same way as the hole in the last test were dug at the apexes of an equilateral triangle of sides 1.83m and centrally arranged as shown in Fig. 6.7. By this arrangement the three holes were equidistant from each other and the effect of the percolation of water in any of them on the other two, if any, would be the same. Furthermore, each hole was 1.07m from the walls of the pit as comapred with the 0.20m distance between the hole and the sides of the timber box in the tests in 6.3.2.

The present arrangement was aimed at eliminating any boundary effects on the percolating water. The tests were performed for ten consecutive days at a rate of one test per hole per day. The time taken for all the water to disappear in each hole each day was noted and the time per cm of depth computed. The results are shown in Table 6.8 and Fig. 6.8.

6.3.4 TESTS FOR COMPARING PERCOLATION RATES IN LINED AND UNLINED HOLES IN MADE SOIL:

The tests were practically the same as those in 6.3.2. They were however carried out in the test-pit used for the tests in 6.3.3. Here a fourth hole was dug at the centre-of-area of the equilateral triangle to the usual dimensions and prepared in the usual way.

The tests were performed in this central hole on four consecutive days. There were eight on the first day, five on the second, five on the third and four on the fourth day. The first five tests on the first day were performed with only the expanded metal outer shell in position in the hole. For the sixth test the inner shell of the galvanised iron was placed in the hole thus providing a solid lining for this test. The seventh and eight tests that day were done with only the outer shell of the expanded metal in position. On the second day the outer shell only was used

TABLE 6.8: TIME TAKEN FOR WATER TO COMPLETELY DISAPPEAR IN EQUIDISTANT HOLES IN TEST-PIT

DAY	A	B	C
	T i	m	e
	Mins/cm	Mins/cm	Mins/cm
1	0.070	0.147	.072
2	0.137	0.260	0.167
3	0.140	0.347	0.583
4	0.139	0.365	0.982
5	0.167	0.585	1.499
6	0.205	0.721	1.073
7	0.349	0.800	1.377
8	0.386	1.096	1.785
9	0.401	1.280	2.111
10	0.477	1.556	2.712

NOTES:

(a) A, B and C are equidistant holes at apexes of an equilateral triangle with sides 1.83m long.

(b) Water in hole A was more than 3 times and nearly 6 times as fast as water in B and C respectively on Day 10.

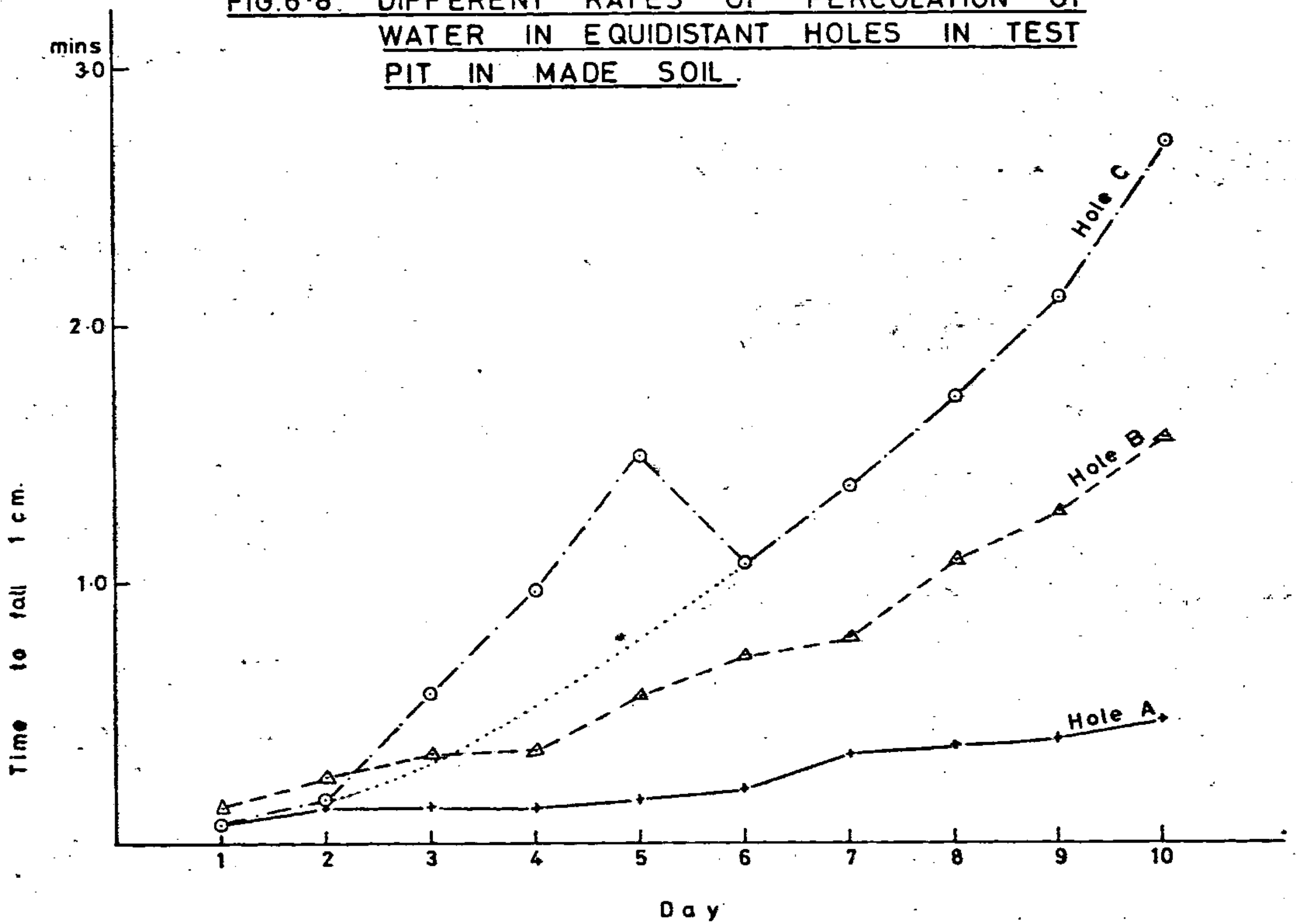


FIG.6·8. DIFFERENT RATES OF PERCOLATION OF WATER IN EQUIDISTANT HOLES IN TEST PIT IN MADE SOIL.

for all the five tests except the third in which the inner shell was used. Similarly the inner shell was used in the third test on the third day as well as in the second test on the fourth day, while the outer shell only was used in all the other tests. The results are shown in Table 6.9 and Fig. 6.9.

6.3.5 RESULTS AND DISCUSSION OF TESTS IN MADE SOIL:

The results of Test 6.3.1 shown in Fig. 6.5, confirm the assumption that the time taken by the water level to fall through a fixed distance in a hole increases with the number of times the test is performed in the hole.

From the way t increased at a decreasing rate with the order of test n in the first 4 curves in Fig. 6.6 it was assumed that the curve of t against n on the fifth day would have followed a similar pattern if all the 6 tests performed that day had been done in an unlined hole. On this assumption, a smooth curve has been drawn passing through the 1st, 3rd, 4th and 6th points. These 4 points were the actual time measurements made during each of the tests. It is assumed that C and D on the curve, vertically above A and B respectively, would have been the measurements of t if the 2nd and the 5th tests had been done in an unlined hole as in the other tests that day. However, the

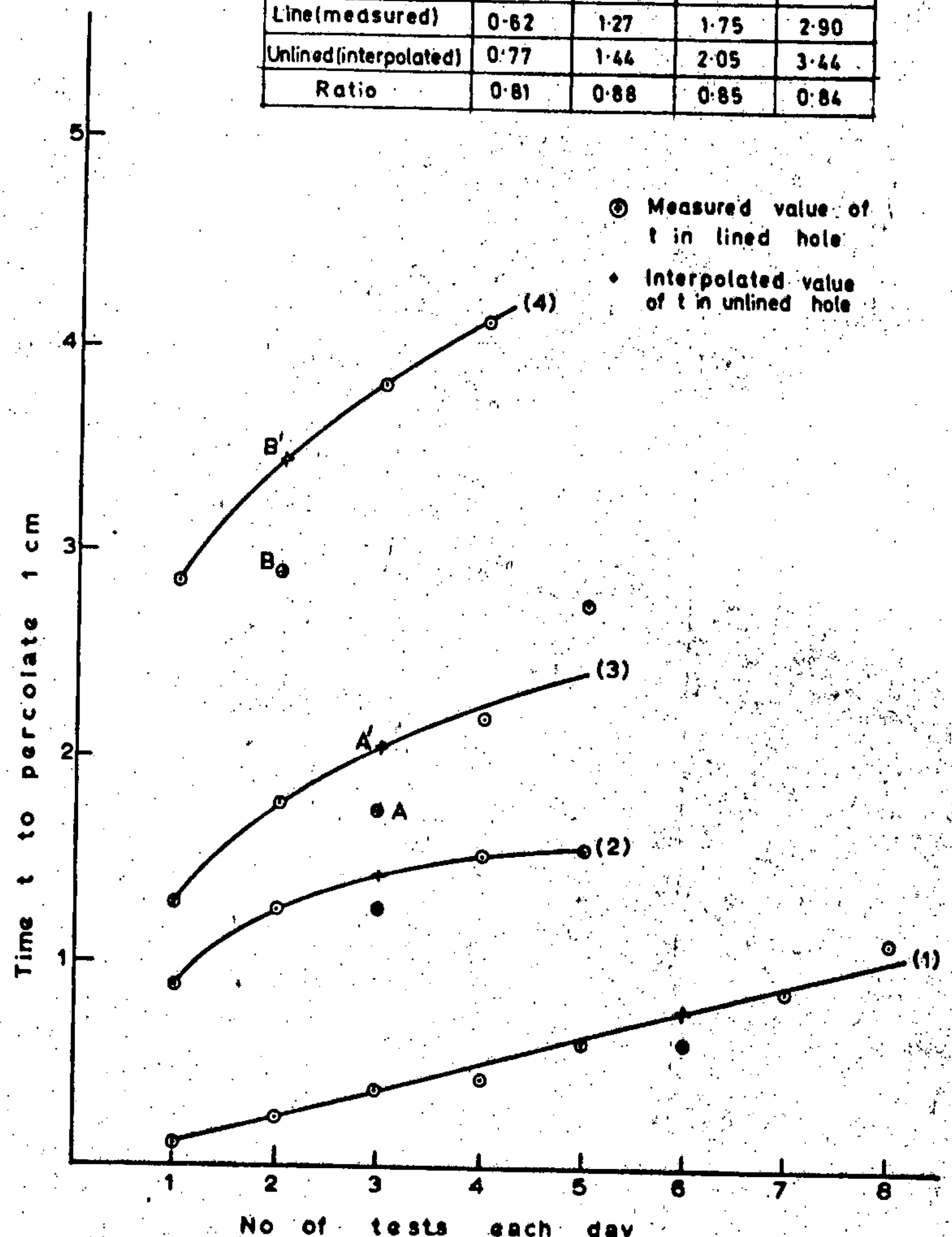
TABLE 6.9: DETERMINATION OF RATIO OF PERCOLATION RATES IN REPEATED TESTS IN MADE SOIL

TEST NO.	1	2	3	4	5	6	7	8	Nature of Hole	Ratio
Day	Time to percolate 1cm in hole (mins)									
1	0.12	0.25	0.38	0.44	0.63	0.77*	0.89	1.12	Unlined	0.81
						0.62			Lined	
2	0.89	1.26	1.44*	1.49	1.52				Unlined	0.88
			1.27						Lined	
3	1.29	1.76	2.05*	2.20	2.76				Unlined	0.85
			1.75						Lined	
4	2.85	3.44*	3.81	4.13					Unlined	0.84
		2.90							Lined	

All measurements each test day were made in unlined holes except one, e.g. test 3 on day 3. That test was performed in a lined hole when actual measurement was 1.75 minutes. Asterisked figure 2.05 (mins) was corresponding time (read off curve 3, Fig. 6.11) for percolation if test had been performed in unlined hole. Ratio $\frac{1.75}{2.05}$ is 0.85, shown in last column of this table.

FIG.6·9 DETERMINATION OF RATIO OF PERCOLATION RATES IN LINED AND UNLINED HOLES IN MADE SOIL ON 4 CONSECUTIVE DAYS

Day	1	2	3	4
Test No	6	3	3	2
Line(measured)	0·62	1·27	1·75	2·90
Unlined(interpolated)	0·77	1·44	2·05	3·44
Ratio	0·81	0·88	0·85	0·84



hole was lined for the two tests during which the actual t recorded was at A (20.00 mins) for the 2nd test and B (27.40 mins) for the 5th test, both below the curve.

From curve (5) C is 22.8 minutes and D, 31.6 minutes. The ratio of t in the lined hole to t in the unlined hole in the 2nd test on day 5 is $\frac{20.00}{22.8}$ or 0.88. The corresponding ratio in the 5th test is $\frac{27.40}{31.6}$ or 0.87. These two figures are significantly the same even though the tests did not follow immediately after each other. They confirm the earlier observation made in Test 6.2.3 in natural soil at the University sites A and B that lining made water disappear faster in a hole. They also quantify the ratio of the rates of discharge in lined and unlined holes.

Both Fig. 6.8 and Table 6.8 show that the time taken for the water to completely disappear in each of the three equidistant holes in Test 6.3.3 increased with the order of the test in each hole. This confirms the observation in Test 6.3.2 mentioned in the immediately preceeding test. The fall in the observed value of t in hole C from day 5 to day 6 in Fig. 6.8 is difficult to explain. It is not considered an error as t subsequently traced a new and uniform curve after day 6. If this new trace is continued back as shown in the dotted curve from day 6 to day 2, the resulting curve from day 1 to day 10 is more uniform than the curve through days 3 - 5.

It is observed that in spite of all the precautions taken to achieve homogenuity in the test soil and in spite of the short distance of each hole from the other two (1.83m) the rate of percolation of water in each hole each day was different from the rate of percolation in the other two holes.

The results of Test 6.3.4 (Table 6.9 and Fig. 6.9) confirm the observation in the previous tests that the time taken by the water level to fall a fixed distance increases with the number of times the test is performed in a hole. Each curve in Fig. 6.9 shows the relationship between the time taken by the water to completely disappear in the hole with the order of the test (cf. Fig. 6.6). It is seen both in Fig. 6.9 and Table 6.9 that each time the hole was lined by placing the inner shell in position the observed time t for the water to completely disappear does not fall on the appropriate curve. Rather on each of the four days of the test the observed time during the test in the lined hole fell below the appropriate curve. On day 3 for instance the observed time in the third test when the hole was lined was 1.75 mins., plotted at A (Fig. 6.9). It is assumed that if the third test had been performed in an unlined hole as in the immediately preceeding two tests and the immediately following test the observed time would have been at A^1 which is on the curve and vertically above A. A^1 is 2.05 mins/cm from the graph. By similar reasoning B^1 (2.90 mins/cm) is the position on the curve for day 4 where the actually observed time B (3.44 mins/cm) would have fallen in the second test that day if the test had been performed in an unlined hole as in the immediately preceeding test and the immediately following test.

The ratio $\frac{A}{A^1}$ is $\frac{1.75}{2.05}$ or 0.85. This means that the time taken by the water level to fall a fixed distance of 1cm in the lined hole on day 3 was 0.85 of the time it would have taken to fall the same distance if the hole had been unlined. Again the ratio $\frac{B}{B^1}$ is $\frac{2.90}{3.44}$ or 0.84. This means by similarly reasoning, that the time actually observed for the water level to fall 1cm in the lined hole in the second test on day 4 was 0.84 of the time it would have taken if the test had been performed in an unlined hole. The corresponding figures for the ratio on the first two days are 0.81 and 0.88. They are all shown in the inset table in Fig. 6.9. The ratio varied little, with an average of 0.85, which is only 4.7% higher than the lowest value and 3.5% lower than the highest value.

It is recalled that the values obtained for this ratio in the second and fifth of the tests in 6.3.2 on the fifth day were 0.88 and 0.87. These are practically the same as the ratio now established on each day of the tests in 6.3.4 now under consideration. This is significant because even though the two test samples were obtained from the same construction site they came from different batches and were taken on different days.

6.4 CONCLUSIONS:

The results of the tests reported in this Chapter lead to the following conclusions:

(a) That the rate of percolation of water varies appreciably in holes spaced only short distances apart both in natural soil and in made soil;

(b) that the rate of percolation of water in the same hole varies from day to day in both natural and made soils, the rate decreasing at an increasing rate in made soil while no definite pattern was observed in the variation in natural soil;

(c) that repetition of tests in an unlined hole leads to erosion of the sides and the clogging of the soil pores at the bottom;

(d) that in the particular laterite soil used in the made soil in these tests the rate at which the water percolated in a lined hole was 0.85 of the rate at which the water percolated in an unlined hole; the corresponding ratio in a sandy-laterite natural soil was 0.85 and 0.97 for 2 sites 10m apart;

(e) that the results fit into the mathematical model $s = pt^q$ which is simpler to operate than the equations 5.8 - 5.11 in Chapter 5.

CHAPTER VII

TESTS ON THE WATER TABLE IN LAGOS

7.1 INTRODUCTION:

The disposal of the septic tank effluent is usually accomplished by leaching into the soil in the soakage trenches or, more commonly in Nigeria, in a soakaway. The effluent either percolates through the soil pores to the water table, or in the case where the water table is at a considerable depth below the soakage trench or the so-akaway, it is just held in the soil pores. However where the water table is close to the bottom of the soakage pit, or trench, the water table affects adversely the performance of the pit. This is because immediately above the groundwater level there is a zone of capillary water. Water from the groundwater reservoir is sucked into this zone and held by surface tension in the pores between the soil particles. This zone is sub-divided into a lower sub zone of closed capillary water in which all the pores are filled with water and are all connected to the groundwater, and an upper sub-zone of open capillary water immediately above this in which the pores are partially filled with water and partially with air.

In general, the capillary height or the thickness of the zone of capillary water depends on many physical and chemical properties of the groundwater and the soil particles. Other things being equal this height varies inversely with the size of the voids or pores in soil, which itself is a function of the particles size and the density of the soil (JUMIKIS, A.R. 1962). In fine gravel the particles are so large that the soil possesses little capillary properties and there is hardly any capillary zone above the groundwater level. The thickness of the capillary zone could however be considerable in fine sands, silts and clays. Table 7.1 shows the dependence of capillary

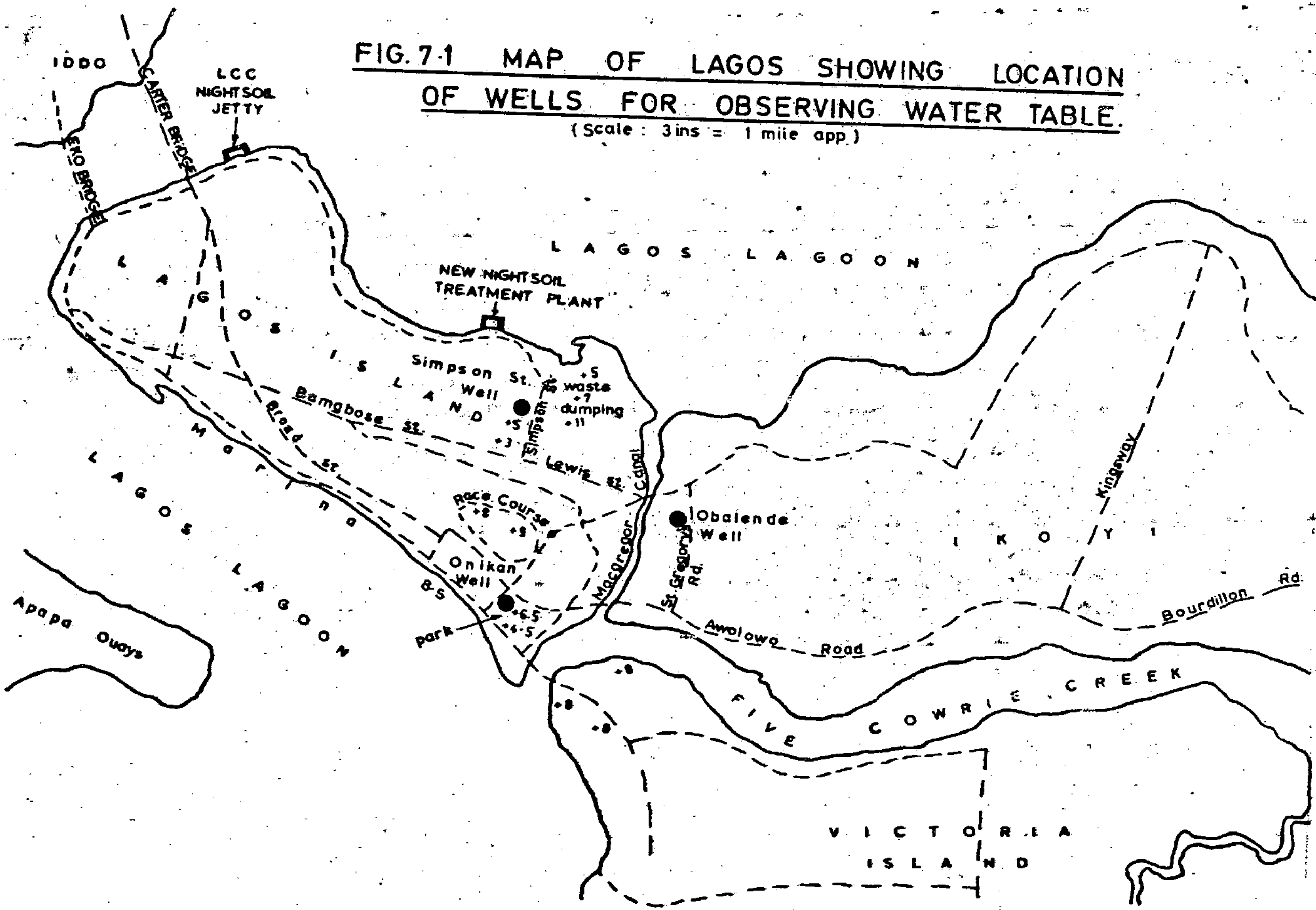


FIG. 7·1 MAP OF LAGOS SHOWING LOCATION OF WELLS FOR OBSERVING WATER TABLE.
(Scale : 3 ins = 1 mile app.)

TABLE 7.1

RELATION BETWEEN CAPILLARY HEIGHT AND GRAIN SIZE OF SOILS

SOIL	FRACTIONS (mm)	CAPILLARY HEIGHT cm
Fine Gravel	2-1	2-10
Coarse Sand	1-0.5	10-15
Medium Sand	0.5-0.25	15-30
Fine Sand	0.25-0.05	30-100
Silts	0.05-0.005	100-1000
Clay	0.005-0.0005	1000-3000
Colloids	under 0.0005	above 3000

(adapted from JUMIKIS, A.R. 1962)

height on the particle size in soils. The values are for a closed capillary fringe and are average for average conditions.

The significance of the existence of the capillary fringe on top of the water table lies in the fact that the movement of the leaching water from the soakaway towards the water table is affected during its passage through the capillary fringe where a substantial number of the pores are already filled with water. For this reason, it is always recommended that the depth of the water table below the bottom of a soakage pit or trench should not be under 1.22m (WAGNER, E.G. and LANOIX, J.N. 1958; SALVATO, J.A. 1971).

Fig. 7.1 is an outline map of part of the Lagos Area with a number of spot heights shown. There are many parts of Lagos Island, Ikoyi and Victoria Island, where the ground level is under 10 ft. (3.05m) above the ordnance datum. Since the groundwater eventually flows into the Lagoon (and thence to the sea) the groundwater level in these areas must be higher than the ordnance datum. Therefore, the depth of the water table below the ground surface is nowhere up to 3.05m in the areas under consideration.

Again, the water table is subject to diurnal variation due to tidal movements, the depth below ground level decreasing at high tide. It is also subject to variations due to precipitation leading to a recharge of groundwater through infiltration into the ground. Finally, it is subject to seasonal variations reflecting the seasonal variation in the quantity of infiltration and the level of the receiving water in the Harbour into which the groundwater flows.

7.2 TESTS ON THE WATER TABLE AT ONIKAN

A fairly detailed study of the daily vertical movement of the water table at one spot on Lagos Island and another on the Lagos Mainland over a period of six years has been made. The point selected on the Island was in the Lagos City Council's King George V Memorial Park, Onikan, 32.0m South of the Park Superintendent's Lodge. The ground level is 1.62m above OD and the soil for some depth is made up of very soft medium grained sand. A 22.9cm diameter everite pipe was buried vertically in a pre-dug hole to a depth of 0.61m below the water table. The pipe was held vertically by a short length of 6.3.1 Portland Cement concrete surround carefully laid dry in view of the considerable amount of groundwater encountered in the hole. Great care was taken to see that no cement got inside the pipe itself which, on curing, might block the ingress of groundwater into the well. The pipe was made 15cm proud of the ground to prevent storm water entering the well during heavy rains, when the site was subject to flooding. Next was erected a sight rail consisting of two uprights of 7.6cm x 5.1cm timber some 0.76m apart to which a 5.1cm x 2.5cm timber horizontal rail was nailed at a height of 1.52m above ground level (Fig. 7.2).

A graduated measuring stick 3.05m long was used for measuring the water table. The stick was lowered into the well. When it pierced the water surface ripples were observed. The stick was then lifted about a centimetre till the ripples disappeared. The stick was again lowered slowly and gently till the surface of the water was again pierced as evidenced by the reappearance of ripples. The marking on the graduated stick against the upper edge of the horizontal rail both at the appearance and the disappearance of the ripples was noted and the mean was recorded. From the known height of the rail above ground level and the known reduced level of the latter the reduced level of the water table was calculated. The readings were taken about the same time every day usually about 9.00 a.m. Readings were however not taken on Sundays and Public holidays. Table 7.2 shows the field form for recording daily readings.

7.3 TESTS ON THE WATER TABLE AT LAGOS UNIVERSITY

The site selected for the test on the Lagos Mainland waa at the University of Lagos Campus, Yaba, a few meters south of the Faculty of Engineering buildings and between the temporary buildings formerly used as the Health Centre and the Department of African and Asian Studies.

TABLE 7.2

REDUCED LEVEL OF WATER TABLE AT ONIKAN, JUNE 1972

(to be read together with Fig. 7.2)

Date	Reading h m.	Depth of water table h_2 ($h_2 = h_1 + 0.305$) m.	Reduced level of water table = $1.616 - h_2$ m
1	0.482	0.787	0.829
2	0.473	0.777	0.839
3	0.473	0.777	0.839
4	-	-	-
5	0.476	0.781	0.835
6	0.476	0.781	0.835
7	0.476	0.781	0.835
8	0.473	0.777	0.839
9	0.473	0.777	0.839
10	0.476	0.781	0.835
11	-	-	-
12	0.482	0.787	0.829
13	0.482	0.768	0.848
14	0.407	0.710	0.906
15	0.366	0.671	0.945
16	0.369	0.674	0.942
17	0.369	0.674	0.942
18	-	-	-
19	0.369	0.674	0.942
20	0.372	0.677	0.939
21	0.330	0.635	0.981
22	0.314	0.635	0.981
23	0.314	0.635	0.981
24	0.332	0.637	0.979
25	-	-	-
26	0.305	0.610	1.006
27	0.308	0.613	1.003
28	0.254	0.558	1.058
29	0.257	0.561	1.055
30	0.257	0.561	1.055

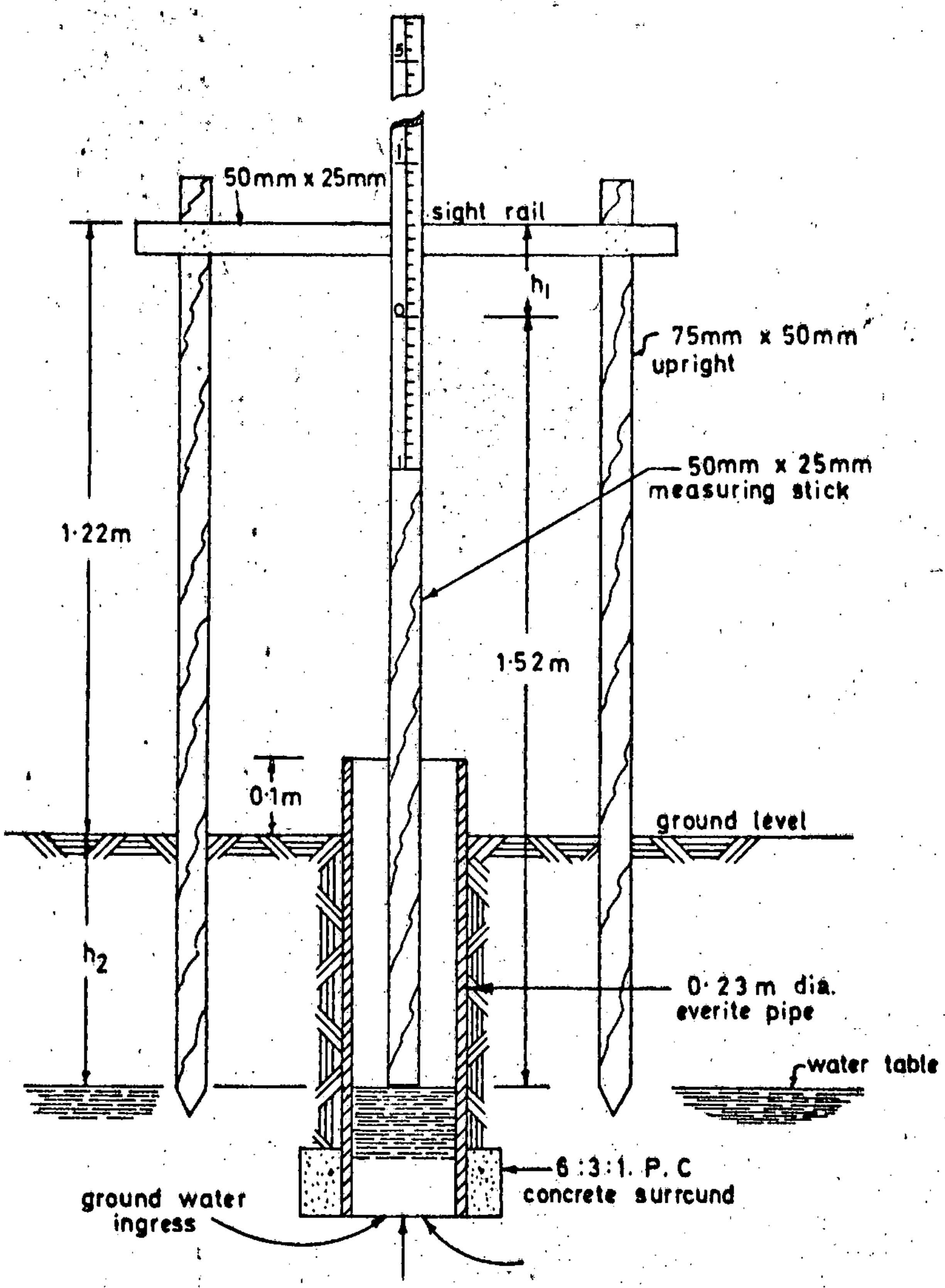


FIG.7·2 MEASURING DEVICE FOR WATER TABLE AT ONIKAN WELL

TABLE 7.3

MAXIMUM, MINIMUM AND ANNUAL MEAN ELEVATIONS OF WATER TABLE AT ONIKAN AND LAGOS UNIVERSITY FROM 1968 TO 1974.

YEAR	ONIKAN					LAGOS UNIVERSITY				
	Ground Level = 5.30 feet.					Ground Level = 23.00 feet.				
	Maximum		Minimum		Annual Mean	Maximum		Minimum		Annual Mean
	Date	Ft.	Date	Ft.	Ft.	Date	Ft.	Date	Ft.	Ft.
1968-69	19-6-68	5.45	28-2-69	2.46	3.65	10-9-68	13.73	1-3-69	3.46	8.55
1969-70	2-7-69	4.72	Incomplete	Records		10-7-69	13.25	31-3-70	3.37	7.35
1970-71	17-7-70	4.78	13-4-70	2.40	3.55	2-7-70	10.06	13-4-70	3.34	5.44
1971-72	12-7-71	4.55	14-5-71	2.62	3.22	25-9-71	9.24	24-3-72	3.83	5.56
1972-73	3-7-72	3.70	21-3-73	1.48	2.21	Measurements discontinued				
1973-74	6-7-73	3.24	5-5-73	1.82	2.62	Measurements discontinued				
	Depth of water table below G.L. Maximum = 5.30 - 1.48 = 3.82 feet Minimum = 5.30 - 5.45 = -0.15 feet i.e. 0.15 ft. flood depth.					Depth of water table below G.L. Maximum = 23.00 - 3.34 = 19.66 feet Minimum = 23.00 - 13.73 = 9.27 feet				

The well at this site was constructed in August 1967, at the end of an unusually long period of drought. The depth below ground level was 6.71m, which was about 0.61m beyond the water table at the time of construction. A pad 0.91m square in plan 0.30m thick, with a 0.30m dia hole in the centre was first cast in 1:3:6 Portland cement concrete at the bottom of the excavation. This served as foundation as well as seating for the first of the seven precast concrete culvert rings, each 0.61m dia, 0.91m high and 5cm thick, which were used in the construction of the well.

Here also the concrete foundation was laid dry to allow for the considerable quantity of ground water encountered in the well. Again care was taken to see that cement did not get inside the bottom ring. The topmost of the rings was constructed 5cm proud of a 7.6cm concrete slab which was constructed round the well. This formed the seating for a 0.78m dia, 2.5cm thick timber cover, which prevented lizards etc., from getting into the well. A shed of cement asbestos walls and roof 7.32m by 5.49m interval floor dimensions and average height of 2.10m was built over the well.

Measurement of the depth of the water table was done by means of a battery operated electric contact gauge. It consisted of two electrodes housed in a 2cm dia and 17cm long copper cylinder at the end of a 30m long metallic tape winding round an 8cm drum mounted on a table about 0.7m vertically above the top of the well. On unwinding the tape the electrodes were lowered into the well. When they made contact with the surface of the water the electrical circuit was completed and a small Osram lamp (5v 0.15 amp) was lit. The depth of the water table below ground level was the difference between the reading on the tape and the height of the gauge drum above ground level. Fig. 7.3 shows the well and the device for measuring the depth of the water table.

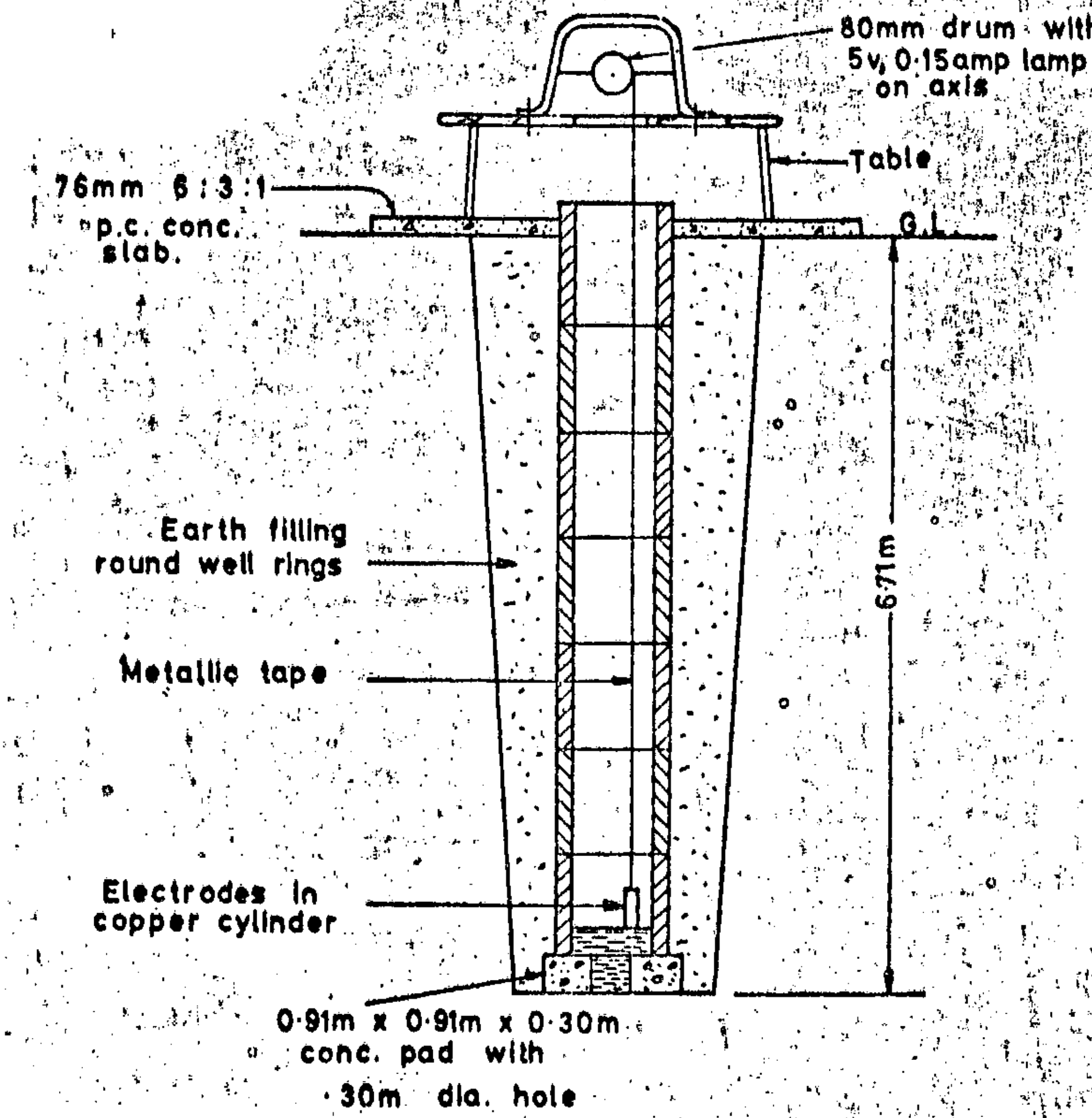


FIG.7·3 MEASURING DEVICE IN WELL AT LAGOS UNIVERSITY SITE

7.4 RESULTS

The results are shown in Tables 7.2 - 7.6 and Figs 7.4 - 7.7. Table 7.3 shows that the minimum depth of water table below ground level at the Lagos University site was 2.83m, which occurred on 10/9/68, that is in the wettest of the six years under study. As the minimum depth of water table below ground level for a soakaway to be effective is known to be 1.83m, it is obvious that the depth of the water table below ground level at the Lagos University site would always be adequate for a septic tank soakaway installation. It was therefore decided to discontinue observations on the water table in this well in October 1972, that is after over 4 years of daily observations.

Fig. 7.4 shows the monthly average of the daily level of the water table for the six years under consideration. It shows a general trend of the water table rising from April the beginning of the rainy season to a peak in July, and then falling to a minimum in March, which is the end of the dry season. Fig. 7.4 shows two maxima, the normal one that occurs in July and a less pronounced one occurring in October.

Fig. 7.5 shows the actual daily level of the water table from May to August in 1968, together with the actual daily rainfall at Tafawa Balewa Square for the same period. The highest recorded level of the water table in the six years under consideration occurred in this particular period of 1968, the water-table actually rising above ground level on three occasions, that is, 1.357m on June 19th,, 1.646m on July 9th and an unrecorded height known to be above 1.662m which occurred after the very heavy rainfall of 8.85 inches* on a single day, July 30th which left the site so severely flooded that it was inaccessible for measurement of the water-table on the 31st day.

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* This part of the work was done before 1972 when the country went metric, and the measurements were taken in Imperial units.

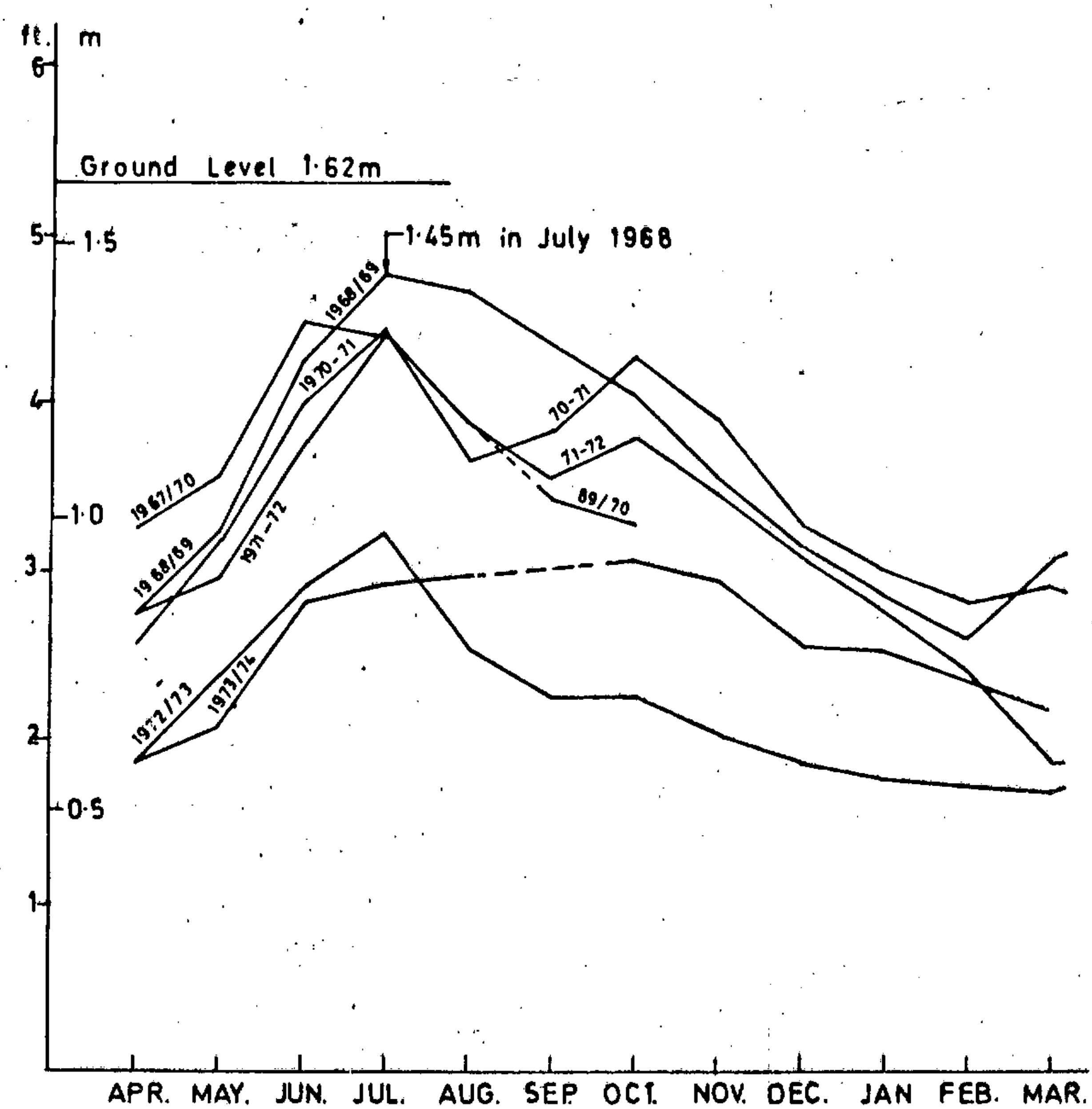


FIG. 7·4 MONTHLY AVERAGE OF DAILY ELEVATION OF WATER TABLE IN ONIKAN WELL FROM 1968 TO 1974

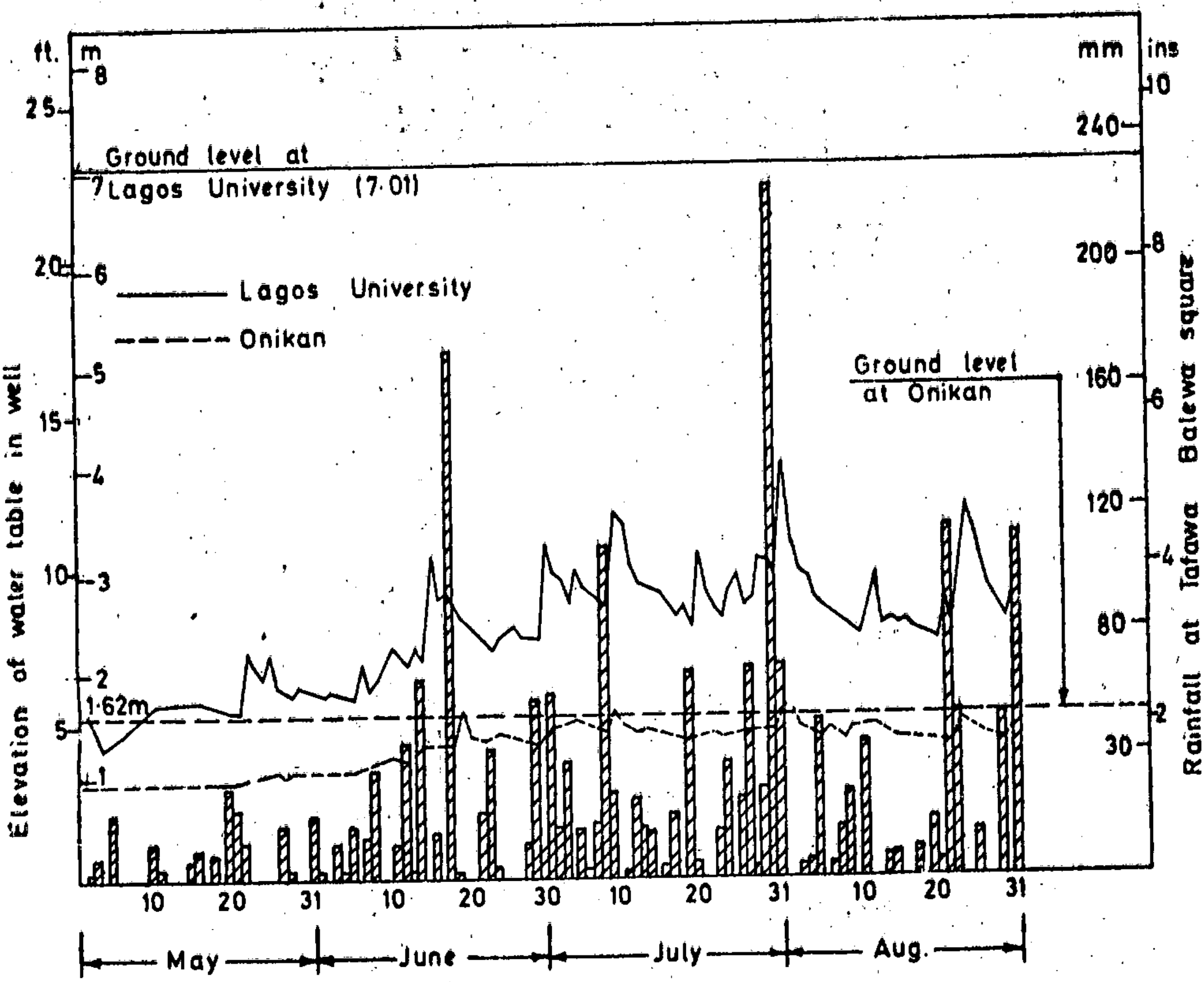


FIG.7·5 ELEVATION OF WATER TABLE AT ONIKAN AND LAGOS UNIVERSITY AND DAILY RAINFALL AT TAFAWA BALEWA SQUARE. MAY—AUGUST 1968

The year 1968/69 was also the wettest year, with a total of 135.00 inches of rain (343cm, Table 7.4). Again Table 7.4 shows that 1972/73 was the driest year with a total of 52.39 inches (133cm) while Fig. 7.4 shows that the lowest level of the water-table generally occurred that year. As would be expected, there is thus great correlation between the rainfall and the level of the water table. The daily level of the water table generally rises with increasing rainfall, with sharp rises occuring after, but lagging some 24 to 48 hours behind; heavy precipitations (Fig. 7.5). From Table 7.5 it is seen that the maximum elevation of the water table varied from 0.988m on 6/7/73 to 1.662 on 19/6/68, a variation of 0.674m. The minimum elevation varied from 0.451m on 21/3/73 to 0.799m on 14/5/71, a variation of 0.348m. The highest variation of 0.912m in the water table in one year occurred in the wettest year 1968/69 while the lowest variation of 0.527m occurred in 1972/73. The highest mean elevation in one year was 1.113m which again occurred in the wettest year 1968/69 while the lowest mean elevation of 0.674m occurred in the 1972/73 year.

P.W.D. Drawing No. 21040 referred to earlier in Chapter III specifies a 2ft. (0.61m) minimum depth of the bottom of a soakaway pit below ground level. WAGNER, E.G. and LANOIX, J.N. (1958) and others recommend a minimum depth of 1.22m for the water table below the bottom of the soakaway. Therefore, a minimum of 6ft. (1.83m) depth of water table below ground level is required for a soakaway installation to be effective. In the six years under study, at no time was the depth of the water table below ground level up to this figure, the greatest depth being 1.16m on 21/3/73 when the water table elevation was 0.45m.

Where sub-surface trenches are used in place of soakaway, the need for earth backfill on top of the gravel surround of the drain pipe (Fig. 3.3) still requires that the trench bottom should be a minimum of .46m below ground level. This is .15m

TABLE 7.4

MONTHLY RAINFALL RECORD AT TAFAWA BALEWA SQUARE, LAGOS APRIL 1968 - MARCH 1973

(inches)

	1968/69		1969/70		1970/71		1971/72		1972/73		1973/74	
	Monthly total	Cumu-lative Total	Monthly total	Cumu-lative Total	Monthly total	Cumu-lative Total	Monthly total	Cumu-lative Total	Monthly total	Cumu-lative Total	Monthly total	Cumu-lative Total
April	7.16	7.16	6.40	6.40	4.00	4.00	2.27	2.27	6.19	6.19	3.54	3.54
May	6.77	13.93	14.51	20.91	11.87	15.87	8.41	10.68	7.75	13.94	8.68	12.22
June	23.51	47.44	20.75*	41.66	23.12*	38.99	14.04*	24.72	18.44*	32.38	10.68	22.90
July	36.47*	73.91	11.14	52.80	18.76	57.75	14.00	38.72	3.59	35.97	3.06	25.96
August	20.75	94.66	5.29	58.09	1.37	59.12	1.08	39.80	0.35	36.32	10.73*	36.69
September	21.06	115.72	2.66	60.75	12.91	72.03	8.64	48.44	4.59	40.91	9.40	46.09
October	7.47	123.19	8.70	69.45	12.04	84.07	2.32	50.76	2.93	43.84	7.65	53.74
November	1.23	124.42	2.05	71.50	2.66	86.73	1.87	52.63	0.79	44.63	1.60	55.34
December	0.71	125.13	0.17+	71.67	0.00+	86.73	0.70+	53.33	0.61+	45.24	2.70	58.04
January	1.23	126.36	4.99	76.66	0.33	87.06	3.35	56.68	1.40	46.64	3.12	61.16
February	0.08+	126.44	0.30	76.96	3.75	90.81	1.02	57.70	1.47	48.11	1.01+	62.17
March	8.56	135.00	1.36	78.32	1.03	91.84	0.93	58.63	4.28	52.39	4.41	66.58
TOTAL	135.00	135.00	78.32	78.32	91.84	91.84	58.63	58.63	52.39	52.39	66.58	66.58

*Wettest month in year

+Driest month in year

NOTE: Figures are inches because the country had not gone metric when the meteorological Department compiled these records.

short of the .61m required below the soakaway pit, and reduces the required minimum depth of the water table below ground level from 1.83m in case of a soakaway to 1.68m in case of trenches. Even then the greatest water table depth of 1.16m in the Onikan well is still 0.52m short of this.

7.5 SIMPSON STREET AND OBALENDE WELLS

It was next decided to investigate how far the movement of the water table in the Onikan well could predict the movement of the water table in other parts of Lagos Island which are reasonably near enough to Onikan to assume that they are affected by the same rainfall. It was decided to study the movement of the water table in wells at two other locations, the first in the premises of the Lagos City Council offices at Simpson Street, and the other in the premises of the Nigeria Police Barracks at Obalende (Fig.7.1). The two wells in Simpson Street, and Obalende were 1.098km and 1.037km distant respectively from the Onikan well. In addition the Obalende well was separated from the Onikan well by MacGregor Canal. Construction details and arrangement for measurement in both wells were identical with those of Onikan.

Measurements on the Simpson Street well started in April, 1972 and continued till March 1974. Measurements on the Obalende well started effectively only in October 1972 and were reliable till only March 1973 after which there were problems with the accuracy of measurement. Measurements were taken on all three wells everyday, the Onikan well being taken as control on the other two. Ground level at the Simpson Street and the Obalende wells were 1.10m and 1.80m above OD respectively.

Fig. 7.6 shows the daily elevation of the water table in all three wells in October 1972 together with the actual rainfall in that month, which was scanty. While a correlation between rainfall and water table elevation in both the Simpson Street and Obalende wells is noticeable, there seems to be no such correlation at the Onikan well. Nor was there any striking similarity between the water table profile at Onikan and those of the other two wells.

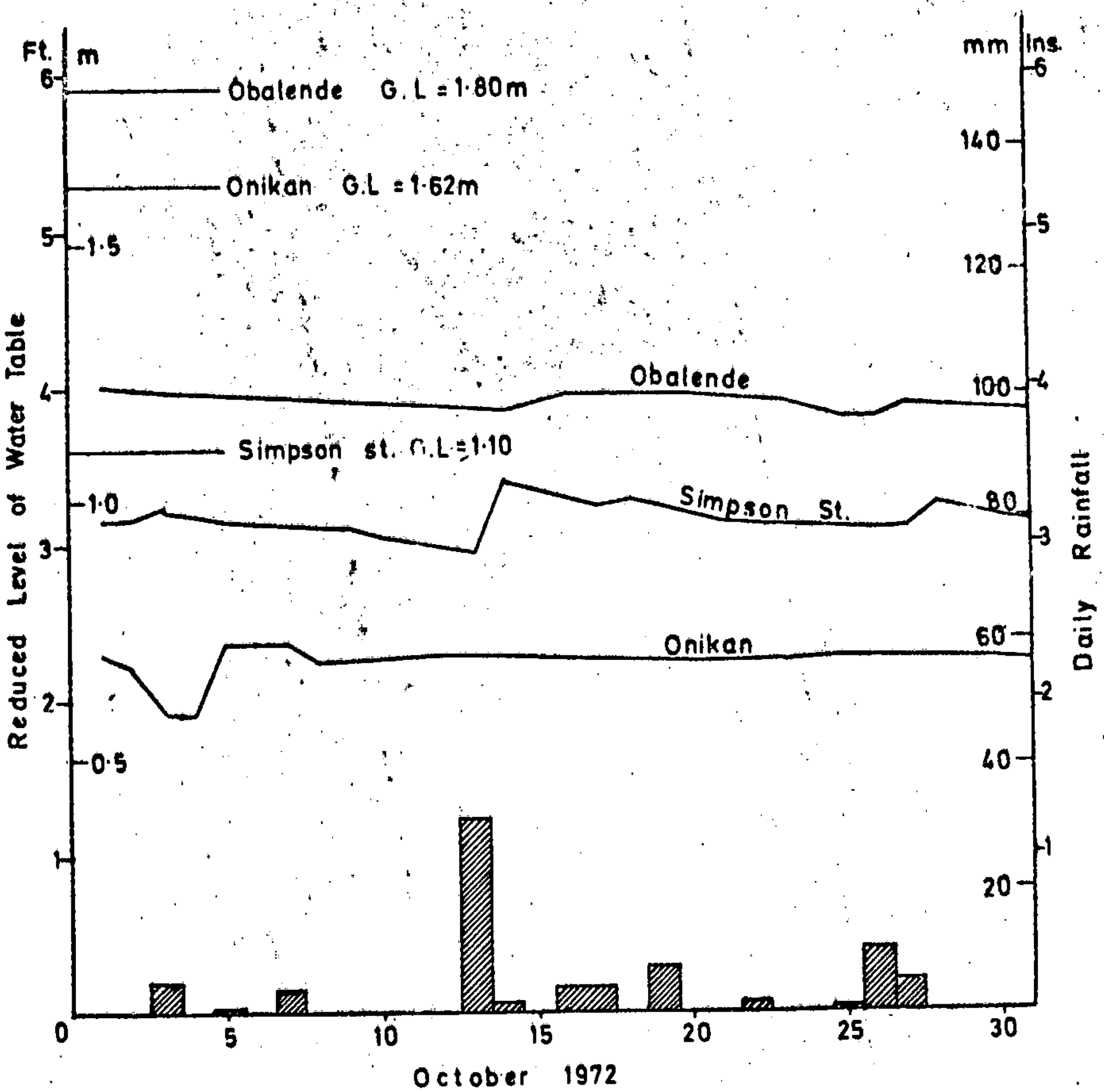


FIG. 7·6 REDUCED LEVEL OF WATER TABLE AT ONIKAN, SIMPSON STREET AND OBALENDE IN OCTOBER 1972

TABLE 7.5 : LEVEL OF WATER TABLE AT SIMPSON STREET IN MARCH 1973 CALCULATED FROM EQUATION $y = 0.52 + 0.55x$, AND COMPARED WITH ACTUAL MEASUREMENT

Date	x (Onikan) m	0.55x m	y=0.52+0.55x (Simpson computed) m	Simpson actual m	Error %
1	0.506	0.278	0.798	0.878	-9.11
2	0.506	0.278	0.798	0.841	-5.19
3	0.555	0.305	0.825	0.848	-2.66
4	0.503	0.277	0.797	0.841	-5.29
5	0.497	0.273	0.793	0.838	-5.42
6	0.494	0.272	0.792	0.838	-5.49
7	0.488	0.268	0.788	0.887	-11.18
8	0.482	0.265	0.785	0.829	-5.34
9	0.478	0.263	0.783	0.826	-5.23
10	0.478	0.263	0.783	0.826	-5.23
11	0.476	0.262	0.782	0.826	-5.32
12	0.473	0.260	0.780	0.826	-5.57
13	0.463	0.255	0.775	0.826	-6.17
14	0.451	0.248	0.768	0.826	-7.02
15	0.463	0.255	0.775	0.826	-6.17
16	0.470	0.258	0.778	0.826	-5.81
17	0.470	0.258	0.778	0.845	-7.87
18	0.470	0.258	0.778	0.841	-7.55
19	0.470	0.258	0.778	0.835	-6.87
20	0.463	0.255	0.775	0.829	-6.55
21	0.451	0.248	0.768	0.826	-7.05
22	0.451	0.248	0.768	0.826	-7.05
23	0.457	0.252	0.772	0.826	-6.54
24	0.451	0.248	0.768	0.826	-7.02
25	0.451	0.248	0.768	0.826	-7.02
26	0.686	0.377	0.897	0.826	+8.60
27	0.674	0.371	0.891	0.823	+8.24
28	0.665	0.366	0.886	0.823	+7.66
29	0.567	0.312	0.832	0.823	+1.09
30	0.466	0.257	0.777	0.948	-18.06
31	0.463	0.255	0.775	0.988	-21.54

TABLE 7.6. WATER TABLE AT SIMPSON STREET IN JUNE 1968 COMPUTED FROM MEASUREMENTS OF WATER TABLE AT ONIKAN THAT MONTH ($y=0.52+0.55x$)

Date	x (Onikan) m	.55x m	0.52+0.55x (Simpson) m	Flood depth m
1	-	-	-	-
2	-	-	-	-
3	1.067	0.587	1.107	0.006
4	1.067	0.587	1.107	0.006
5	1.067	0.587	1.107	0.006
6	1.095	0.602	1.122	0.021
7	1.113	0.612	1.132	0.031
8	-	-	-	-
9	-	-	-	-
10	1.192	0.656	1.176	0.075
11	1.189	0.654	1.174	0.073
12	1.171	0.644	1.164	0.063
13	1.274	0.701	1.221	0.120
14	1.290	0.709	1.229	0.128
15	1.323	0.728	1.248	0.147
16	-	-	-	-
17	1.326	0.729	1.249	0.146
18	1.326	0.729	1.249	0.148
19	1.662	0.914	1.434	0.333
20	1.473	0.810	1.330	0.229
21	1.421	0.781	1.301	0.200
22	1.387	0.763	1.283	0.182
23	-	-	-	-
24	1.439	0.791	1.311	0.210
25	1.409	0.775	1.295	0.194
26	1.390	0.765	1.285	0.184
27	1.369	0.753	1.273	0.172
28	1.351	0.743	1.263	0.162
29	1.335	0.734	1.254	0.153
30	-	-	-	-

Groundlevel at Simpson Street = 1.101m O.D.

flood depth of 0.34m appears to have occurred on June 19th, a day after the very heavy rainfall of 7.79 ins. (17.3cm) of 18th June which no doubt caused it.

It is considered that the high correlation coefficient of 0.9050 in equation 7.2 indicates that the elevation of the water table at Obalende can be predicted with some confidence from observations at Onikan. The two wells are most probably covered by the same rainfall and the same tides the effects of which appear to predominate over whatever effect the MacGregor Canal might have over them in spite of the fact that the wells are situated on opposite banks of the Canal. Unfortunately in spite of this high correlation coefficient between Obalende and Onikan, observations at Obalende covered only the dry season months of October 1972 to March 1973. In the circumstances, predicting the movement of the water table at Obalende from observations at Onikan must be limited to the dry season only.

7.6 CONSIDERATION OF CAPILLARY FRINGE

To obtain an idea of the texture of the soil through which the water percolated under each well, samples were taken from a depth of about 3ft. (0.91m) from each of 3 holes located at approximately equal distances of 0.61m. from each well. The nine samples were later analysed for particle size distribution. Table 7.7 shows the results for the three holes around the Onikan well. As would be expected there was some variation in the particle size distribution in the three holes. As the variation in most cases is, however, under 2%, it is, considered reasonable to assume that the mean distribution shown in the last column will approximate very closely to the actual particle size distribution in the soil in the well. Table 7.8 and Fig. 7.7 drawn from it therefore show the approximate particle size distribution of the soil in the three wells at Onikan, Simpson Street, and Obalende. The soil is predominatly medium sand in each case. For soil of this grain size the capillary height ranges from 15cm to 30cm according to Table 7.1.

If for design purposes a capillary height of 30cm is assumed then a soakage pit the bottom of which is 0.46m above the water table should function reasonably well. This height allows for an additional 0.15m through which the leaching effluent can percolate before entering the capillary fringe. Since Fig. 3.4 shows a minimum soakaway depth of 0.61m the required minimum depth of the water table below ground level becomes 0.46 + 0.61 = 1.07m. This new figure of 1.07m is considerably less than the 1.83m minimum depth established earlier in Chapter III as well as in this Chapter.

At Onikan where the ground level was 1.62m above OD, the highest acceptable level of water table to satisfy the new criterion is 1.62 - 1.07 = 0.55m. The periods in which the water table at Onikan was not above this figure during the six years study were the 16 days from 1/4/72 to 16/4/72, the 71 days from 22/12/72 to 2/3/73, the 21 days from 4/3/73 to 25/3/73, and the 31 days from 30/3/73 to 30/4/73. This amounts to 139 days 123 of which were in the unusually dry, dry season of 1972/73. The water table being under 0.55m OD in 139 days in the period of six years means that the water table was low enough to satisfy this new and less rigorous criterion only 6.4% of the period of investigation.

The lowest water table recorded at Onikan was 0.45m OD on 21/3/73. If it is assumed that the lowest water table at Simpson Street and Obalende also occurred on that day, equations (7.1) and (7.2) can be used to calculate the water table at the two places on that date in spite of the misgiving already noted in the application of equation (7.2). The calculated values of the water table at the two places are 0.77m and 0.80m respectively. Considering that the ground level at the two locations is 1.10m and 1.80m above OD respectively, the highest levels of water table to satisfy the criterion of a minimum depth of 1.07m below ground level are 0.03m and 0.73m respectively. As these latter are lower than the values calculated above, it is concluded that if there had been soakage pits at Simpson Street and Obalende, they would not have functioned properly at any time during the six years investigation.

FIG. 7·7 PARTICLE SIZE DISTRIBUTION OF SOIL AT ONIKAN, SIMPSON STREET & OBALENDE.

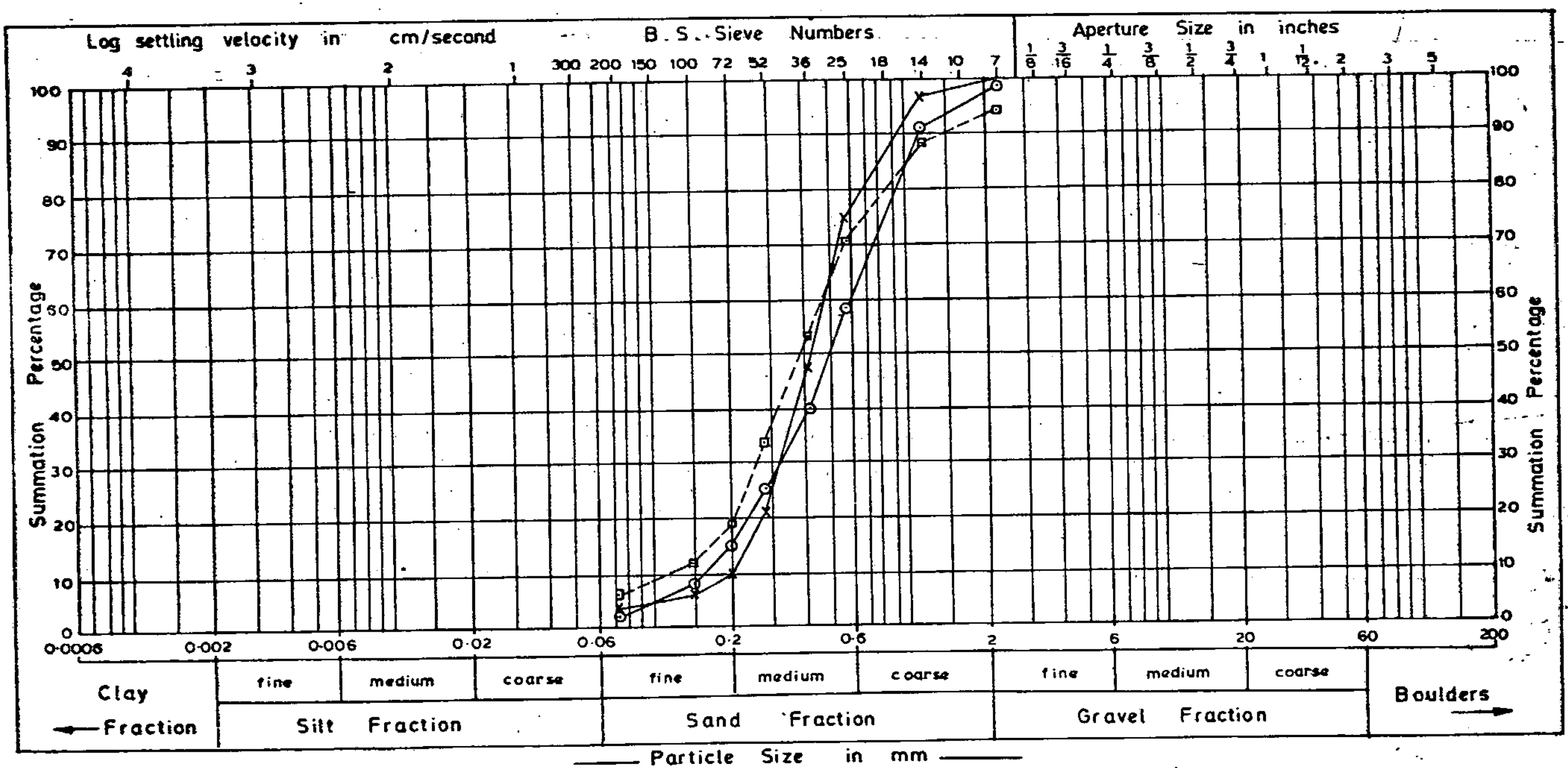


TABLE 7.7

SIEVE ANALYSIS TEST ON SOIL SAMPLES AT ONIKAN

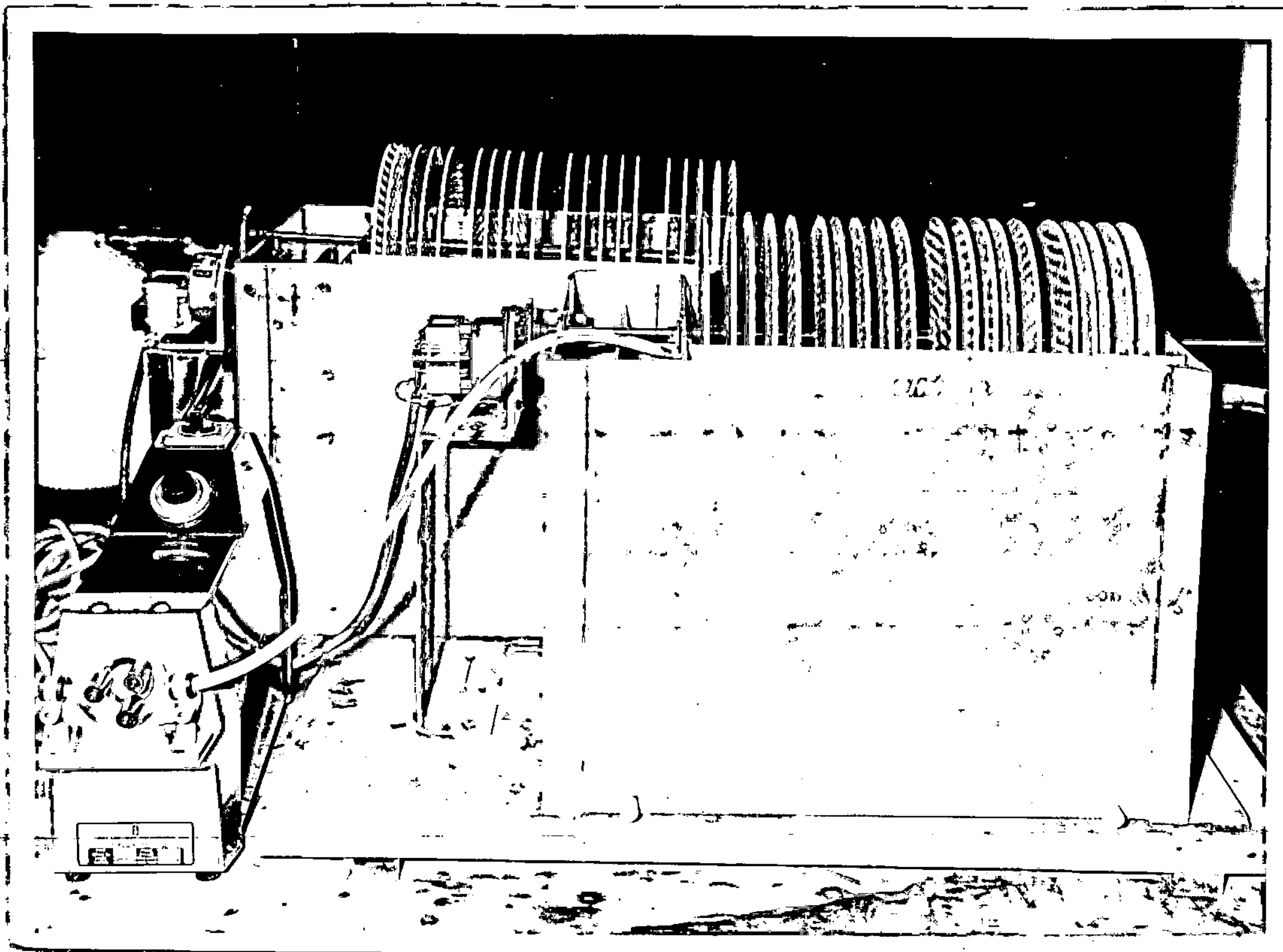
(Total Wt. of dry sample used in each case = 100.00gm)

	SPECIMEN NO. 1				SPECIMEN NO. 2				SPECIMEN NO. 3				
Sieve No.	Weight ret'd (gm)	% ret'd	Cum. % ret'd	% passing	Weight ret'd (gm)	% passing	Cum. % ret'd	% passing	Weight ret'd (gm)	% passing	Cum. % ret'd	% passing	Mean % passing
7	17.5	1.75	1.75	98.25	19.00	1.90	1.90	98.10	20.00	2.00	2.00	98.00	98.12
14	71.0	7.10	8.85	91.15	87.50	8.75	10.65	89.35	63.00	6.30	8.30	91.70	90.73
25	330.0	33.00	41.85	58.15	354.00	35.40	46.05	53.95	304.00	30.40	38.70	61.30	58.11
36	173.0	17.30	59.15	40.85	180.00	18.00	64.05	35.95	181.00	18.10	56.80	43.20	40.00
52	148.0	14.80	73.95	26.05	136.00	13.60	77.65	22.35	160.0	16.00	72.90	27.10	25.28
72	107.0	10.70	84.65	15.35	95.00	9.5	87.15	12.85	99.0	9.90	82.80	17.20	15.13
100	70.0	7.00	91.65	8.35	65.00	6.5	93.65	6.15	88.00	8.80	91.60	8.40	7.55
200	60.0	6.00	96.65	2.35	48.00	4.8	98.45	1.55	66.0	6.60	98.20	1.80	1.90
Receiver	9.0	0.90			7.00	0.7			9.5	0.95			
Total Wt.	975.5 gm				991.5 gm				991.5 gm				

TABLE 7.8

APPROXIMATE SOIL PARTICLE SIZE DISTRIBUTION AT ONIKAN, SIMPSON STREET & OBALENDE

SIEVE No.	ONIKAN		SIMPSON STREET		OBALENDE	
	% Wt. retained	% Wt. passing	% Wt. retained	% Wt. passing	% Wt. retained	% Wt. passing
7		98.12		94.0		99.27
14		90.73		88.0		97.03
25		58.11		70.7		74.57
36		40.00		53.2		47.32
52		25.28		33.9		20.63
72		15.13		19.4		10.40
100		7.55		12.4		6.40
200		1.90		5.9		2.47



LABORATORY SCALE MODEL BIODISC WITH GROWTH ON DISCS

7.7 C O N C L U S I O N

The conclusion from this study is that the water table in the three parts of Lagos investigated is too near ground level to make any soakaway constructed in them function effectively. This disadvantage applies not only in the wet season but in the dry season as well. By implication all areas in Lagos that are not much higher than the areas investigated must necessarily suffer from the same disadvantage. Therefore the septic tank - soakaway system is an unsuitable method of sewage disposal in the low lying areas of Lagos Island, Victoria Island, Apapa and the Mainland.

CHAPTER VIII

THE BIODISC PROCESS FOR WASTE TREATMENT

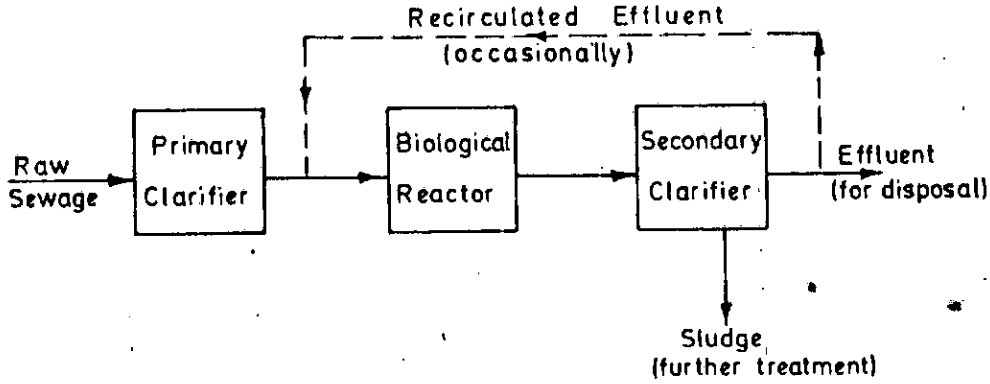
8.1 INTRODUCTION

It was shown in Chapter I that the network of sewers for collecting and transporting sewage is the most expensive component of a municipal waterborne sanitation system and that it is largely due to this that the system will remain an unrealisable ideal for some time yet in Nigeria. While the cost of treatment plant is not high relative to the cost of sewers, the cost of the conventional waste treatment systems like the Activated Sludge and the Trickling Filter is however high enough in itself to make these established conventional systems prohibitive in the circumstances of a developing country. The biodisc process is a low-cost form of waste treatment for which a number of advantages are claimed by several authors and equipment manufacturers. The process is discussed in the present Chapter in the hope that it may be introduced to Nigeria for the treatment of wastes from suitable areas in existing towns as advocated in Chapter I, and from villages and other small communities.

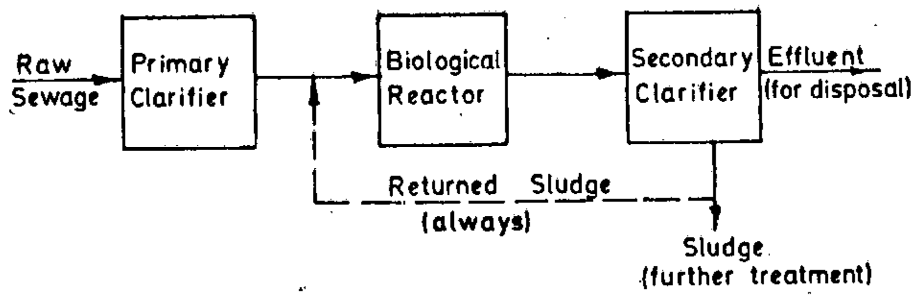
8.2 EVOLUTION OF THE BIODISC PROCESS

Like the trickling filter and the activated sludge process the biodisc is bascially a biological process of waste treatment. Common to all three processes is a biological reactor in which the process of biological decomposition takes place through the agency of aerobic microorganisms. Whereas the return of sludge to the biological reactor is an essential feature of the activated sludge process and the recirculation of effluent sometimes a feature of the trickling filter, neither is sludge returned nor effluent recirculated to the reactor in the basic biodisc process (JOOST, R.H. 1969). The reactor here consists of one or more batteries of discs rotating in a tank of waste-water as compared with the biological support media of the trickling filter or the aeration tank of the activated sludge process. Fig. 8.1 shows the basic difference between the biodisc on the one hand and the trickling filter and the activated sludge process on the other.

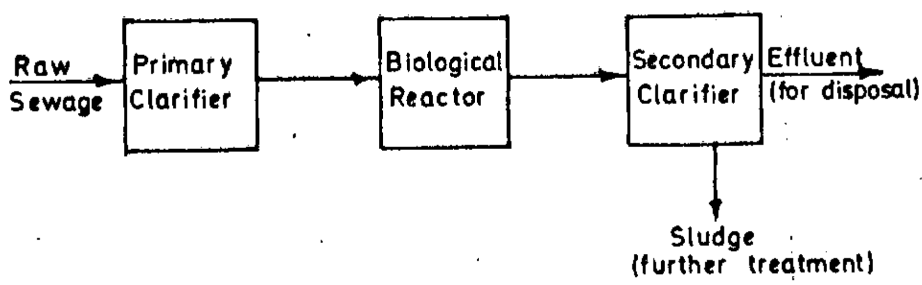
GLOYNA et al (1952) described an experimental trickling filter consisting of rotating tubes in each of which a film of micro-organisms on the inner surface is alternately submerged in a flowing waste stream in the lower segment of the tube section, and alternately exposed to the air above the waste in the upper segment. Gloyna's tube was rotated at a speed of 16 r.p.m. and was claimed to have among others the advantage of permitting a more rational analysis of filter function. HARTMAN, H. (1960 and 1961) described discs which when rotating slowly in a sewage basin were soon covered with biological growths which could absorb organic compounds from the waste during immersion and, on emerging from the waste, could take up and



(a) The Trickling Filter: Part of the effluent from the secondary clarifier is occasionally recirculated through the biological reactor



(b) The Activated Sludge Process: Part or all of the Sludge from the Secondary Clarifier is recirculated through the biological reactor.



(c) The Biodisc Process: No recirculation of either effluent or Sludge through the Biological reactor

FIG. 8.1 BIODISC PROCESS COMPARED WITH ACTIVATED SLUDGE PROCESS AND TRICKLING FILTER store up the oxygen necessary for the oxidation of the organic compounds. Hartman's 'dipping filters' consisted of a series of such discs revolving on a common shaft held slightly above the water line. SIMPSON, J.R. (1963) evolved a model comprising of twenty vertical perspex discs carried on a common shaft and dipping into a rectangular tank divided into five compartments by round-the-end baffles. In a subsequent development of Simpson's laboratory model the perspex discs were replaced first with netlon mesh and later with perforated I.C.I. davic 1.2mm thick discs with 6.35mm openings. The rougher surface of the new material allowed a firmer attachment of the micro-organisms and algae film, which improved the efficiency of oxygen transfer.

One of the factors responsible for the high efficiency and dependability of the biodisc process is the environment provided by the oxygen-rich waste film on the disc which supports high densities of aerobic bacteria, reported in the Literature to be in the order of 18,000 to 30,000mg/litre. This results in low food-micro-organism ratio of some 0.02 - 0.05 as compared with 0.3 for the conventional activated sludge process (JOOST, R.H. 1969). These high biomass densities enable the process to absorb organic shock loads and make it ideally suited to the treatment of high concentration wastes.

8.3 DESCRIPTION AND OPERATION OF LABORATORY SCALE MODEL BIODISC

The laboratory unit used in the studies embodied in this thesis is a modification of Simpson's model. It consists of three parts: an aluminum Imhoff Tank, a battery of 20 discs rotating on a stainless steel shaft, and a fractional horse-power motor for rotating the shaft (Fig. 8.2).

The tank is divided into three sections: a small primary settling chamber, an aerobic chamber sub-divided into four compartments by round-the-end baffles, and a relatively deep anaerobic chamber to afford ample sludge storage. The battery of discs, five dipping into the waste in each compartment, are spaced 15mm centres along the 7mm dia shaft which rotates 15mm above the water line in the aerobic chamber and is connected through a gearing system to the motor (Type CK. from Messrs. Edgecumbe Peebles Ltd. of Glasgow, 230 Volts, 50 cycles, 1 rpm).

The shaft is rotated at one revolution every two minutes. This results in the film of micro-organisms on each of the two faces of each disc being first immersed in the waste fluid in the aerobic chamber and next exposed to the atmosphere for aeration once every two minutes. Some of the organisms on the discs draw upon atmospheric oxygen on emergence from the waste fluid while others draw upon the oxygen dissolved in the waste fluid on immersion in it. Aerobic decomposition of the organic compounds results in the oxidation of some of the compounds into carbon dioxide, water, ammonia etc. and the synthesis of the others into new organisms which grow in the slime on the rotating discs. As new organisms grow some of the older ones fall off the discs and settle to the bottom of the waste in the anaerobic chamber to join the sludge already accumulated there and undergoing anaerobic digestion by faculative and anaerobic organisms.

As the film of micro-organisms grows thicker on each disc atmospheric oxygen finds it more and more difficult to penetrate the thickness to reach the layer nearest to the disc surface. This lack of oxygen leads to anaerobic decomposition resulting in the production of organic acids and alcohols at the disc-micro-organisms interface, a process which destroys the attachment of the micro-organisms to the disc surface. The whole mass of micro-organisms then sloughs off the disc and drops into the waste fluid to join the accumulated sludge at the bottom. The build-up of sludge is faster in the sloughing period. The sloughing period lasted from four to six days in the tests done for this thesis.

The total 'wetted' area exposed by the forty surfaces of the discs in this model is 6100cm^2. The area of the air-water interface above the aerobic chamber in the tank is 633cm^2. The twenty discs have therefore increased the available oxygen diffusion area 9.5 times.

In the unit just described the rotation of the discs is at right angles to the direction of flow of the waste fluid. There is also relatively large tank storage for anaerobic digestion of sloughed material from the biomass on the discs. In the units described by JOOST, R.H. (1969) and TORPEY, W.N. et al (1971) the discs rotate parallel with, and in the opposite direction to the direction of waste water flow, while there is no storage space for sludge digestion at the bottom. Indeed the bottom of the tank described by BORCHARDT, J.A. (1971) just clears the submerged edges of the discs to which it closely approximates in dimensions. This causes high enough local velocities to carry all sloughed solids out of the tank into the final clarifier which is an essential component of the unit where provision for sedimentation and anaerobic digestion is not made in the bottom storey of an Imhoff Tank. In either case the overall removal in the unit is a combination of more than one single operation: sedimentation, biological flocculation, aerobic decomposition and, in the case of the unit with the Imhoff Tank, anaerobic decomposition.

8.4 MODIFICATIONS IN FIELD UNITS AND PILOT PLANTS

Field units and pilot plants differ from laboratory scale models in respect of material, size, number and arrangement of discs, the speed of rotation, retention time and other design and constructional details dictated by the type of waste, loading and the degree of treatment desired.

Some of the disc materials reported in the literature are plastic (low density polysterene described by JOOST), aluminium (TORPEY et al), asbestos cement (SIDDIQI, R.H. 1971), and expanded metal (DOWNING, A.L. 1971). TORPEY'S discs were 0.91m dia, DOWNING'S 1m dia, SIDDIQI'S 2m dia while HARTMAN'S were 3m dia. DOWNING'S discs were rotated at 1 r.p.m.,HARTMAN'S at 2-3 r.p.m., SIDDIQI'S at 3.8 r.p.m. and TORPEY'S at 10 r.p.m.

The laboratory scale model shown in Fig. 8.2 is a single-shaft 4-stage unit as the discs rotate on a single shaft but in four compartments connected in series. Discs on field units are normally mounted on 2, 3, or 4 shafts rotating parallel with one another in one plane.

8.5 ADVANTAGES OF THE BIODISC PROCESS

HARTMAN (1960) claims the following advantages for the dipping filter process:

(a) extreme insensitiveness to fluctuations of the hydraulic loading and of sewage concentration;

(b) organic wastes are transformed into removable solid substances and can therefore be removed from the sewage;

(c) the process works practically without maintenance and is operationally absolutely reliable;

(d) any degree of purification is attainable, (partial or total);

(e) the over-loading capacity of the discs is unlimited, i.e. even when the hydraulic load is increased limitlessly there occurs no change-over into anaerobic conditions. Only the percentage reduction efficiency decreases slowly and continuously.

(f) the power and energy required are reduced to a minimum since the sewage flows through the installation almost without loss of head and no energy is required for the aeration of the sewage. To drive the discs only the bearing friction of the shaft has to be overcome, therefore the operating costs are below those of any other of the known biological purification processes;

(g) the cost of construction is considerably below that for a comparable low rate trickling filter and even below that for a high rate trickling filter where a pump is required to lift the sewage on top of the filter.

Hartman's claim of the non-existence of anaerobic conditions is probably an exaggeration as such conditions will exist when the slime becomes thick and diffusion of oxygen from the atmosphere does not reach the layer next to the disc face.

The advantages mentioned by BULUSU, K.R. (1967) include:

(a) achievement of a continuous solid contact surface at which the physiological activities of micro-organisms growing both at the surface and in the feed solution is furthered;

(b) the active surface film on the discs helps in concentrating and lifting the colloidal matter out of the waste fluid in which it would have remained in a fluid state;

(c) the biological film sloughs off the disc when the thickness becomes excessive;

(d) the use of many discs to obtain a total large surface area giving opportunity for a large number of micro-organisms to come into contact with the organic compounds in the liquid;

(e) the exposure of a known area of the disc surface for a known time makes the condition approach the Fick's Law of Diffusion;

(f) owing to gravity effects the wetted band of slime is flowing and so provides a mobile surface during the period of exposure which should be more effective than a stationary surface;

(g) as it is gravity that is predominant at the low speed of rotation, the film gets thicker towards the axis giving opportunity for new growth to take place in layers over the film before sloughing takes place.

In addition to the above advantages the laboratory model is a useful device in determining the treatability of wastes, as the model is little different in design and operation from an actual field plant.

In the next Chapter are described a number of tests done with the laboratory scale model described earlier in this chapter and shown in Fig. 8.2.

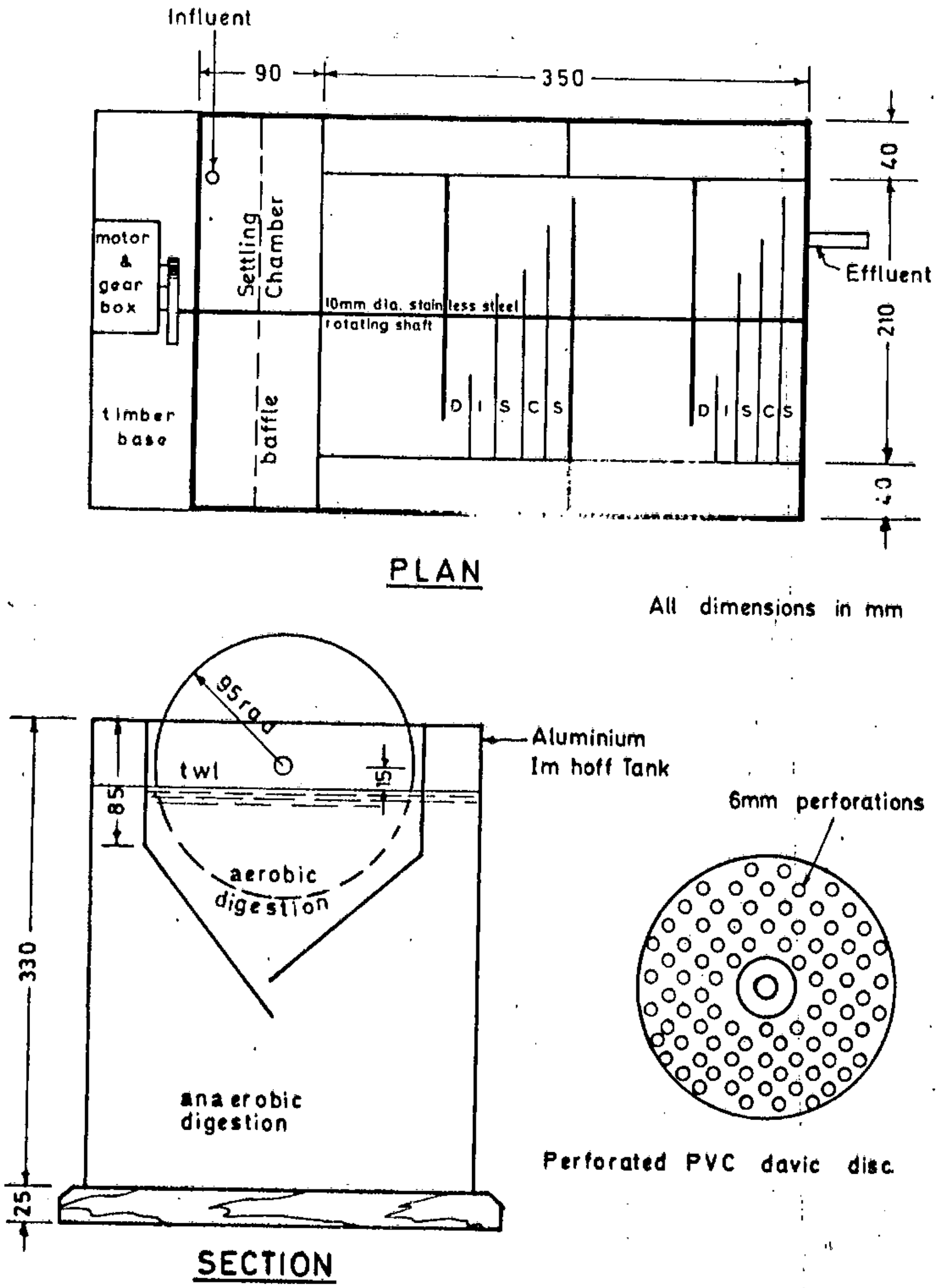


FIG. 8.2 LABORATORY SCALE MODEL OF BIODISC

CHAPTER IX

TESTS WITH THE BIODISC PROCESS IN THE TREATMENT OF WASTES

9.1 It was shown in Chapter III that the nightsoil conservancy system and the septic tank are the two methods of sewage disposal used most extensively in Lagos and other towns of Nigeria. The shortcomings of both of them were described in that chapter. In Chapter VII it was shown that the water table in the low lying areas of Lagos is too near ground level to make the septic tank soakaway effective in those areas. In Chapter VIII the biodisc process was discussed together with its merits over the more established methods of sewage treatment. The experiments reported in the present Chapter were designed to verify the efficiency of the biodisc process in the treatment of milk, domestic sewage, nightsoil and industrial wastes all of which were obtained in the Metropolitan Lagos Area. The results could indicate whether or not the process could be introduced in Lagos to overcome some of the shortcomings of the nightsoil conservancy system and the septic tank mentioned earlier.

More of these tests were done with milk than with the other wastes for the following reasons which the author adduced in an earlier work with the biodisc: milk is a balanced substrate with respect to the essential nutrients in it; it can be reproduced to a relatively high degree of consistency in its chemical composition; its preparation requires a relatively short time

(ALUKO, T.M. 1969). Marvel, Peak and Fussel's condensed milk were all tried at the beginning of the investigation. Of these, Fussel's condensed milk gave the best result because it did not clot. Table 9.1 shows the chemical composition of Fussel's milk (TURNBULL, A.J. 1969). Data on the chemical composition of the other brands of milk was not considered worth collecting since they were unsuitable and therefore not used in these tests.

The first three of the tests were conducted to determine some essential parameters for the main tests with the laboratory scale model biodisc unit. They involved the effective rate at which the waste flowed through the unit and the quality control of the feed in the tests with milk.

All the COD, BOD and suspended solids tests done in the tests reported in this Chapter were in accordance with Standard Methods (APHA, AWA, WPCF 1971).

9.1.1 DETERMINATION OF THE FLOWING-THROUGH PERIOD OF THE LABORATORY SCALE MODEL BIODISC

(a) Aim: Comparison of the theoretical Retention Time with the Flowing Through Period.

(b) Theory: The retention time of a treatment device is the time in which a given volume of the waste is resident in the device. It is equal to the volume of the device divided by the flow. There is usually short-circuiting in the flow between the influent point and the effluent point which results in the actual flowing through period being usually shorter than the theoretical retention time.

TABLE 9.1

CHEMICAL COMPOSITION OF FUSSELS CONDENSED MILK

CONSTITUENT	% BY WEIGHT
PROTEIN	10.2
SUCROSE	44.0
LACTOSE	14.8
MINERALS	2.3
FAT	0.2
MOISTURE	28.5

(Compiled from figures obtained from
A.J. Turnbull in personal communication, July 1969)

There is in this case a device which has both an aerobic chamber and an Imhoff tank. While a substantial amount of the flow would be expected to be confined to the aerobic Chamber in which both the influent and the effluent points are situated some of the flow would go through the Imhoff tank. Most of the water in the tank however would be expected to be dead water. The test would indicate how much of the flow goes through the aerobic chamber.

COX, C.R. (1964) described a method of determining the flowing-through period of a basin which consisted in the rapid application for one minute of a concentrated sodium chloride solution to the influent and the determination of the concentration of sodium chloride in the effluent at frequent intervals. The sodium chloride concentration was plotted against time and the centre of area determined. The distance of the centre of area from the y-axis represented the flowing-through period as shown in Fig. 9.1

The specific gravity of a 1% solution of sodium chloride is about 1.0071, which is higher than that of water. If such a solution was used in the flowing-through test some of the sodium chloride solution would tend to sink to the bottom of the tank instead of flowing-through normally. This could have an adverse effect on the test. It was therefore

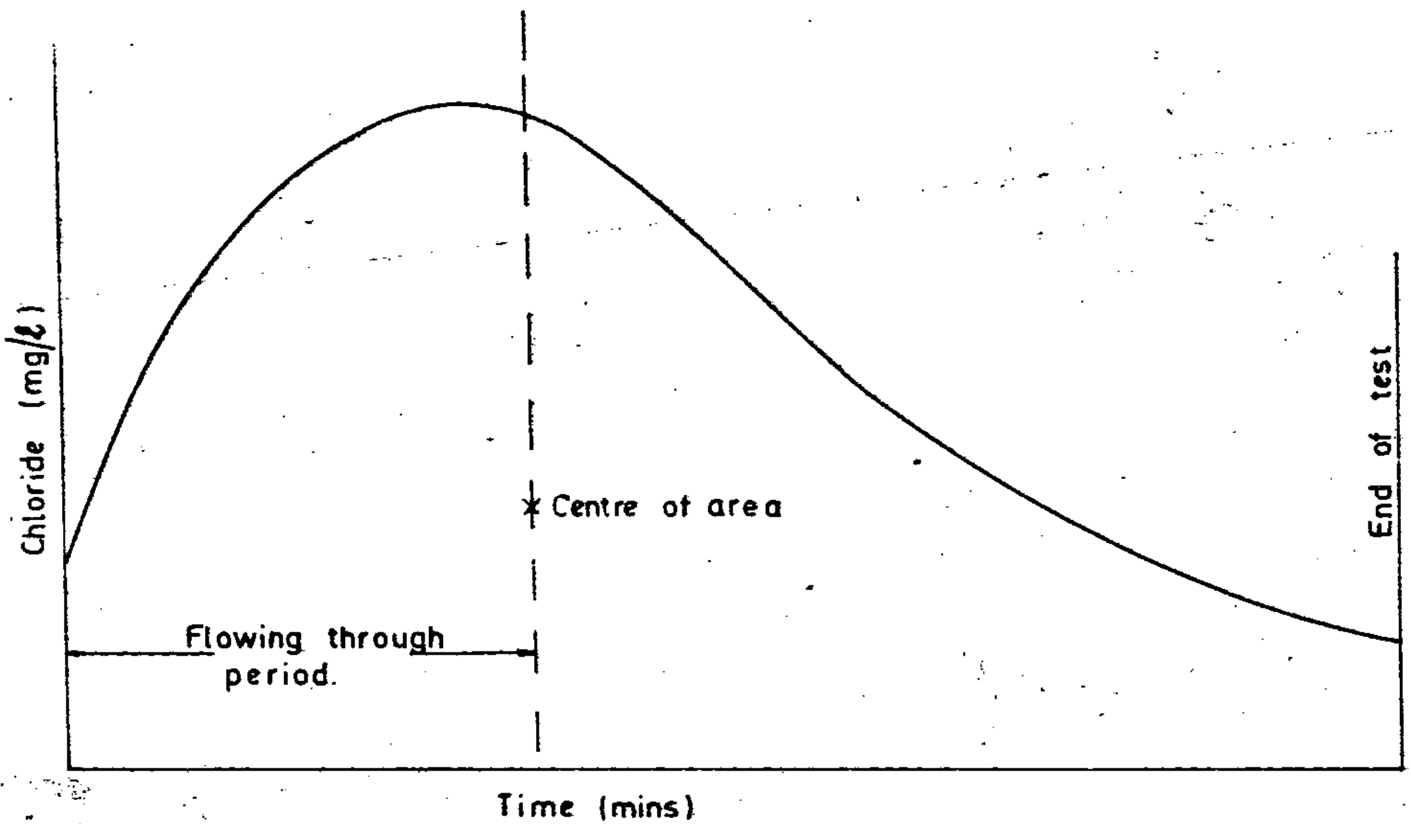


FIG.9·1 SODIUM CHLORIDE SOLUTION METHOD FOR DETERMINATION OF FLOWING THROUGH PERIOD IN SEDIMENTATION TANK.

decided to use ammonium chloride, which is a lighter salt with a specific gravity of 1.0031. It was also known t sugar dissolved in water has a slightly higher specific gravity than water. A solution of 26mg/litre of sugar in the tank and another of 25mg/litre of ammonium chloride in the influent had approximately the same specific gravity of 1.00. If therefore a 25mg/litre solution of ammonium chloride was run into a 26mg/litre solution of sugar in the tank the tendency for the chloride solution to sink in the solution in the tank would be eliminated. Finally it was decided to measure the specific conductance of the effluent samples instead of the concentration as it was found that this was a much more rapid and convenient way of determining the salt content since the concentration and the specific conductance are known to have linear relationship over the short range of concentrations involved.

(c) <u>Method:</u> The 25mg/litre solution of ammonium chloride was run from a 10 litre aspirator into the 26mg/litre solution of sugar in the biodisc at a controlled speed of 120ml/min. After 1 minute the chloride solution was cut off and was simultaneously replaced with the 26mg/litre sugar solution running at the same controlled speed of 120ml/min. from a 25 litre aspirator. Effluent samples were taken every 5 minutes and the specific conductance determined with the conductivity meter, correction being made for temperature at each reading. The test lasted 450 minutes at the end of which the specific conductance had fallen from a maximum of 370 mhos to the original base value of 161 mhos in the sugar solution (Fig. 9.2). The distance from the vertical axis of the centre of area of the area between the curve and the horizontal line showing the base specific conductance of the sugar solution gave the flowing-through period.

(d) Results: The results are shown in Table 9.2 and Fig. 9.2. The flowing-through period for this flow of 120ml/min. as determined from the graph in Fig. 9.2 was 80 minutes. The theoretical retention period is obtained as follows.

Gross volume of aerobic chamber = 10,106cm^3.

(a) Case 1, Test without discs:

Volume of chamber = 10106cm^3.

Volume of 4 baffles = 260cm^3

Net volume of chamber = 9846cm^3

Retention Time = $\frac{9846}{120}$ = 82.1 mins.

(b) Case 2, Test with clean discs:

Volume of chamber = 10106cm^3

Volume of 20 clean discs and 4baffles = 1790cm^3

Net volume of chamber = 8316cm^3

Retention Time = $\frac{8316}{120}$ = 69.3 mins.

TABLE 9.2

VARIATION OF SPECIFIC CONDUCTANCE WITH TIME IN 1% NH_4Cl SOLUTION IN DETERMINATION OF FLOWING-THROUGH PERIOD

Time (mins)	Meter Reading	Temp. (°C)	Temp. factor	Corrected Reading	Specific Conductance (μmhos)
0	117	26	0.98	115	161
1	117	26	0.98	115	161
5	116	26	0.98	114	160
10	199	26	0.98	197	276
15	257	26	0.98	252	353
20	270	26	0.98	266	372
22	260	26	0.98	255	357
25	260	26	0.98	255	357
30	256	26	0.98	251	351
35	256	26	0.98	251	351
45	235	26	0.98	230	322
50	225	26	0.98	220	308
55	218	26	0.98	214	300
60	215	26	0.98	210	294
65	207	26	0.98	203	284
70	203	26	0.98	199	278.6
75	195	25.5	0.99	193	270.2
80	192	25.5	0.99	190	266
85	190	25.5	0.99	188	263.2
90	188	25.5	"	186	260.4
95	185	25.5	"	183	256.2
100	184	25.5	0.99	182	254.8
105	180	25.5	0.99	178	249.2
110	177	25.5	0.99	175	245
115	177	25.5	0.99	175	245
120	172	25.5	0.99	170	238
125	170	25.5	0.99	168	235.2
130	168	25.5	0.99	166	232.4
135	166	25.5	0.99	164	229.6
140	165	25.5	0.99	163	228.2
145	164	25.5	0.99	162	226.8
150	162	25.5	0.99	160	224.0
155	160	25.5	0.99	159	222.6

Contd.

TABLE 9.2 CONTD.

Time (mins)	Meter Reading	Temp. (^{o}C)	Temp. factor	Corrected Reading	Specific Conductance (µmhos)
160	155	25.0	1.0	155	217.0
170	152	25.0	1.0	152	212.8
180	150	25.0	1.0	150	210.0
190	147	25.5	0.99	146	204.4
200	144	25.5	0.99	143	200.2
210	143	25.5	0.99	142	198.8
220	140	25.5	0.99	139	194.6
230	138	25.5	0.99	137	191.8
250	136	25.5	0.99	135	189.0
270	133	25.5	0.99	132	184.8
290	129	25.5	0.99	128	179.2
310	127	25.5	0.99	126	176.4
330	125	25.5	0.99	124	173.6
350	123	25.5	0.99	122	170.8
370	120	25.5	0.99	119	166.6
390	119	25.5	0.99	118	165.2
410	117	25.5	0.99	116	162.4
430	117	25.5	0.99	116	162.4
450	117	25.5	0.99	116	162.4

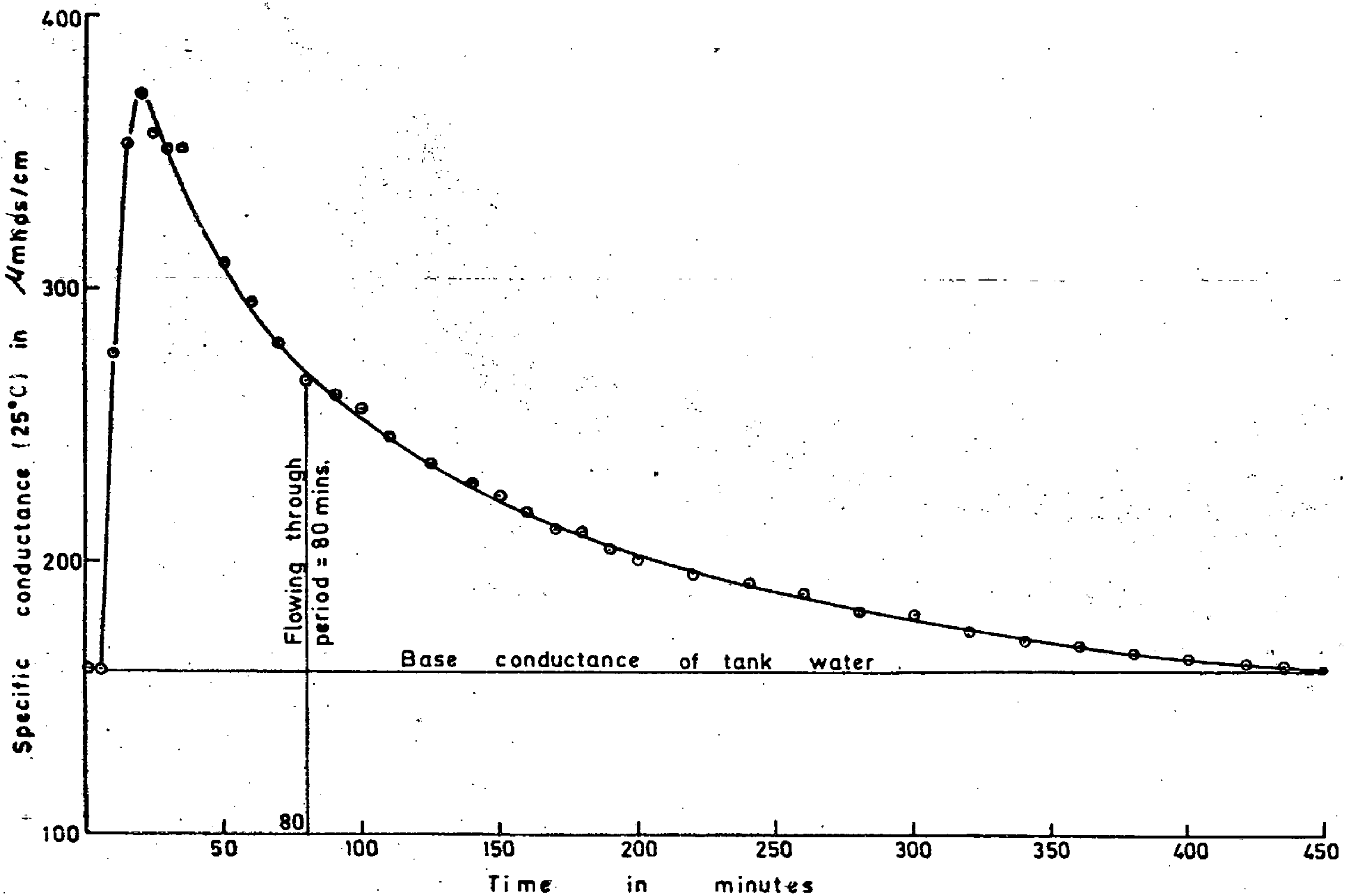


FIG. 9·2 DETERMINATION OF FLOWING THROUGH PERIOD OF BIODISC BY SPECIFIC CONDUCTANCE METHOD

(c) Case 3, Test with discs covered with 0.15cm thick growth either side:

Volume of chamber = $10106cm^3$

Volume of 20 covered discs and 4 baffles = $3320cm^3$

Net volume of chamber = $6786cm^3$

Retention Time = $\frac{6786}{120}$ = 56.6 mins.

Case 1 retention time of 82.1 minutes is very close to the experimentally determined flowing-through period of 80 minutes for this flow of 120ml/min. The fact that the flowing-through period is slightly smaller than the retention time is accounted for by dead pockets of water in the aerobic chamber. This case applies only to Test 9.2.7 in which discs were removed from model B to observe the effect of the plant running without discs . No plant would be run without discs in practice.

Case 2 represents the situation at the beginning of each test in which the discs are clean while Case 3 represents the situation when the discs are already covered with growth. The tests were run in all cases but one at a flow of 14ml/min. At this flow retention times for Cases 1, 2 and 3 were 11.73 hours, 9.90 and 8.09 hours respectively. The average retention time for a test starting with clean discs and finishing with discs fully covered with growth is the average between the last 2 figures, i.e. 9.0 hours.

This compares well with the retention time of 8 - 12 hours for the conventional activated sludge system (MARA, D. 1976).

9.2 TESTS WITH MILK

9.2.1 DETERMINATION OF THE QUANTITY OF SODIUM BICARBONATE REQUIRED FOR pH CONTROL IN THE MILK SUBSTRATE AT DIFFERENT CONCENTRATIONS:

(a) Object: It was observed early in the tests with milk that even though the pH of the milk solution varied between 7.2 and 7.6 at preparation the pH dropped to values which sometimes were as low as 4.6 after three or four days. The effluent also showed similar drop in pH. These observations are consistent with the formation of organic acids during the decomposition of the milk. As these pH values were considerably below the optimum of 7.0 $\pm$ 0.5 at which micro-organisms were most effective in the metabolism of organic wastes (OGINSKY, E.L. and UMBREIT, W.W. 1959) it was decided to use sodium bicarbonate to compensate for the lowering observed in the pH. The object of this test therefore was to find the quantities of sodium bicarbonate required to add to a 25 litre aspirator of milk substrate at different concentrations so that the final pH of the influent remained close to neutral even after a few days storage.

(b) Method: A standard stock solution of 1 tin of milk made up with tap water to 2 litres was made as described in Test 9.2.3.

Different quantities of this solution as shown in the table below were placed in eleven clean 250ml flasks. The contents of each flask were made up with tap water to the 250ml mark. The diluted solution was next transferred in each case into a 500ml beaker. It was then titrated with 0.1N sodium bicarbonate solution run in slowly from a 50ml burette while the solution was kept constantly stirred by a magnetic stirrer. Phenolpthalein indicator was used to determine the end point in each titration.

Flask	1	2	3	4	5	6	7	8	9	10	11
Vol. of stock solution from 2 litre jar (ml)	0	2	4	6	8	10	12	14	16	18	20
Add tap water to make volume, 250 ml.											
Vol. of 0.1N sodium bicarbonate added (ml)											

(c) Results: Table 9.3 and Fig. 9.3 show the quantities of the 0.1N sodium bicarbonate solution for the eleven different milk concentrations. They also show the weight of sodium bicarbonate required for different volumes of the standard stock solution in the feed in a 25 litre aspirator.

TABLE 9.3

QUANTITY OF $NaHCO_3$ BUFFER REQUIRED FOR DIFFERENT CONCENTRATIONS OF MILK FEED

Vol. of Stock Solution Milk in Sample (ml)	Initial pH	Titration with .1N $NaHCO_3$ (ml)	Final pH	Adjusted Titration (ml)	Wt. of $NaHCO_3$ in 25 litre Feed (gm)
1	2	3	4	5	6
0	7.40	3.50	8.00	3.50	2.940
2	7.40	7.10	8.00	6.40	5.377
4	7.40	10.50	8.00	9.80	8.233
6	7.40	12.00	8.00	13.60	11.426
8	7.40	16.40	8.05	18.00	15.122
10	7.40	23.50	8.00	22.90	19.238
12	7.35	26.30	8.00	27.80	23.355
14	7.35	32.70	8.00	32.70	27.471
16	7.35	42.00	8.00	37.20	31.252
18	7.35	43.30	8.00	41.00	34.444
20	7.30	44.35	8.00	44.35	37.258

(a) Column 5 is obtained from graph in Fig. 9.3

(b) Column 6 is obtained by multiplying column 3 by 0.8401 (.1N $NaHCO_3$ = 8.401 gm/l).

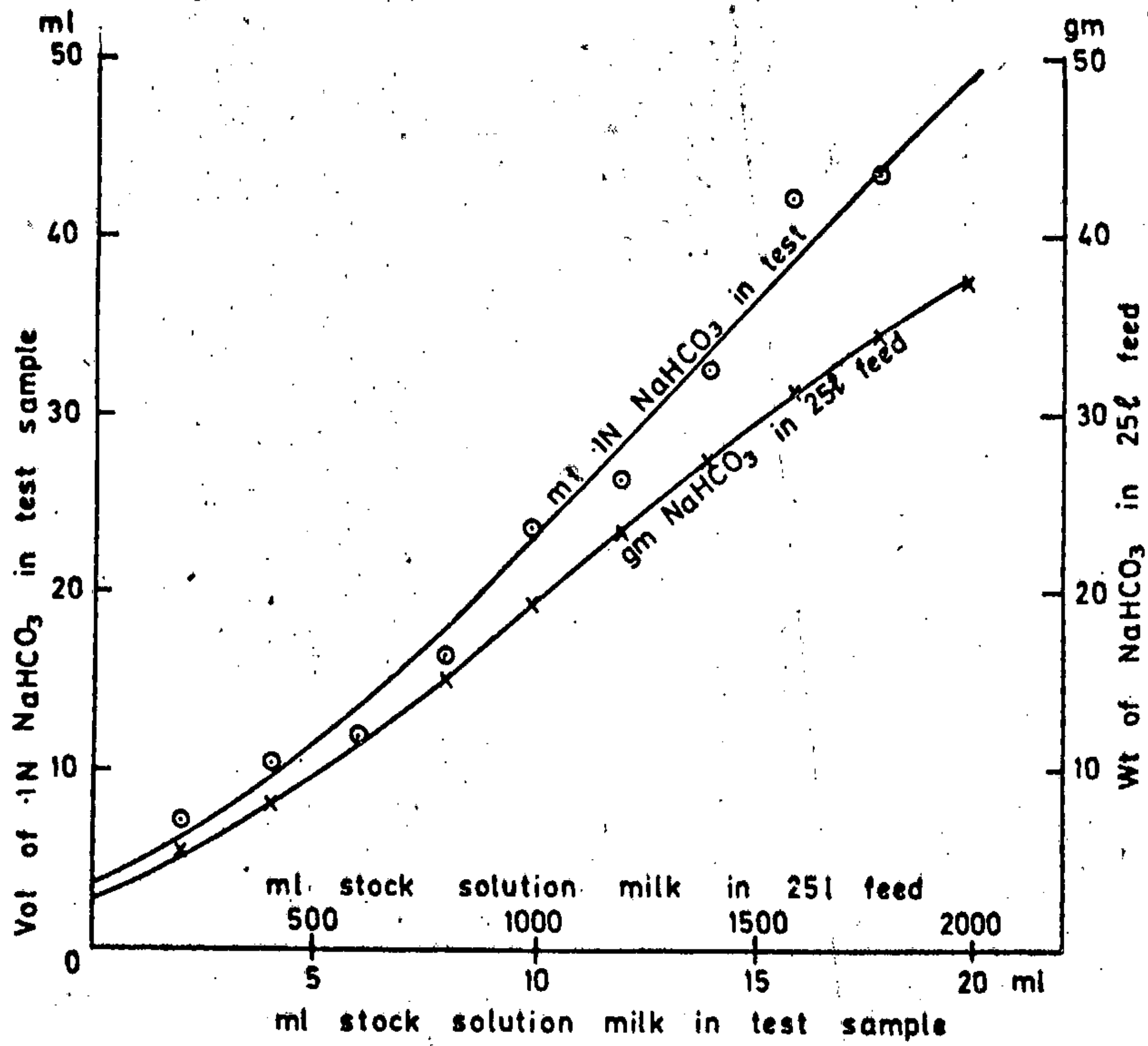


FIG. 9·3 QUANTITY OF $NaHCO_3$ REQUIRED FOR DIFFERENT CONCENTRATIONS OF MILK FEED

9.2.2 QUALITY CONTROL TEST: INVESTIGATION OF VARIATION OF COD AND pH WITH LENGTH OF TIME OF FEED STORAGE

Because of the low flow rate, the first few tests were done on milk solution that had stayed in the aspirator for sometimes as long as 3 days. It had been observed that even though the feed concentration had remained constant from one preparation to another the influent COD and pH appeared to vary with the length of time that the milk solution had been in storage. The object of this test was to determine the nature of this variation.

(a) Method: A 250ml volume of the standard stock solution of 1 tin of milk made up with tap water to 2 litre as described later in Test 9.2.3 below was placed in a clean 10 litre aspirator. The weight of sodium bicarbonate required to compensate for pH drop in this quantity of milk feed at this particular concentration was 2.4gm as determined from Fig. 9.3. This quantity of bicarbonate in solution was added. The contents of the aspirator were then stirred while tap water was run in to bring the whole to the 10 litre mark. The aspirator was kept on the bench in the laboratory for nine days. 100ml samples of this solution were withdrawn by pipette at intervals of 0, 1, 6, 10, 24 hours; 2, 3, 4, 5, 6, 7, 8 and 9 days. The COD Test was performed on a 2ml portion of each of these samples while the pH of the remaining portion of each sample was determined on the Pye Unicam pH meter (model 292).

(b) Results: The results are shown in Fig. 9.4. The COD rose rapidly from an initial value of 3200 mg/l to 3600mg/l in 3 hours. It then dropped to 3000mg/l in the next 7 hours, and to 2300mg/l in the next 14 hours i.e. 24 hours after the test was started. It remained approximately constant at 2300mg/l for most of the remaining 8 days of the test. The pH dropped fairly sharply from the initial 8.4 to 6.3 in 24 hours. It then rose fairly uniformly back to 8.4 during the remaining 8 days of the test.

The fall in both the COD and the pH in the first 24 hours of the test was due to the fermentation of some of the organic compounds in the milk solution resulting in the formation of organic acids and alcohols. The organic acids depressed the pH to the low value of 6.3. The subsequent rise in pH is considered due to the cessation nof the formation of the acids and the subsequent digestion of these acids, together with the possible formation of some ammonium bicarbonate in accordance with equations 9.1 and 9.2 developed later in this Chapter in Section 9.2.10. The bicarbonate raised the pH of the solution. The initial rise in the COD immediately after the preparation of the feed needs further study.

It was concluded from this test that since the substrate COD rose and fell sharply in the first few hours of preparation COD tests should not be performed within the first 24 hours of the preparation of a new feed. As the COD became steady after 2 days it was concluded that it was ideal to do COD tests on feeds that had been in storage from 2 to 4 days. The pH at this time was considered not too low to have an adverse

Time	hours					days							
	1	3	6	10	24	2	3	4	5	6	7	8	9
COD (mg/l)	3200	3600	2920	3020	2440	2320	2240	2280	2280	2540	2320	2380	2180
pH	8·4	8·4	8·4	8·3	6·3	6·4	6·5	7·2	7·2	7·5	7·9	8·1	8·4

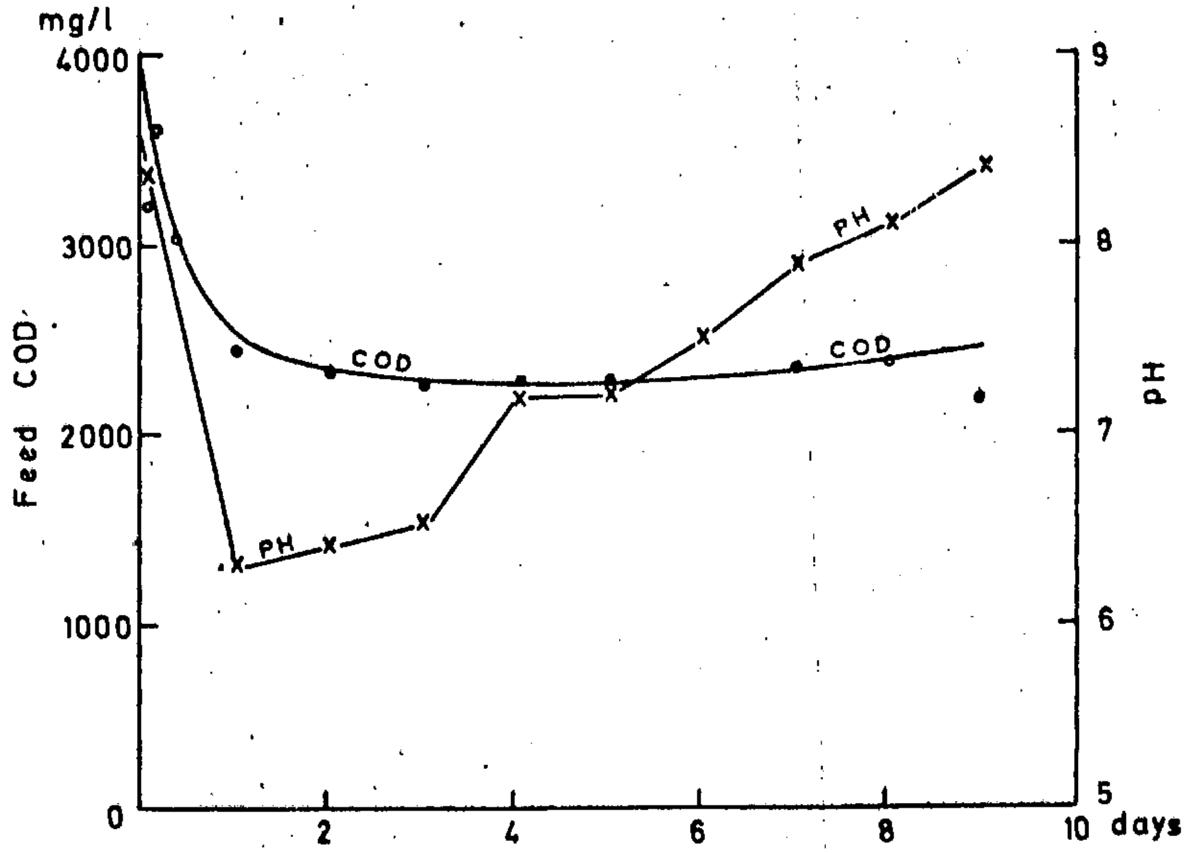


FIG. 9·4 VARIATION OF FEED QUALITY WITH STORAGE effect on the effluent pH.

9.2.3 PREPARATION OF STOCK SOLUTION OF MILK SUBSTRATE

(a). The object was to achieve a thorough mixing and a fair control of the concentration of the milk solution at each preparation of the substrate.

(b). The contents of each tin of milk were tipped into a clean plastic 1 litre beaker. Tap water under pressure was used to flush into the beaker all milk remnants inside the tin as well as on the lid. The contents of the beaker were thoroughly mixed with a clean glass stirrer. They were then transferred into a clean 2 litre pyrex measuring cylinder. The beaker and the stirrer were rinsed once. All this ensured that the entire milk contents of the tin were transferred into the measuring cylinder. The solution in the cylinder was then made up to the 2 litre graduation mark with tap water and again thoroughly mixed. From this stock solution of 1 tin of milk in a 2 litre solution would be drawn, at each feed preparation, the pre-determined volume which would be made up to 10 litres in Test 9.2.2 or to 25 litres in tests 9.2.4 - 9.2.8 described immediately below.

In each of the following tests the feed was prepared by placing in a 25 litre aspirator the pre-determined quantity of this stock solution required to give the desired concentration of substrate. The appropriate quantity of sodium bicarbonate as determined from Fig. 9.3 was added to this. Tap water was run in to bring the solution to the 25 litre mark, the contents being stirred with a clean stick 1.8m long to ensure good mixing. The solution was then made to drip through a 4mm bore plastic tube about 2m long on to the first disc in the influent compartment of the biodisc. The effluent was disposed as waste in a sink, but where it was required for re-use in Test 9.2.6 it was collected in another 25 litre aspirator. Fig. 9.5 is a flow diagram in this and the other tests with the laboratory scale model of the biodisc.

COD tests were performed on influent and effluent samples and, where required, on samples taken from the three intermediate points shown in Fig. 9.6. Suspended solids tests were omitted as milk is a colloidal waste in which suspended solids in appreciable quantities could not be expected as compared with the other wastes dealt with later in this chapter.

In view of the result of Test 9.2.2 which showed that the feed was not stable in the first 24 hours after preparation only samples from solutions that had stayed in storage up to 2 but not longer than 4 days were used in these tests.

The flow was kept constant at 14ml/min. The room temperature was most of the time between 19°C and 20°C.

For each test influent and effluent samples were taken at the same time. The difference in the COD of the samples was assumed to be a measure of the amount of treatment that had taken place in the solution in its passage from the influent to the effluent point. This assumption is not quite correct as each effluent sample originated from another influent sample older than the one then under consideration by the retention time of the device. The more correct procedure would be to take an effluent sample at a time equal to the retention time after the time the influent sample was

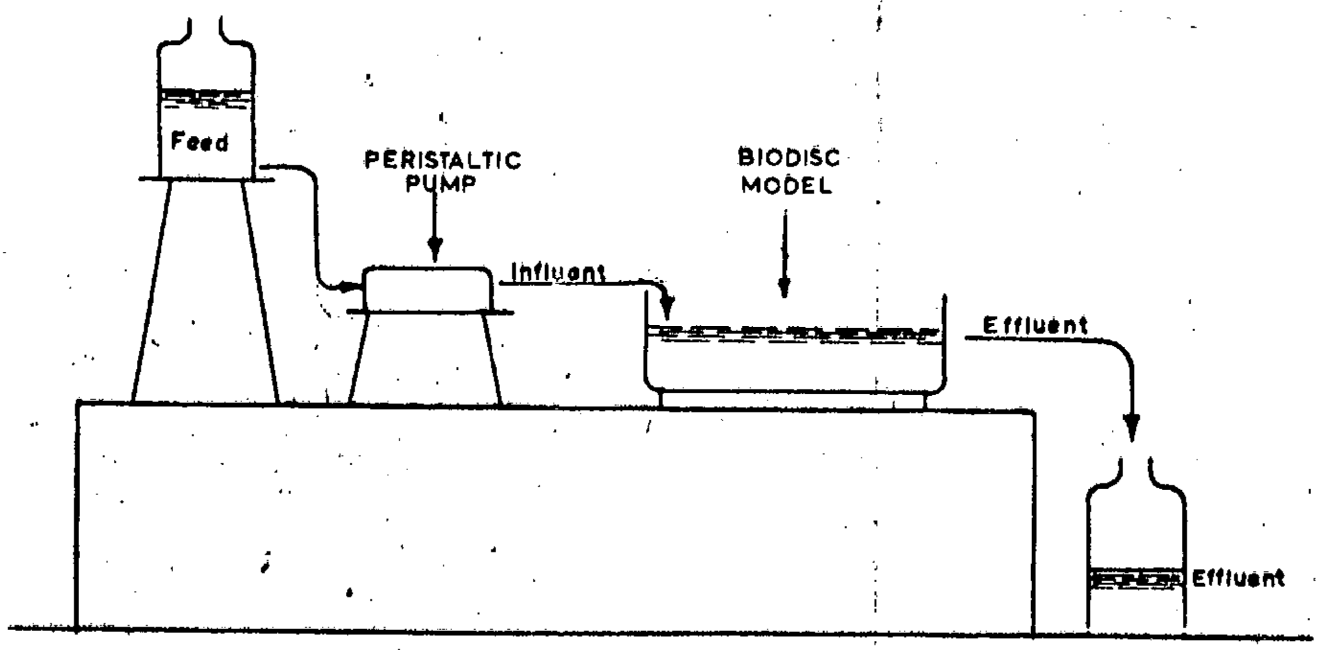


FIG. 9·5 LABORATORY ARRANGEMENT OF BIODISC PROCESS

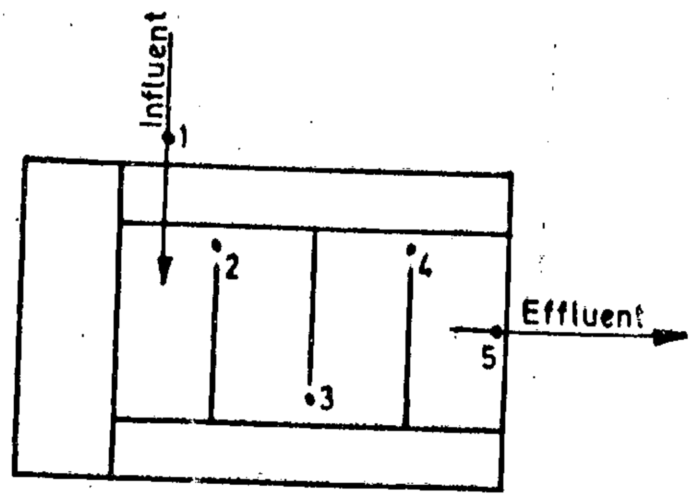


FIG. 9·6 SAMPLE POINTS FOR TREATMENT PROFILE TEST taken. The method used in these tests is, however, often used in waste-water treatment practice.

9.2.4 DETERMINATION OF THE TREATMENT EFFICIENCY AND TREATMENT PROFILE OF THE BIODISC ON A MILK SUBSTRATE:

The object of this test was to determine both the treatment efficiency and the treatment profile in the passage of a milk substrate through the four compartments of the Biodisc.

A concentration of 600ml of the stock solution made up to 25 litres as described earlier in 9.2.3 was used in this test. After the biodisc has been running for 7 days and the discs covered with growth, COD tests were performed on influent and effluent samples as well as on additional samples taken from points 2, 3, and 4 in Fig. 9.6. The results are shown in Table 9.4 and Fig. 9.7.

9.2.5 DETERMINATION OF THE VARIATION IN THE TREATMENT EFFICIENCY OF THE BIODISC ON TWO CONSECUTIVE DAYS:

The procedure in this test was exactly as in Test 9.2.4 described earlier. COD tests were however performed on two different sets of samples taken from the same feed at 24 hours interval. The first tests were done after the biodisc had been running for 9 days and the second after it had been running for 10 days. The results are shown in Table 9.5 and Fig. 9.8.

9.2.6 THE PERFORMANCE OF THE BIODISC ON RECIRCULATED EFFLUENT

The object of this test was to find out if the Biodisc could treat further a waste that has been treated once by the device.

A 25 litre quantity of effluent from a first passage of the feed through the device was transferred into the 25 litre influent aspirator and was used as

TABLE 9.4

TREATMENT EFFICIENCY AND TREATMENT PROFILE OF BIODISC ON MILK SUBSTRATE

Sample Point	C O D		Cumulative
	COD at Point (mg/1)	COD Reduction (mg/1)	% COD Reduction
1	2	3	4
(Influent)	3380	-	-
2	1152	2228	65.9%
3	903	2477	73.3%
4	595	2785	82.4%
5 (Effluent)	587 −	2793 +	82.6% +

(a) Table shows a progressive decrease in the COD and corresponding increase in COD reduction along length of biodisc.

(b) Column (4) is obtained by dividing the appropriate COD reduction in column (3) by 3380 the influent COD and multiplying by 100.

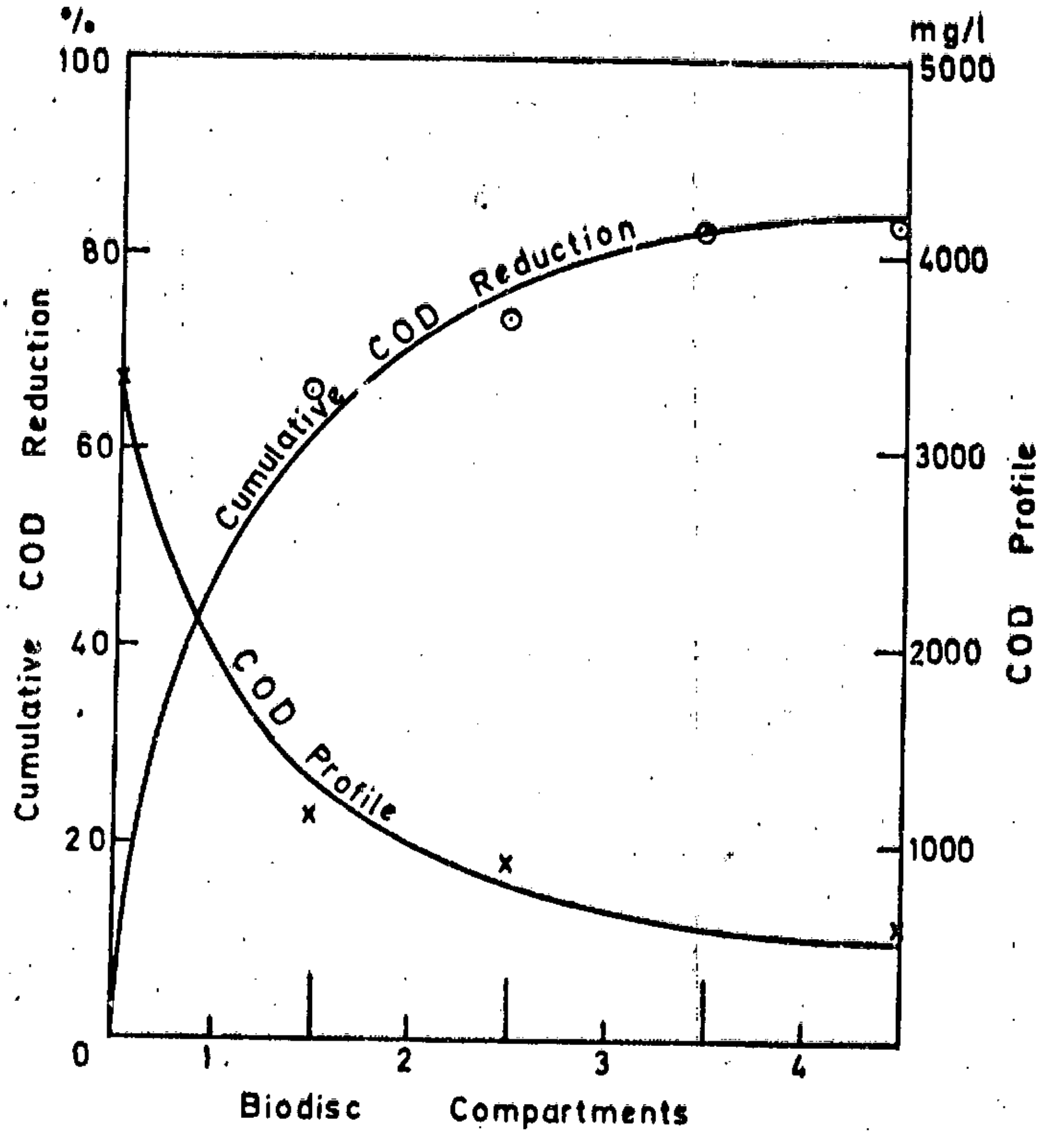


FIG.9·7 TREATMENT EFFICIENCY AND TREATMENT PROFILE IN BIODISC

TABLE 9.5

PERFORMANCE OF BIODISC ON SAME FEED ON 2 CONSECUTIVE DAYS

Sample Point (Fig. 9.2)	7/10/74			8/10/74		
	COD (mg/1)	COD Reduction (mg/1)	Cumulative % COD Reduction	COD (mg/1)	COD Reduction (mg/1)	Cumulative % COD Reduction
1	5660			6053		
2	2180	3480	61.5	2560	3493	57.8
3	2050	3610	63.7	2384	3669	60.5
4	1960	3700	65.2	2333	3720	61.5
5	1830	3830	67.6	2310	3743	61.9

There is difference in the performance of the Biodisc on the same feed within 24 hours.

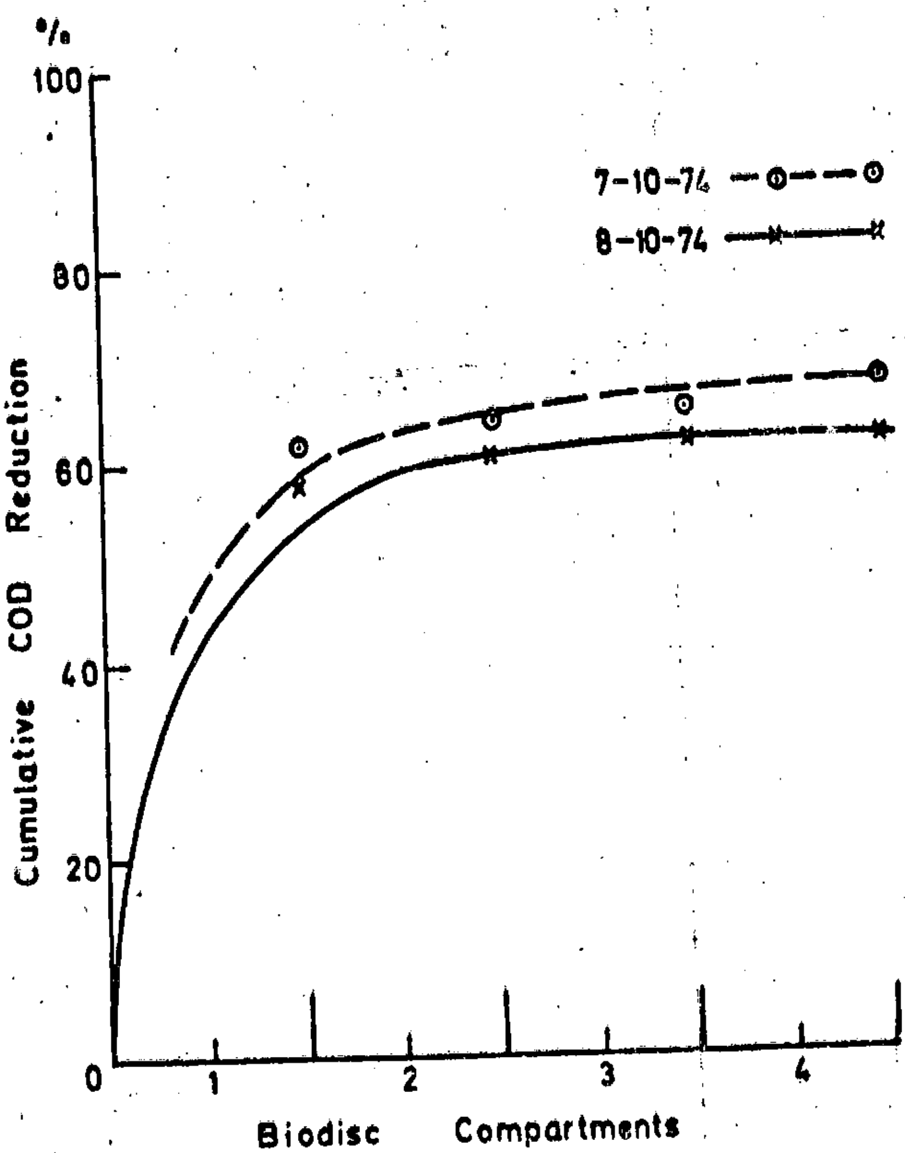


FIG. 9.8 PERFORMANCE OF BIODISC ON SAME FEED ON 2 CONSECUTIVE DAYS

the new feed in the Biodisc. It was necessary to store in the first instance sufficient quantities of the effluent to ensure that quantities for topping up the new feed came from the same stock and were therefore of the same characteristics.

COD tests were performed on the following four different dates: 24/8/70, 29/8/70, 24/9/70 and 13/1/71. The test on 24/8/70 was performed both on samples taken from the three intermediate points as described in Test 9.2.4 as well as on influent and effluent samples. The test on each of the other three occasions was limited to influent and effluent samples. All the tests were done with the biodisc that had been running for several days and the discs covered with growth. The results of the tests are shown in Tables 9.6 and 9.7.

9.2.7 ASSESSMENT OF THE PERCENTAGE TREATMENT ATTRIBUTABLE TO THE ORGANISMS ON THE DISCS:

The object of this test was to obtain an idea of what percentage of the overall treatment was attributable to the organisms on the revolving discs in the Biodisc.

Two Biodisc models A and B of identical construction were used in this test which ran for 32 days. Both A and B received at the same rate feed from the same 25 litre aspirator. The concentration of the feed was 600 ml. of the stock solution in 25 litre. Both A and B were started the same time with clean discs rotating at the normal speed of ½ revolution per minute. On day 4 discs were removed from B which tontinued for 16 days without them.

TABLE 9.6

PERFORMANCE OF BIODISC ON RECIRCULATED MILK EFFLUENT ON 4 DIFFERENT DAYS

TEST NO.	9.3.1.1	9.3.1.2	9.3.1.3	9.3.1.4
DATE	24/8/70	29/8/70	24/9/70	13/1/71
Influent COD* (mg/l)	1450	1140	820	430
Final effluent COD(mg/l)	850	770	540	250
COD Reduction in Final effluent (mg/l)	600	370	280	180
% COD Reduction in Final effluent.	41.5%	32.5%	34.2%	42.0%

* Influent in these tests was recirculated effluent.

TABLE 9.7

TREATMENT PROFILE OF BIODISC ON RECIRCULATED MILK EFFLUENT ON 24/8/70

Sample Point	COD		Cumulative
	COD at Point (mg/1)	Cumulative COD Reduction (mg/1)	% COD Reduction
1 (Influent)	1450	-	-
2	950	500	34.5
3	900	550	38.0
4	?	?	?
(Effluent) 5	850	600	41.5

Table shows a progressive decrease in COD and correspondingly progressive increase in COD reduction along length of biodisc.

COD tests were performed on influent and effluent samples from each biodisc about twice a week. On day 20 clean discs were restored in B so that for the last 13 days of the test both biodiscs ran normally with discs. The results are shown in Table 9.8 and Fig. 9.9.

9.2.8 VARIATION IN THE PERFORMANCE OF THE BIODISC WITH TIME

The object of this test was to study the variation of treatment efficiency with time in the biodisc. The biodisc was started with clean discs and run for several weeks on the same waste and at the same concentration till sloughing occurred in the biomass on the discs. COD tests were done approximately every other day throughout the run.

In the first run sloughing started on most of the discs on the 40th day and was complete on the 45th day, when the test was discontinued. After a 4 months' gap the second run was started, greater attention being paid to pH control this time. While sloughing started on some of the discs on the 55th day and was complete on 18 of them by the 60th day there was no sloughing at all on the last two discs till the test was discontinued on the 72nd day. At this time new growths had appeared on most of the other discs. The results are shown in Tables 9.9 and 9.10 as well as in Figs. 9.10 and 9.11.

9.2.9. COD - BOD CORRELATION TESTS:

It is known that there is a simple relationship between the COD and the BOD of most wastes. The COD (Dichromate Value) Test is very much quicker, and possesses a higher degree of accuracy than the standard

TABLE 9.8

ASSESSMENT OF PERCENTAGE TREATMENT ATRIBUTABLE TO ORGANISMS ON DISCS

Date	Days	C O D IN A			C O D IN B		
		Inflt. (mg/1)	Efflt. (mg/1)	% Reduction	Inflt. (mg/1)	Efflt. (mg/1)	% Reduction
Sept. 29, '71	6	5800	850	85.3	5900	1900	67.8*
Oct. 2,	9	4500	1200	73.4	4500	2350	47.8
6,	12	4200	1000	76.2	4500	2550	43.3
9,	15	3800	700	81.6	3900	2550	34.6
13,	19	4900	750	84.7	5600?	2900	40.7
16,	22	5800	1100	81.0	5700	2200	61.4**
20,	26	5100	300	94.1	5100	1000	80.4
23,	29	3900	650	83.4	4000	1150	71.2
25,	32	3400	1200	64.6	3300	1300	60.5

* Discs in Biodisc B were removed on Day 6.

** Discs were replaced in Biodisc B on Day 20.

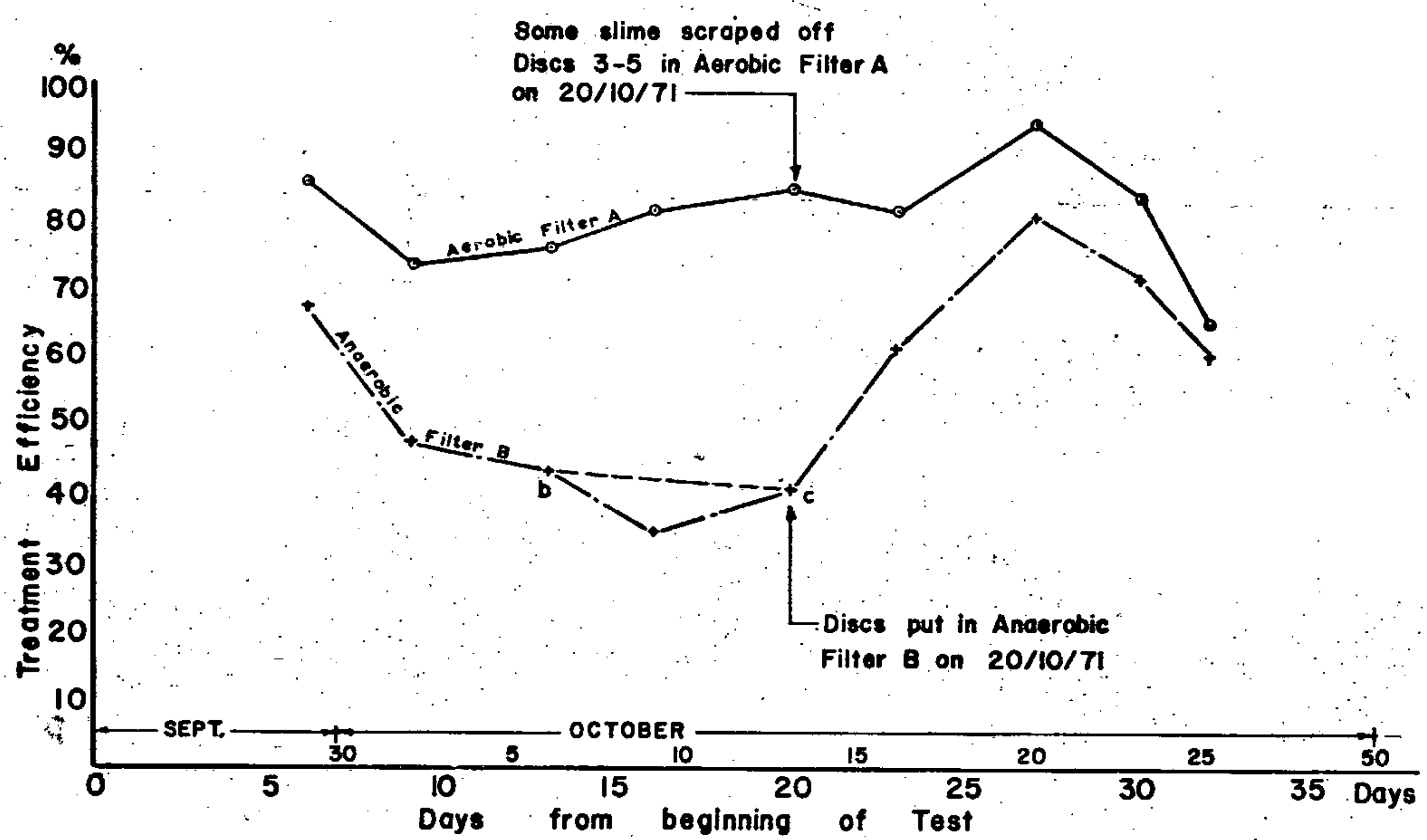


FIG. 9·9 ASSESSMENT OF PERCENTAGE TREATMENT ATTRIBUTABLE TO DISCS.

TABLE 9.9

PERFORMANCE OF BIODISC ON MILK SUSTRATE FOR A PERIOD OF 45 DAYS

Date (1971)	Days from beginning	COD Influent (mg/1)	COD Effluent (mg/1)	COD Reduction (mg/1)	COD Reduction (%)
May 20	4	2150	1125	1025	48
22	6	2250	675	1575	70
24	8	1775	613	1162	65
25	9	1800	576	1224	68
27	11	2100	425	1675	80*
29	13	2100	863	1237	59
30	14	2900	1213	1687	58
31	15	2750	1213	1537	56
June 2	17	1975	875	1100	56
3	18	1925	725	1200	62
5	20	2950	400	2550	86*
6	21	2450	900	1550	63
9	24	1400	750	650	46
10	25	1200	375	825	69
12	27	1450	375	1075	74
14	29	2500	400	2100	84
15	30	2200	500	1700	77
17	32	2250	675	1575	70
19	34	1650	500	1150	70
21	36	1800	500	1300	72
24	39	1850	250	1600	86
25	40	1850	150	1700	92
28	43	1825	225	1600	88
30	45	1750	675	1075	61

* Feed flow stopped overnight.

TABLE 9.10

PERFORMANCE OF BIODISC ON MILK SUBSTRATE FOR A PERIOD OF 72 DAYS

Date (1971)	Days from beginning	C O D Influent (mg/1)	C O D Effluent (mg/1)	C O D Reduction (mg/1)	C O D Reduction %	pH Influent	pH Effluent
Nov. 11	4	2480	1080	1400	56.5	7.6	8.8
13	6	2540	760	1780	70.1	5.8	8.7
14	7	3240	1000	2400	69.1	5.9	8.7
16	9	2600	670	1930	74.2	6.7	8.7
18	11	1900	530	1370	72.1	?	?
19	12	1650	400	1250	75.8	7.3	8.5
22	15	1670	390	1280	76.7	3.8	8.5
24	17	2520	295	2225	88.3	3.8	8.0
26	19	2140	285	1855	86.7	4.2	7.3
30	23	3950	445	3505	88.7	5.9	8.3
Dec. 3	26	3540	1360	2180	61.6	6.6	8.6
6	29	2895	630	2265	78.2	6.3	8.6
8	31	2700	570	2130	78.9	6.6	8.6
10	33	3400	450	2950	86.8	8.2	8.5
12	35	3400	395	3005	88.4	6.0	8.7
14	37	3080	280	2800	90.9	6.3	8.7
16	39	3160	520	2640	83.5	6.7	8.7
17	40	2800	290	2510	89.6	7.1	8.7
19	42	3500	280	3220	92.0	5.6	8.6
21	44	2900	305	2595	89.5	6.5	8.6
24	47	2970	370	2600	87.5	6.1	8.5
25	48	2750	230	2520	91.6	6.3	8.6
27	50	2840	290	2550	89.8	8.1	8.7
29	52	3120	370	2750	88.1	6.2	8.5
31	54	2880	295	2585	89.8	?	?
Jan. (1972) 2	56	2880	440	2440	84.7	6.8	8.8
4	58	2600	340	2260	86.9	6.1	8.8
8	62	3080	400	2680	87.0	6.4	8.8
13	67	1960	300	1660	84.7	6.5	9.2
16	70	3440	380	3060	88.9	?	?
18	72	2960	755	2205	74.5	6.4	8.8

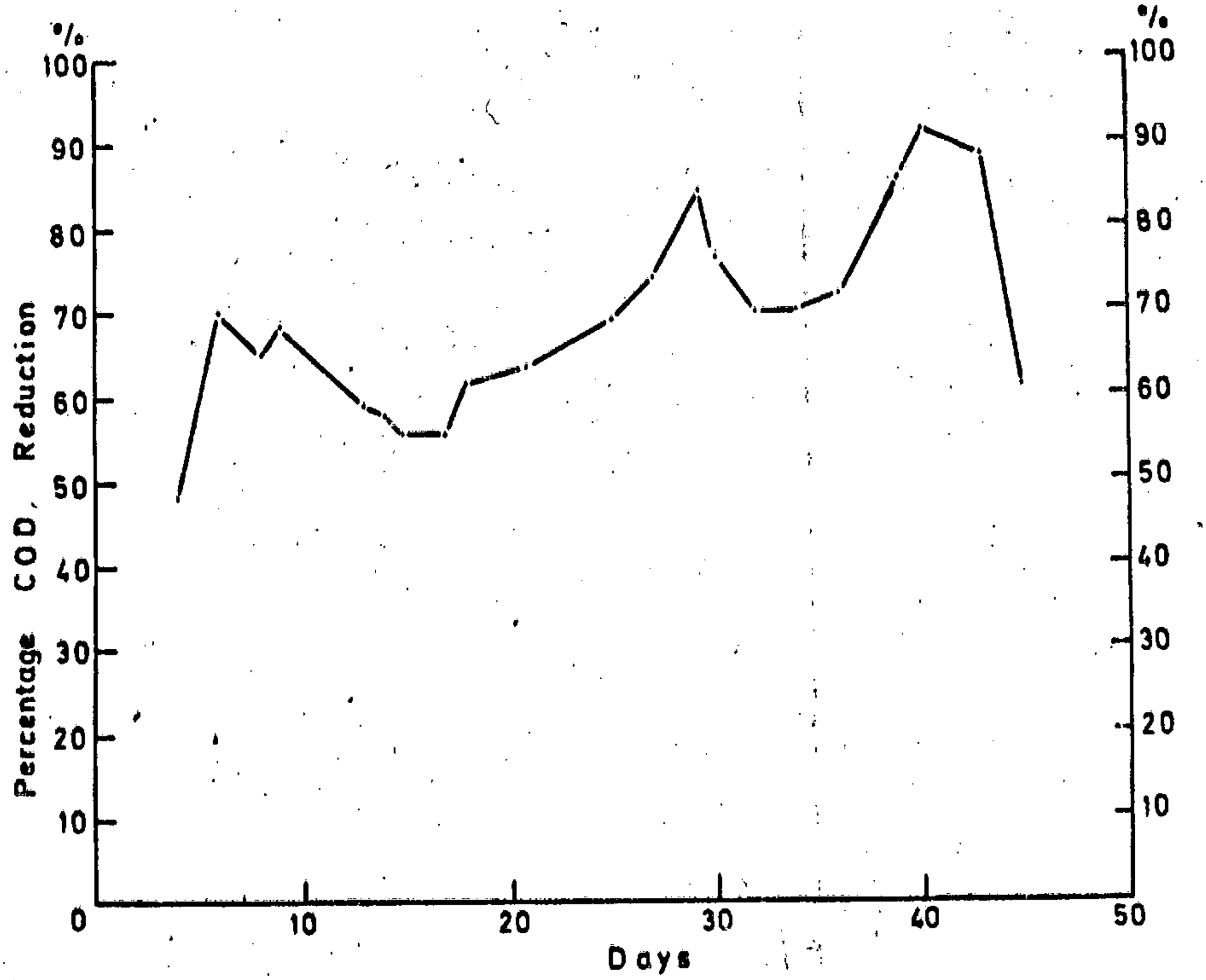


FIG.9·10 PERFORMANCE OF BIODISC ON MILK SUBSTRATE THROUGH PERIOD OF 45 DAYS

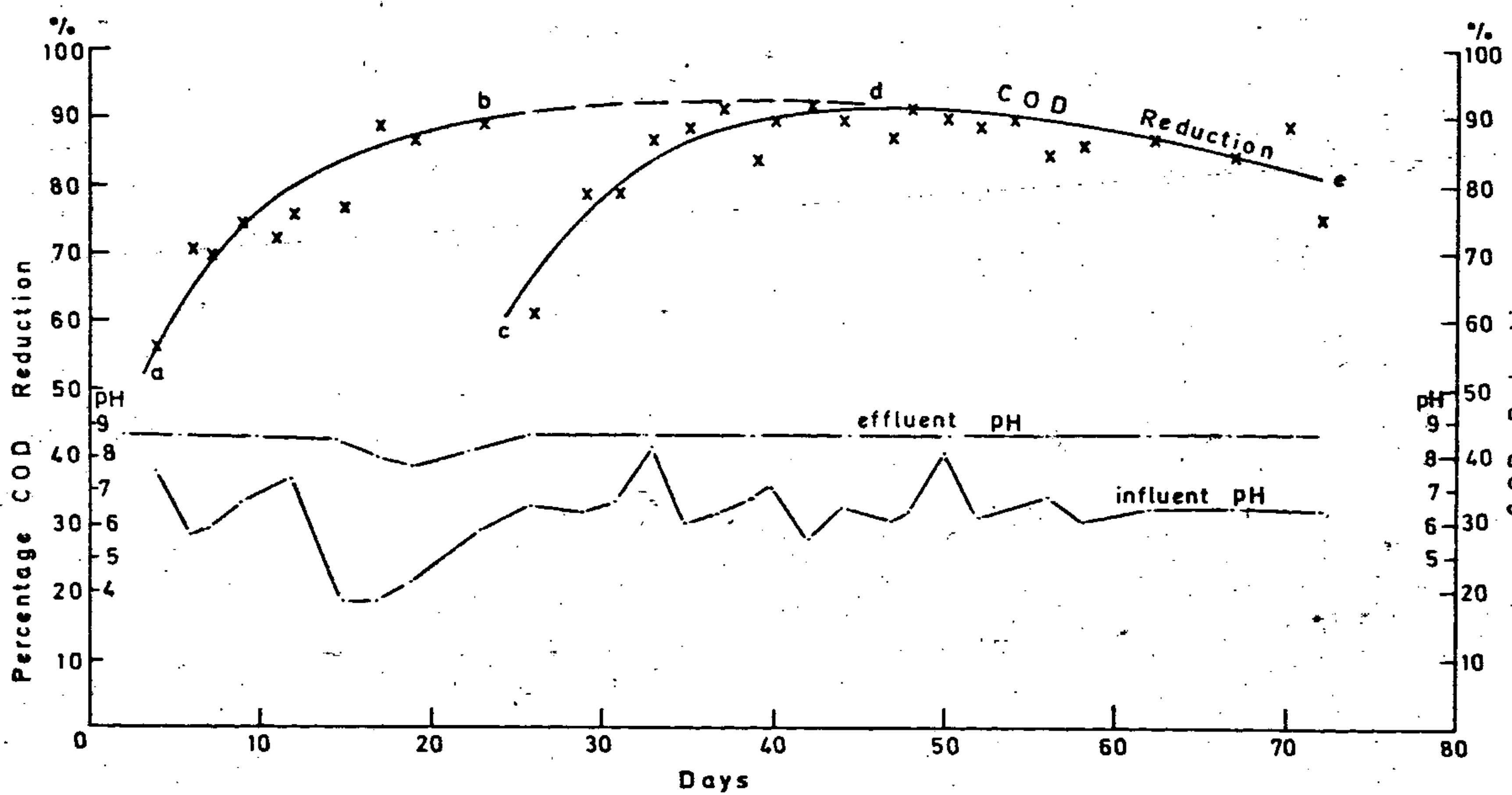


FIG.9.11 PERFORMANCE OF BIODISC ON MILK SUBSTRATE THROUGH PERIOD OF 72 DAYS

5-Day BOD Test. For this reason it is usual to establish this relationship at the beginning of a research programme by performing both COD and BOD tests on different portions of samples of a waste at different concentrations and plotting the BOD values against the corresponding COD values. After this relationship has been established the rest of the programme can be completed by doing only COD tests the values obtained from which can be converted to BOD values by applying the equation already established.

From a stock solution freshly prepared as described in Test 9.2.3, 11 quantities of 0, 2, 4, 6 ..... 16, 18 and 20 mls were placed each in a 250ml conical flask. The contents of each flask were made up with tap water to the 250ml mark after the appropriate quantity of $NaHCO_3$ as determined from Fig. 9.3 had been added. After stirring, a 25ml sample was taken from the contents of each flask. The COD test was performed on one portion of this quantity and the BOD test on another.

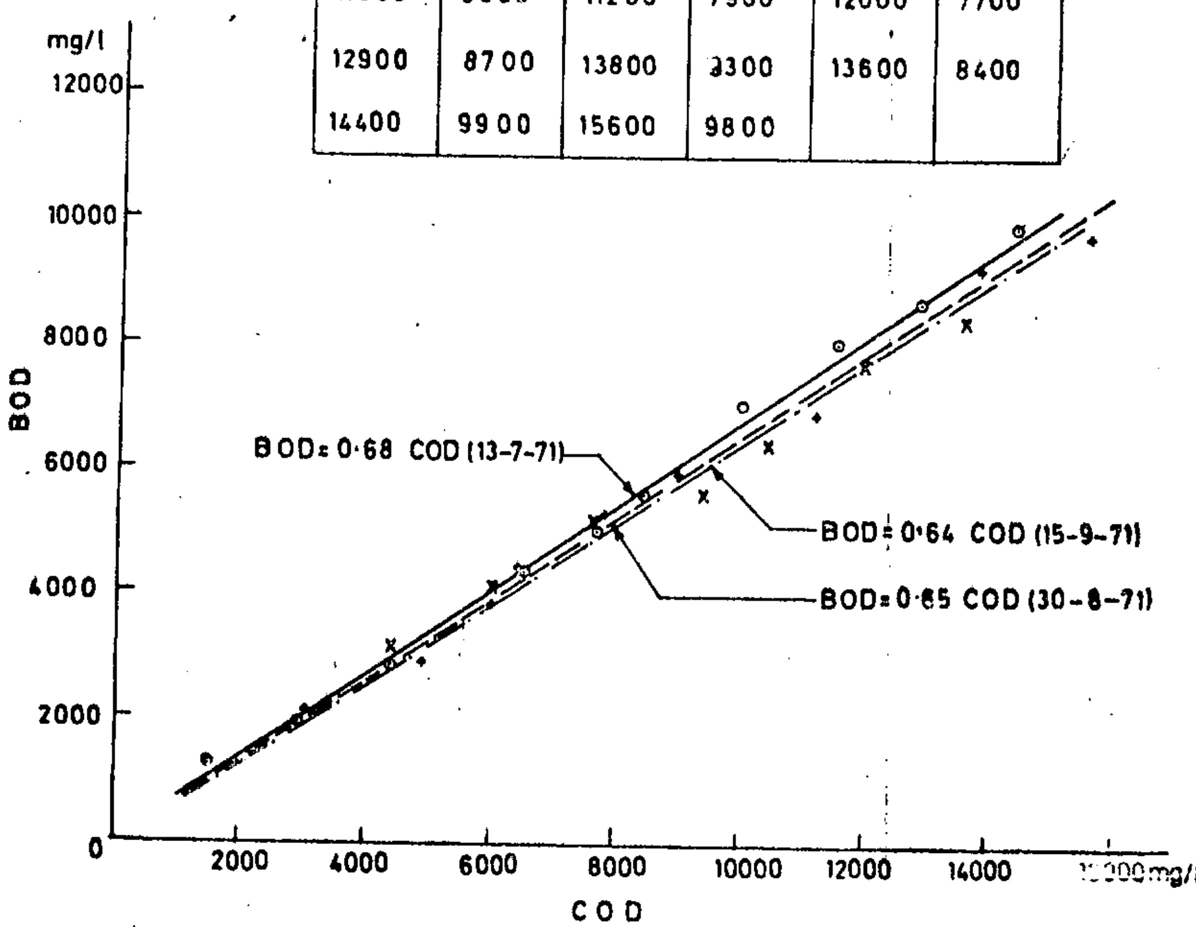
The results of tests performed on samples from stock solutions prepared on three different dates are shown in Fig. 9.12.

9.2.10 DISCUSSION:

The results of the tests with the biodisc in the treatment of milk are discussed in this section. Table 9.4 and Fig. 9.7 show that a final effluent concentration of 587mg/l COD was achieved after a waste of an initial concentration of 3380mg/l COD had passed through the biodisc. Applying equation 9.1 developed later to these figures, the corresponding BOD figures become

FIG. 9·12 COD - BOD CORRELATION FOR MILK

13 – 7 – 71		30 – 8 – 71		15 – 9 – 71	
Concentration (mg/l)					
COD	BOD	COD	BOD	COD	BOD
1500	1275	1450	1250	1600	1200
2900	1950	3100	2150	3000	1900
4400	2900	4900	2900	4400	3100
6500	4400	6000	3800	6000	4100
7700	5000	6400	4500	7600	5200
8400	5600	9000	6000	9400	5600
10000	7000	7800	5300	10400	6400
11500	8000	11200	7900	12000	7700
12900	8700	13800	9300	13600	8400
14400	9900	15600	9800		



2232mg/1 and 387mg/1 for the influent and effluent respectively. The treatment efficiency was 86.6%. Higher atment efficiency figures have been recorded in other tests with milk at the same concentration, particularly when the test had been running for several weeks. In Table 9.10 a COD reduction of 92% was achieved on Day 42.

Both Table 9.4 and Fig. 9.7 show that some 80% of the total treatment had been achieved before the waste entered the second compartment, and that only an additional 20% treatment took place in the remaining three compartments. This phenomenon of most of the treatment taking place in the first compartment is feature which the biodisc shares with other aerobic treatment devices. This is due to the fact that the organics in the waste reach the biomass on the discs in the first compartment first. Here the organisms remove most of the more easily removable components of the waste and pass on only the left-overs to the organisms in the remaining three compartments downstream. The need for compartmentation beyond two or three is open to doubt in view of this feature of the biodisc.

Both Table 9.5 and Fig. 9.8 show that the treatment efficiency of the biodisc on the same waste on two consecutive days dropped from 67.6% on the first day to 61.9% on the second. The influent COD increased from 5660mg/1 to 6053mg/1, a rise of 6.5% in 24 hours. The drop in efficiency of 5.7% was probably due in part to this rise in influent concentration. It is however thought to be due more largely to some change like loss in the volume of the biomass caused possibly by sloughing which could affect adversely the performance of the biodisc on the second day.

Tables 9.6 and 9.7 show that the biodisc reduced further the organics remaining in the effluent of a waste that had once passed through the device. The treatment efficiency in this second pass of the waste ranged from 32% to 42%. The cumulative percentage COD reduction in Table 9.7 shows the same feature of most treatment taking place in the first compartment which has been noted earlier in discussing Tests 9.2.5.

At no time was treatment efficiency achieved on recirculated effluent as high as that achieved on raw waste. It is considered that this is due to the fact that the most easily degradable components of the waste had been decomposed in its first passage through the biodisc. Also the chemical composition of the recirculated effluent would be different from the chemical composition of the original waste though a few components would still be common to both. This change in chemical composition probably resulted in a new waste more difficult to treat than the original waste. Again the microorganisms that had been adapted to the treatment of the original waste probably needed more time than had been allowed in these tests to acclimatise before the biodisc could treat the recirculated effluent with reasonable efficiency.

Describing this as a test on recirculated effluent is open to criticism becuase during collection and subsequent storage the effluent would have undergone a certain amount of decomposition. Recirculation normally implies pumping an effluent back to the influent point immediately it discharges at the effluent end. The arrangement used in these tests was used to ensure that sufficient quantities of effluent were at hand before the start of each run with the biodisc.

Both Table 9.8 and Fig. 9.9 show that even though the two biodiscs were of identical construction and were started on the same feed at the same time Biodisc A had achieved 85.3% COD reduction while B had achieved only 67.8% reduction six days after the test began. This again is attributable to some difference in the biomass on the discs in each biodisc which made A perform better than B. More importantly however both Table 9.8 and Fig. 9.9 show that three days after discs were removed from B the COD reduction had dropped by exactly 20%, and that it continued to drop for another 10 days when it reached 40.7%. The lowest COD reduction figure of 34.6% on day 15 is probably an error in view of the fact that the rise from 34.6% to 40.7% in 4 days in a continuing condition of anaerobic digestion was most unlikely in the biodisc. A line from b to c in Fig. 9.9 appears to fit in better in the curve of falling COD reduction in B. Two days after discs were restored in B the COD reduction rose dramatically by 20.7% from 40.7% to 61.4% and rose yet by another 19% in the next four days to 80.4%, which was the highest reached during the test.

The initial drop from 85.3% to 73.4% in the COD reduction in A from day 6 to 9 is probably due to some change in the condition of the biomass adversely affecting its performance in those 3 days. The treatment efficiency however rose steadily from day 9 to 26 when the highest figure of 94.1% occurred on the same day as the highest figure of 80.4% in B.

It is likely that if B had been allowed to continue to run without discs beyond day 20 the treatment efficiency would have continued to fall. If the 40.7% figure obtained on day 19 is however accepted as the minimum treatment efficiency obtained under anaerobic conditions and the 80.4% obtained on day 26 after the biodisc began to run aerobically as the highest then it could be concluded that approximately 50% of the treatment was due to the organisms on the revolving discs and the remaining 50% to all the other treatment processes like sedimentation, bio-flocculation and anaerobic decomposition.

Tables 9.9 and 9.10 as well as Figs. 9.10 and 9.11 show that waste reduction increased through each of the two tests for treatment efficiency through the period of the test. The jerky nature of the graph in Fig. 9.10 is considered due to inadequate pH control which was made good in the latter and longer of the two tests. Table 9.10 shows that while the influent pH varied from about 7.5 on the day of a new feed mix to an average of about 6.0 after about 4 days storage, the effluent pH averaged 8.6 most of the time. The fall in the pH of the feed during storage which is responsible for the regular troughs and crests in the pH curve in Fig. 9.10 is due to some degree of biological decomposition of organic compounds taking place in the stored feed, with the formation of ammonia which together with carbon dioxide forms NH_4^+ and HCO_3^- ions which react to form ammonium bicarbonate buffer according to the equations:

$$NH_3 + CO_2 + H_2O = NH_4^+ + HCO_3^- \quad \ldots\ldots(9.1)$$

$$NH_4^+ + HCO_3^- = NH_4HCO_3 \text{ (buffer)} \quad \ldots\ldots(9.2)$$

For most bacteria the pH for growth ranges from a minimum of 4.5 to 5.0 to a maximum of 8.0 to 8.5 with an optimum of 7.0 $\pm$ 0.5 (OGINSKY, E.L. and UMBREIT, W.W. 1959). Table 9.10 and Fig. 9.11 show that the effluent pH in the 72 days test was most of the time above 8.5, which is getting outside the upper limit of the range of maximum pH values. As the COD and the BOD reduction were above 80% and 90% respectively most of the time it is concluded that operating the biodisc at the fringe of the maximum pH does not appear to have affected its treatment efficiency adversely.

The influent pH inadvertently fell below 4.5 from day 15 to day 19 (Table 9.10). It was in fact only 3.8 on day 15 and day 17. The unacceptably low pH in this 4 day period depleted the NH_4HCO_3 buffer in the waste and caused the depression in the effluent, which fell to 7.3 on day 19. The rather steep fall in the percentage COD reduction from 88.7 on day 23 to 61.6 only 3 days later is due to the delayed adverse effect of the fall in both the influent and the effluent pH on days 15 and 19 respectively. Soon after the influent pH rose to the normal level both the effluent pH and the percentage COD reduction regained their normal values. It is clearly demonstrated in this that low influent pH had an adverse effect on COD reduction. There appears to be a time lag of about 11 days between the fall in influent pH and the consequent fall in treatment efficiency. A close examination of the points from which the COD reduction curve was drawn in Fig. 9.7 shows that while the treatment efficiency dropped rather steeply on day 26 due to the fall in the influent pH 11 days earlier a new treatment efficiency curve fairly quickly re-established itself and by day 37 had attained the figure of 90.91% which was slightly higher than the figure of 88.75% from which the original curve fell 18 days earlier. This is considered evidence of the capacity of the biodisc for quick recovery from adverse conditions after the restoration of a suitable environment.

The discs started sloughing about four at a time about day 42 even though the last two did not slough before the test was discontinued on day 72. It is considered that the fall off in the reduction curve from day 42 was due to the sloughing on the discs.

9.3 TESTS WITH DOMESTIC SEWAGE

Having established that the biodisc has a high efficiency in the treatment of a milk substrate, it was decided to extend the study to the performance of the device on domestic sewage which is known to be less uniform than milk in its chemical composition, consistency and concentration.

The sewage was obtained from the 15cm (6") dia influent sewer of the 22,500gd. (102.4m^3/d) capacity 20B27 Oxigest treatment plant serving the staff quarters at the University of Lagos and described in Chapter 2. Samples were taken from the plant at 9 a.m. each test day. This procedure of taking samples at the same time of day on each test day was aimed at reducing the variation in the concentration and consistency of the sewage. Particles which by their size were likely to block the tube from the feed aspirator to the biodisc were removed by running the sample through a No. 85 BSS sieve. COD, BOD and suspended solids tests were done and the amount of treatment taken as the difference between the organic loading in the influent and the effluent samples.

9.3.1 PRELIMINARY RUN ON DOMESTIC SEWAGE:

In a preliminary run the biodisc was first made to treat a milk substrate for 24 days. The milk was then replaced with domestic sewage and without cleaning the discs the run was continued for another 19 days. The flow was 7ml/min. giving a retention period of 18 hours. COD, BOD and suspended solids tests were done 5, 13 and 19 days after the change of substrate. Sloughing of the organisms on the discs started on the 13th and was complete on the 19th day after the change.

It was discovered five days after the change from milk to domestic sewage that no bicarbonate addition was required to keep the influent pH from falling to undesirably low levels. This is considered due to the absence of organic acids in the decomposition of the components of the domestic sewage, which might depress the pH to dangerous levels as occurred in the decomposition of the milk waste where such organic acids were formed.

The relatively high treatment efficiency figures obtained in the preliminary run (Table 9.11) indicated that the flow rate of the sewage through the biodisc could be increased without the treatment efficiency falling to unacceptable levels.

9.3.2 MAIN TESTS WITH DOMESTIC SEWAGE:

The tank and the discs were cleaned out for the full run on domestic sewage which lasted 49 days. The tank was initially filled with sewage of one quarter the strength of the feed sewage. The flow was increased to 14ml/min. (9 hours retention time). Twice a week COD and suspended solids tests were done on two portions of the influent sample as well as on two portions of the effluent sample.

Once a week BOD tests were done on yet a third portion of the influent sample as well as on a third portion of the effluent sample. These

TABLE 9.11

PRELIMINARY RUN OF BIODISC ON DOMESTIC SEWAGE ON 3 OCCASIONS IN JUNE 1971

	No. of days from change from milk to domestic sewage	3/6/71	11/6/71	17/6/71
		5	13	19
COD	Influent (mg/1)	430	480	590
	Effluent (mg/1)	95	93	68
	Treatment Efficiency	78.4%	80.6%	88.6%
BOD	Influent (mg/1)	250	200	175
	Effluent (mg/1)	24	10	7
	Treatment Efficiency	90.4%	99%	96%
Suspended Solids	Influent (mg/1)	528	Test	248
	Effluent (mg/1)	133	not	14
	Treatment Efficiency	74.8%	done	94.3%

NOTES:

(1) Milk substrate was replaced with domestic sewage on 29/5/71.

(2) Influent pH was constant at 7.3 while effluent pH varied between 8.1 and 8.5 after the change in substrate.

(3) Addition of sodium bicarbonate to influent was discontinued 5 days after change of substrate in view of high influent and effluent pH.

weekly BOD tests thus yielded in the seven weeks run 8 BOD values for influent samples for which the COD values are known and 8 BOD values for effluent samples for which the COD values are also known. The BOD values compared with the COD values are shown in Table 9.13. The results are shown in Tables 9.13 and 9.14 as well as in Figs. 9.13 - 9.15.

9.3.3 DISCUSSION:

Figs. 9.13 - 9.15 show that there is a wide variation in both the COD, BOD and SS of the influent through the 49 days of the full run on domestic sewage in spite of the fact that the samples were taken from the same plant and at the same time of the day.

The maximum BOD recorded was 355mg/litre and the minimum, 50mg/litre with an average value of 166mg/litre. The BOD of domestic sewage is known to vary over a wide range. McKINNEY, R.E. (1962) indicated that the organic components of domestic sewage average approximately 300mg/litre, with a range from 100 to 500mg/litre. BABBIT, H.E. and BAUMANN, E.R. (1965) gave an average of 262 with a range from 121 to 473mg/litre obtained from a survey of 22 sewage treatment works in the U.S.A. It would appear that the organic loading of the test samples was low in the light of these latter figures. It is observed further in Figs. 9.13 - 9.15 that the COD, BOD and suspended solids fell substantially in the latter half of the test period. This is probably due to the fact that

TABLE 9.13

COD and BOD REDUCTION IN DOMESTIC SEWAGE IN 49 DAYS RUN

Date (1971)	COD *					BOD *				pH	
	Day	Influent (mg/1)	Effluent (mg/1)	Reduction (mg/1)	% Reduction	Influent (mg/1)	Effluent (mg/1)	Reduction (mg/1)	% Reduction	Influent	Efflu-
June 23	-	400	400	-	-	160	160	-	-	7.20	7.20
26	3	490	118	372	76.1					7.20	7.60
30	7	450	87	363	80.9	200	10	190	95.0	6.80	7.70
July 3	10	520	75	445	85.4					7.20	7.90
7	14	430	94	336	78.0	138	12	126	91.5	7.30	7.50
10	17	500	74	426	85.2					6.70	7.80
14	21	400	74	326	81.5	167	5	162	96.8	7.60	7.80
19	26	400	92	308	77.0					7.70	7.70
21	28	590	88	502	85.2	355	15	340	95.6	6.90	7.60
24	31	540	78	462	84.5					6.90	7.60
28	35	402	79	323	80.4	140	16	124	88.6	6.90	7.60
Aug. 4	42	265	96	169	63.8	120	4	116	96.7	6.90	7.60
7	45	173	62	111	64.2					7.00	7.00
11	49	217	66	151	69.8	50	3	47	94.0	7.00	7.00

* The COD-BOD correlation is computed by the method of linear regression of BOD on COD, with the following results:
Influent: BOD = 0.71COD-112. Correlation coefficient r = 0.9156.
Effluent: BOD = 0.16COD-4. Correlation coefficient r = 0.3173 (weak)

TABLE 9.14

SS REDUCTION IN DOMESTIC SEWAGE IN 49 DAYS RUN

Date (1971)	Day	Suspended Solids				pH	
		Influent (mg/1)	Effluent (mg/1)	Reduction (mg/1)	% Reduction	Influent	Effluent
June 23	–	126	126	–	–	7.20	7.20
26	3	230	20	210	91.5	7.20	7.60
30	7	134	20	114	79.3	6.80	7.70
July 3	10	148	5	143	91.5	7.20	7.90
7	14	147	15	132	89.8	7.30	7.50
10	17	–	–	–	–	6.70	7.80
14	21	215	12	203	94.5	7.60	7.80
19	26	173	78	95	54.9	7.70	7.70
21	28	176	16	160	90.8	6.90	7.60
24	31	267	29	238	89.3	6.90	7.60
28	35	136	25	111	81.6	6.90	7.60
Aug. 4	42	184	14	170	92.4	6.90	7.60
7	45	39	13	26	66.7	7.00	7.00
11	49	54	7	47	83.0	7.00	7.00

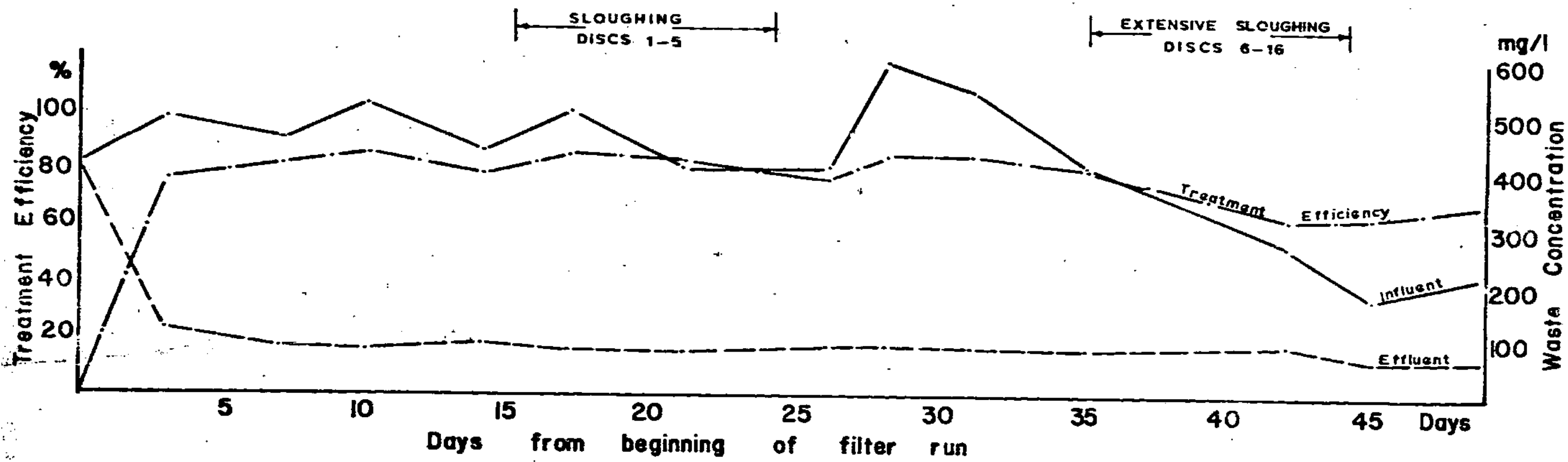


FIG.9-13 COD TESTS ON DOMESTIC SEWAGE THROUGH PERIOD OF 49 DAYS

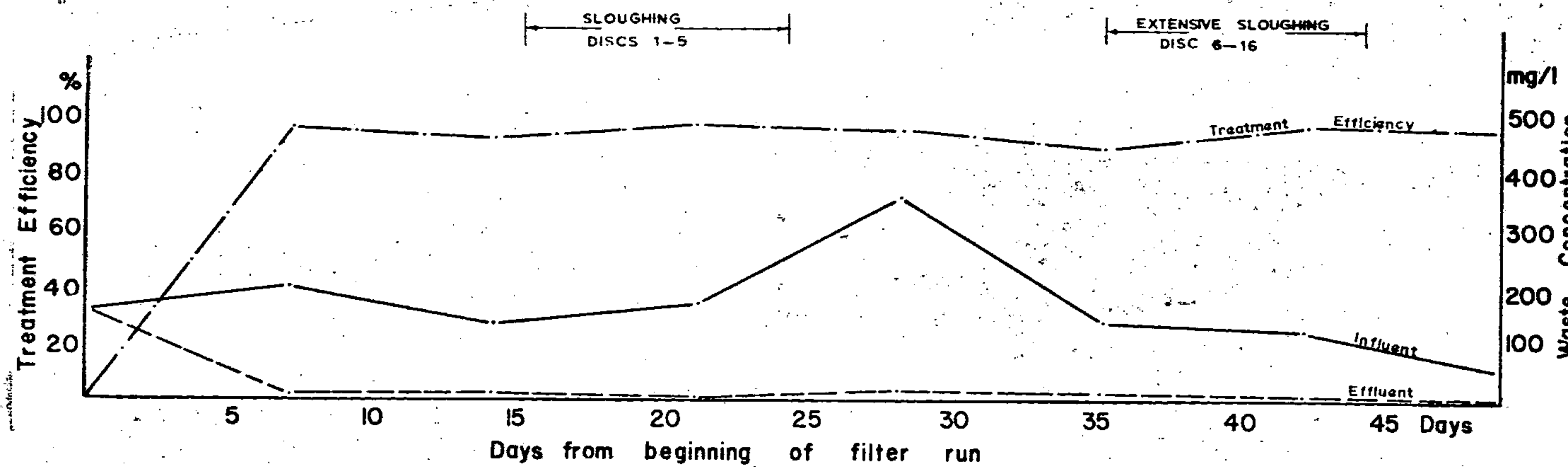


FIG. 9.14 BOD TESTS ON DOMESTIC SEWAGE THROUGH PERIOD OF 49 DAYS

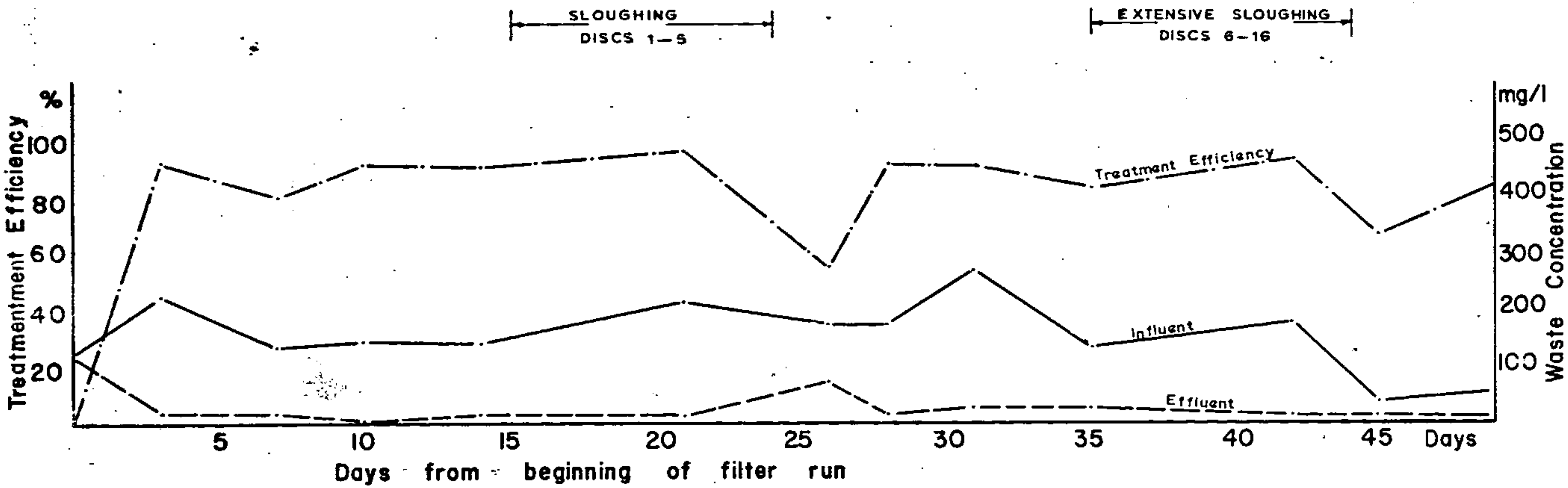


FIG.9·15 SS TESTS ON DOMESTIC SEWAGE
THROUGH PERIOD OF 49 DAYS

this period, 24th July to 11th August 1971 was in the first half of the University vacation when staff were away from the University Campus, possibly resulting in a reduction of both the organic and hydraulic loading in the sewage treatment plant.

Table 9.13 shows that a substantial treatment efficiency was achieved with the biodisc on 3/6/71 which was only 5 days after the change in substrate from milk to domestic sewage indicating that the microorganisms that originally grew on milk could adapt themselves to domestic sewage and still give good performance. This finding is in keeping with the result of the analysis of the growth on the discs in a run on milk in February 1971 when the organisms identified were mostly those usually found in domestic sewage, which explains why adaptation from a milk substrate to domestic sewage had not been difficult for them.

Table 9.11 shows that the treatment efficiency increased through the 19 days of the preliminary run with the biodisc in which milk substrate was replaced with domestic sewage. This is attributable to the organisms getting more and more adapted to their new environment. Figs. 9.13 - 9.15 show that the treatment efficiency was high in COD, BOD and SS throughout the period of the test and that these high values were attained early. The efficiency however, dropped in the SS and COD after the first sloughing had occurred (days 16-25). It again dropped in both the SS and COD during the second and more extensive sloughing (days 35-44). Similarly fall in treatment efficiency in the earlier tests with milk usually preceded sloughing by about two days. There was however no appreciable fall in the BOD treatment efficiency during the 2 sloughings in this test with domestic sewage.

The highest effluent BOD was 16mg/litre which occurred on day 35. The highest effluent SS was 78mg/litre and this occurred on day 26; the average effluent SS was 21.2mg/litre. These average figures are well below the Royal Commission Standards of 20mg/litre BOD and 30mg/litre SS for wastes discharging into rivers and other receiving waters.

In these tests and in the tests with the other wastes reported in this chapter the discs were rotated at a constant speed of ½ rpm, representing a peripheral speed of 0.30m/min. The rotational velocity used in these tests was probably low. Excessive velocity however would cause high centrifugal forces which could make slime slide off the discs. It could also result in an excessively high peripheral velocity where disc diameters are large, which could cause the erosion of the slime off the discs when the discs are immersed in the waste. AUTOTROL CORPORATION (1972) suggest an optimum figure of 18.3m/min. for the peripheral velocity which is nearly 60 times the speed used in these tests with discs of small diameter. The treatment efficiency could therefore be expected to be higher in a field plant in which the discs are rotated at a higher speed.

ANTONIE, R.L. and others (1970) reported favourably on the performance of a 0.5MGD Biodisc Municipal Wastewater Treatment Plant at Pewaukee Wisconsin, U.S.A. At a hydraulic loading of 2.0gpd/ft^2 (81.3 litres/d/m^2)BOD_5 removal from a 100mg/litre average BOD_5 wastewater was 83% resulting in a 17mg/litre BOD_5 effluent. At a hydraulic loading of 1.0gpd/ft^2 (40.6 litres/d/m^2) the BOD_5 removal increased to 92% and the effluent BOD_5 decreased to 8mg/litre. Wastewater temperatures above 55°F (12.8°C) were reported to have no effect on treatment efficiency but below this temperature lower hydraulic loadings were necessary to achieve the same degree of treatment. Sludge production was reported to be 0.4lb/lb (.4kg/kg) of BOD_5 removed.

9.4 TESTS WITH NIGHTSOIL:

The study of the biodisc was next extended to its performance on nightsoil. This waste has been chosen because it is a much more concentrated type of domestic sewage and because its treatment has a very direct practical application in Lagos and other towns in Nigeria. The nightsoil conservancy system has already been described in Chapter III.

9.4.1 PRELIMINARY TEST WITH NIGHTSOIL:

In a preliminary test to determine the amount of dilution required and the likely or probable order of COD and SS concentration a 2 litre sample was obtained from a tanker discharging into the Lagoon at the Lagos City Council Jetty at Ebute-Ero on 11/6/74. 250 ml of this quantity was first diluted and made up to 1 litre. The diluted quantity was run through a fine mesh raffia basket to remove trash like paper and leaves. After a thorough mixing 100ml of the filtrate was made up to 1 litre, now giving a resultant concentration of 1 in 40 in the nightsoil sample. COD and SS tests were then performed on samples of this.

The results of this preliminary test indicated that for this sample diluted 40 times the COD was approximately 3,300mg/litre and the SS was 2,000mg/litre.

About twice a week after this preliminary test a composite sample made up of quantities of approximately 1 litre from each of 12 tankers discharging at the Jetty at Ebute-Ero was collected in a plastic 20 litre bucket. It was sifted first with a rough texture raffia basket and later with a No. 85 B.S.S. sieve. After stirring with a stick some 6 litre of the liquor was taken in a 10 litre aspirator to the laboratory where it was diluted down to the required concentration for the main tests described below.

It was discovered during the collection of the sample for the preliminary test that the aspirator must not be filled to the top and that the lid must not be screwed on too tight during transportation to allow for the escape of gases resulting from the decomposition of the organic compounds continously taking place in the sample. The same precaution was necessary in the storage of nightsoil in the laboratory.

9.4.2 MAIN TESTS WITH NIGHTSOIL:

There were three main tests. In the first test which ran for 17 days the nightsoil samples were diluted with tap water. The dilution was approximately 40 times at first but this was gradually reduced to about 10 times, towards the end of the test. The substrate entered at the influent end of the settling chamber and therefore underwent primary sedimentation before entering the aerobic chamber. COD tests were performed about thrice a week on the raw influent, the settled influent and the effluent. SS tests were performed once a week on the raw influent and on the effluent. This run was started with clean discs.

In the second test which ran for 16 days lagoon water was used for diluting the nightsoil. The water was obtained from the University of Lagos foreshore and its salinity as determined on the conductivity meter was 14,500mg/litre. The raw influent was led straight into the aerobic chamber in this test. COD tests about thrice a week and SS tests once a week were performed on both the influent and the effluent. While the tank was cleaned out at the change over, the discs were not. The new run was therefore started with discs carrying biomass which had formed in the first test.

In the third test, effluent resulting from the second test was made to recirculate through the biodisc, and COD and SS tests were performed on this 5 times in the 19 days in which the test lasted. This third run was started with clean discs.

The flow was 14ml/min. (9hours retention time) in all the three tests. As in the tests with domestic sewage it was not found necessary to buffer up the feed for pH control.

There was a supplementary test to determine the COD-BOD correlation for nightsoil. The procedure was as described for determining the same relationship in domestic sewage described earlier. The results of the tests are shown in Tables 9.15 - 9.20 and Figs. 9.16 - 9.20.

9.4.3 DISCUSSION:

Table 12.1 shows that the tests with nightsoil differed from the tests with the other wastes already investigated as regards concentration which ranged widely in these and which were on the whole much higher than those in the other tests. A wide range was selected deliberately to study how treatment efficiency varied with concentration. The mean concentration of raw influent samples obtained from Table 9.15 was 12,620mg/litre COD and 4,350mg/litre SS. The corresponding figures for effluent samples were 1578mg/litre and 411mg/litre respectively.

Table 9.16 shows substantial COD settled in the settling chamber, ranging from 43.7% on day 15 to 83.7% on day 25. Applying equation 9.7 developed later in this Chapter gives the corresponding BOD values for both the raw influent and the settled influent. The settled % BOD values so obtained are slightly higher varying from 46.7% on day 15 to 85.5% on day 25. Further the COD settled in the settling chamber increased with raw influent concentration as seen in Fig. 9.16.

TABLE 9.15

PERFORMANCE OF BIODISC ON NIGHTSOIL DILUTED WITH TAP WATER

Date (1974)	Day	COD			SUSPENDED SOLIDS	
		Raw Influent (mg/1)	Settled Influent (mg/1)	Effluent (mg/1)	Raw Influent (mg/1)	Effluent (mg/1)
1	2	3	4	5	6	7
June 22	4	3820	2100	672		
26	8	9340	4900	1870	3830	560
27	9	12260	4590	1665		
29	10	12700	4120	1570		
July 3	15	8700	4900	1825	3000	355
6	18	9640	3880	1475		
10	22	9840	4200	1510	3460	210
13	25	26800	4360	1635		
17	29	20480	5067	1980	7100	520

TABLE 9.16

PERFORMANCE OF BIODISC ON NIGHTSOIL DILUTED WITH TAP WATER (COD AND SS REDUCTION)

Days	Influent COD (mg/l)	% COD settled in settling Chamber	% COD Reduction in Biological Chamber	Total % COD Reduction in Biodisc (mg/l)	Influent SS (mg/l)	Total % SS Reduction in Biodisc
4	3820	45.0	68.0	82.4		
8	9340	47.5	61.8	80.0	3830	85.4
9	12260	62.6	63.7	86.4		
10	12700	67.6	61.9	87.7		
15	8700	43.7	62.8	79.0	3000	88.2
18	9640	59.8	62.0	84.7		
22	9840	57.3	64.0	84.7	3460	93.9
25	26800	83.7	62.5	93.9		
29	20480	75.3	60.9	90.3	7100	92.7

TABLE 9.17

PERFORMANCE OF BIODISC ON NIGHTSOIL DILUTED WITH LAGOON WATER

Date (1974)	Day	C.O.D. Raw Influent (mg/l)	C.O.D. Effluent (mg/l)	SUSPENDED SOLIDS Raw Influent (mg/l)	SUSPENDED SOLIDS Effluent (mg/l)
July 20	3	4070	855		
24	6	5500	890	2070	195 (9.4%)
25	7	6345	772		
31	13	16400	1450	7090	550 (7.8%)
Aug. 3	16	11640	1150		

TABLE 9.18

PERFORMANCE OF BIODISC ON NIGHTSOIL DILUTED WITH LAGOON WATER (COD REDUCTION)

Date (1974)	Day	C O D		C O D REDUCTION	
		Raw Influent (mg/1)	Effluent (mg/1)	(mg/1)	%
1	2	3	4	5	6
July 20	3	4070	855	3215	79.0
24	6	5500	890	4610	83.8
25	7	6345	772	5573	87.8
31	13	16400	1450	14950	91.2
Aug. 3	16	11640	1150	10490	90.1

TABLE 9.19

PERFORMANCE OF BIODISC ON RECIRCULATED NIGHTSOIL EFFLUENT DILUTED WITH LAGOON WATER

Date (1974)	Day	C O D		C O D REDUCTION	
		Raw Influent (mg/l)	Effluent (mg/l)	Reduction (mg/l)	% Reduction
1	2	3	4	5	6
July 17	2	1335	950	385	28.8
24	9	960	536	424	44.2
27	12	788	406	382	48.5
31	16	1395	328	1067	76.5
Aug. 3	19	1120	304	816	72.9

TABLE 9.20

COD - BOD CORRELATION FOR NIGHTSOIL

INFLUENT		EFFLUENT	
COD (mg/l)	BOD (mg/l)	COD (mg/l)	BOD (mg/l)
9340	3450	1570	375
4900	2050	1510	290
4120	1900	1310	280
8700	2700	650	58
9840	3350	1980	550
4200	1350	1335	435
-	-	950	215
5070	2320	890	110
5500	1850	960	130

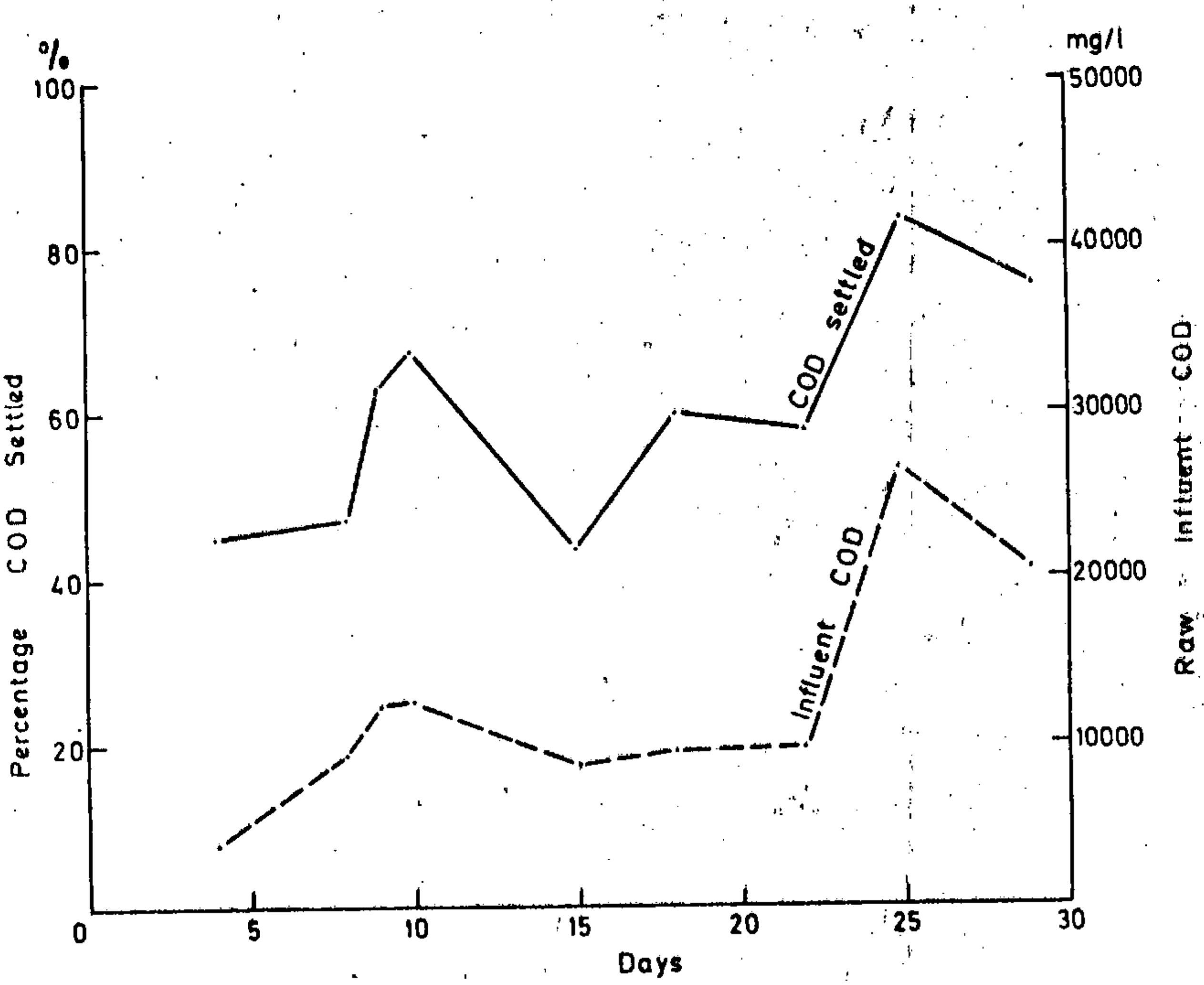


FIG.9·16 COD SETTLED IN SETTLING CHAMBER IN NIGHTSOIL DILUTED WITH TAP WATER

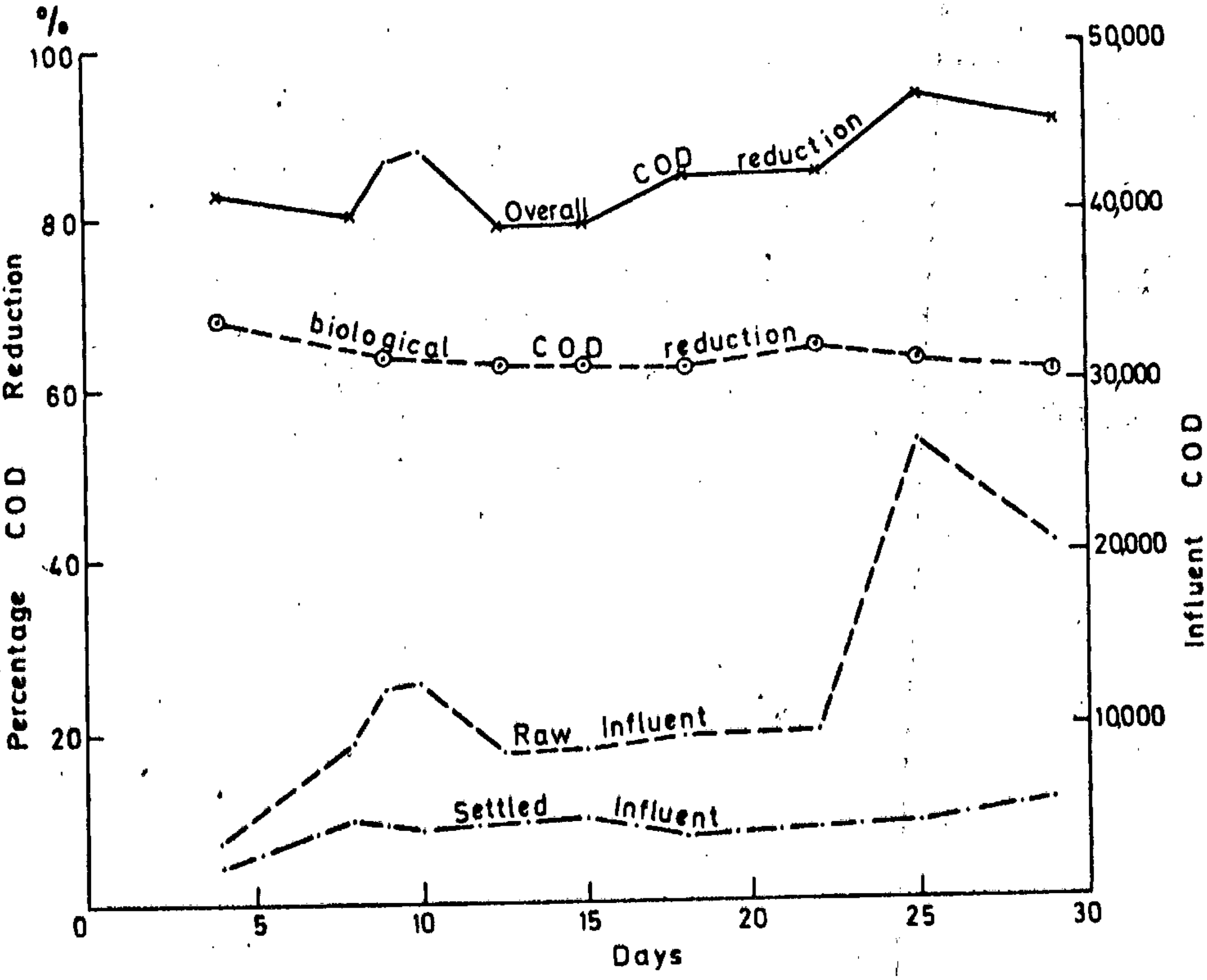


FIG.9.17 COD REDUCTION IN BIODISC IN NIGHTSOIL DILUTED WITH TAP WATER

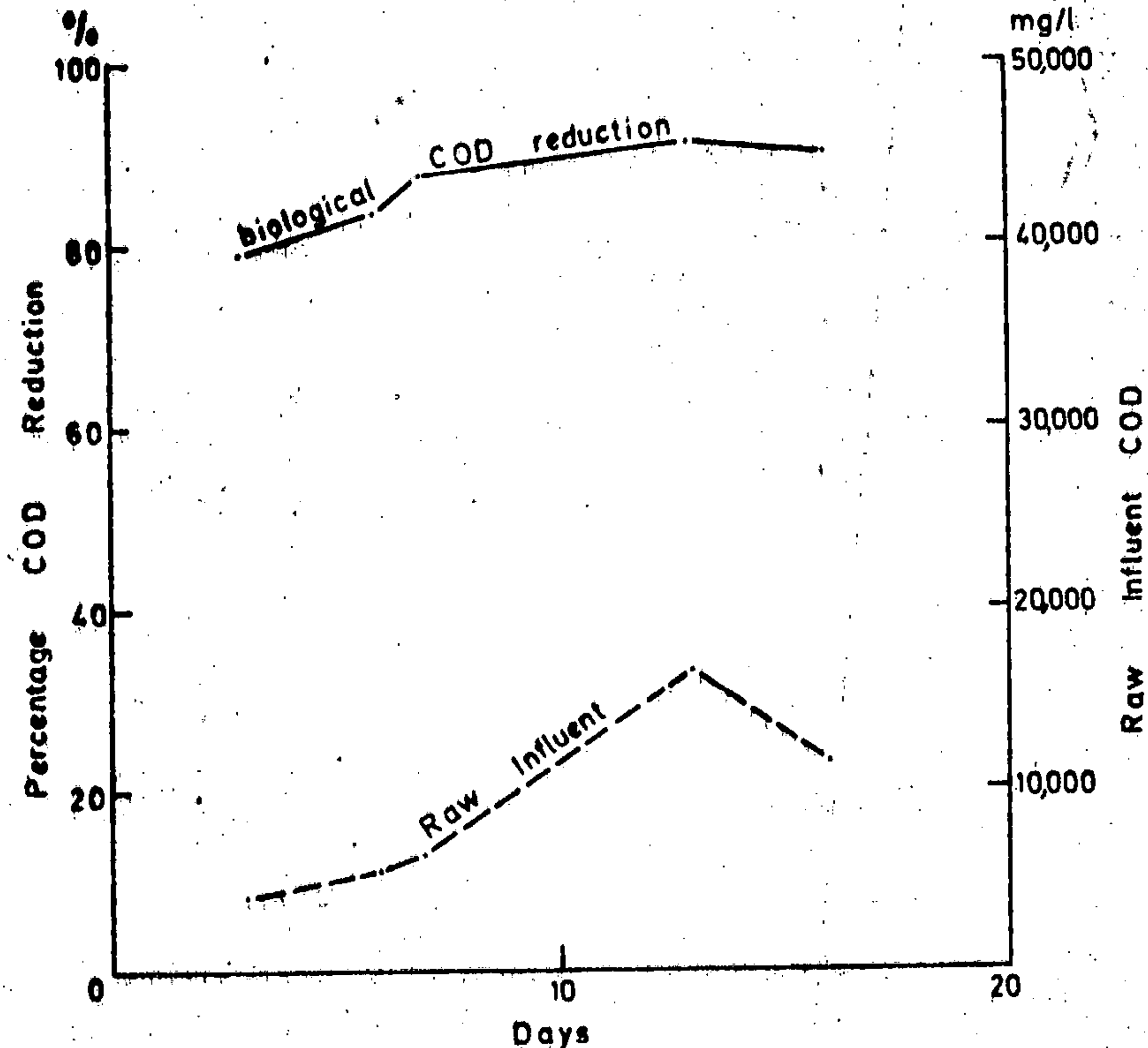


FIG. 9.18 BIOLOGICAL COD REDUCTION IN BIODISC IN NIGHTSOIL DILUTED WITH LAGOON WATER

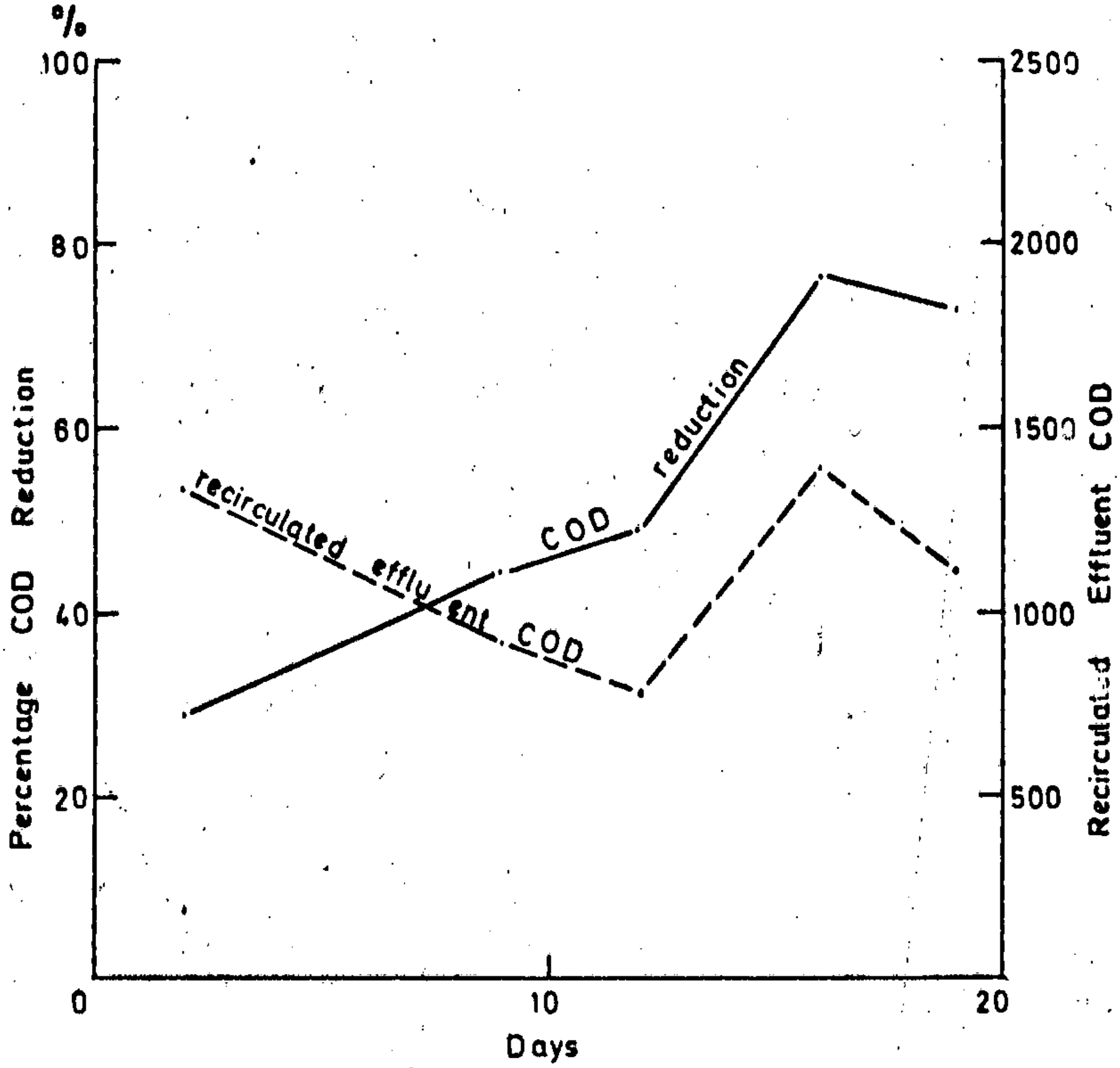


FIG.9·19 BIOLOGICAL COD REDUCTION IN RECIRCULATED NIGHTSOIL EFFLUENT

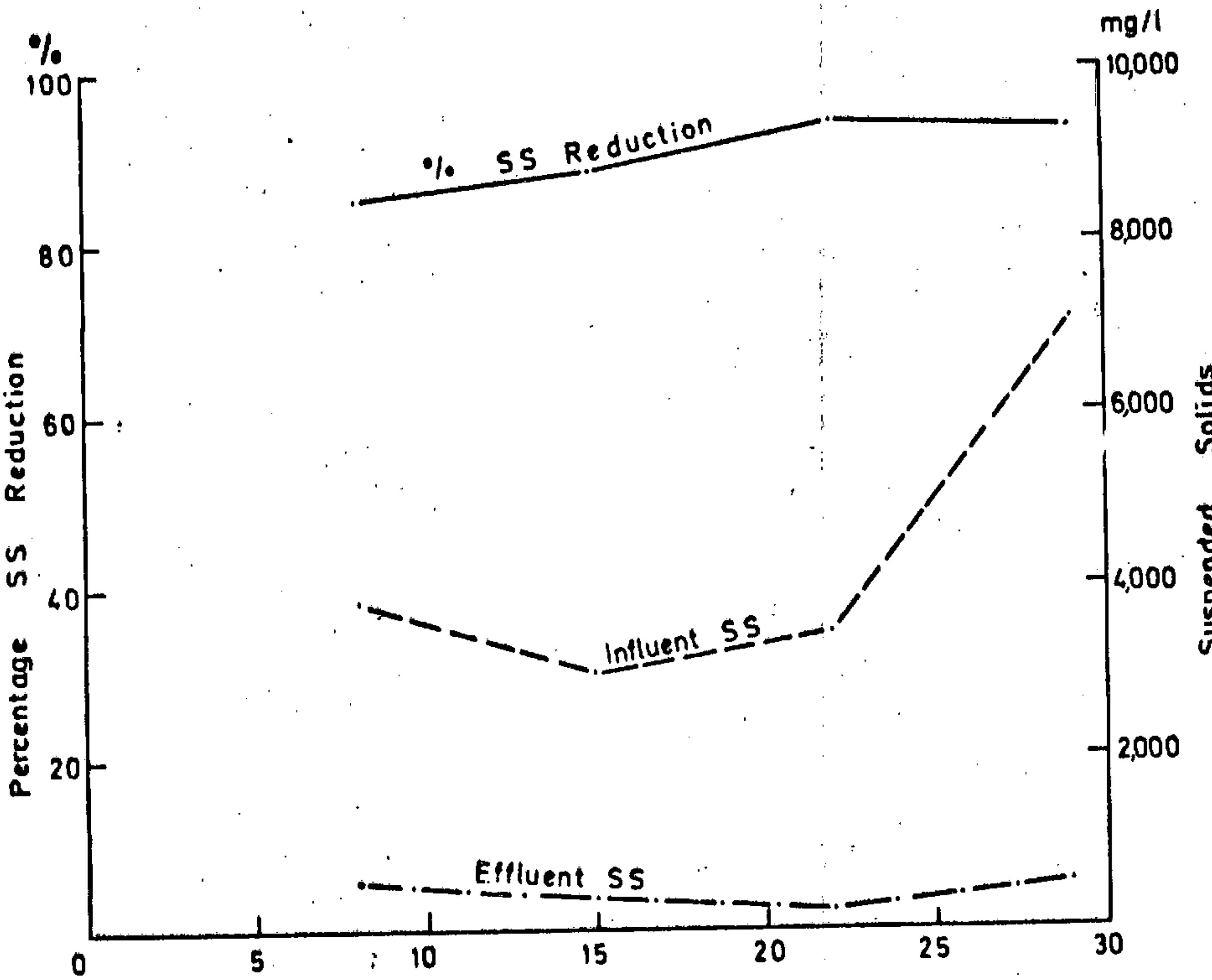


FIG. 9.20 REDUCTION IN SUSPENDED SOLIDS IN BIODISC IN NIGHTSOIL DILUTED WITH TAP WATER

The amount of BOD settled in a sedimentation tank from domestic sewage and wastes of similar concentration is usually about 30% to 40% of the total organic load. Figures obtained in this test therefore are relatively high. It is however considered that there is no particular advantage in this as much of the COD settled in that settling chamber is in fact stored in the sludge which still has to receive further treatment.

Table 9.15 shows further that the range in concentrations in the settled influent is not wide compared with the concentration range in the raw influent. Excluding the figure of 2100mg/litre COD on day 4 as being unrepresentative of the class, there is a 26% difference between the highest figure 5067mg/litre COD in the settled sewage on day 29 and that of 3880mg/litre COD on day 18. Table 9.16 however shows that in spite of this difference of 26% in the influent to the biological chamber, the percentage COD reduction in that chamber was practically steady at about 62%. Again in Table 9.18 where the settling chamber was by-passed there was approximately 400% variation in the concentration of the influent samples. In spite of this very wide variation there was only 12% variation in the COD reduction figures. Both these are evidence of the capacity of the biodisc to give uniform performance in spite of variations in the organic loading.

Again Table 9.16 and Fig. 9.16 show that the overall COD reduction varies from 79% to 94% in the test in which nightsoil was diluted with tap water and biological treatment was preceded with settling. Table 9.18 shows that the variation is from 79% to 91% in the other test in which the raw influent by-passed the settling chamber and the nightsoil diluted with lagoon water. The range of treatment efficiency in either case is considered high particularly for the high organic loadings involved.

Both Tables 9.16 and 9.18 show that the range of treatment efficiency of the biodisc in terms of overall COD reduction is approximately the same irrespective of whether the nightsoil was diluted with tap water or with lagoon water. Salinity in the dilution water apparently has no adverse effect on the treatment process. This is important in that a treatment plant can be sited on the lagoon where an unlimited quantity of dilution water is immediately available at little cost.

Tables 9.16, 9.18 and 9.19 as well as Figs. 9.17, 9.18, 9.19 and 9.21 show that COD reduction increased with both time and the concentration of the raw influent. Increase of treatment efficiency with time is due to increasing growth in the biomass on the discs while the increase with influent concentration is due to the COD that is lost in increasing quantities to the sludge in both the settling chamber and in the anaerobic chamber of the biodisc.

The test with recirculated effluent was started with clean discs and the low treatment efficiency figures at the beginning of the test shown in Table 9.19 are due to the fact that the micro-organisms were just starting to grow on the discs at that time. The treatment efficiency was above 70% on each of the last two occasions of the test, when the biomass on the discs had grown properly and adapted themselves to the feed. Compared with the results obtained in the tests with recirculated milk effluent in Tables 9.6 and 9.7 the high treatment efficiency figures on those two occasions indicate that the biodisc gives better performance on recirculated nightsoil effluent than on recirculated milk effluent. This is important in that a treatment plant could be constructed of two biodisc units connected in series, the first unit treating raw nightsoil and passing on the effluent for further treatment in the second unit downstream. A second alternative would be to recirculate a pre-determined quantity of the effluent from a single unit back to the influent chamber to mix with and dilute further influent.

Both Tables 9.16 and 9.17 as well as Fig. 9.20 show that the biodisc achieved high treatment efficiency figures in the removal of suspended solids. This observation is again qualified by the knowledge that a substantial portion of the suspended solids still has to be encountered either in the treatment of the sludge in a separate digester or in the anaerobic digestion taking place at the bottom of the tank of the biodisc.

In these tests nightsoil was diluted with water to aid handling and the separation of the nightsoil from trash and paper. Dilution brought the concentration down to levels where treatment by aerobic methods was feasible. Dilution of any waste before treatment appears to be a contradiction in terms of the important objective of settling out suspended solids in the primary treatment process and the subsequent concentration of the sludge before digestion, as dilution is the direct reverse of this process. It inevitably increases the volume of waste handled, with a corresponding increase in plant capacity and costs, both capital and operational.

The new Lagos City Council Nightsoil Treatment Plant in which the nightsoil is diluted with lagoon water before digestion in aerated lagoons suffers from this disadvantage. In this case however, the dilution water costs nothing and therefore brings down a little the otherwise high operating costs. The simple primitive process of trenching which is still practised in many towns in the developing countries, and has already been described in Chapter III, appears to remain the most economical and effective method of handling and disposal of nightsoil. It is however emphasised that the amount of treatment that takes place in the nightsoil after burial is totally inadequate for ground water derived from the vicinity of the trenching ground to be considered safe. Anaerobic treatment in oxidation lagoons and anaerobic digestion in digesters are reported to have yielded good results in South Africa, South East Asia and Japan.

9.5 TESTS WITH INDUSTRIAL WASTES:

It was decided to conclude the study of the performance of the biodisc in the treatment of local wastes by making it treat industrial wastes from some factories in the Ikeja Industrial Estate. The waste was obtained from the 640,000 gpd ($2912m^3/d$) Trade Effluent Treatment Plant which in the four months of the investigation received effluents only from the following three factories: Dunlop Tyre Factory, Nigeria Textile Mills, Guiness Brewery. There was difficulty in obtaining waste samples from other factories.

9.5.1 TESTS WITH INDUSTRIAL WASTES:

Samples were collected from the mixing tank of the plant about three times a week and at about 11 a.m. each day. This was aimed at collecting samples with approximately the same characteristics each day. It was observed that the waste was coloured green reflecting the colour of the dyes from the textile mills.

There were two tests in each of which the run was started with clean discs. In the first which ran for 26 days both COD tests were done on influent and effluent samples about three times a week and SS tests approximately once a week. There was a single BOD test.

The second test lasted 91 days during which COD and SS tests were done about twice a week and BOD test about once a week on influent and effluent samples. In both tests the influent was made to by-pass the settling compartment by leading it straight into the first of the four compartments in the aerobic chamber.

It was noticed that while two or three discs sloughed at a time there was no mass sloughing in either of the two tests and there was no definite sloughing cycle established. Also there was no desludging even in the longer run of 91 days, compared with the run on milk when desludging was done about every 2 or 3 weeks. The results are shown in Tables 9.21 and 9.22 as well as in Figs. 9.21 - 9.23.

9.5.1 DISCUSSION:

Unlike the cases of the three other wastes investigated in this study, the industrial wastes investigated here were a mixture of effluents from three manufacturing processes: brewery, textile and rubber, together with the effluents from such

TABLE 9.21

COD, BOD AND SS REDUCTION IN INDUSTRIAL WASTE IN 26 DAYS RUN

	C O D			B O D			S S		
	Inflt. mg/l.	Efflt. mg/l	% Reduction	Inflt. mg/l	Efflt. mg/l	% Reduction	Inflt. mg/l	Efflt. mg/l	% Reduction
Day	1	2	3	4	5	6	7	8	9
5	1860	1000	46.2				300	140	53.3
7	2340	580	75.2						
11	1310	440	66.4						
12	1320	400	69.7						
14	1260	390	69.1				230	98	57.4
18	1250	310	75.2				875	80	90.9
21	3800	535	85.9						
23	2930	745	74.6				815	140	82.8
26	2250	590	73.8	730	112	84.66			

TABLE 9.22

COD, BOD AND SS REDUCTION IN INDUSTRIAL WASTE IN 91 DAYS RUN

	COD			BOD			SS		
Day	Inflt mg/l 1	Efflt. mg/l 2	% Reduction 3	Inflt mg/l 4	Efflt mg/l 5	% Reduction 6	Inflt mg/l 7	Efflt mg/l 8	% Reduction 9
3	1800	760	57.8				640	71	89.0
7	1750	430	75.4	615	58	90.60	447	52	88.4
10	1800	400	77.8				610	61	90.0
14	1925	350	81.8	598	24	95.99	656	46	93.7
17	2380	350	85.3				993	55	94.5
21	3560	590	83.4	1088	50	95.40	633	65	89.7
24	2280	690	69.7						
26	1680	540	67.9				254	72	71.7
28	1540	520	66.2	498	26	94.78	207	47	77.3
31	1610	465	71.1				460	46	90.4
35	1970	405	79.4	760	18	97.63	951	68	92.9
38	3850	520	86.6				866	90	89.6
42	4200	920	78.0	2000	280	86.00			
45	4025	930	76.9				960	73	92.4
49	2275	520	77.4	750	45	94.00	590	57	90.4
52	2820	420	85.1				1242	115	90.7
56	3700	540	85.4	1280	60	95.31	1148	95	91.7
59	3560	625	82.4				1129	73	93.5
63	4000	580	85.5	1300	34	97.38	955	79	91.7
66	3200	455	85.9				1022	111	89.2
70	3550	485	86.3	1250	56	95.52	616	103	83.3
73	4550	585	89.2				672	118	82.5
77	4450	950	78.7	1925	255	86.75	877	121	86.2
80	4600	980	78.7				872	103	88.2
84	3080	720	76.6	1160	89	92.34	758	116	84.6
87	3150	620	80.3				1164	101	91.3
91	3300	500	84.9	1040	26	97.50			

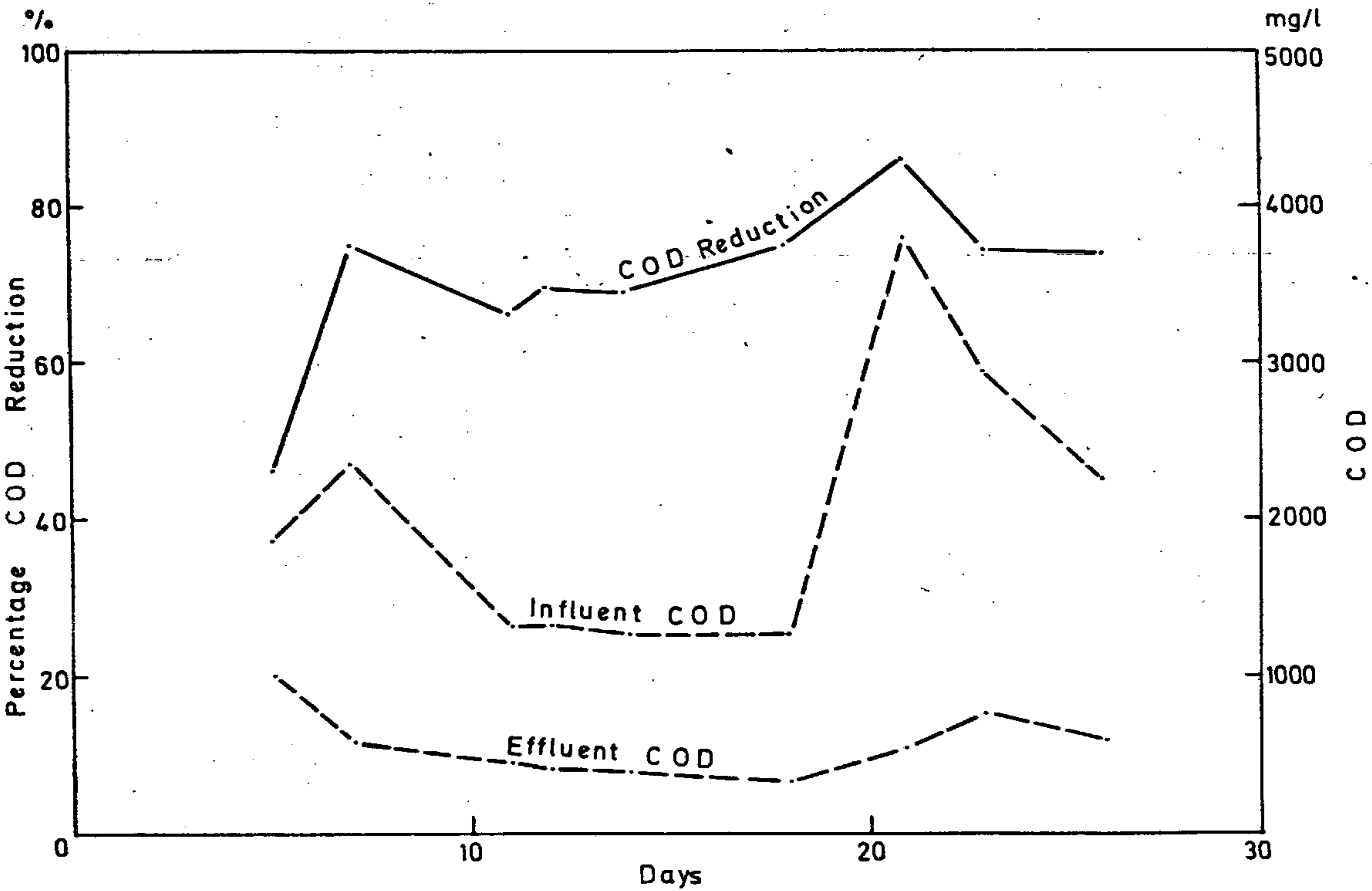


FIG. 9·21 COD REDUCTION IN INDUSTRIAL WASTE IN 26-DAYS RUN

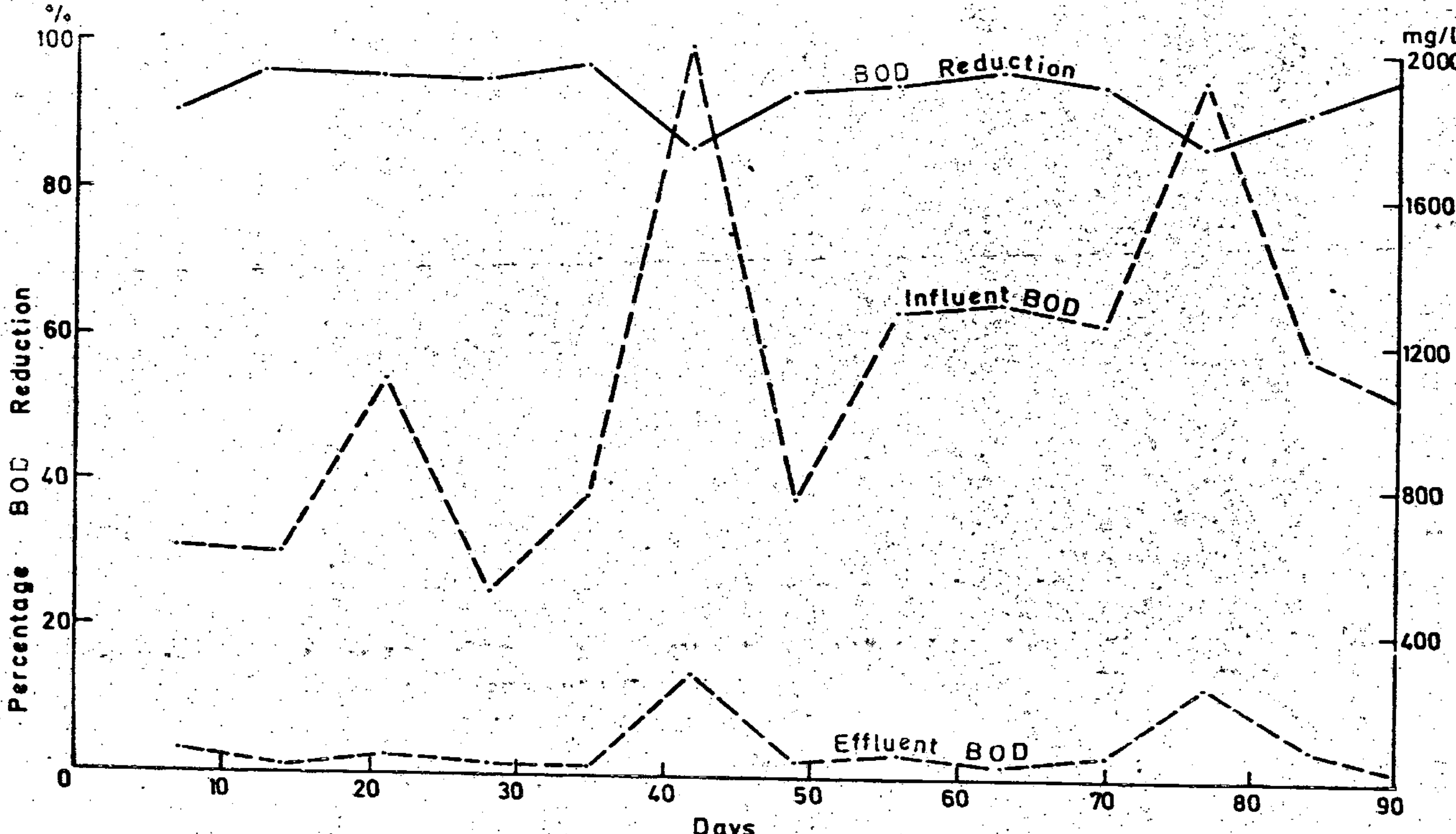


FIG.9·22 BOD REDUCTION IN INDUSTRIAL WASTE IN 91 DAYS RUN

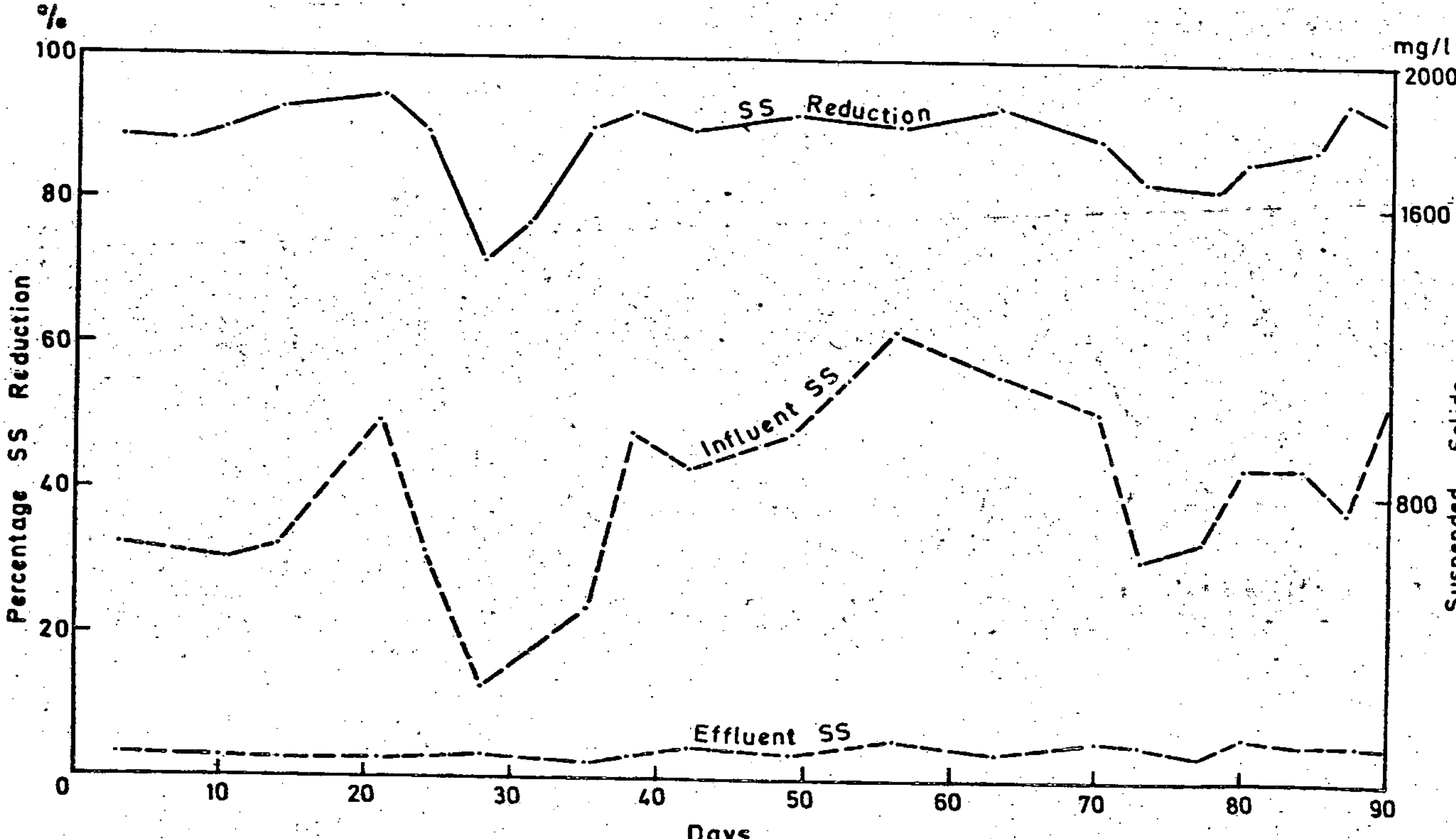


FIG. 9.23 REDUCTION IN SUSPENDED SOLIDS IN INDUSTRIAL WASTE IN 91 DAYS RUN

other supporting minor processes employed in each factory. The influent concentrations in terms of COD were about the same range as those employed in the tests with milk. In Table 9.21 the treatment efficiency in COD removal was low in the first half of this first run of the biodisc on industrial wastes indicating that the micro-organisms were still establishing themselves at this time. The efficiency rose in the latter half. Table 9.22 however shows that high treatment efficiency figures were attained relatively early in both COD, and BOD in the second run.

The figures in COD reduction were on the average slightly higher than those recorded in the tests with milk but the BOD reduction figures were about the same in both cases, with an average well above 90% which is quite comparable with the performance of conventional type treatment plants on wastes of comparable concentrations. Such high performance of the biodisc on a mixture of wastes with an average concentration higher than is usually encountered in domestic wastes is, firstly, evidence of the absence of any components in the three main wastes which could inhibit the biological treatment process in the biodisc and, secondly, further evidence of the already established capacity of the biodisc to give high performance in the treatment of a wide range of wastes.

Again Table 9.21 shows relatively low performance figures in terms of the removal of suspended solids at the beginning of the first run when the micro-organisms were still establishing themselves. The treatment efficiency figures were however high throughout the 91 days of the second run. The figures were on the average higher than the corresponding figures in the tests with domestic sewage and about the same order as those obtained in the latter half of the tests with nightsoil (Tables 9.13, 9.14 and 9.16).

If in Table 9.22 the comparatively large effluent BOD figures of 280mg/litre and 255mg/l on days 42 and 77 are excluded from consideration on the assumption that they were likely adversely affected by errors inherent in the standard method of determining the 5 day BOD, the average effluent BOD in the test was 44mg/litre. The average effluent SS was 84mg/litre. While the biodisc has shown high performance figures in the removal of both BOD and suspended solids, the effluent would require further treatment to make it satisfy the 20/30 Royal Commission Standards. It may however be suggested that in Nigerian circumstances where no legislation at present exists about the quality of trade effluents that could be discharged into rivers and Lagoons a less stringent standard be accepted, in which case the effluent from the Ikeja Treatment Plant might be accepted into the Lagoon. For plants located on the Lagoon where limitless quantities of water are available, dilution of the effluent 2 or 3 times would reduce its concentration to tolerable levels at the point of discharge before further dilution takes place downstream. This of course does not reduce the total pollutional load on the waters of the Lagoon.

In the two runs of the biodisc on industrial wastes no definite sloughing cycles were observed and there was no mass sloughing at any time. As reported earlier in the tests with domestic sewage no sloughing occurred on the last twoee discs, in the 91 days of the longer run. In the tests with nightsoil in which there were three separate runs there was no mass sloughing even when the test was discontinued at the end of the second run after the discs had been running for a total of forty five days. The discs sloughed about four at a time. There was however, mass sloughing between the 32nd and 38th day in the preliminary run in the tests with domestic sewage. In the second run also mass sloughing occurred from the 45th to the 49th day. Finally, in the two runs with the biodisc on milk mass sloughing started on the 40th and 55th day respectively.

The rate of growth of the biomass on the disc which itself depends on the concentration of the waste must be an important factor in sloughing. Even though initially all the discs have equal chances of picking up organics and micro-organisms from the waste, the discs in the first compartment soon get covered with thicker growth than the others. In such circumstances of unequal rate of growth of biomass, mass sloughing in cycles cannot always be expected.

9.6 COD-BOD CORRELATION:

Fig. 9.12 shows that the relationship between the COD and the BOD in milk at different concentrations is a straight line. The line passes through the origin, with a slope of 0.68, 0.65 and 0.64 for the samples tested on the 3 dates 13/7/71, 30/8/71 and 15/9/71 respectively. These slopes are very close, with an average of 0.66. From this the relationship between COD and BOD can be expressed mathematically thus:

$$BOD = 0.66\ COD \quad \ldots\ldots\ldots \quad (9.3)$$

This compares reasonably well with the following equation established by the author in an earlier work with this brand of milk in 1969 (ALUKO, T.M. 1969):

$$BOD = 0.71\ COD \quad \ldots\ldots\ldots \quad (9.4)$$

Plotting BOD against COD from the figures in Table 9.13 in the tests with domestic sewage resulted in too wide a scatter of points for both the influent and effluent samples for an accurate determination of the slopes of the two lines. The following equations were therefore obtained for both from the method of the linear regression of BOD on COD:

$$BOD = 0.71\ COD - 112 \quad \text{(influent)} \quad ....(9.5)$$

$$BOD = 0.16\ COD - 4 \quad \text{(effluent)} \quad ....(9.6)$$

The correlation coefficient r for equation (9.5) is 0.9156 while the corresponding r for (9.6) is 0.3173. The latter of these two values of r is unacceptably small and equation (9.6) must be considered unreliable.

The COD-BOD correlation equations for nightsoil also developed by the method of linear regression from the figures in Table 9.20 are:

$$BOD = 0.29\ COD + 501 \quad \text{(Influent)} \quad ..... (9.7)$$

$$BOD = 0.36\ COD - 176 \quad \text{(Effluent)} \quad ..... (9.8)$$

The correlation coefficient r is 0.9350 and 0.9301 respectively for these two equations.

Finally the correlation equations for the industrial wastes tested calculated from the figures in Table 9.22 are:

$$BOD = 0.45\ COD - 245 \quad \text{(Influent)} \quad ..... (9.9)$$

$$BOD = 0.43\ COD - 171 \quad \text{(Effluent)} \quad .... (9.10)$$

The correlation coefficient r is 0.9304 and 0.8986 respectively for the two equations.

Equations (9.5) - (9.10) obtained by the method of linear regression show various intercepts on the BOD axis instead of passing through the origin as in (9.3) and (9.4). While a negative intercept on the BOD axis could indicate the possibility that a substrate which does not exert a biochemical oxygen demand at a low concentration may in some kcases exert a chemical oxygen demand, a positive intercept would indicate a substance exerting a biochemical oxygen demand without being able to exert a chemical oxygen demand. This would present an unusual situation as most materials that could be oxidised by bacterial action would also be oxidisable by the hot acids in the COD Test. Equations (9.5) - (9.10) obtained by the method of regression analysis will have to be interpreted with some caution for this reason. The indication is that they are not applicable at low concentrations.

CHAPTER X

SEPTIC TANK - BIODISC PLANT DESIGN

10.1 INTRODUCTION

The efficiency of the biodisc process in the treatment of wastes has been demonstrated by the performance of the laboratory scale model in the treatment of the four wastes reported in the last chapter. This Chapter is devoted to the design of a small biodisc plant to fit into an existing septic tank. This upgrading of a septic tank into a biodisc plant is offered as a solution to the problem of high water table which has been shown to render soakaways and soakage trenches ineffective in the low-lying areas of Lagos, particularly during the wet season.

This design exercise differs from the normal in that the shape and capacity of the tank here is already fixed. What remains to be determined is the diameter of the disc that will fit into this shape, the number of discs that will give the required area for the daily flow, and the length of the space that will be occupied by this number of discs. Tank size V which is the largest size in Nigerian building practice is used in this exercise. The dimensions are taken from Table 3.2 in Chapter III.

10.2 DESIGN DATA

Tank Dimensions:

Length = 3.05m

Width = 0.76m

Depth = 1.22m (this is water depth).

Population = 40 persons (soil wastes only)

Organic Loading:

BOD_5 = 65gm/hd/day (total, assumed)

SS = 75gm/hd/day (total, assumed)

Flow:

Assume (i) each user visits toilet 6 times daily,

(ii) capacity of WC cistern is 9 litres.

10.2.1 Organic Loading

Assume (i) 2/3 of total 65gm/hd/day goes through WC system while 1/3 goes through bath, kitchen etc.

(ii) 30% BOD and 60% SS settled in septic tank.

BOD contribution per person per day $= 65 \times 2/3$

BOD loading on discs $= 65 \times 2/3 \times .7 = 30.5$gm/hd/day

Total BOD on discs $= 30.5 \times 40 \times .001 = 1.22$kg/day .. (10.1)

Total SS on discs $= 75 \times 2/3 \times .4 \times 40 \times .001$

$= 0.80$kg/day .. .. (10.2)

Flow per person $= 9 \times 6 = 54$ litres.

Total flow $= 54 \times 40 \times \frac{1}{1000} = 2.16m^3$ ... (10.3)

BOD concentration $= \frac{1.22}{2.16} \times 100 = 565$mg/l ... (10.4)

SS concentration $= \frac{.80}{2.16} \times 1000 = 370$mg/l ... (10.5)

10.3 DESIGN OF BIODISC

10.3.1 Disc Design (BOD)

(i) Average loading on plant of comparable disc size, from Table 10.2 = 10.4mg/m²/day (this is in a temperate climate).

(ii) Loading at 20^{oC} ambient temperature could be greater than 20gm/m²/day (ELLIS, K.V. & BANAGA, B.S. 1976).

(iii) Loading suggested for warm climate of Nigeria = 15 - 20gm/m²/day (SIMPSON, J.R. 1976).

A loading of 15gm BOD/m²/day will be used.

Total disc area required $= \frac{1.22}{15} \times 1000 = 82m^2$

Tank width = 0.76m

Use a disc diameter 0.69m rotating 5cm above waterline. This gives a clearance of 3.5cm between tank wall and disc edge.

Disc area $= \frac{\pi}{4} (D^2 - d^2)$

$= .785 (.69^2 - .1^2) = 0.366m^2$

$= 0.732m^2$ (2 faces)

No. of discs required

$$= \frac{82}{.732} = 112$$

Provide 114 discs in 3 compartments .. .. (10.6)

10.3.2 <u>Length of Biodisc Chamber</u>

Space discs at 2.5cm along shaft, allowing 5cm at either end.

Length of each compartment = $37 \times 2.5 + 2 \times 5 = 102.5$cm
$= 1.03$m.

Length of 3 compartments $= 3.09$m.

1 compartment will be provided at the effleunt end of the existing septic tank and the reaining 2 in an extension to the tank.

10.3.3 <u>Retention Time in Biodisc Chamber</u>

From Fig. 10.1, cross-sectional area of biodisc chamber $= 76\ (22.0 + 12.5) - 2 \times \frac{1}{2} \times 22 \times 22$
$= 2138\text{cm}^2 = .214\text{m}^2$

$L = 3 \times 1.03 = 3.09$m

Vol. $= 3.09 \times .214 = 0.661\text{m}^3$ .. .. . (10.7)

Assume that at thickest growth each disc is 7.5mm thick

Area of immersed disc $= \frac{\pi}{4} \times .69^2 \times \frac{1}{2} - .69 \times .05$
$= .1524\text{m}^2$

Volume of 114 discs $= .1524 \times 114 \times .0075$
$= 0.130\text{m}^3$
$= 0.132$, allowing for 2 baffles.

Net volume of chamber $= .661 - .132 = 0.529\text{m}^3$ .. (10.8)

Flow $= 2.16\text{m}^3$ in 24 hours

Retention Time $= \frac{.529}{2.16} \times 24 = 5.88$ i.e. 6 hours..(10.9)

10.3.4 <u>Sludge Production in Biodisc</u>

Assume (i) biological solids = 0.4 BOD, (ELLIS & BANAGA)
(ii) 2% sludge concentration

Sludge weight $= .4 \times 1.22 = .488$kg/day.

Volume at 2% concentration $= .488 \times \frac{100}{2} = 24.4$ litre/day ....(10.10)

10.4 DESIGN OF SECONDARY SETTLING TANK

Design for 4½ hours detention in rectangular tank.

Volume of tank required = $\frac{2.16}{24} \times 4.5 = .405m^3$

Double this to allow for sludge storage = $0.81m^3$ ... (10.11)

No. of sludge storage days assuming 30% sludge digested in storage = $\frac{.81}{2} \times \frac{1000}{24.4} \times \frac{1}{7} = 26.64$ (small)

Use cross-section 0.76 x 1.22 (m),

Area = $0.93m^3$

Length = $\frac{.91}{.93}$, say 1m.

Use 1.6m length, which gives a better length - width ratio (2.1), and provides more storage for sludge.

10.5 DESIGN OF POWER UNIT

Hydraulic loading on discs

$$= \frac{2.16}{114x.366} \times 1000 = 51.8 \text{ litres/m}^2$$

HP consumed per MGD plant capacity at 52 litres/m^2 hydraulic loading = 48.0 (Autotrol Corporation)

Add 331/3% to make up for occasional load imbalances and starting up operation.

Installed hp = 48 x 1.33 = 64 MGD

Total flow = $2.16m^3$

= 568 U.S. gallons

$$\text{HP required} = \frac{64 \times 568}{10^6}$$

$$= .036$$

Instal a ¼ hp plant.

10.6 DISCUSSION

The plant is most probably over-designed as regards both the hydraulic and organic loading. It is certain that all the 40 occupants of a building will not stay home all 24 hours of a day. The use of toilet facilities by adults at work and children at school will reduce significantly the average sewage flow to the home treatment plant, with a corresponding reduction in the loading on the discs. This will also increase the already generous allowance made for sludge storage in the secondary settling tank.

The decision to design for soil wastes only in the upgraded septic tank might be criticised on the grounds that relatively highly polluting discharges from kitchen sinks etc., would go directly into the public drains, which is undesirable from public health considerations. Unfortunately the present position in Nigerian building practice is to exclude kitchen discharges from septic tanks. The origin of the practice is not clear. Health authorities at the present stage of development have reconciled themselves to this practice.

A defect in this design is the absence of a gas vent at each side of the biodisc compartment in cross-section. These vents are a usual feature of the Imhoff Tank and it is through them that gases from the digesting sludge at the bottom of the tank escape into the atmosphere. In the present design such gases will bubble through the sewage undergoing biological oxidation in the biodisc compartment with a possible lowering of treatment efficiency. To make provision for even an 8cm vent at either side of the biodisc would reduce the diameter of the disc from 0.69m to 0.53m, which is a reduction of 41% in the disc area, with a corresponding increase in the number of discs and the length of the biodisc compartment. In the circumstances of the rather small width of the existing tank gas vents cannot be provided.

One problem of the upgraded tank is the disposal of the effluent from the secondary settling tank. Effluent disposal from the normal septic tank takes place in the soil and, while posing the dual problem of subsoil and groundwater pollution, it does not constitute a nuisance. In the upgraded plant however, the effluent will have to be discharged into a water-course where one is near and is of sufficient capacity to accept the effluent, or into the usual open drain along the street. It is believed that the design will produce an effluent of sufficient quality to make it accaptable into these drains which are known to carry heavily polluted water now anyway. There will have to be good liaison with the health authorities in this regard.

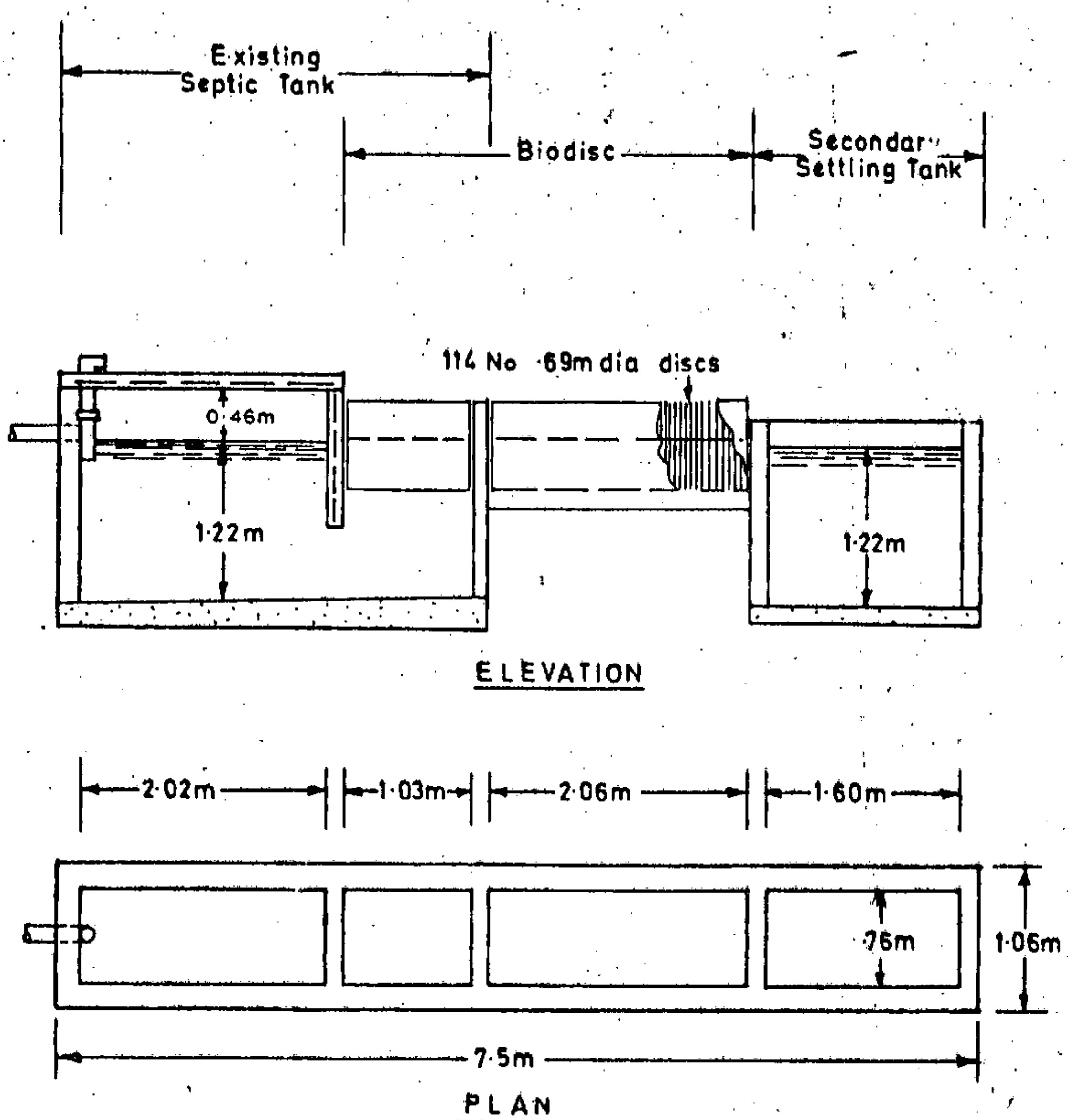
TABLE 10.1

OPERATING CHARACTERISTICS OF 4 EXISTING SMALL DIAMETER BIODISC PLANTS

No.	Material	Disc diameter (m)	Loading m discs		Degree of treatment (BOD_5)	Rotational speed r.p.m.	Authority
			gm/m²	Litres/m²			
1	Polysterene	0.85	8.1	40.9	94%	11	Antonie
2	Polysterene	1.75	8.2	41.2	95%	2-4.6	Antonie
3	Galvanised XPM	1.00	6.0	?	96%	apprx. 1.0	Bruce, Brown and Mann
4	?	0.61	19.24	65.0	?	?	Bennett

FIG. 10.1 BIODISC PLANT FROM EXISTING SIZE V SEPTIC TANK

(Scale. 1:50)



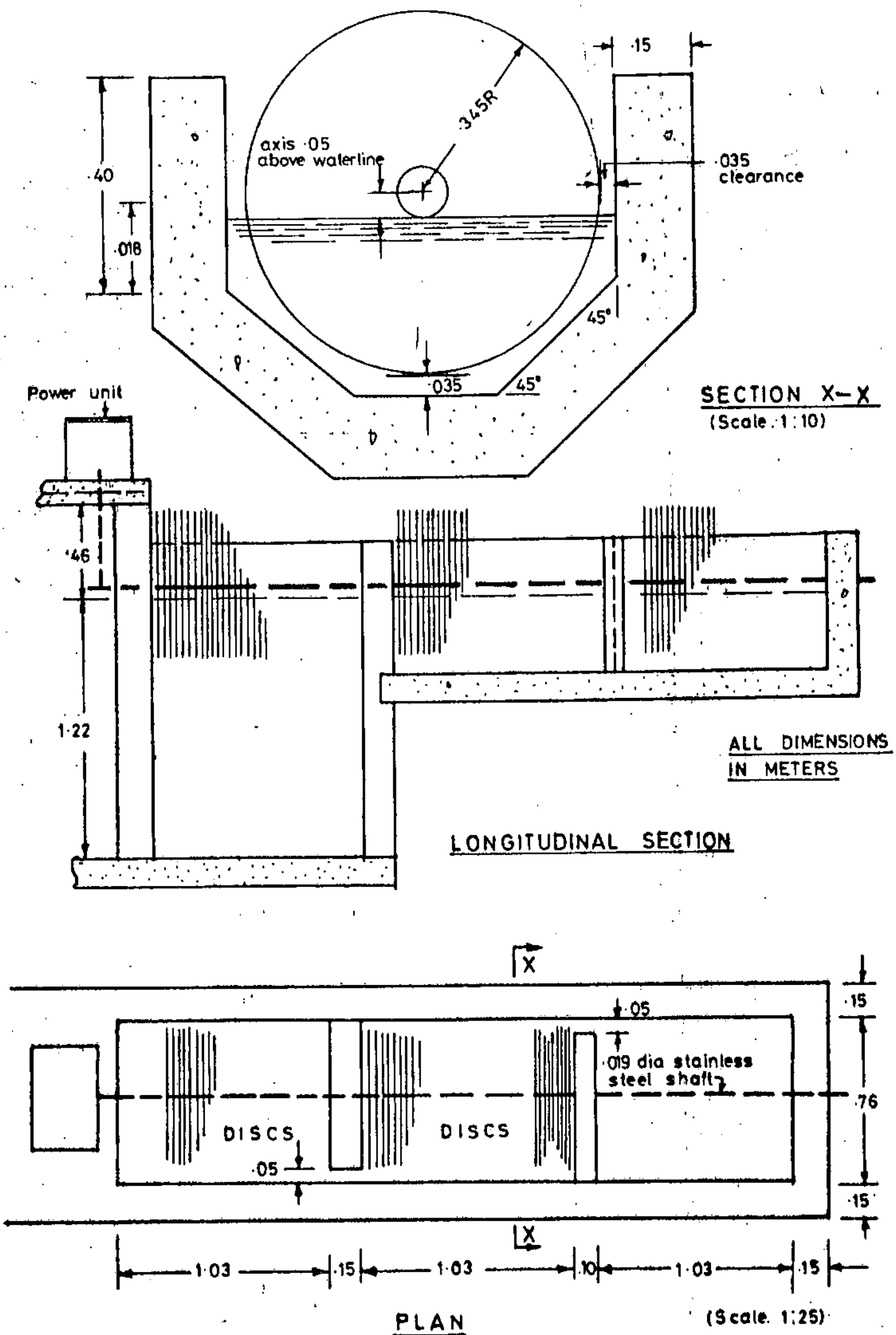


FIG.10·2 DETAILS OF SEPTIC TANK
-BIODISC PLANT FOR 40 PERSONS

CHAPTER XI

DISCUSSION AND CONCLUSIONS

11.1 INTRODUCTION

This thesis has highlighted in the introductory chapter the serious difficulties which will make the central sewerage system remain an unrealisable ideal in towns in Nigeria for a long time to come (1.1 - 1.4). It has also made a case for an interim programme of limited waterborne sanitation facilities in certain areas of towns which meet a number of criteria which are defined in the thesis (1.5). However such facilities in the Lagos Area are shown to cater for only 1.2% of the population while the remaining 98.8% depend on either the nightsoil conservancy system or the septic tank for their sewage disposal (3.1). The thesis discusses the serious shortcomings which are inhereht in the nightsoil conservancy system and make it the least satisfactory method of sewage disposal. Much of the work however is devoted to a study of the septic tank leaching system in Nigerian building practice particularly in the peculiar circumstances of the high water table in the Lagos Area.

1.2 PERCOLATION TESTS IN SOILS FOR SEPTIC TANK LEACHING SYSTEM

The treatment of sewage is only partial in the septic tank and is not complete till the sewage has gone through the leaching system. For this reason the leaching system is an integral part of the septic tank installation.

The method of performing percolation tests for the determination of the percolation rate of water in soils was developed by researchers at the Robert A. Taft's Sanitary Engineering Center at CinCinnati, U.S.A. in holes in which water discharged into the soil from both the sides and the bottom. Such holes were found early in this study to suffer from a number of disadvantages: they could not be dug to the required depth in the sandy - laterite soil in Lagos without the sides caving in; they could not be dug to true and uniform cross-sectional dimensions; the erosion of material from the sides of the hole tended to cause early clogging of the soil at the bottom. A new method of percolation tests in lined holes was developed in this study to eliminate or minimise these disadvantages (6.2.3).

Since the percolation rate of water determined from an unlined hole in a soil has been linked by the Cincinnati researchers with the rate of sewage application in an empirical formula, it was necessary to establish in this study a relationship between the percolation rate in a lined hole and the percolation rate in an unlined hole before the empirical formula can be used to evaluate percolation test results from lined holes. Tests in both natural and made soil in this study indicated that water percolated faster in a lined hole than in an unlined hole, (6.2.3, 6.2.6, 6.3.5). In a made soil of laterite the rate in an unlined hole was found to be 0.85 of the rate in a lined hole. The corresponding rates in a sandy-laterite natural soil was found to be 0.85 at one point and 0.97 at another point located only 10m away. This is a variation of 14% in this ratio in the same soil in the short distance of only 10m, and is a relatively mild example of the great variability in the percolation characteristics within short distances in the soils tested in this study.

A mathematical formulation for percolation tests was made in Chapter V of the Thesis. The expression for s in terms of H and t (defined at page 84) was found to be unwieldly, and depends on a constant of permeability which itself varies with time t(5.4.3).

The results of the percolation tests done both in lined and unlined holes however indicated that the observed values of s fitted into the mathematical model:

$$s = pt^q$$

where s = progressive drop in the level of water in the hole from the beginning of the test (cm)

t = time from the beginning of the test (mins),

p, q = constants which varied from hole to hole.

This was found to be correct within 5% of the measured value of s most of the time, and was found to be more reliable in the calculation of the percolation rate in the last 10 minutes of a 60 minutes test as any error made in the measurement of s either at $t = 50$ or $t = 60$ is distributed through the whole range of readings from $t = 0$ to $t = 60$. This contrasts with the original Senator Taft's Sanitary Engineering Center's method in which the measurement of s at $t = 50$ is just subtracted from the measurement of s at $t = 60$, which would give a result that would absorb all the error in s either at $t = 50$ or $t = 60$. (5.4.3)

11.3 Empirical Formula for Sewage Application Rate:

The empirical formula developed by Senator Robert A Taft's Sanitary Engineering Center, Cincinnati, U.S.A., linking the percolation rate of water in a soil with the rate of application of sewage on the soil is:

$$q = 5t^{-\frac{1}{2}}$$

where q = rate of sewage application, in U.S. gallons per sq. ft.

t = percolation rate of water, in munutes. per inch (time for level of water to drop 1 inch in last 10 minutes of a 60 minutes test).

This formula is criticised in this thesis for the following reasons. Firstly the researchers did not appear to have given sufficient consideration to the non-homogenous nature of soils which results in considerable variation in the water absorption properties of soils as observed in holes spaced only short distances apart (5.4.2). Secondly the researchers appeared to have put much reliance on the evidence of householders in evaluating leaching field efficiency (5.4.2). Finally it is difficult to justify the use of a formula developed in one part of one country for universal application in other countries several thousand miles away and comprising a wide range of climates (5.4.2).

A suggested test for developing a formula would be to make sewage effluent and water percolate simultaneously in a number of holes spaced some 3m apart in the same soil. Percolation tests would be made in each hole twice or thrice a day and the tests conducted over a period of 3 months. In that period, the microorganisms in the holes in which sewage effluent was percolating would have been subjected to a number of growth and decay cycles which would give sufficient indication of the worst effluent percolation rate to be expected in an operational leaching field in that soil. This information could be used in designing the leaching system in that soil. Comparison of the sewage percolation rate so obtained with the percolation rate of water from the tests in the holes in which water was percolating would lead to the desired link between the two rates of percolation.

11.4 THE P.W.D. STANDARD SOAKAWAY DESIGN

The P.W.D. Standard designs for both the septic tank and its soakaway and sockage trenches have been used in Nigerian building practice for some four decades. The basis for the designs is now obscure. The shape and size of the soakaway has been criticised in this study.

The results of tests done on a laboratory scale model soakaway lead to the conclusion that the effluent issuing from the saw-cuts spaced along the invert of the sewer penetrating the diameter of a soakaway wets only a narrow strip at the bottom of the soakaway. The area of this strip amounts to only a small fraction of the total area at the bottom. As it is only this strip that is being effectively used, theoretically only this amount of area need have been provided in the first instance. The existing P.W.D. design is therefore considered uneconomic

Assuming however that the empirical formula linking the percolation rate and the rate of sewage application on a soil already described above is applicable to soils in Nigeria, the three alternative sewage effluent loading rates of 98, 196 and 294 litres/m^2 used in the P.W.D. Standard design correspond to a percolation rate range of 0.49 - 4.39 minutes* which represents only 7% of the 60 minutes range in which the authors of the empirical formula consider soils can be used for leaching systems. The vast majority of soils in which these standard designs are used in Nigeria are known to have percolation rates far in excess of 4.39 minutes. They must therefore be considered overloaded in terms of the empirical formula. Such overloading most probably takes place on the narrow central strip mentioned above, leading to the excess effluent which the central strip cannot absorb flowing sideways into the area now considered ineffectively used both sides of the strip, from where it leaches away. These two shortcomings, that is the uneconomic use of area and the overloading of the central strip, would tend to cancel out. In these circumstances it appears reasonable to continue using the existing P.W.D. design <u>area</u> pending further work on the rate of sewage application on soils. The foregoing however applies only to the <u>area</u> and not the <u>shape</u> of the soakaway. The rectangular shape, preferably long and narrow is decidely more effective than the existing circular shape (4.4).

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*definition at page 274.

11.5 THE WATER TABLE IN LAGOS

Various authorities recommend that the depth of the water table below the bottom of the soakage trench or the soakaway should be at least 1.22m (7.1). Also the P.W.D. standard design for the soakaway specifies a minimum depth of 0.61m. This means that for a soakaway not to be affected by the water table, the minimum depth to the water table should be 1.83m (7.2). The results of six years daily observation on the water table at a point at Onikan on Lagos Island reported in this thesis show that not once during the study was the depth of the water table below ground level up to this minimum.

From a consideration of the capillary water held in the soil at the Onikan location obtained from a determination of the grain size distribution a smaller figure of 1.07m was calculated for the minimum depth of the water table required below ground level. The results of this study show that the water table was low enough to satisfy this less rigorous criterion only 6.4% of the period of investigation and that this short period occurred mostly during the exceptionally dry, dry season of 1972-73 (7.6).

A method was evolved in this part of the study of predicting from measurements of the water table at Onikan the depth of the water table at two points about 1 km away giving results that were most of the time within 10% pf actual measurements at the new sites (7.5). The results seem to indicate that the two new locations at Simpson Street and Obalende were affected by the same rainfall.

It was concluded from the study on the water table at the points selected on Lagos Island that the shallow depth of the water table in the low-lying areas of Lagos Island, Victoria Island, Apapa and the Mainland makes the septic tank soakaway system an unsuitable method of sewage disposal in these areas (7.5).

11.6 THE BIODISC PROCESS IN THE TREATMENT OF WASTES

The efficiency of the biodisc process in the treatment of liquid wastes already established by various authors was demonstrated in this study in the treatment of milk, domestic sewage, nightsoil and industrial wastes with the laboratory scale model. The distinguishing feature of the process is the use of many discs rotating on a horizontal axis to obtain a total large surface area giving opportunity for a large number of micro-organisms to come into contact with the organic compounds in a waste (8.4). In addition the laboratory scale model is a useful device for determining the treatability of a waste as the model is little different in design and operation from an actual field plant (8.4). Among the advantages claimed by several authors and plant manufactures are low initial and operational cost, uniform performance irrespective of variation in loading, simplicity in operation and suitability for the treatment of wastes from small populations (8.4).

In the tests with each waste a COD - BOD correlation was established which facilitated the work in view of the much greater speed at which COD tests can be done relative to the standard 5-day BOD Test. The equation determined graphically in the tests with milk (BOD = 0.66 COD) agrees reasonably well with an earlier equation (BOD = 0.71 COD) established by the author in an earlier work with the same brand of milk. Correlation equations for the other wastes developed by the method of linear regression in view of the wide scatter of the points when BOD and COD were plotted against each other indicated in all cases small intercepts on the BOD axis. While a small negative intercept could be associated with a substance exerting a COD while not exerting a BOD, which some substances do, a positive intercept would represent a rare situation in which a substance would be exerting a BOD without exerting a COD. For this reason the equations would not be applicable to low concentrations.

In all the tests treatment efficiency figures above 90% in terms of BOD and SS reduction were obtained. The tests were run at a flow of 14 ml/min corresponding to a retention time of 9 hours in the aerobic chamber. The laboratory scale model was operated at a low speed of ½ r p m corresponding to a peripheral velocity of 0.30m/min. This is well below the 18.3m/min optimum peripheral velocity recommended by Autotrol Corporation of U.S.A. who have had wide experience in the development and marketing of biodisc equipment. Increasing the speed from the low figure of 0.30m/min would most probably increase the efficiency.

Most of the treatment was observed to have occurred in the first compartment in all the tests. This makes the need for having more than two or three compartments questionable except for the marginal advantage of additional sedimentation that occurs in a multi-compartmental device in which the waste flows through in a tortuous path.

The tests with the milk confirm the need for proper pH control in the biodisc process. Addition of pre-determined quantities of 0.1N sodium bicarbonate kept the influent pH at the desired level of approximately 6.5. The effluent pH was however most of time about 8.8, which was outside the range of 8.0 - 8.5 which Oginsky and Umbreit state to be the upper limit for bacteria growth.

The suggestion made in the last Chapter for converting the septic tank into a small biodisc plant upgrades the tank generally. It also overcomes the particular problem of the ineffectiveness of the soakaway in the low lying areas of Lagos. The need to fit the disc into the width of the existing tank is a severe constraint which makes the disc diameter small and uneconomic, resulting in a larger number of discs and a greater length of plant than would normally be necessary (10.5). This design follows the existing Nigerian building practice which excludes kitchen wastes and bath water from the septic tank. This practice results in a relatively high level of pollution in the discharges into the public drains. Health authorities in Nigeria have already reconciled themselves to this undesirable situation.

The recent installation of a nightsoil treatment plant is admission by the authorities in Lagos that this unsatisfactory method of collection and treatment of sewage will continue for some time yet. The very good results obtained in the treatment of nightsoil with the biodisc process would indicate that the process could be used as an alternative to the aerated lagoons which are yet to be proved in Lagos. These very good results are however qualified by the realisation that high concentration of BOD and SS in the sludge in the settling tank will still be encountered during sludge treatment.

The features of particular significance in the perfor ance of the biodisc in the treatment of nightsoil are, firstly, the high efficiency of the process in the treatment of recirculated effluent,/secondly, the fact that salinity in the dilution water has no adverse effect on the treatment efficiency. The significance of the first lies in the fact that as a single-pass treatment of nightsoil is most unlikely to result in a satisfactory effluent such effluent could be passed on for further treatment in a second treatment unit downstream. The importance of salinity having no adverse effect on the treatment process lies in the fact that lagoon water could be used as dilution water in plants installed in coastal areas.

/and

The need for dilution before nightsoil can be treated in the biodisc process is considered a shortcoming of the process in the treatment of this waste. Dilution is the reverse of sedimentation which aims at concentrating the solids in a waste before further treatment in a digester. Diluting nightsoil increases the volume of the waste handled, with a corresponding increase in plant capacity and cost. Anaerobic treatment in oxidation lagoons and digesters will be more satisfactory than any aerobic process in this regard. The widely used method of trenching is economical, simple and effective. There is however the need to recognise the danger of ground water pollution in this method and to ensure that wells are not sited near or downstream of a nightsoil trenching ground.

11.7 SUGGESTION FOR FURTHER WORK

The following suggestions are made for further work in areas related to some of the topics covered in this Thesis:

(a) Investigation into the average BOD produced per person per day in Nigeria;

(b) A full scale survey of existing water-borne sanitation facilities in Nigeria, with particular attention to capital and operational costs, the capital costs distinguishing between sewerage costs and equipment costs wherever possible;

(c) Determination of relationship between the percolation rate of water and the percolation rate of sewage effluent in the same soil;

(d) Investigation into the use of locally produced materials for discs in full scale biodisc plants;

(e) A study of the effect of humidity, high temperature and insect breeding on full scale operating biodisc plants.

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