"A Study of Upward Flow Pilot Plant Filters"

by

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DECLARATION

THIS THESIS IS MY ORIGINAL WORK AND HAS NOT BEEN PRESENTED FOR A DEGREE IN ANY OTHER UNIVERSITY.

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SUPERVISOR

A B S T R A C T

In general, it has been observed from past experience that there are numerous limitations and operational problems associated with the conventional downflow filters. Intensive research in many parts of the world, particularly in Europe, America, and Russia lead to the introduction of many improvements on the operation and design of filters. An upward flow filter is one such innovation which was found to offer many advantages over the conventional downflow filters; in terms of compactness of plant, turbidity removal, higher filtration rates and the generally low operational and maintenance costs.

It has been identified that one of the major causes of slow sand filter failures, in Kenya is high raw water turbidity. There is therefore a need for research to alleviate this problem.

Two pilot plants were set up. One was at the Environmental Health Engineering Laboratory. This pilot plant utilised artificially prepared turbid raw water. The parameters of turbidity, colour, headloss development and pH variations were monitored. Nine test runs were carried out in all using variable filtration rates of 0.2 m/hr to 1 m/hr. The

influent turbidity varied from 10 FTU to 170 FTU. No pretreatment of the raw water was provided.

The second pilot plant was set up at Kabete water works with naturally turbid water. No pretreatmen was also provided. The parameters of turbidity colour and pH were monitored. An attempt on microscopic investigation into the presence or absence of Plankton life in the filter was eventually abandoned when the slides inserted into the filter became constantly covered with sand. Four test runs were made using this filter.

Experiments with fine sand media of effective size 0.22 mm and coefficient of uniformity of 2.46 gave unsatisfactory results. The sand media was too fine and lower depth of the filter media tended to clog rapidly with high values of raw water turbidity.

Recommendations on areas of future research have been made.

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SYMBOLS AND NOTATIONS

Most symbols have been explained where they appear in the text. Some of these symbols and notations used in the text are listed below:

μ	-	Dynamic Viscosity
ν	-	Kinematic Viscosity
ϕ	-	Diameter
Ac	-	Cross-sectional area
d	-	Average grain diameter
ρ	-	Porosity
f	-	Initial Porosity
g	-	Acceleration due to gravity
FTU	-	Formation Turbidity Unit
^oH	-	Degree Hazen
K	-	Headloss Constant
λ	-	Filter Coefficient
α, β, γ	-	Filter Coefficient
C	-	Concentration of suspension
Co	-	Initial Concentration of suspension
WHO/IRC	-	World Health Organisation/International Reference Centre for Community Water Supply.

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PARTS I

PART I

LITERATURE REVIEW

CHAPTER I INTRODUCTION

1.1 Kenya's National Requirement

A safe and convenient water supply is of paramount importance to human health and the well-being of society as a whole. A satisfactory water supply for domestic purposes, such as human consumption and personal hygiene is characterised by adequate standards regarding the availability of water, its quality, quantity and the reliability of the supply. Data collected periodically by International Agencies show that a substantial part of the World's population, in particular a great many people in developing countries, do not have reasonable access to an adequate water supply. In recent years many efforts have been made to improve this situation.

In her efforts to supply safe and wholesome water to her population, the Kenya Government created in 1974 a Ministry of Water Development which is charged with the overall responsibility for water development and supplies; water conservation and control of water catchment and water quality and pollution control. The long term objective of the Ministry is to provide safe and wholesome water to the entire population by the year 2000 A.D. This is in keeping with Kenya Government's health programme which is geared towards the achievement of universal health by the year 2000, a goal set by the World Health Organisation.

To achieve this gigantic task, the Government has been greatly assisted by many Governments and International Agencies. This help is usually in the form of financial support as well as technical manpower. The current emphasis is to provide piped water to convenient points for distribution to people's homes. In most of the water supplies, treatment given to the raw water varies with the quality of raw water like in some supplies disinfection in the form of chlorination is the only treatment provided. In a country where many people in the rural areas still walk for many kilometres in search of water, providing piped water to a reasonable walking distance from their homes is a great relief indeed. But there is no doubt that as the country moves forward in her development efforts, more elaborate treatment processes for raw water are to be applied to satisfy the improved standard of living of her population.

1.2 Water Sources in Kenya

Kenya is a country of great contrasts. Two thirds of the country are semi-arid to arid land and support less than one third of the total population. This is the region that receives less than 500mm of rainfall annually. The main source of water in these areas is ground water extracted from wells and boreholes. In many cases, no treatment is provided for this type of water source. Treatment like removal of iron, fluorides or desalination may be necessary. But what generally happens is that around the springs, wells and boreholes, proper fencing at a sufficient distance from the source is applied. This is to protect the source from contamination. As for bore-holes, submersible pumps are usually located at least 2 metres above the bottom to protect them from silt and sand perticles.

In contrast to this is the well-watered part of the country supporting more than half of the country's population. Here are to be found many streams and rivers which originate from the highland mountains and are the main source of water to both the rural and urban population. In all these rivers and streams, the quality and quantity of the water varies from season to season. Thus in raining seasons, the rivers and streams have high volumes of water with very high turbidities. In dry seasons the flows are low and the water have low turbidities. These great variations have been characterised with large amount of chemicals being used during the raining seasons for coagulation. In most treatment plants where slow sand filters are in use, filter breakdowns are a common phenomenon.

1.3 Water Standards and Treatment Criteria in Kenya

Kenya, like many other developing countries has adopted the WHO standards for drinking water. The general characteristics of good drinking water may be summarised as follows:- It must be free of pathogenic organisms toxic substances, and an overdose of minerals and organic material; to make it pleasant, it should be free of colour, turbidity, taste and odour; moreover, it should contain a high enough oxygen content and it should have a suitable temperature

However, due to financial constraints it has not been possible to strictly follow the WHO standards in all aspects. With such an enormous task undertaken by the Government of Kenya of supplying piped water within a short period of less than twenty years to the whole nation, this selective attitude is quite understood. For the immediate need for the population of this nation is to obtain water within reach of their homes. For a long time, they have been used to fetching untreated raw water from the rivers.

But the government aware of the danger of water related diseases has directed that all projects shall supply disinfected water. The normal procedure is to dose the water with chlorine at such a concentration that after one hour there is a residual of 0.2 ppm free chlorine (pH 6.0 - 8.0). To allow for sufficient disinfection at least 30 minutes shall be allowed between the point of application and till water reaches the first consumer. Normally a solution of tropical chloride of lime (TCL) or calcium hypochlorite is used. Where slow sand filtration is applied, no further treatment is usually provided.

A general classification of the relationship between water-related diseases and water quantity and quality with examples for each category and preventive strategies against the occurence of such diseases is illustrated in table IA.(see page 8).

As is shown in the table, first the water quantity, the accessibility and reliability of supplies should be improved, then efforts should be made to improve the bacteriological quality of the water. This may be done by measures aimed at the prevention of pollution of raw water sources with faecal material or treatment of the water with purification methods which allow for a considerable improvement of the bacteriological quality such as slow sand filtration or chlorination. Hence the current government's emphasise of disinfection as the only compulsory form of treatment required.

One other constraint apart from the financial aspect on the part of the government in the provision of safe and wholesome water supply is the prevailing difference between the urban and rural environment, cultural and socio-economics. The concept of commercialisation of water supply as a self-supporting proposition, let alone a profitable one, can seldom gain acceptance among most of the rural population. The rural population has viewed water supply as a social ammenity which the government should provide free.

Hence the approach and strategy adopted for rural water supply programmes calls for special techniques which are simple, reliable and economical rather than "scaled down" versions of urban installations. For it is noticeable in this country, like so many other developing countries, that priority has almost invariably been given to urban projects, even though the rural communities may represent the major proportion of the country's population.

There are a number of reasons for thiswhich include the greater political pressure that can be brought by large communities, the increased hazards of epidemics and the capacity to pay for an improved water supply.

1.4 Need for Research

As mentioned earlier, the immediate objective of the government is to provide safe and wholesome water to all homes by the turn of the century. Treatment of raw water has been kept generally to the minimum possible, particularly in the rural areas. This involves in many cases disinfection with chlorine. The major constraint being encountered is the lack of technical manpower and to some extent lack of finance. Provision of complete treatment works in all water supply projects cost a lot of money which could be used to finance other new water schemes and at the same time no enough technical personnel is available to operate these treatment works satisfactorily.

Category of Transmission Mechanism	Examples	Preventive Strategy
1. Water borne infections		
- classical - non-classical	Typhoid, cholera Infective hepatitis	Improve water quality and prevent casual use of water from other unimproved sources.
2. Water washed infections - skin and eye infections	trachoma, scabies.	improve water quantity and water accessibility; improve hygiene.
3. Water based infections - Penetrating Skin - inpested	Hookworm Schistoso-miasis Guinea-worm	decrease need for water contact, control snail population and improve water quality.
4. Infections with water related insect vectors.	Malaria	Improve surface water management, destroy breding sites of insects and decrease need to visit breeding sites.
- biting near rivers	sleeping sickness	
- breeding in water	yellow fever.	

TABLE 1A

Ref. (WHO/IRC for Community Water Supply
Technical Paper series No. 1I.

Though this reasoning was plausible in the early stages of the programme, it can no longer be accepted in its totality as more and more water schemes are commissioned. Presently, a large fraction of the money earmarked for the Ministry is used on maintenance and operation work. It is also apparent that a thorough follow up work is needed to assure the effectiveness of the whole policy. This calls for highly skilled manpower, which the Ministry has been undertaking by providing various courses to her personnel within and outside the country. This is in the hope of strengthening the local manpower supply to the Ministry to be able to carry on effectively the development of water supplies, operation and maintenance and the required research work. It is also a clear manifestation of the association between the University and the Ministry of Water Development in their common search for ways and means of developing and provision of water to the Kenya Community.

As mentioned earlier, one of the most important sourcesof water is surface water; particularly river water. With increasing human activities, particularly in the field of agriculture and livestock development, soil erosion has become a major concern. Not only is soil ercsion becoming a major source of pollution, but the result of high turbidities have had a marked effect on the raw water treatment works. In cases where pretreatment of raw water is applied before filtration, increasing amount of chemicals is being used, whereas in cases where no pre-treatmentof raw water is applied, high turbidities have significantly contributed to the ineffectiveness of filtersas treatment units. This is very apparent during the raining seasons.

A survey carried out by the Ministry of Health in 1976 into the state of some of the slow sand filters in the country illustrates the extend to which high raw water turbidities have contributed to the ineffectiveness of the filters (see table 2B pages 11-13).

As seen from the table, it is obvious that the cause of most of the filter failures is due to high raw water turbidities. There is therefore a need for research into ways of alleviating this problem. One method is to institute river catchment management to lessen the impact of soil erosion. Such a move, particularly if applied strictly, will be beneficial in the long run. But for the present moment, there is an urgent need to find immediate solutions to this problem.

Province & District	Name of Scheme	Condition of filters	Remarks
1. <u>Central</u>			
Kiambu	Nembu	Not working	very turbid raw water particularly during rainy season, commissioned 1976.
Nyeri	Karembu	Good performance	Raw water clear, commissioned 1972.
	Tumu Tumu	Good performance	Commissioned 1972.
	Mihuti	Under Construction 1977.	
	Mutathi-ini	abandoned	Raw water turbidity is high commissioned 1976.
Nyandarua	Ol joro-Orok	Poor Performance	Poor construction
	Githunduti	abandoned	High raw water turbidity
Muranga	Gatheru	Poor Performance	High raw water turbidity, commissioned 1975.
2. <u>Western</u>			
Bungoma	Kapsakwony	not working	Poor construction high raw water turbidity commissioned 1976.

TABLE 2B.

RUNJI, Cecil; Performance of Slow Sand Filters in Kenya (M.Sc. Thesis 1978) Department of Civil Engineering, University of Nairobi.

Province & District	Name of Scheme	Condition of filters	Remarks
	Khasoko	under construction (1978)	
	Myanga	"	
4. <u>Nyanza</u>			
Kisii	Ogembo	Fair	Commissioned 1965
South Nyanza	West Karachuo-nyo	Fair	Commissioned 1973 high raw water turbidity, but settling of raw water before filtering is done.
5. <u>Eastern</u>			
Embu	Riakanau	-	under construction (1978)
Machakos	Mutituni		"
	Mitaboni		"
6. <u>Rift-Valley</u>			
Baringo	Kabarnet	Good Performance	clear raw water commissioned 1954.
Kericho	Sigor	Not working	commissioned 1970. Break-down of operational organisation.

Province & District	Name of Scheme	Condition of filters	Remarks
	Ndanai	Not working	No specific reason given. commissioned 1970.
	Longisa	Poor Performance	High raw water turbidity. commissioned 1973.
Samburu	Maralal	Poor Performance	High raw water turbidity

The more conventional way is to apply pretreatment to the raw water before filtration. In the case of rapid filters, the use of chemical coagulant aids and the provision of coagulation, flocculation and sedimentation basins lessen this problem. In water works where slow sand filters are used, the conventional ways of applying pretreament are the use of the following systems: storage basins, plain sedimentation basins, river bed filtration, rapid roughing filtration, and horizontal-flow coarse - material prefiltration. Another way of dealing with this problem is by the use of coarse to fine filtration process. One filter that has the advantage of coarse to fine filtration system is an upward flow filter. The performance of upward flow filters has not been fully investigated particularly in use with raw water with high turbidities and no pretreatment provided for.

This applies particularly in developing countries, such as Kenya .Presently, there is only one upward flow filter in the country. This is at Mumias and is used for treatment of potable water for the community living in Mumias Sugar Estate.

1.5 Scope of the Investigation

Two upward flow pilot plants were constructed. One pilot plant was installed at the University of Nairobi and the other was at Kabete Water Works. The investigations were on the performances of these pilot plant filters using raw water with no pretreatment provided for.

The pilot plant at the Environmental Health Engineering Laboratory was operated with artificially prepared raw water turbidities ranging from 10 FTU to 170 FTU.

The pilot plant at Kabete water works used natural raw water gravitated from Ruiru dam having turbidities ranging from 1 FTU to 10 FTU. Plankton investigation inside the pilot plant was carried out. These pilot plants were operated at flow rates of between 0.2 m/hr and 1 m/hr. The filter media used had the effective size 0.22 mm and unformity coefficient of 2.46. In using a large range of turbidity values (1-170 FTU); it was hoped that enough operational data would be obtained in order to determine the usefulness of upward flow filters.

CHAPTER 2

PRINCIPLES AND HISTORY OF WATER FILTRATION

2.1 General Definition of Filtration

Filtration is the process by which water is separated from suspended and colloidal impurities it contains. The number of bacteria is materially reduced and changes in the chemical characteristics of the water are brought about by passing it through a porous substance. In principle this porous substance may be granular bed of sand, crushed stone, anthracite, glass, charcoal, gravel etc. In the field of water purification, sand is almost exclusively used as a filtering material because of its availability, its relative low cost and the satisfactory experience that it has given over the many years it has been in use.

2.2 History of Sand Filtration

Water filtration is probably the most important single unit operation of all the treatment processes and it appears that more technical papers have been written on filtration than on any other water treatment subject. Man's use and dependence on filtration as a mean of producing potable water is well documented in literature. As early as

2000 B.C. the Hindus of India used charcoal to purify water, and the use of porous vessels and infiltration galleries has been described by Aristottle and documented in the bible. The importance of filters, however, developed slowly until 1600 A.D. and by late 1800's filtration was established as an integral part of water treatment. Infact the first known Engineering design of a filter was done in 1804 by John Gibb when he built a small experimental slow sand filter in Paisley Scotland. He and others improved on the design and practical details, and in 1829 the method was first adopted for the water supplied by the Chelsea Water Company in London. By 1852 the practice had become established and its advantages so evident that the Metropolis Water Act was passed requiring all the water derived from river Thames within 5 miles of St. Paul's Cathedral to be filtered before being supplied to the public.

All that time the existence of pathogenic bacteria was unknown and the slow sand filter was regarded as a mechanical means of straining out turbidity and suspended solids. John Snow, however, in his studies of cholera transmission had come to the conclusion that the disease was water borne, and postulated the existence of

Materies Morbi - a material derived from previous cases that could transmit infection to those who ingested it. This Materies-Morbi was removed with other solids, by filtration or could be avoided by drawing the supply from a point upstream of any sewer discharge. As a result the first regular examinations of water supplies including chemical analysis were initiated in London in 1858.

As water filtration developed into an established integral part of the water clarification process, the use of slow sand filters was superceded by the introduction of rapid sand filters which became the accepted mode of water polishing and advances were limited to the refinements in design that improved flushing capabilities. In the late 1940's emphasize shifted towards improving the operational economics of filtration.

The success of rapid sand filteration was followed closely by the development of dual and multi media filters (see page 36) the cross sections). Investigations in this area revealed that filtration first through coarse media and then through progressively finer media is superior to conventional fine to coarse filtration. Researchers indicated that it offered the advantages of low headloss, greater solids capacity and higher filtration rates.

One such filter which has been found to offer all these advantages is an upward flow filter. Upward flow filtration offers all the in-depth filtration advantages as multi-media down flow filters and the additional advantages resulting from simple design and operation. Filtering and in many cases flushing in the same upward direction enable the use of conventional sand and gravel media. In addition, the simpler direction of both operating modes eliminates the need for separate wash water troughs and influent manifolding.

But in recent years, more attention has been given once again to the use of slow sand filters, particularly in the developing countries.

Thus in 1968, the World Health Organisation (WHO) in collaboration with the Government of Netherlands established the WHO International Reference Centre for community water supply (IRC) at the Netherland's National Institute for water supply in Voorburg (The Hague). In close contact with WHO, the IRC operated as a world wide network of Regional and National Collaborating Institutions, both in developing and industrialized countries.

In the developing countries, particularly within the tropics, WHO/IRC recommended the use of slow sand filters for community water supply due to its excellent low-cost purification technique for surface water and the simplicity of the operation and maintenance of the filters. But the main drawback in the use of this filter is its inability to function effectively with the changing qualities of raw water particularly its ineffectiveness in the purification of water with high turbidities. Several pretreatment methods were suggested to cope with this problem and these are summarized in chapter 2.6.

2.3 Mechanisms of Sand Filtration

The removal of impurities during the filtration process is brought about by a combination of many factors. The main factors can be grouped into two main mechanisms.

2.3.1 Physico-Chemical Mechanisms

Physico-chemical mechanisms encompass transport and attachement mechanisms. They involve the removal of particles from the flowing fluid and the attachement of these particles to the surfaces of the sand grains.

2.3.1.1. Transport Mechanisms

The transport mechanisms involved in filtration are documented as follows.

2.3.1.1.1. Mechanical Straining

This is the removal of particles of suspended matter which are too large to pass through the interstices between the sand grains.

This occurs entirely at the surface of the filter where the water enters the pores of the sand bed and is independent of the flow rate. By the twisting movement of the water through the filter, velocity gradients are created, bringing the suspended particles in contact with each other. Thus fine suspended matter aggregate into bigger flocs and as such are able to be retained within the filter bed at greater depth. In the case of slow sand filters, the coarser suspended solids caught at the surface of the bed form a fine porous layer (schmutzdecke) which greatly enhances straining efficiency and as a result reduce effluent turbidity. Straining also is enhanced with time by deposits on the grains in the body of the filter bed, constricting pore openings.

2.3.1.1.2. Sedimentation Mechanisms

This removes particulate suspended matter of finer sizes than the pore openings by precipitation upon the sides of the sand grains. The process can be likened to the process in settling tanks for downward flow filtration.

But here the particulate matter deposits on the combined surface area of all the filter grains (Prof. L. Huisman June 1977).

The magnitude of the area involved can be illustrated by the following calculations. The following assumption are made (Huisman 1977).

a) all the sand grains are perfectly round.

b) area of each sand grain = Πd^2

c) $\frac{\text{area of sand grain}}{\text{volume}} = \frac{6}{d}$

Taking $1m^3$ of sand bed with porosity P

$$\text{Total volume of grains} = (1-P)m^3$$

$$\text{Total area} = \frac{6}{d}(1-P)m^2$$

Taking an example of a slow sand filter media whose average porosity is normally in the region of 38% and average grain size of 0.25 mm.

$$\text{The gross area} = \frac{6}{0.25 \times 10^{-3}} (1 - 0.38)$$

$$= 14.88 \times 10^{3} m^{2}/m^{3}$$

But for sedimentation, the effective area is the upward facing area which is not in contact with other grains.

According to Prof. L. Huisman (1977) this area is unlikely to be less than 1000 m^2/m^3. In his investigation Prof. L. Huisman (1977) stated that sedimentation is a function of the ratio between the surface loading and the settling velocity of the suspended particles.

According to Stokes law, settling velocity s of a particle in a laminar flow is given by

$$S = \frac{1}{18} \frac{g}{v} \frac{\Delta\rho}{\rho} d^2 \quad \text{-------------} (1)$$

where s = settling velocity

g = gravity constant 9.81 m/sec^2.

v = kinematic viscosity of fluid.

$\Delta\rho$ = $(\rho_s - \rho)$

d = effective diameter of settling particle

ρ, ρ_s = specific mass density of water and suspended matter respectively.

Taking as an example temperature at $20.2^{o}c$;

$v = 10^{-6}\ m^2/s$

Equation (1) becomes

$$S = \frac{1}{18} \times \frac{9.81}{18} \times \frac{10^3 \times 3.6}{10^{-6}} \quad \frac{\Delta\rho}{\rho} \ d^2$$

$$= 1.96 \times 10^9 \ \frac{\Delta\rho}{\rho} \ d^2 \ m/hr.$$

Usually, the ratio $\frac{\Delta\rho}{\rho}$ is small and is in the order of 0.001 since density of suspended matter is slightly more than that of water.

Thus the equation above becomes

$$1.96 \times 10^9 \times 10^{-2} \times d^2 \ m/hr.$$

$$= \quad 19.6 \times 10^6 \ d^2 \ m/hr.$$

Assuming a usual filtration rate of slow sand filtration of the order of $0.2\ m^3/m^2/hr$ and taking $1\ m^3$ of sand.

Surface loading $(\frac{Q}{A})$

$$= 0.2\ m^3/1000\ m^2$$

$$= 2 \times 10^{-4}\ m/hr.$$

Complete removal of particles is expected when

$$19.6 \times 10^6 d^2 \quad > \quad 2 \times 10^{-4}$$

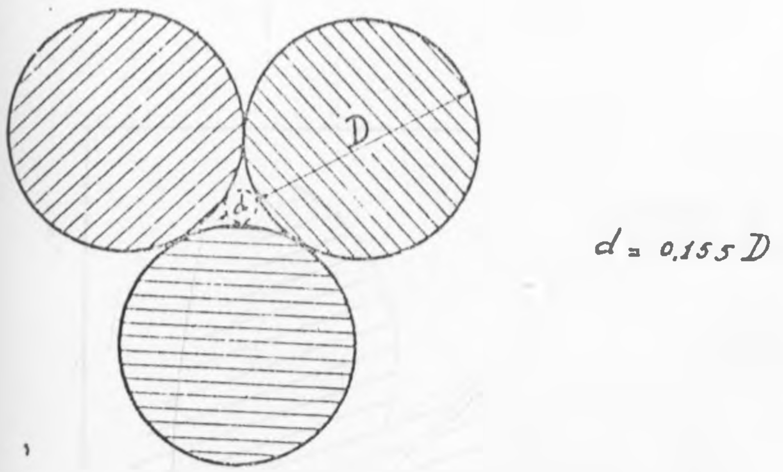
$$\text{i.e.} \quad d^2 > \frac{2 \times 10^{-4}}{19.6 \times 10^6}$$

$$d \quad > \quad 0.32 \times 10^{-5} \text{ m}$$

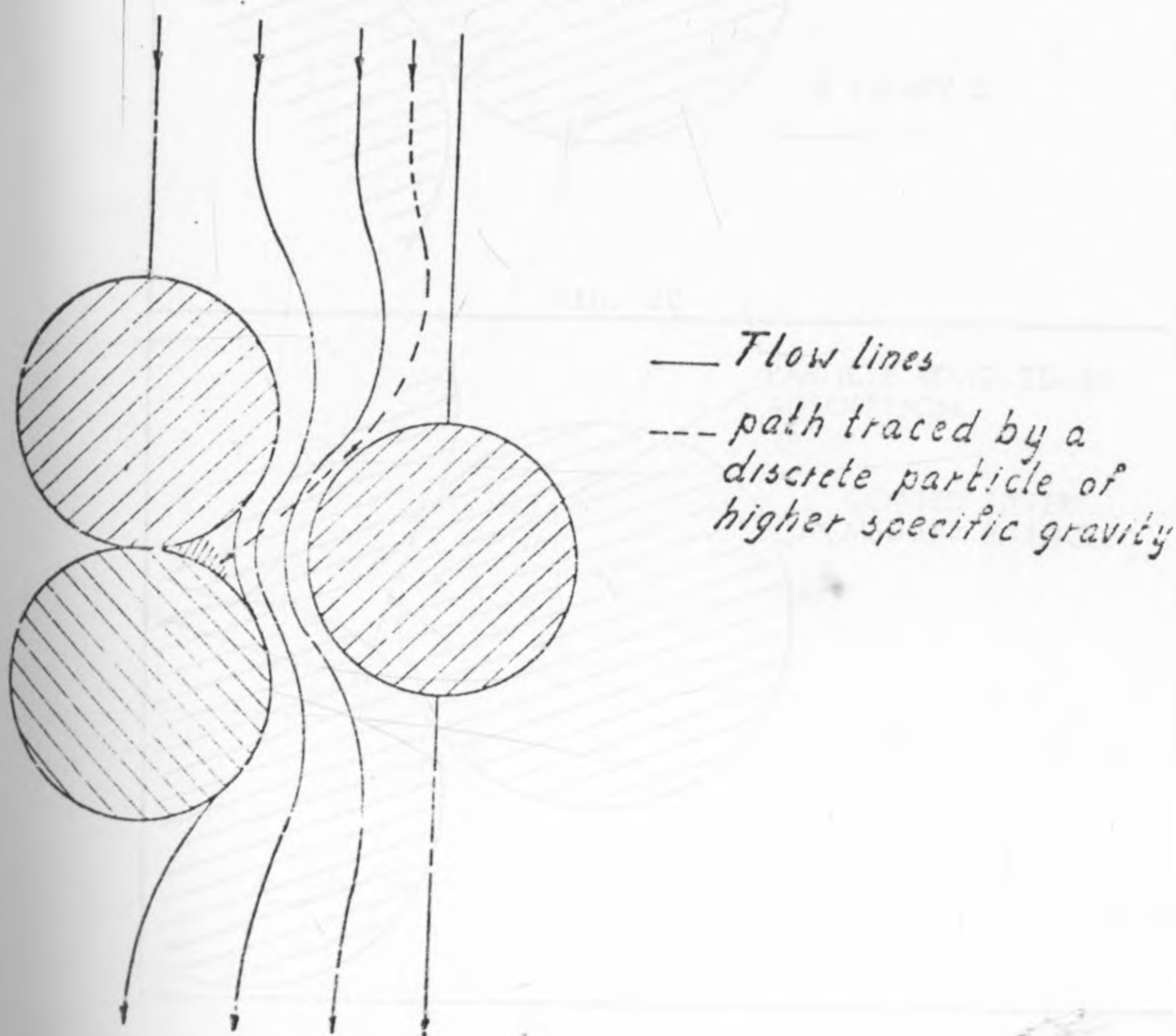
$$> \quad 3.2\mu\text{m}.$$

This implies that the particles with diameters greater than 3.2µm and satisfying the above assumptions would be removed by this mechanism. Also smaller and larger particles can be partly removed due to flocculation of these particles in the filter bed as the filtrate moves downwards. However, trully colloidal matter cannot be removed completely in this way.

Other forms of transport mechanisms which are not so pronounced and yet are important in the overall removal of particulate matter in the filter bed are interception, inertia, diffusion and hydrodynamic action mechanisms. These are fully illustrated in figures 2A and 2B.



RELATION BETWEEN GRAIN SIZE AND PORE SIZE FIG. 2A



REMOVAL OF DISCRETE PARTICLES BY CENTRIFUGAL FORCE FIG. 2B.

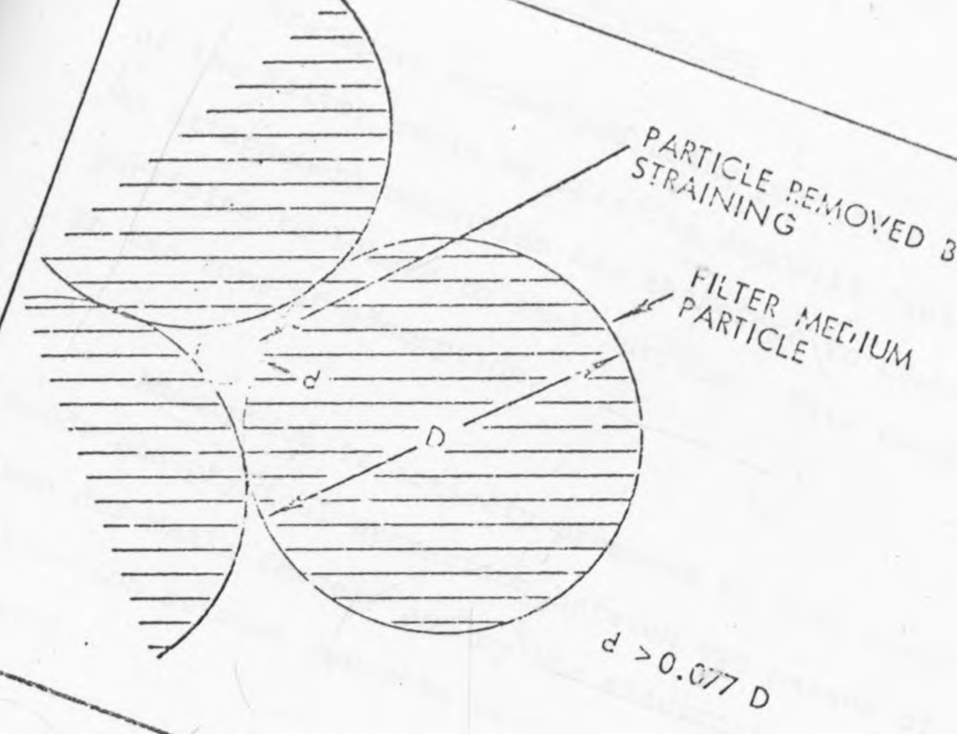

PARTICLE REMOVED BY
STRAINING
FILTER MEDIUM
PARTICLE
d
D
d > 0.077 D

FIG. 2C

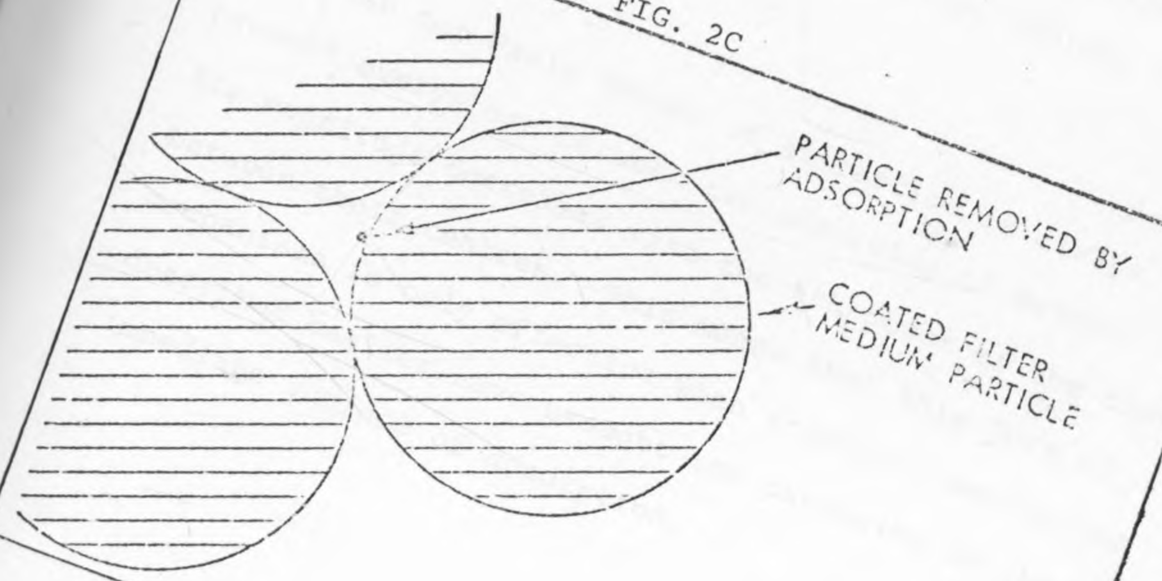

PARTICLE REMOVED BY
ADSORPTION
COATED FILTER
MEDIUM PARTICLE

FIG. 2D.

2.3.1.2 Attachement Mechanisms

Transport mechanisms bring particles to the surface of the filter media or existing deposits (see fig. 2D). An attachement mechanism has therefore to cause the particles to adhere to that surface. This takes place in the form of adsorption.

Adsorption is actively promoted by both chemical bonds and physical attraction between two masses of matter (van der Waals forces) and by the electrostatical attraction between opposite electrical charges (Coulomb forces).

Van der Waals forces of attraction is always present everywhere between two particles of matter. Its magnitude decreases with the 6th power of the distance between their centres. This means that this form of mechanisms is only effective when transport mechanisms described earlier have brought the particles to the immediate surface of adsorption.

Electrostatic forces only occur between oposite charges play a more emminent role than van der waals forces. By the nature of its crystalline structure,(Huisman 1977) clean quartz sand has a negative charge and this is able to adsorb positively charged particles. These are in the form of colloidal matter such as flocs of carbonates, iron, aluminium hydroxides as well as cations of iron, manganese and aluminium. Colloidal matter of organic origin and bacteria are mostly negatively charged and hence are supposed to be repulsed by the charge on sand grains. But this is not absolutely true because this colloidal matter is also adsorbed.

The accumulation of positively charged particles around a sand grain, may be followed by a neutralization of the negative zeta-potential on the sand and in some cases a reversal of the charge may occur (Huisman 1977). A reversal of the charge would favour adsorption of negatively charged particles until another reversal occurs.

Moreover ions on the sand grains may be dragged away by the flowing liquid, by which again a reversal of charge will occur (flow potential). Thus we have primary adsorption taking placefollowed by secondary adsorption.

2.3.2 Biochemical Mechanisms

By the transport and attachment mechanisms described earlier, impurities are transferred from the water to the surface of the filter grains. This contributes much to the improvement of physical and to some extent chemical quality. However, after some time some material will have accumulated to the extent that at some time these impurities are scoured away by the flowing water and hence deterioration of effluent quality becomes evident. This process is very much evident in rapid filters.

But in filters like slow sand filters, it takes time for such impurities to accumulate to such an extent. What happens is that chemical and micro-biological reactions take place and thereby destroy the impurities collected on the surface of the grains.

In this way the chemical and the bacteriological quality of the filtrate is also improved. In general metabolisms of bacteria, other aquatic micro-organisms and algae (as in the case of slow sand filters) contribute to this situation.

During the break-in period, bacteria from the raw water are adsorbed on the filter grains where they multiply selectively using the organic matter deposited on the grains as food. This food is partly oxidised to provide energy for bacteria for their metabolism and partly for cell formation. The by-products of this reaction are carried on by the water to be used again at greater depths by other bacteria. In this way degradable organic matter present in raw water is gradually broken down and finally converted into stable inorganic salts like nitrates, phosphates and constituents like water and carbon dioxide and discharged in the filter effluent.

The bacterial activity described is most pronounced in the upper part of the filter bed and gradually decreases with depth as food becomes scarce (Frank, 1965)

Below a depth of 30 - 40 cm (depending on the filtration rate) bacterial activity is small. With depth a change in the type of bacteria also takes place and at greater depths, true water bacteria predominate . According to Schmidt (1963), different types of bacteria are found at different depths, leading to a division of the filterbed into bacterial zones. In each of these zones, specific bacteria predominate and have specific functions. Even then at greater depths chemical reactions still occur, converting micro-biological degradable products such as amino-acids into ammonia, nitrites into nitrates (nitrification) and reduce the permanganate consumption.

2.4 Types of Sand Filters

There are many ways of classifying filters and these are as follows;

1) due to direction of flow e.g. downflow, upflow, biflow, radial flow, horizontal flow, fine-to-coarse flow, or coarse to fine flow.

2) due to type of Media use e.g. sand, coal (or anthracite) Coal-sand, Multilayered, mixed-media or ditomaceous earth filters.

3) due to flow rates; e.g. 0.1-0.5 m/hr are slow filters; 5-15 m/hr are rapid filters and 15-45 m/hr are high rate filters.

4) due to pressure or gravity flow, gravity filters are usually built of concrete or steel with open tops. Pressure filters are fabricated from steel tanks on cylindrical form with closed tops.

In the field of Community Water Supply Engineering sand is exlusively used as the main filter media. For the purpose of this investigation, two main sand filters are identified. These are upward sand flow filters and the conventional downward flow filters.

The details of the upward flow filtration is extensively covered in chapter 3. The two basic types of the conventional sand filters are slow sand filters and the rapid sand filters.

2.4.1 Rapid Sand Filters

These are again subdivided into two main groups viz. gravity (open) and pressure filters. Pressure rapid sand filters are usually fabricated from steel or concrete and are covered at the top. The gravity rapid sand filters are built with open top and constructed of concrete or steel. The water flows through granular bed of medium to coarse sand at high velocities of 5 - 15 m/hr. This rate is so high that rapid clogging of the filter bed occurs, necessitating cleaning every one to a few days. The effective grain size varies from 0.5 to 2 mm.

There have been various problems in rapid filters operation and performance which arise either from poor design or poor operation. Some of these potential filter problems are listed as follows:-

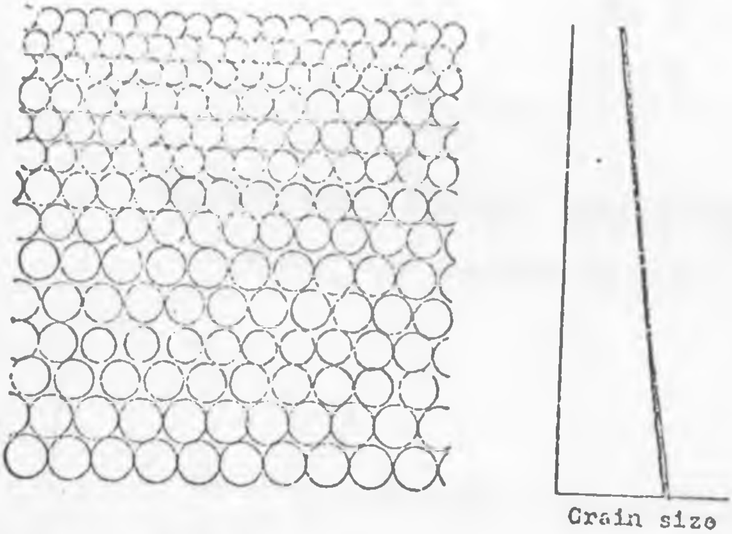
- Surface clogging and cracking usually caused by rapid accumulations of solids on the top surface of the fine media.
- Short runs due to rapid increase in headloss.
- Short runs due to floc breakthrough and high effluent turbidity.
- Variations in effluent quality with changes in applied water flow rate or quality.
- Gravel displacement or mounding.
- Growth of filter grains, bed strinkage and media pulling away from side walls.
- Sand leakage.
- Loss of media.
- Negative head and air binding.

2.4.2 Slow sand filters

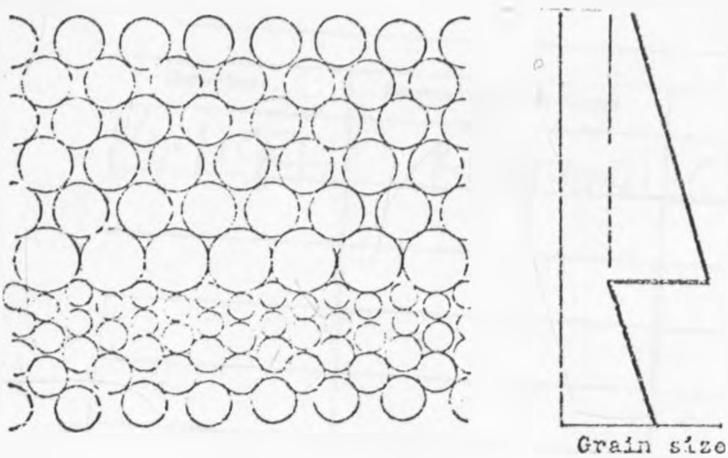
In slow sand filtration, water passes by gravity downwards through a layer of fine sand at low velocities. The velocities vary from less than 0.1 to about 0.5 m/hr.

The filter bed is composed of fine grains of inert and durable sand. This filter medium is characterized by its effective size and uniformity coefficient. Normally an effective size in the range of 0.15-0.35 mm is used and the uniformity coefficient should preferably be less than 2.0, although values of upto 5 are acceptable (Huisman 1977).

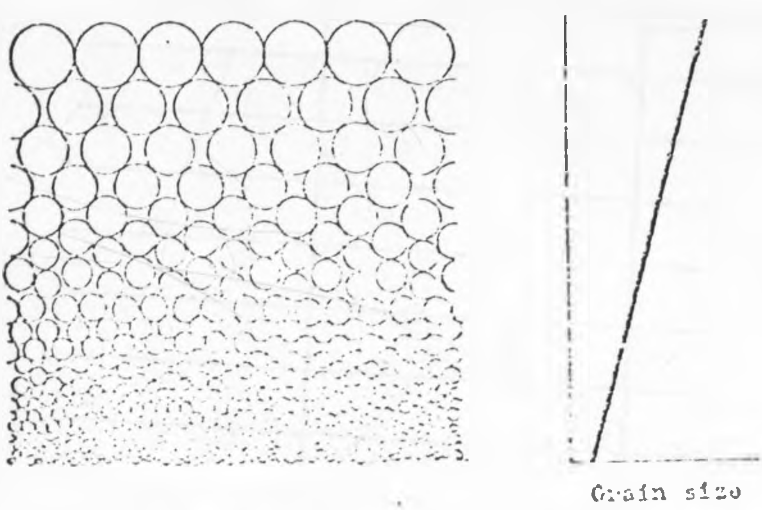
Basically, a slow sand filtration unit consists of a box, containing a supernatant raw water layer, a bed of filter medium, a system of under drains and a set of filter regulation and control devices. A typical diagramatic section of a slow sand filter is shown in fig. 2E. The effect of the purification process on the effluent water quality depends on many factors, such as raw water quality, the rate of filtration, grain size of the filter medium, the temperature and the oxygen content of the water. For normal operational conditions, the average performance of slow sand filters with regard to the removal of certain impurities is summarized in table 2F (from WHO/IRC tech. paper no. 11) on page 38 .



. . Cross section through single-medium bed such as conventional rapid sand filter.



Cross section through dual-media bed: coarse coal above fine sand.



Cross section through [illegible] graded from [illegible] top to

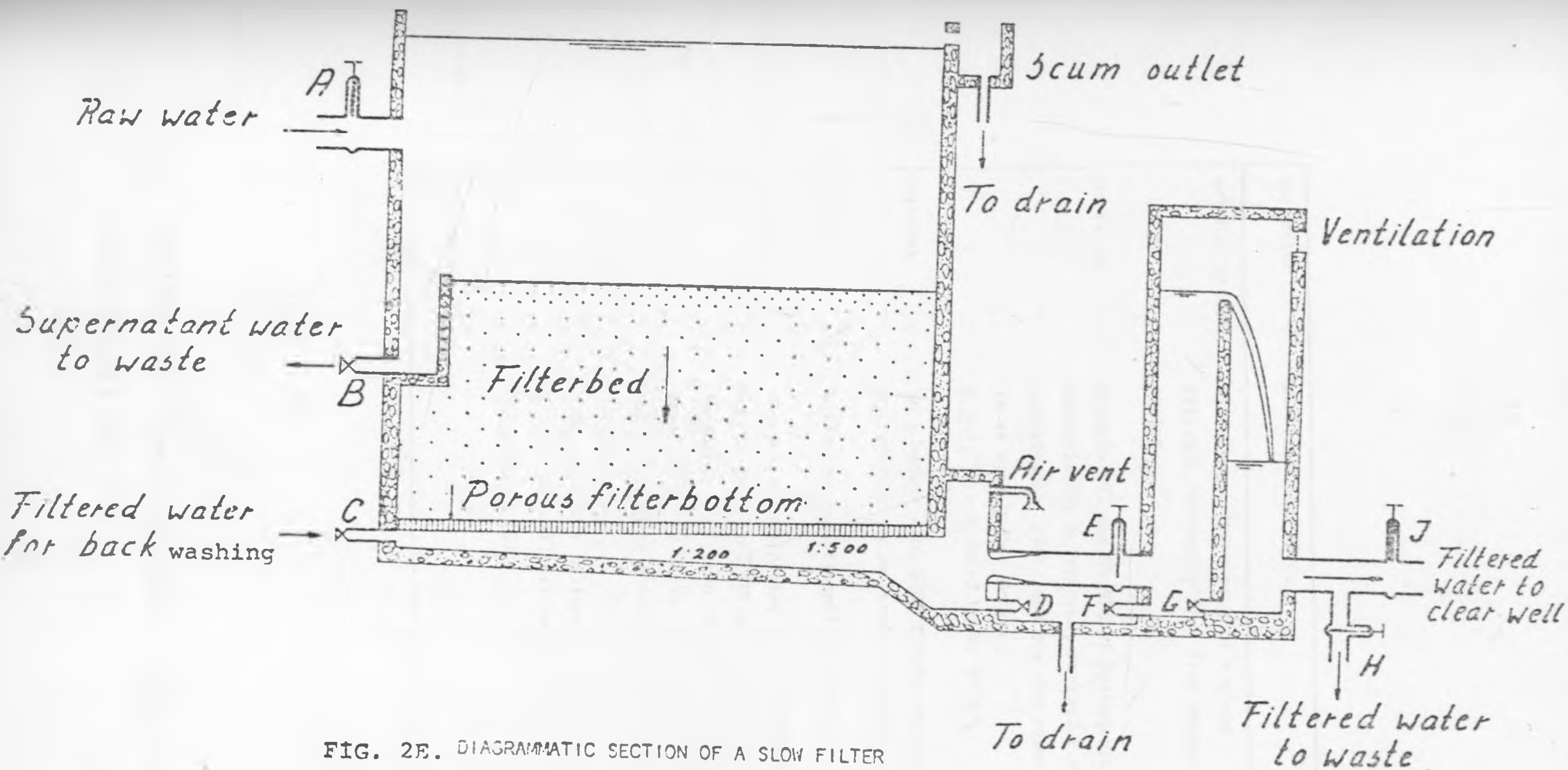


FIG. 2E. DIAGRAMMATIC SECTION OF A SLOW FILTER

parameter	purification effect
organic matter	slow sand filters produce a clear effluent, virtually free from organic matter
bacteria	between 99% and 99.99% of pathogenic bacteria may be removed; cercariae of schistosoma, cysts and ova are removed to an even higher degree; E.Coli" are reduced by 99-99.9 %
viruses	in a mature slow sand filter, viruses are virtually completely removed
colour	colour is significantly reduced
turbidity	raw water turbidities of 100-200 NTU" can be tolerated for a few days only; a turbidity more than 50 NTU is acceptable only for a few weeks; preferably the raw water turbidity should be less than 10 NTU; for a properly designed and operated filter the effluent turbidity will be less than 1 NTU

Table 2F.

Ref. WHO/IRC for Community Water Supply Technical Paper Series No. 11.

CHAPTER 3

UPWARD FLOW FILTERS

3.1 INTRODUCTION

As is widely accepted in the field of water engineering, the main basic objectives for treating water are:-

- Production of water safe for human consumption.
- Production of water appealing to the consumer.
- Production of water using facilities reasonable with respect to capital and operational costs.

In the design of water treatment plants, therefore, the attainment of the above basic objectives is the prime goal, anything less is unacceptable. A properly designed plant though is not a guarantee of safety, but skillful and alert plant operation and attention to the sanitary requirements of the source of supply and distribution system are equally important.

The design of a treatment plant to be used in any water works will depend on the quality of raw water and the desired effluent for use. Some sources of public water supply yield water of excellent quality, requiring little in the way of treatment.

In most parts of the world, public water supplies use conventional downward flow rapid or slow sand filters for filtration of raw water. The basic forms of these filters have remained unchanged for well over fifty years. It is generally recognised that the removal of suspended solids from the liquid being filtered takes place within the top surface layer of the bed. The reverse flow of water during filter washing tends to grade the sand grains, so that the finest are at the top and the largest at the bottom of the bed. This produces the basic limitation of the conventional downflow rapid filter, since most of the filtering capacity is lost due to the increase in pore size in the direction of flow of the liquid being filtered. It also follows that any materials passing the top few inches of the bed will quite likely pass completely through the filter. It is apparent that an ideal filter would be the inverse of the conventional downflow filter; that is it would have the coarsest material on top and the finest on the bottom. The grain size and pore space would be uniformly graded from coarse to fine from top to bottom.

One avenue of research has led to the introduction of an upward flow filter. An upward filter resembles the conventional downwards filters, but the liquid to be filtered flows upwards through the filter medium. The sand medium becomes graded during backwashing with the coarse media at the bottom and successively finer media at the top. Hence the water being filtered meets coarser material at the bottom and progressively finer material towards the effluent end. This ensures that solids being removed are distributed throughout the filter bed. This makes it more of an ideal filter than the conventional down flow filters. Reported major advantages of upward flow filtration over the conventional downflow filtration have been listed as:

- No filter rate controllers are required.
- Filter cannot be run dry while in use.
- Filter cannot become air-bound or generate a negative head.
- The underdrain system and subgrade perform their proper function in that the water passes through the coarse elements first, and through the finer medium last.

When conventional downflow filters have been dismantled and the sub-grade examined, it has usually been found to be full of debris and sediment which have been strained out of the water. This suggests that during the normal filter runs, some strained-out material must have passed to the clear water tank. The advantages of upward flow filters are:

- Mudball formation is absent.
- Different layers of various types and sizes of medium can be maintained in their proper position better than in downflow filters. The finer medium tends to move to the upper layers where it is more effective and is in the correct position for the last stage of filtration.
- The "Spread" due to the upward direction of flow tends to give efficient filtration "in depth" and the whole of the medium including the coarser grades, is made to work. Thus the filter has greater solids retaining capacity.
- High rates of filtration upto 25 m/hr can be achieved without the difficulties of fluidization if proper design is adopted.

- The upflow filter is quite flexible in permitting alteration of filtration rates without adverse effects to the filtrate.
- Headloss is lower compared to downflow filtersbecause of more even distribution of solids removal, as a result the clear water tank can be sited at a higher level.
- The singular direction of filtering and backwashing which is generally practised eliminates the need for a separate wash water tank.
- Capital and operational costs of upflow filters is low.
- If a "break through" occurs, it can be readily observed and the filter put out of action. This is not possible with a downflow filter, it is more likely that floc passes through to the clear-water tank unobserved, during the end of the run. This is particularly common with filters that have been in use for a long time.
- Reduction in the quantity of coagulated chemicals and stability of the process towards temperature changes of the raw water.

- Reduction in the physical size of the plant and capital cost due to the possible elimination of pre-treatment units.

Though upward flow filtration seems to have many advantages, some disadvantages have been noted as follows:

- Risk of local fluidisation unless proper precaution is taken.
- Clogging of underdrain may be a potential problem.

- Risk of losing finer media during backwashing.
- Ineffective cleaning of the lower layers of sand and gravel has been noted.
- Due to the singular direction of flow for both filtration and backwashing potential risk of contaminating the filtrate exists unless proper disinfection is done where such a danger exists.

Upward flow filtration has found a lot of scope in water and waste-water engineering. It has been used for potable water production; industrial water use; as a pre-treatment unit for highly turbid water; filtration of swiming pool water; and in the tertiary treatment of effluent waste water.

3.2 THEORY OF UPWARD FLOW FILTRATION

3.2.1 Principle Mechanisms of Filtration

The mechanisms involved in suspended solids removal in any filter are complex. Some of these mechanisms have been discussed in chapter 2. In upward flow filtration, suspended solids removal is primarily within the whole filterbed (depth filtration) some of the large solids may be removed by the interstitial straining. But the dominant types of mechanisms are transport mechanisms and attachement mechanisms which have been briefly discussed - chapter 2.

3.2.2. Theoretical considerations

Many theories on upward flow filtration have been developed from the existing theories on filtration in general. The generally accepted theoretical equation was developed by K.J. IVES and defines the change in suspension concentration C, as it follows through the filter length L. This is expressed as:-

$$C = C_o e^{-\lambda L} \qquad \text{----------------} \qquad (1)$$

where C_o = inlet Conc. at $L = 0$

λ = filter coefficient.

Filter coefficient varies with the degree of filter pore clogging. This is defined by the specific deposit σ (the volume of deposited material per unit filter bed volume).

Specific deposit σ varies with position L in the filter and with time of filtration t. This implies that the filter coefficient λ varies also with position L, and time t.

The change in concentration δc of suspension through a layer of filter length δL is given by the differential equation

$$\frac{\delta c}{\delta L} = \left[\frac{\alpha + \beta\sigma - \gamma\sigma^2/f - \sigma}{d^m}\right] C \qquad \text{----------} \qquad (2)$$

α, β, γ are filter constants

f is initial porosity

d is grain size of media in layer δL

σ is specific deposit

C is concentration of suspension entering layer δL

m is constant depending on suspension.

Sholji et al (1965) showed in their experiments that $m = 1$ over the range of grain sizes normally used for filter media.

Thus for a size - graded filter whose grains vary along the length of the filter the following equation describes the filtration process.

$$\lambda = \left[\frac{(\alpha + \beta\sigma - \gamma\sigma^2)/f-\sigma}{ds}\right] C \quad \text{---------} \quad (3)$$

where ds is the sieve size of the grains.

Across any filter, the increase in headloss is due to accumulation of deposits in the pores. Hence headloss is a function of specific deposit.

This can be expressed as follows:-

$$\frac{\partial h}{\partial L} = \frac{dh}{dL_o} + k\sigma \quad \text{---------} \quad (4)$$

where h is headloss

$\frac{dh}{dL_o}$ is initial headloss per unit depth

k is headloss constant.

For a givensuspension, water temperature and filtration velocity, the constants can be worked out and will depend solely on the nature of the granular media in use.

3.3 HISTORY OF UPWARD FLOW FILTRATION

The principle of upflow filtration or filtration by ascent is not quite new and it is reported that the idea was concieved as early as 1685 by Porzio, an Italian Physician. This was a multiple-filter whereby water was passed through a sand medium in downward direction at first then followed by an upward direction. The principle was followed further by Amy, a Frenchman in 1754 and later by Von Wuthwehr, an army surgeon general in 1790. They built a number of such filters for use depending on the desired filtrate quality. However, Peacock, a British architect was one of the first to recognise the advantages of coarse to fine filtration and to obtain a patent for such a unit in 1791. The filter consisted of sand, gravel, broken glass and other suitable materials. The filter media had different layers and each layer placed had to have a grain diameter about half of the layer below it. A false bottom of wood slats was used to support the filter media. Washing was done by reversing the flow.

The first known Municipal Installation was at Greenock Scotland in 1827 and in America at Richmond, Va in 1832. This utilised the highly turbid water of the James River which eventually overworked the filter and was abandoned in 1835. Many more installations followed in U.S.A., Britain, and elsewhere. The major filter media, commonly used in all these filters were either gravel, charcoal, sand or a combination of any of the above materials.

In 1883, an upflow filter was constructed at Pawtucket R.I. The filter media consisted of gravel graded from egg size at the bottom and Pea gravel at the top. A larger filter was built in 1889 consisting of 46 cm of graded stone at the bottom, 30 cm of brick and mapple charcoal and another 46 cm of stone graded to the size of Pea gravel at the top. It was noted that it successfully removed all suspended matter and a large fraction of micro-organisms. The average filtration rate was 9.4 m/hr for the whole year. Washing was done by draining the filter and hosing the surface until the top gravel was clean.

An upward flow filter was also constructed in 1891 at Storm lake, Iowa for filtration of lake water. The filter media were graded in five layers of 30 cm depth with coarse gravel at the top followed by layers of small gravel, coarse sand, charcoal and broken stone respectively. Washing the filter was done by reversing the direction of the flow. Another filter was incorporated in a small water treatment plant at Bart-besville, Oklahoma in 1904. The plant employed alum coagulation, sedimention and upflow filtration for treatment of high turbidity water.

In New York, an experimental upflow filter was installed in 1907 to treat water from Croton Reservoir. The filter used wooden sticks at the bottom covered with wire mesh to support the filter media. 18.0 cm of screened charcoal covered with a copper mesh screen was used as bottom support for 51 cm sand layer used as filter media. The filter operated at a flow rate of 0.7 m/hr and was cleaned at an interval of two weeks. Washing was again done by reversing the flow downwards.

As is evident from the foregoing history of the use of upflow filters, the common way of washing the filter bed was by reversing the direction of flow. This method of filter washing was not effective since it provided no agitation or scrubing of the media. Also lack of expansion of media during washing made it difficult to remove completely the suspended matter that had penetrated deeply into the media.

3.4 Development of Upflow Filters

While upflow filtration produces perfect filtering conditions, the washing of upflow filters by reversing the direction of flow resulted in poor cleaning of the clogged sand bed and consequent failure of the process. This became a major problem for some time.

However, post-war intensive research on filtration led to the revival of the principle of upward flow filtration once again.

In Russia, this led to the development by Minz of upflow "Contact clarifiers" in 1953-1954 and he had them installed at Moscow, Lenigard, Cheliabinsk, Gorkii, Kiev and other places for purifications of turbid water of upto 1.50 mg/l". These installations had facilities such as flocculation, coagulation and sedimentation units combined.

The coagulant is added to the raw water before it enters the filter bottom. Flocculation and coagulation take place in the bottom layers of gravel followed by clarification and filtration in the overlying sand bed. The most important thing was that washing of the filter was accomplished by passing water through the filter in the same direction i.e. from bottom upwards but at a much higher rate than filtration.

The original contact clarifiers contained gravel and sand at a depth of 2.3 to 2.6 m. The normal design filtration rate ranged from 5.8 m/hr to 8.7 m/hr and upward flow washing of 36.5 m/hr. Later designs of contact clarifiers used higher flow rates. In Leningrad, a contact clarifier was operated at filtration rates of 20.5 m/hr with a sand depth of about 2.6m. The coagulant dosage was added to the raw water immediately ahead of the filter since large, easily settleable flocs are not desired. The aim is the formation of "Micro-aggregate" flocs with coagulation within the filter bed being more rapid and complete on contact with the surface of sand grains as compared to the normal coagulation. In this way coagulant dosage was found to be reduced by 30%; space requirement by more than 50% and initial costs reduced by 20-25%.

The arrangement of washing the filter bed by increasing the flow rate in the same direction as filtration process introduced the problems of how to prevent the bed from expanding in the upward direction and the finer sand particles being washed away as the pressure differential increases with the clogging of the bed.

3.4.1 <u>A Centre-Collector System</u> (see Fig. 3A page 57).

The first successful solution to the above problem was the production of a filter in which the raw water was introduced at both top and bottom of the unit simultaneously and the filtrate collected somewhere in the centre of the filter. A centre collector system, some 0.46 m or so below the top of the sand was used to collect and discharge the total filtrate from the unit. Downflow water through the upper part of the filter kept the sand bed in compression, thereby avoiding the lifting and separation of the sand grains. A number of such filters operating on this principle have been developed. But in the 2-way flow, the quality of the filtrate depends on the effectiveness of each part of the filter and can never be as good as that from a similar bed in which flow is in one single direction and filtrate is refined progressively. The construction of a 2-way filter with a deep bed and a centre collector system was found to be cumbersome and expensive. This type of filter has now been superseded and abandoned.

3.4.2 Grid-type Immedium Filter (see Fig. 3B page 57)

The problems of fluidization of the finer layers of filter media during washing of the bed called for more research to improve upflow filters. The centre-collector system did not improve the situation very much. This led to a second innovation by Peter Smit of Holland in 1961 in which the filter bed is maintained in a compact condition by a steel grid located a short distance below the surface of the sand bed.

This unit is known as the grid-type immedium filter. The improvement by Smit consisted of placing a steel grid at a short distance of 0.1 m below the sand surface. The grid consists of vertical plates spaced at 100-150 times the size of the smallest grain in the sand bed. During filtration the sand forms natural compression arches against the grid preventing occurance of fluidisation. After the filtration cycle, the bed is washed by first lowering the water level to a point just above the sand surface and then the bed is vigorously agitated using compressed air to break the compression arches.

The sand bed is expanded by 20% during washing. This expansion is important as accumulated dirt is to be removed from the whole depth of the sand bed.

Immedium filter can be operated either as an open unit or as a closed pressure unit. The grading of the pebble and sand media may be varied according to the application. The following grading, given in the direction of flow is typical of many, immedium filter installations in many parts of the world.

Depth of layer (m)	Grading (mm)
0.15	40-50
0.25	8-12
0.25	2-3
1.58	1-2

3.4.3 Declining Rate Upflow Filter (see Fig. 3E page 59.)

This type of filter is basically a modified conventional grid immedium upflow filter whose sidewalls are approximately sloping to form a diverging filter cross-section. It has been shown theoretically that the performance of a declining rate upflow filter is superior to all other systems, since it is the only system where both the grain size and the velocity of flow decreases in the direction of flow. The reasoning is based on the basis of the

surface area availability and the

velocity of flow past a section. Declining rate upflow filter provide more surface area as compared to the conventional upflow filter since the lower portion of the filter, having coarse media, is replaced by an equal portion at the top, containing fine grain media. Beside this it is noted that the increase in surface area is in a region where the local velocities are less than the average velocity. Headloss development is also gradual through the filter bed as compared to that in the conventional upflow filters because of the reduced pore velocities in the direction of flow.

3.5 Application of upflow filters

Most of the practical experiences gained on the use of upflow filters has been done on immedium filters. The use of the "contact clarifiers" has been mainly confined to Russia where it was first designed and put into use by Minz. Declining Rate upward flow filters have not been extensively used. In his contribution on the use of declining rate upflow filters, Dhabadgaonkar,(1976) of the Regional College of Engineering NAGPUR, has presented a design of a "package declining rate upflow filter" in which the overall treatment scheme consists of raw water pump, chemical design chamber, mixing baffled channel, dosed water influent well, package

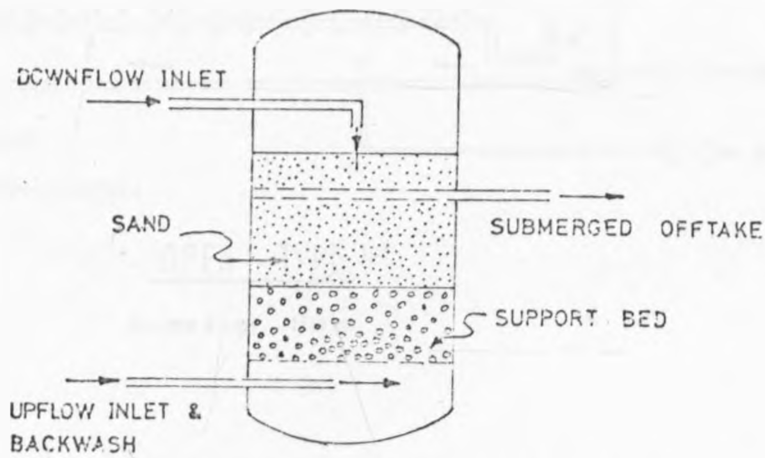


Fig. 3A. PRINCIPE OF OPERATION OF CENTRE COLLECTOR.

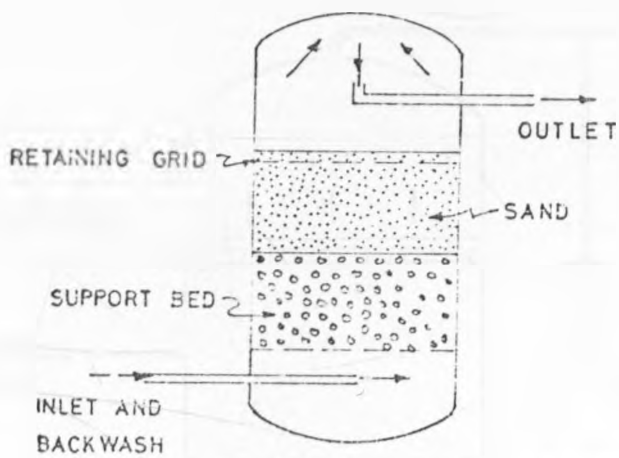


Fig. 3B. PRINCIPLE OF OPERATION IMMEDIUM GRID FILTER.

UPFLOW FILTER DESIGN.

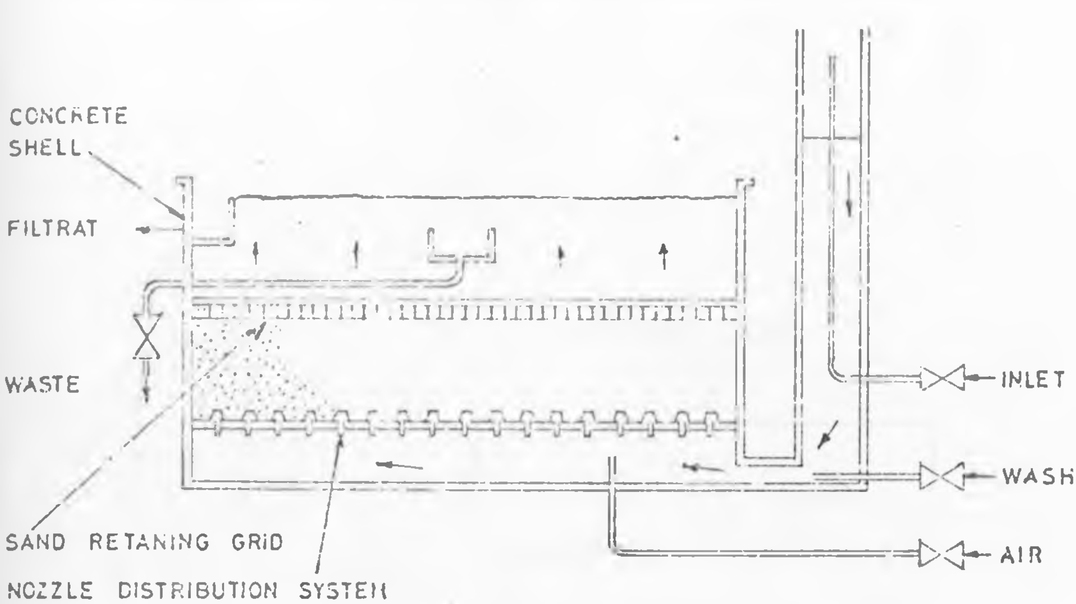


Fig. 3C. OPEN TYPE.

Immedium filter

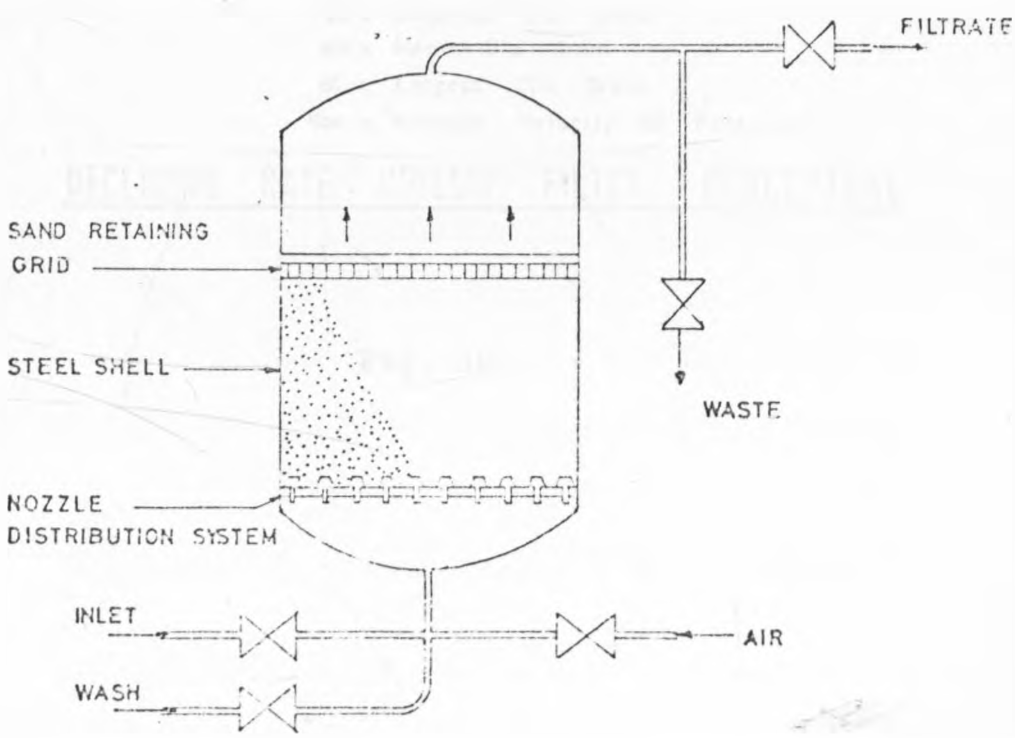
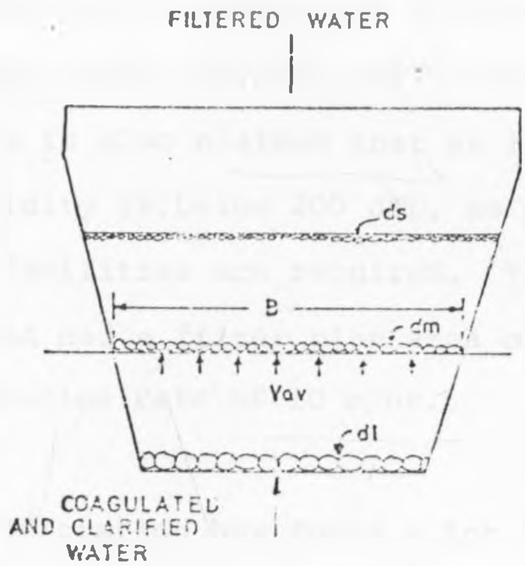


Fig. 3D. PRESSURE TYPE.

Inmedium filter



ds = Smallest Dia Grain
dm = Mean Dia Grain
dl = Largest Dia Grain
Vav = Average Velocity Of Filtration

DECLINING RATE UPFLOW FILTER CONCEPTUAL.

Fig. 3E.

declining rate upflow filter, chlorination and filtered water storage tank which provides the necessary contact line for disinfection. It is claimed that the water supplied to the community is bacteriologically safe and will have turbidity less than 1 JTU. It is also claimed that as long as the raw water turbidity is below 200 JTU, no separate pre-treatment facilities are required. The typical design presented had a filter plan area of one square metre and filtration rate of 10 m/hr.

Upflow filtration has found a lot of scope both in the field of water and waste water engineering. It has been used in some parts of the world as a prefilter or a polishing unit to water with very high turbidity. It has also been used successfully for potable water production, industrial water treatment, tertiary treatment of sewage effluent and swimming pool filtration. The following are some selected examples on the use of upflow filters.

3.5.1 Filtration of Potable Water

Potable water production has been the main consideration in the use of upflow filtration. In Russia, extensive use of "contact clarifiers" has been made. The original contact clarifiers tested contained gravel and sand to a depth of 2.4 to 2.6 m. It was noticed that the headloss increase during filtration was slow. Some studies at the Rublevsky water plant at Moscow indicated that the use of upflow filters could produce potable water of high quality comparable to that from conventional down-flow filtration. Table 3A shows some of the results of the tests.

The first immedium filter installed in great Britain is at the Treforest Industrial Estate, South Wales, A 3.5 m diameter steel unit is used to filter a water supply used for both potable and industrial uses. The filter operatesat a flowrate of 9.8 m/hr though at times this is raised to 14.7 m/hr. The raw water is first coagulated and clarified in sedimentation tanks before passing to the immedium filter, which operates in parallel with conventional downflow sand filters. The downflow filters operate at 4.9 m/hr. In producing similar quality of filtrate, the immedium filter at 9.8 m/hr is found to give 6.25 times the total output between washings. At flowrate of 12.2 m/hr, the immedium filter gives five times the output of the downflow filter and at a rating of 14.7 m/hr, it gives over twice the output of the downflow filter. These results, carried out by Boby W.W. et al (1967) are tabulated in table 3B.

Type of WATER	Turbidity, Colouration Indices Mean Value per Cycle				Bacteriological Data			
	Mean Conditions		Flood Conditions		Mean Conditions		Flood Conditions	
	Turbidity mg/l	Colour ^{o}H	Turbidity mg/l	Colour ^{o}H	Gaster Bacter Index	Bacter Count Index	Gaster Bacter Index	Bacter Count Index
River Water	6.4-9.8	22°-72°	16.7-21.5	42°-115°	30-3000	200-2000	200-200000	15-120
Effluent from Con-tact Fil-ters	0.4-1.5	9°-18°	0.5-1.5	12°-18°	4-72	700-800	7-200	7-3000
Effluent from Con-ventional downward Filters	1	11°	0.5-1.8	8°-19°	4-16	15-590	4-1400	6-4000

TABLE 3A.

(After Minz)

T A B L E 3B

COMPARATIVE RESULTS OF OPERATION OF IMMEDIUM AND DOWNFLOW FILTERS, TREFOREST INDUSTRIAL ESTATE

		Downflow Filter	Immedium Filter		
Rate of Flow	$gal/ft^2/hr$	100	200	250	300
	m/hr	4.9	9.8	12.2	14.7
Total output between washings	gal/ft^2	2400	15,000	12000	5000
	m^3/m^2	117	734	587	245

"After Boby W.W. and Alpe G. 1967"

In 1965, pilot plant trials were carried out by the consulting engineers for Kincordine County Council (Messrs Crouch & Hogg, Glasgow) on the water of river Dye at Glendye, Scotland. Dye is a typical flashy mountain river which can change rapidly in flow and in nature. The water is soft and acidic with a P^H varying from 4.5 to 6.8 and colour varying from 40^{o}H to over 200^{o}H. The pilot plant comprised of an immedium filter 2'6" Ø and mounted on a steel skid base, together with an air blower and a dual purpose pump to deliver raw water either at normal filtering rate or at the wash rate.

The filter was operated as a single stage plant, the coagulants were injected immediately before the raw water entered the immedium filter, and was operated at two flow rates (4.9 m/hr or 9.8 m/hr). Many trials covering a wide range of river conditions were carried out and a sample of typical results taken during one week are shown in table 3C. As a result of these trials, the full scale plant was constructed at Glendye to treat 4.5×10^3 m^3/day of water at a flow rate of 4.9 m/hr.

These pilot plant trials verified an interesting point made by Minz in describing experiences with upflow filtration of similar coloured water in

Russia. He found that in applying coagulants before the raw water enters an upward filter (referred to as "contact clarifiers") the chemical dosage used may be less than half those required for floc formation and precipitation in sedimentation tanks. It is found that the passage of the chemically treated water through the coarse pebble underbed of the immedium filter induces flocculation.

In 1969, the Thames Valley Water Board approached the plant Development Division of the Water Research Association on the possible methods of treating increased water abstracted from the river Kennet. After some discussion, the W.R.A. suggested the possible use of the immedium upflow filter as an alternative method of pretreatment. Pilot plant studies were carried out on this basis at the Fobney Works.

The pilot installations consisted of 0.76 m diameter by 3.33 m high immedium upflow filter and two 0.3m diameter by 3.66 m high perspex downflow columns, each with suitable ancilliary equipment. The immedium filter was a standard Boby pilot plant unit while the downflow columns were constructed to the specifications drawn up by W.R.A.River water was supplied to the plant by a submmersible pump fitted with a coarse strainer. On the immedium filter, chemical dosing was by direct injection into the inlet pipe of the unit while on the downflow filters, coagulant addition was made into flash mixing tanks which provided upto 5 minutes contact time. The whole programme was geared to carrying out a progressive series of trials to determine the performance of:-

(a) Anthracite/sand beds in the downflow columns with and without chemical dosing.

(b) The immedium upflow filter, again with and without chemical dosing.

(c) The upflow and down flow filters in series on either or both stages.

The experiments were carried out during the period 30.1.70 to 24.5.71 covering practically all river conditions likely to be encountered. Some of the results obtained are tabulated in the tables 3D-3F. In general, very interesting observations were made in respect to the upflow filter. These are summarised as follows:-

a) It was generally found that upflow filtration without coagulant dosing gave a reduction in turbidity of approximately 50%.

b) The upflow filter, used without a coagulant, did not perform satisfactorily as a primary filter under all conditions.

c) The upflow filter, used with an adequate coagulant dose, functioned extremely well, as an alternative method of primary treatment. Possible advantages were mentioned as

(i) More compact plant.

(ii) Lower capital costs.

(iii) Lower running costs.

d) The combined upflow/downflow system, operated at 12.5 m/hr flow rate, with primary and secondary dosing worked well with raw waters upto approximately 20 JTU.

e) The system was capable of dealing with rapid changes, though it was felt that some raw water storage would be desirable and would assist plant operation by reducing flocculation.

Table 3D: Prelimary results of upflow Filtration without coagulation.

Date	Flow Rate m/h	Coagulant dose	Turbidity J.T.U.		Duration of run
			Raw	Filtrate	
21.12.69	12.5	Nil	4.5	2.5	Not less than 7 days
26.12.69	"	Nil	8.5	4	" " "
25.12.69	"	Nil	13	6	" " "
10.12.69	"	Nil	22	10	2-3 days

Table 3E Prelimary results of upflow filtration with coagulation

Date	Flow rate m/h	Aluminium sulphate mg/l	Turbidity J.T.U.		Duration of run
			Raw	Filtrate	
10.2.70	12.5	1.5	6.5-7.0	1.0	72 h minimum* decreasing to 18 h approximately
"	"	3	"	0.6	
"	"	6	"	0.34	
"	"	12	"	0.28	
16.3.70	"	8.5	10	1.0	24 h
15.4.70	"	7	30	1.2	16 h

*Duration estimated from head loss readings.

After Fox and Priest (1960)

Table 3F: Preliminary results of upflow and downflow filtration with coagulation aluminium sulphate

Date	Raw water Turbidity J.T.U.	UPFLOW				DOWNFLOW			
		Rate m/h	Aluminium sulphate mg/l	Turbidity J.T.U.	Duration h	Rate m/h	Aluminium sulphate mg/l	Turbidity J.T.U.	Duration h
1.2.70	10-16	12.5	6	1.6	24	5	24	0.4	24
	"	"	"	"	"	10	10	0.4-0.8	"
3.2.70	11-17	12.5	7.5	0.8-2.5	"	5	11	0.2	"
	"	"	"	"	"	10	8	0.2-0.4	"
16.3.70	10	12.5	8.5	1.0	"	10	5	0.2	"
27.6.70	1.5	12.5	11	0.25-0.3	"	12.5	Nil	0.22	"
	"	"	"	"	"	12.5	4.5	0.15	"

After Fox and Priest (1960)

As early as 1960, F.G. Hill used an upflow filter for the filtration of Singapore and Johore water supplies. The plants incorporated the provision for dosing and mixing chemicals, coagulation, settlement and filtration. The upflow filter was also adopted for use with three Singapore city council swimming pools. It is reported that satisfactory results were given by these filters.

At the National Environmental Engineering Research Institute, India; a laboratory investigation of anupflow pilot plant was carried out.

The filter unit 1' 0" square in plan and 16' high was fabricated out of M.S. sheets with perspex on one side of facilitate direct visual observations. The grading and depth of gravel and sand used in the filter were as follows:-

4" - 6"	size stones ---	3' 0"
2" - 4"	stones	2' 0"
1" - 1½"	gravel	1' 0"
½" - 1"	gravel	6"
⅛" - ⅜"	gravel	6"
sand of E.S. 0.7m and U.C. of 1.2		4' 6"
Total depth of filter media		11' 6"

Water with artificial turbidity in the form of a suspension of locally available clay added, served as raw water for the experiments. A solution of filter alum was injected into the raw water inlet by means of a positive displacement pump. Hourly samples of filter influent and effluent were collected and tested for turbidity and pH. In all a total of 17 runs were made with different raw water turbidity and filtration rates. The results are tabulated in table 3G. Based on these investigations, the following observations were made:

a) Upflow pilot plant filter with this particular type of bed media and raw water quality; high rate of filtration (more than 1.5 gpm/sq.ft) were not practical and short filter runs were observed.

b) Alum requirement was found to be less than the optimum dose as determined by jar test. This was in agreement to findings by other researchers like Boby W.W.T., et al.

c) Deep gravel layers, such as used in the study, though playing a definite role in the coagulation of influent turbidity do not remove turbidity to any significant level, as indicated by the headloss pattern obtained. It was also observed that only about 2ft of sand bed overlying gravel accounted for major turbidity removal.

d) Cleaning of upflow filter required application of wash water at higher rates than conventional downflow filters and accounted for wash-water consumption of 7-8 percent of water treated.

In the U.S.A., Hamann and McKiney (1972) of the University of Kansas carried out some feasibility study of purifiying Kansas River water by use of upflow filtration with coagulation. The researchers used 6" diameter filters of similar design to the Soviet filters and concluded that the upflow filter had a definite advantage over gravity filtration and in some cases could replace conventional flocculation sedimentation and filtration. In many other parts of the world, the use of upflow filters for portable water production both on pilot and full scale plants have been undertaken. The few examples presented show the extend to which upflow filtration has taken root.

Turbidity during the study ranged from about 100 to 200 ppm (SiO_2) but for individual runs more or less a constant turbidity was maintained.

TABLE 3G

NO.	Rate of Filtration gpm/sq.ft (ImP)	Average Turbidity ppm(SiO_2) Inf.	Eff.	Alum dose ppm	pH Inf.	pH Eff.	Alkalinity mg/l $CaCO_3$ Inf.	Eff.	Filter run hrs.	Final Head loss (Inches)
1.	3.75	152	1.2	34.6	8.0	7.3	95	91	3	18.45
2.	3.75	136	0.34	28.0	8.2	7.4	134	121	4.25	28.56
3.	3.25	99	0.45	38.7	7.7	7.4	86	77	16	40.44
4.	3.25	130	0.63	33.0	8.3	7.3	114	108	8.5	33.2
5.	3.25	129	0.65	-	3.2	7.4	134	121	9	29.2
6.	2.25	160	0.72	45.0	8.3	7.5	50	35	8	22.4
7.	2.0	144	0.63	55.0	8.3	7.2	-	-	9	25.6
8.	2.0	181	0.60	44.5	8.3	7.5	-	-	9	23.8
9.	2.0	204	0.77	58.0	8.25	7.3	-	-	8.25	23.7
10.	1.75	123	0.56	66.0	8.30	7.5	-	-	13	25.6
11.	1.75	130	0.70	50.6	8.3	7.4	-	-	17	34.2
12.	1.75	186	0.30	57.0	8.3	7.3	208	200	8	25.6
13.	1.5	108	0.45	61.5	8.3	7.4	182	166	38	73.2
14.	1.5	143	0.73	77.3	8.3	7.4	172	144	37	45.9
15.	1.5	146	0.25	46.0	8.2	7.5	-	-	30	59.3
16.	1.5	162	0.51	72.3	8.3	7.4	-	-	37	52.1
17.	1.5	141	0.36	51.0	8.1	7.2	-	-	33	57.13

3.5.2 Filtration of Sewage Effluents

Upflow filters have been used in a number of tests on sewage effluents. These have included effluents from humus tants following percolating filters and effluents from sewage works operating on activated sludge aeration systems. A paper quoted by Boby and Alpe (1967) on filters at LUTON describes the use of immedium upflow filter for tertiary treatment tests at Luton County Borough's East Hyde Sewage Disposal Works. The immedium upflow filter was operated in parallel with rapid gravity filters and micro-strainers. Comperative results of the immedium filter and the existing rapid gravity sand filters in the treatment of the same effluent led to the following conclusions by the authors concerning the advantages of immedium filter over downflow filters.

a) For a given filtration rate a better quality of effluent is obtained.

b) With poor feeds good effluent can be obtained while maintaining comparatively high flow rates.

c) For a given rate of flow and clogging head, longer filter runs between backwashes are possible.

d) The use of feedwater for backwash represents a direct saving.

e) The system permits a smaller area of bed to be used and possibly a smaller number of filters.

Further investigations on the flocculating effect of the flow path through the underbed of an immedium filter has been observed during tests on tertiary treatment.

An indication of this was given by comparing the relative settling times of the suspended particles in the wash water taken from an immedium upflow filter with those taken from a downflow sand filter, after both had operated on the same sewage effluent for similar periods.

Fast settlement was achieved in the case of washwater from immedium upflow filterswhile solids removed from the rapid gravity filter remained as a light suspension in the wash water; indicating that the immedium effluent filter provided better treatment of sewage effluent than the conventional downflow filter.

The examples of experiments already described refer to straight filtration without addition of coagulants. In most cases, this is satisfactory. But in cases where sewage effluent reuse is practiced (e.g. for industrial use) a higher quality of sewage effluent might be desirable. Boby and Alpe (1967) have reported the findings of a laboratory investigation with the use of immedium filter pilot plant and sewage effluent. The coagulant aid was injected into the sewage effluent before filtration. The filter was operated at a rate of 9.8 m/hr and was found to remove approximately 85% of suspended solids and 40% of BOD.

3.5.3 Other Applications

Examples which have been cited indicated the range of results which can be expected from the use upflow filtration in water and waste water treatment. A number of other applications have been investigated. These include filtration of industrial effluents, the removal of iron and manganese from potable water supplies the removal of fine chalk suspensions after lime softening and the filtration of water from impounding reservoirs subject to algae growth.

In U.K. pilot plant trials on the performance of immedium upflow filter on paper mill effluent has been reported by Messrs Edward Towgood and sons ltd (1967) .The investigations had the following conclusions to make based on these pilot plant trials.

a) for 90% of the time all the suspended solids and turbidity were removed.

b) when colloided starch was present in the effluent, very little of the suspended solids were removed and the turbidity was only slightly reduced.

c) when dyes and pigments were present in the effluent most of the colour was removed.

d) the maximum flow rate was found to be 14.7 m/hr. which is double the flow rate possible on a downward flow filter.

e) Cleaning of the filter was simple when the period of filtration was restricted to 24 hours.

f) There was an average of 30% reduction in the B.O.D.

The introduction of upflow filtration can therefore be said to have extended the range of sand filtration. This extension is two fold.

1) Filtration efficiency is improved on applications where in the past downflow sand filters have been used.

2) Upflow filters may be used on certain applications where the suspended solids load is too heavy for conventional downflow sand filters.

The greater filter efficiency, higher filtration rates and longer filter runs lead to saving of capital expenditure on new plants. By using upflow filters, elimination of sedimentation tanks from water treatment projects may be possible due to in-line coagulation which can take place in the filter. This can also lead to reduction in chemical doses compared to the dosage required in forming flocculants that can be removed by sedimentation tanks. Research is still going on in different parts of the world on the use of upflow filters for different applications and under different conditions.

CHAPTER 4:

CONSTRAINTS DURING THE INVESTIGATION

During the period of December 1979 to April 1980, when the experiments were being carried out, a number of problems were encountered which had significant impact on the way the investigations were carried out.

The first major problem was the effect of the severe drought which fell on the country during this period. As a result, there was a serious crisis of shortage of water in the Nairobi City. This led to rationing of water in some sections of the city and the use of water was diverted to only the essential services. Most storage tanks had low volume of water and taps ran dry in most places. One of the places severely affected by this crisis was the University of Nairobi. The Environmental Health Engineering laboratory had no tap water for the period December 1979 to February 1980 when the department had to obtain water by using hose pipes connected to a source 60 metres away from the laboratory.

Also as a result of the drought, power rationing was introduced during this same period. Since the country's largest source of electricity is hydropower generated, the prolonged drought had direct impact on the amount of electric power available to the nation. One of the places affected by rationing of power was the University. Thus it became exceedingly difficult to run the experiment on continuous basis. It also resulted in having short filter runs when the filtration process was carried out. The University authorities had to step in and explain to the East African Power Limited Company the adverse effects power rationing had on research work which was being conducted within the University. Though the University was eventually exempted from being affected by power rationing, none the less at some occassions power rationing was being extended to the University until the University authority had to intervene once again.

The constant interruptions by power cuts were not conducive to continuous filtration operations and a lot of time was wasted during these interruptions.

The second major problem was due to the failure of the joints of the perspex pilot plant. Though the filter box was externally reinforced using flat iron and perspex bars, the use of chloroform in joints did not prove to be strong to sustain the weight of the filter media as well as the pump pressure. In all these were five major bursts and numerous water leakages during the time of running the experiment. Again, alot of time was wasted in stopping the operation and carrying out repairs where needed on the pilot plant. It also contributed to short filter runs during the experiment. Short circuiting was another factor which was very pronounced. The walls of the filter column were too smooth and short circuiting was very evident.

There was no major technical problem with the pilot plant of Kabete. The only major problem was the distance. Kabete water works is about 10 km away from the Department. The analyses were to be done in the Environmental Health Engineering laboratory. On many occassion, to obtain transport from the department to transport the samples from Kabete water works was not easy. So in most occassions, public transport means were used and this meant that a few selected analyses could be carried out. The few analyses which could be conveniently carried out were found to be Turbidity, Colour, Iron Content, pH and the Microscopic Investigations. The Microscopic investigation was later on covered inadequately because the slides were constantly covered with sand. This point is adequately explained in chapter 7.

Notwithstanding the above constraints, the investigations were carried out and the results and analysis are subsequently covered in the next chapters.

CHAPTER 5:

THE DESIGN AND OPERATION OF THE PILOT PLANT

PILOT PLANT I (UNIVERSITY OF NAIROBI)

5.1 Design of Filter Column

The filter column was built of Perspex Material of a rectangular plan with dimensions of 350 mm x 250 mm and a vertical height of 2680 mm. It was reinforced with perspex and flat iron bars at varying intervals along its length (see plates 5D to 5F). The column was further partitioned into two smaller columns of dimensions 250 m x 250 m and 250 m x 100 m respectively. This partitioning extended for 1800 mm along the filter length. The purpose of such partitioning was to have the filter media in the larger column of dimension 250 mm x 250 mm so that the filtrate could overflow over the weir and hence lead to waste through the smaller column of dimensions 250 mm x 100 mm. But during the first filter run, the partitioning perspex wall failed on its joints. Chloroform was used for joints. This seemed to have been weak to withstand the applied pressure. As a result, the filter sand poured into the filtrate chamber, thereby making the whole system fail

So the whole perspex column was filled with filter media and used for filtration purposes. The internal partition was neglected. An overflow outlet was made at about 50 mm above the original overflow weir. This outlet was of 25.4mm diameter and attached to a 25.4mm diameter G.I. pipe which was joined to the drain pipe that led the filtrate to waste.

To support the gravel and sand media, there was a perforated perspex plate with perforated perspex supports. This acted as the support for filter media as well as a distribution plate releasing water uniformly over the filter media. The inlet G.I. pipe was of the diameter of 1½" (38.1 mm (see fig. 5A-5C).

5.2 Raw Water Preparations

Two water tanks of 900 litres (200 galls) capacity each were provided; one for turbid water and the other for clear tap water. The turbid water was created by mixing tap water and red coffee soil obtained within the University compound. Vigorous mixing was applied by stirring the content inside the tank manually using a wooden stick. To ensure constant mixing, a pump was used to recirculate the turbid water from bottom to top of the turbid water tank. But this system was found to be extremely inadequate. There was constant breakdown of pump operation due to brushes getting worn out so quickly that much of the time, the stirring was done manually.

Due to the initial high concentration of turbidity in the turbid water tank, dilution was considered to be necessary before pumping the raw water into the filter. A third smaller perspex tank was provided. Both the contents of clear water and turbid water storage tanks emptied into this third tank whereby dilution was effected. To facilitate proper mixing, some of the raw water was circulated back into the mixing tank. The two storage tanks were continuously being replendished by water from the mains (see Fig. 5A).

5.3 The Filter Media

A base layer of 100 mm of ¾' gravel and 150 mm ⅜" gravel was placed respectively on top of the perforated perspex supporting plate. Then the filter sand of 1000 mm depth was placed on top of the gravel. The filter sand used had effective size of 0.22 mm and a coefficient of uniformity of 2.46. This sand had previously been prepared for use in pilot plant investigations of slow sand filters at Kabete water works. The results of the sieve analysis of the sand is given in table 5.1 and the grading curve is given in figure 5D.

5.4 Filter Control Devices

Several control devices were provided. The storage tanks were provided with float valves which controlled the amount of water pouring into the tanks. The smaller perspex mixing tank was provided with an overflow pipe leading to the drainage system. Several gate valves were provide as shown in Figure 5A. In obtaining and maintaini the desired flow rates, both the speed of the pump and some of the gate valves were adjusted.

5.5 Sampling points during filtration

Samples of inlet raw water, the filtrate and the downward flush water were taken. Three sampling points were provided for. One sampling outlet for influent raw water was provided just before the raw water enters the filter column; the filtrate sampling point was provided at a point just after the effluent outlet and the down-ward flush water sampling point was also provided a flush water drain pipe. All these facilities are shown in figure 5A, and plates 5E - 5G

5.6 Procedures adopted during filtration operations

5.6.1 Filtration procedures

Before the start of any filter run, the raw water pump had to be primed and it was necessary to make sure that it did not run dry. This was done by having the perspex mixing tank constantly full of water and a little overflow from the tank was allowed to maintain constant flow and also as a precaution against accidentally pumping all the water from the tank, thereby exposing the pump to sucking in air. To ensure automatic inflow of water into both storage tanks, the valves V1 and V20 (see fig. 5A) were fully opened and float valves were provided in the tanks so that when these tanks were full, the valves close automatically and when the level of water has gone down, the valves open automatically and water is released into the tanks.

Due to constant breakdown of the pump which was meant to keep a continuous recirculation of the water in the turbid water storage tank, the tank was manually stirred at desired intervals to keep the soil suspension well distributed throughout the tank. Valves V3 and V19 were opened continuously so that the initially desired turbidity is obtained. It was not possible to maintain constantly a desired turbidity. What was possible was to maintain a certain range of turbidities during a given filter run. But often, the raw water turbidities became very high each time the turbid water storage tank was stirred manually or when fresh soil is added into the tank and stirred vigorously. Any new soil added into the tank was first ground to fine porer using a pestle so that a more even distribution of suspension could be achieved in the tank.

To obtained the desired filtration rates, the speed of the raw water pump was accordingly adjusted. Valves V14 and V7 were always fully opened during a filter run. Valves V5, V16 and V17 were partially opened and by proper manipulation of these valves, the desired flow rates would be obtained and maintained. The opening of valves V16 and V17 also provided water recirculation into the mixing tank. This ensures the raw water is constantly stirred and the turbid water suspension uniformly maintained in the tank. Valve V15 was always kept closed unless the filter column was required to be filled from the top. This was done once during the downward flushing when investigating the effect of downward flushing only.

5.6.2 Sampling procedures

During any filter run, the first sample to be taken was for the filtrate. This was because collecting the filtrate by opening valve V8 did not affect the rate of filtration. So valve V8 was fully opened and valve V9 closed. A large graduated measuring cylinder was used to collect the sample over a given time. A stop watch graduated in seconds was used to measure time. A constant time period of 15 seconds was chosen.

Having taken the filtrate sample, the raw water sample was then taken by opening valve V6. Again the same measuring cylinder and stop watch were used.

Usually before taking any samples, the manometer readings were taken first. This was because it was noticed that by opening and closing some of these valves, flow rates and pressure differences were affected.

5.6.3 Backwashing procedures

In treatment plants employing the use of rapid filtration, backwashing is usually done when a predetermined headloss has been achieved. The level of water in the filter tank would then be lowered to a point just above the sand surface and the bed would be vigorously agitated using compressed air. The air would bubble through the filter bed loosening and scouring the bed in preparation for washing. This expansion is important as accumulated dirt is removed from the whole depth of the bed.

The backwash flow rate is kept such that the fine sand is not washed away.

In backwashing the pilot plant, the above outlined methods were not followed. No air was applied to scour the bed; but backwashing upwards and downwards at relatively low flow rates was done. The bed could not be expanded because of the fragile nature of the perspex joints. Each time the backwash flow rate was high, the perspex joints for the filter, would burst open. Secondly, the filter sand was relatively fine and any high flow rates of backwashing upwards would simply wash away the fine sand. No grid under the surface of the sand was provided for as is the case in the actual field conditions.

After nearly every filter run, backwashing of the filter was done. Backwashing was done both upwards and downwards. The sequence of backwashing was as follows: The filter run was stopped and the filter flushed downwards. When all the water above the sand had been flushed out, the influent water was pumped upwards at a relatively higher rate for some times. This depended on how clogged the bed was as seen visually through the perspex filter wall. Then downward flushing was repeated. The sample for downward flushing was taken at a interval of 3 minutes for every downward flushing. The above sequence was repeated once or twice and then upward backwashing was continued till the effluent water became clear. The next filter run was then started and the necessary adjustments of the pump speed and the gate valves were made to obtain the desired flow rate. The results and analysis of the downwards flushing of the filter after some of the chosen filter runs are given in the appendix. The precise sequence followed in backwashing is described for each particular filter run separately.

particularly for low colour values. The equipment used for colour analysis was a Lavibond-Nessleriser. The colour values of 5, 10, 15, 20, 30, 40, 50, 60, 70 degrees Hazen could be accurately determined but any value between these numbers or anything above the value of 70^{O}H was to be determined through interpolation or by dilution method. But it is apparent, that the equipment used for colour measurement was not as accurate as a turbidimeter. As such no attempt was made to precisely determine the actual colour of the samples, apart from taking the sample and reading the values on the Lavibond disc. The method followed is also in accordance to "standard methods for examination of water and waste water".

5.6.4.3. pH

pH was not considered an important parameter of any significance for this investigation. However, pH was monitored for the sake of knowing the acidity/alkalinity of the raw water sample used. Due to the short interval of time of sampling, it was not anticipated that any appreciable pH changes could occur. Indeed by looking at the pH readings, this point is apparent. A pH meter, chemitrix type 40E was used. Buffer solution of known pH values (i.e. 7, 9 and 4) were used to calibrate the meter before making any readings.

5.7 Summary of filter runs conducted

The following is a tabulated brief summary of the filter runs conducted in the Environmental Health Engineering laboratory (see Table 5).

5.8 Abbreviations used

In the remaining chapters, the following abbreviations have been used:-

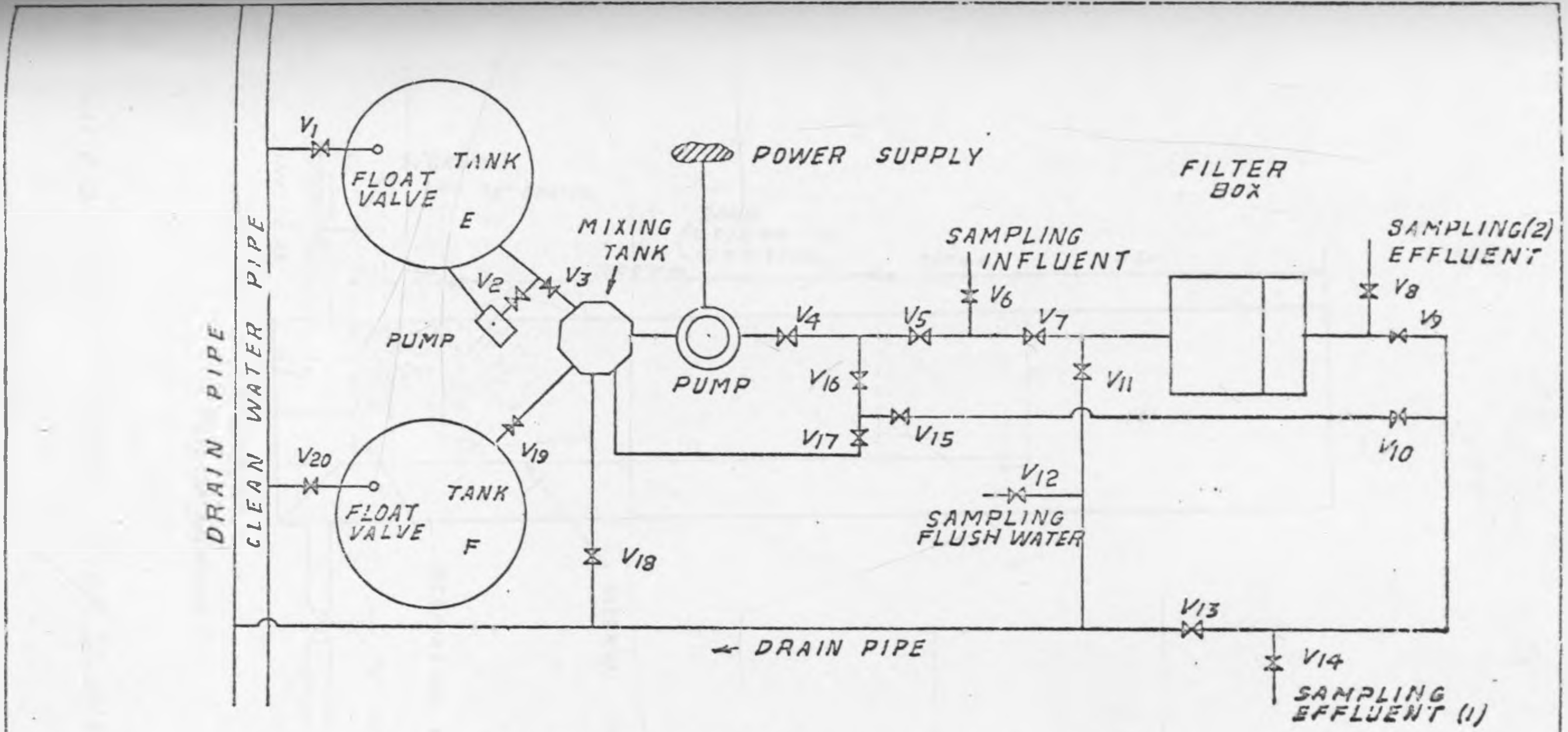
TURB	-	Turbidity
COL	-	Colour
EFF	-	Effluent
INF	-	Influent
REM	-	Removal
FTU	-	Formazin Turbidity Unit
^{o}H	-	Degree Hazen.

Table 5

FILTER RUN	FLOW RATES m/hr	LENGTH OF RUN HRS	BACKWASHING AFTER FILTER RUN	MODE OF BACKWASHING	OTHER REMARKS
1	0.96	9	Yes	Downward flushing followed by upward flushing.	No headloss measurents taken. Backwashing stopped after an Internal burst. This was a preliminary filter run
2	0.52	10	Yes	Downward followed by upward flushing. Flush water analysis carried out.	after backwashing, accumulated sand under the filter could be seen. A minor burst developed which was sealed.
3	0.36	8	Yes	Downward followed by upward flushing. Flush water analysis carried out.	An attempt made to wash off accumulated sand in the filter underdrain. This was done by pumping in raw water at maximum pump speed, then openi-ning the drain pipe.

FILTER RUN	FLOW RATE m/hr.	LENGTH OF RUN HRS	BACKWASHING AFTER FILTER RUN	MODE OF BACKWASHING	OTHER REMARKS
4	0.6	27	Yes	Upward flushing only. No flush water analysis	-
5	0.41	7	No	-	Crack developed on the perspex wall. Run stopped and repair done.
		25	Yes	Upward flushing for 30 minutes. No flush water analysis	
6	1.0	50	Yes	Down followed by upward flushing. The process repeated three times. Flush water sampling done	There was a minor crack of perspex joint at the start of the run which was sealed of.
7	0.78	11	Yes	Downward followed by upward flushing. Flush water sampling done.	-

Filter Run	Flow Rate m/hr	Length Run HRS	Backwashing After Run	Mode of Backwashing	Other Remarks
8	0.27	103	Yes	Upward flushing for over 3 hours. No flush water sampling carried out.	It was seen that the lower layers of the gravel and sand were heavily clogged, hence the long backwashing period
9	0.2	37	No		-

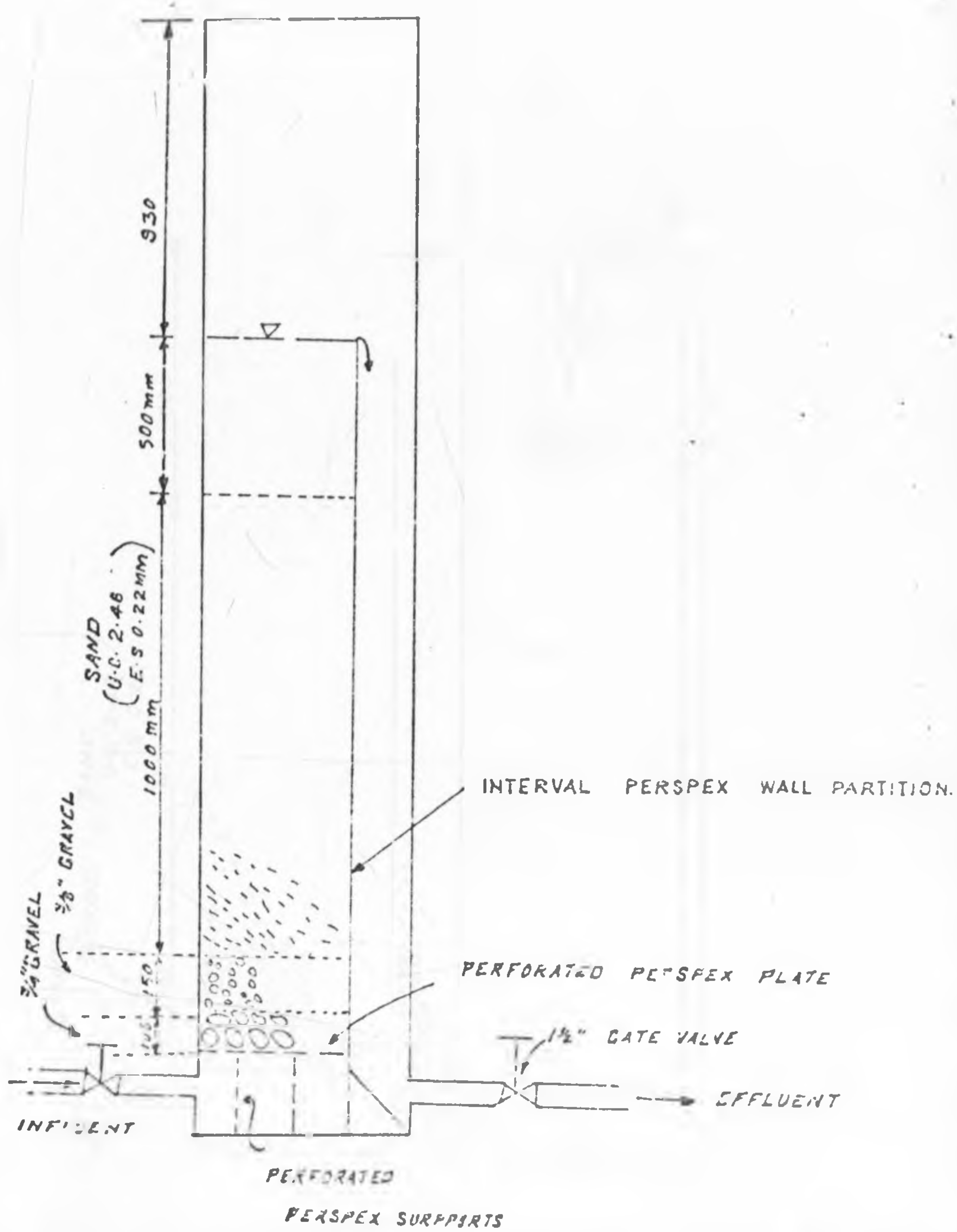


PLAN OF THE PILOT UPFLOW FILTER

FIG. 5 A

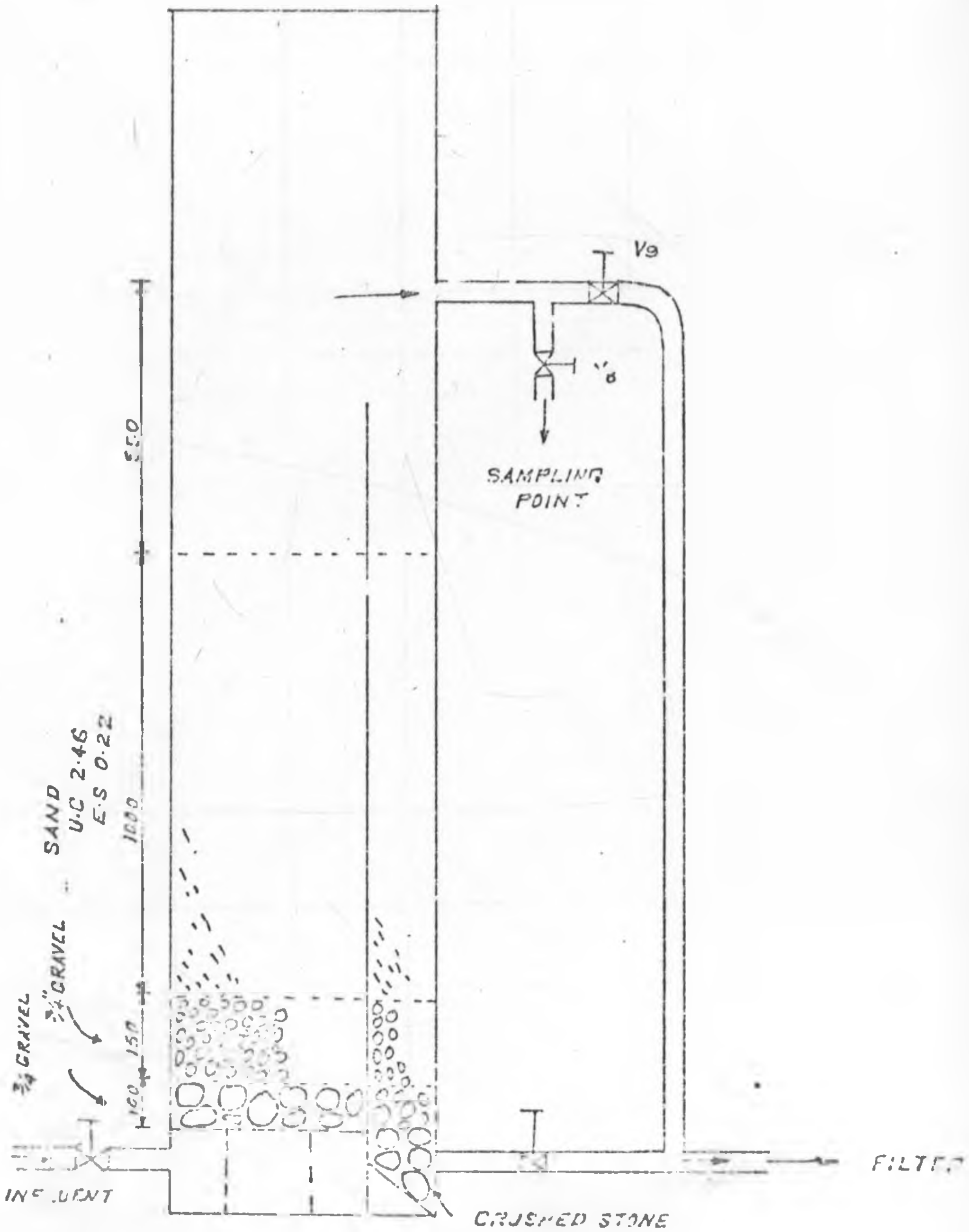
Tank E is the turbid Water Tank

Tank F is the clear Water Tank



SCHEMATIC DIAGRAM OF PERSPEX FILTER COLUMN (EARLY VERSION

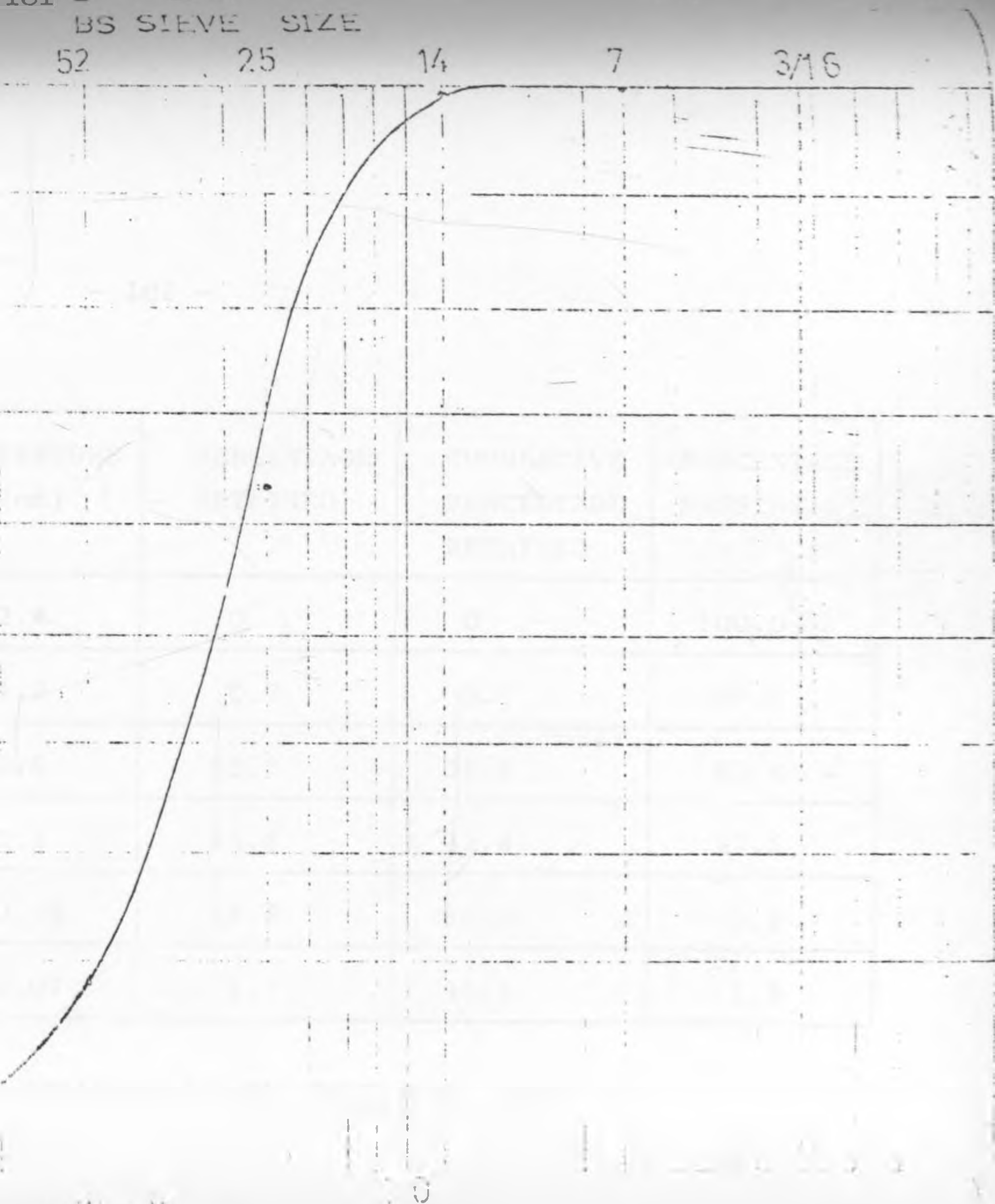
FIG. 5B



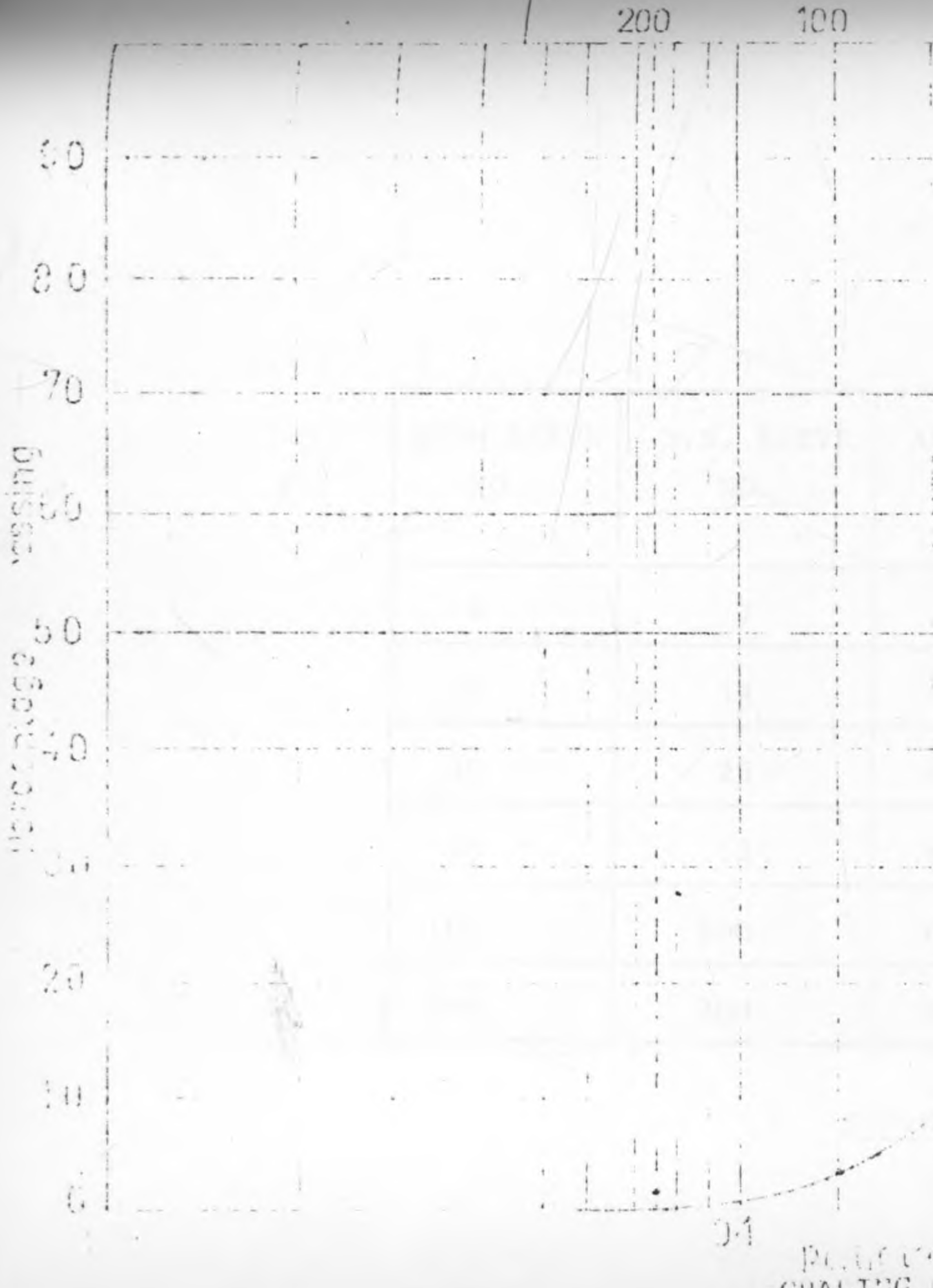
SCHEMATIC DIAGRAM OF PERSPEX FILTER COLUMN

(LATER VERSION)

FIG. 5C.



CURVE FOR THE FINE SAND


200
100
GRADING

ASTM SIEVE NO.	B.S. SIEVE NO.
8	7
16	14
30	25
50	52
100	100
200	200

SI

APERTURE (mm)	PERCENTAGE RETAINED	CUMULATIVE PERCENTAGE RETAINED	PERCENTAGE PASSING
2.4	0	0	100.0
1.2	0.7	0.7	99.3
0.6	35.9	36.6	63.4
0.3	45.8	82.4	17.6
0.15	14.4	96.8	3.2
0.07	1.7	98.5	1.5

EVE ANALYSIS OF THE FINE SAND USED

Fig. E.

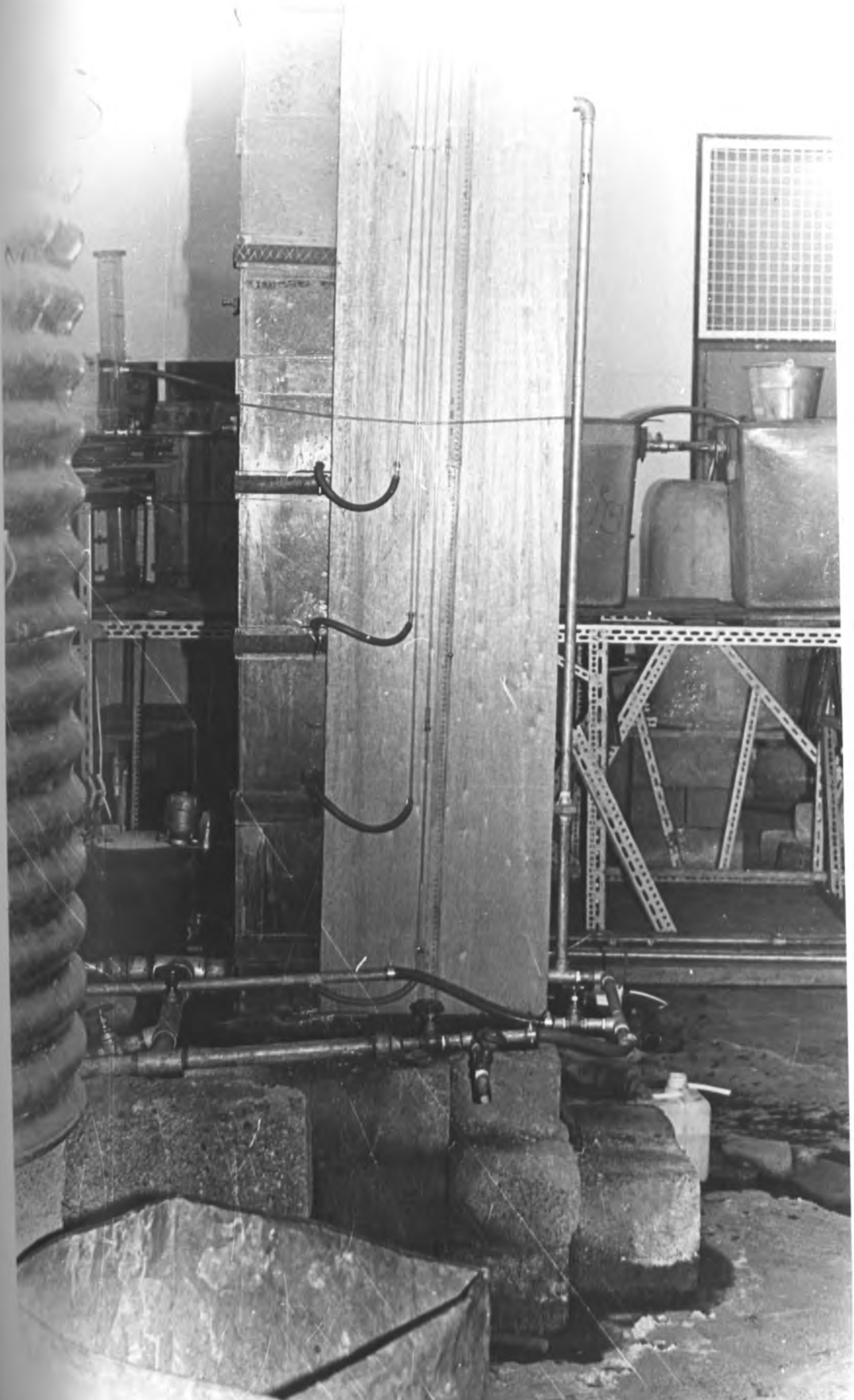


Plate 5E

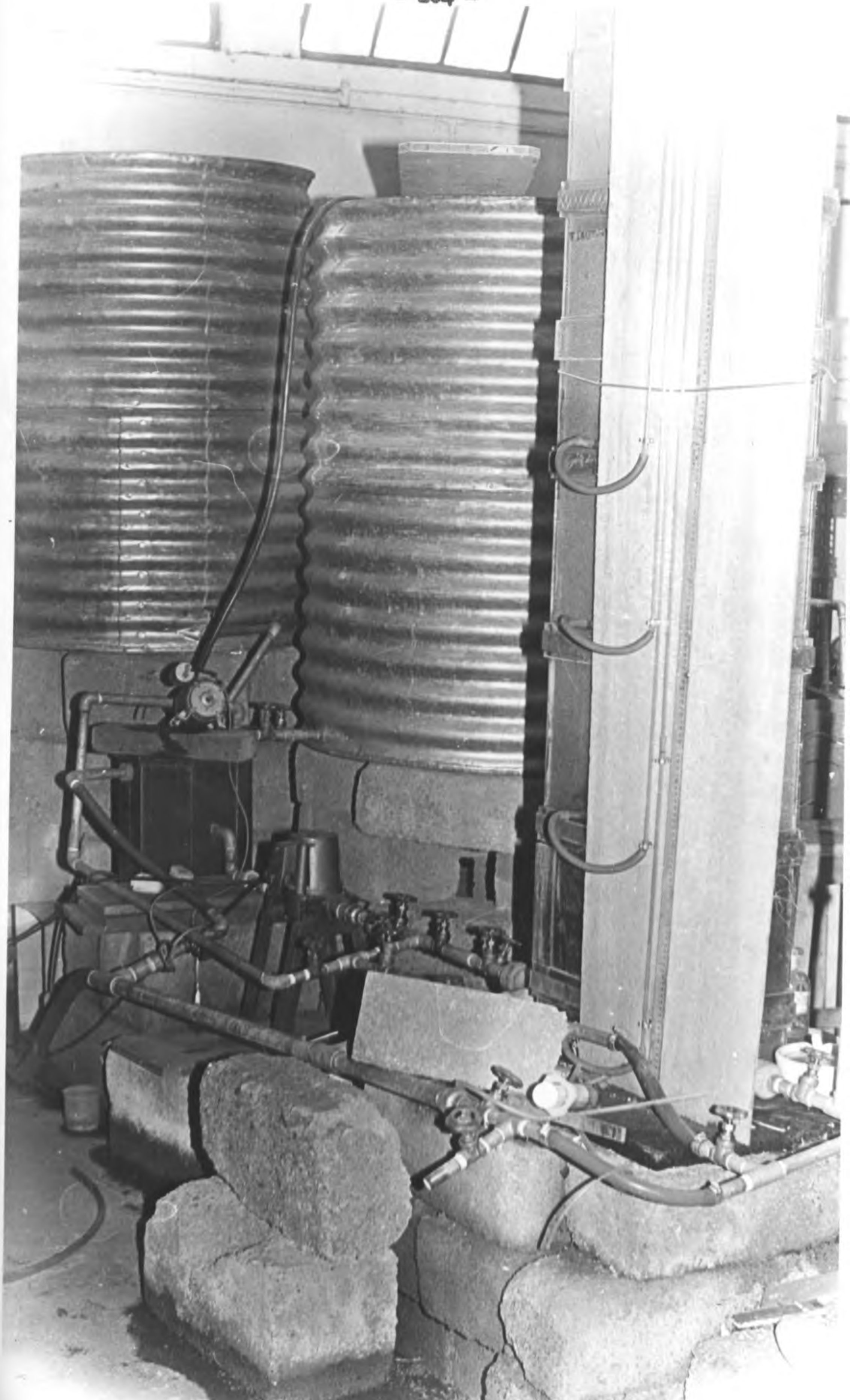


Plate 5F

Plate 5G

CHAPTER 6

RESULTS OF TESTS AND ANALYSIS OF DATA

6.1 Turbidity Variation and Removals

The variations of raw water and filtered water turbidity during the filter runs are outlined graphically in figures 6.1 - 6.9. The results of the readings are tabulated and are found in the appendix. It was observed during the investigation that to reach steady conditions before taking samples was not easy due to short periods of sampling. Sampling was done hourly (or less in some cases).

6.2 Headloss development

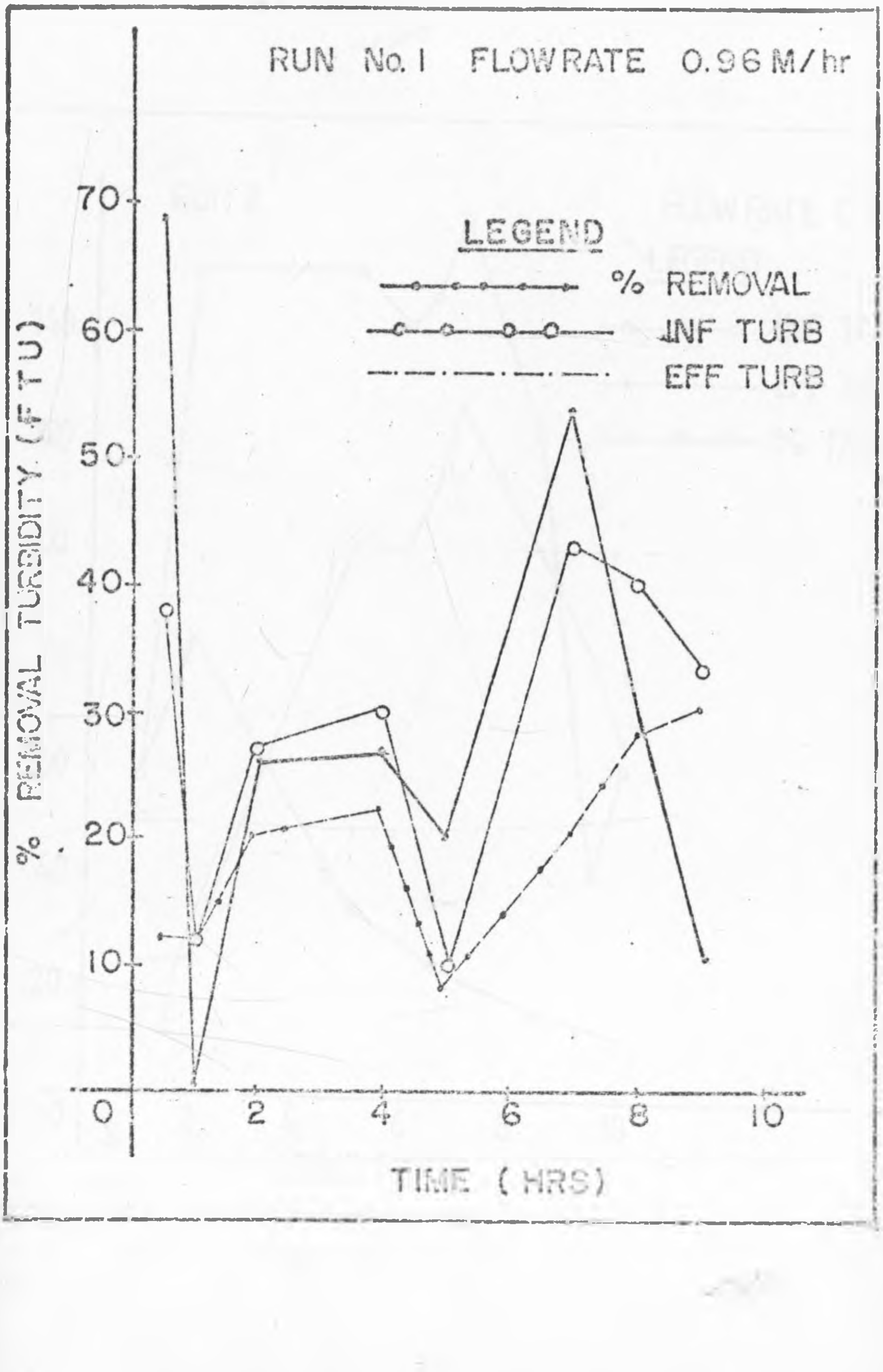
The headloss over the gravel depth of 350 mm (denoted by A-B); sand layer of depth 300 mm (denoted by B-C) and sand layer of depth 350 mm (denoted by C-D) were monitored. The results are graphically outlined in figures 6.10 - 6.18). The manometer readings and calculated headloss values are tabulated and are found in the appendix.

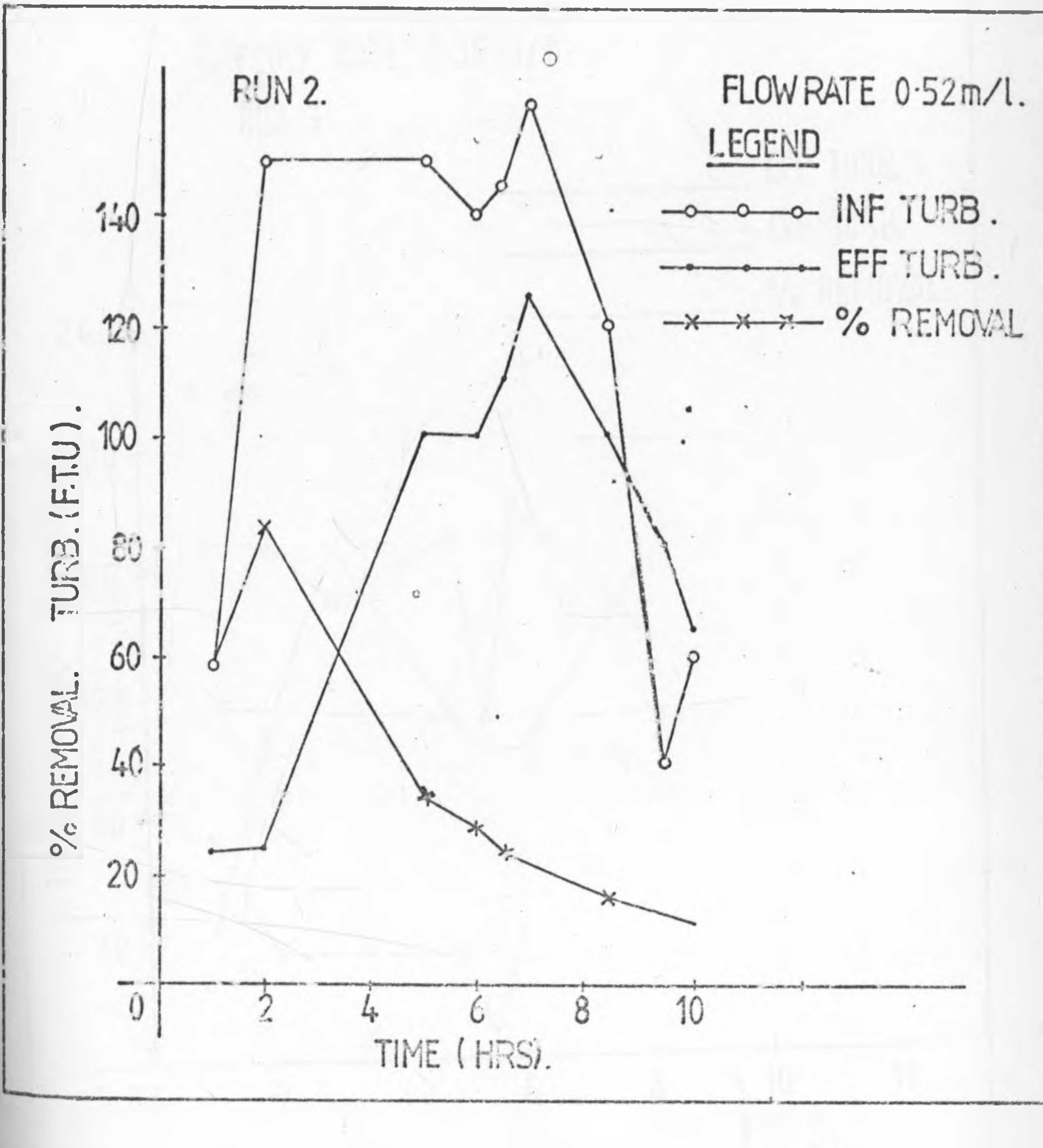
6.3 Colour and pH Variations

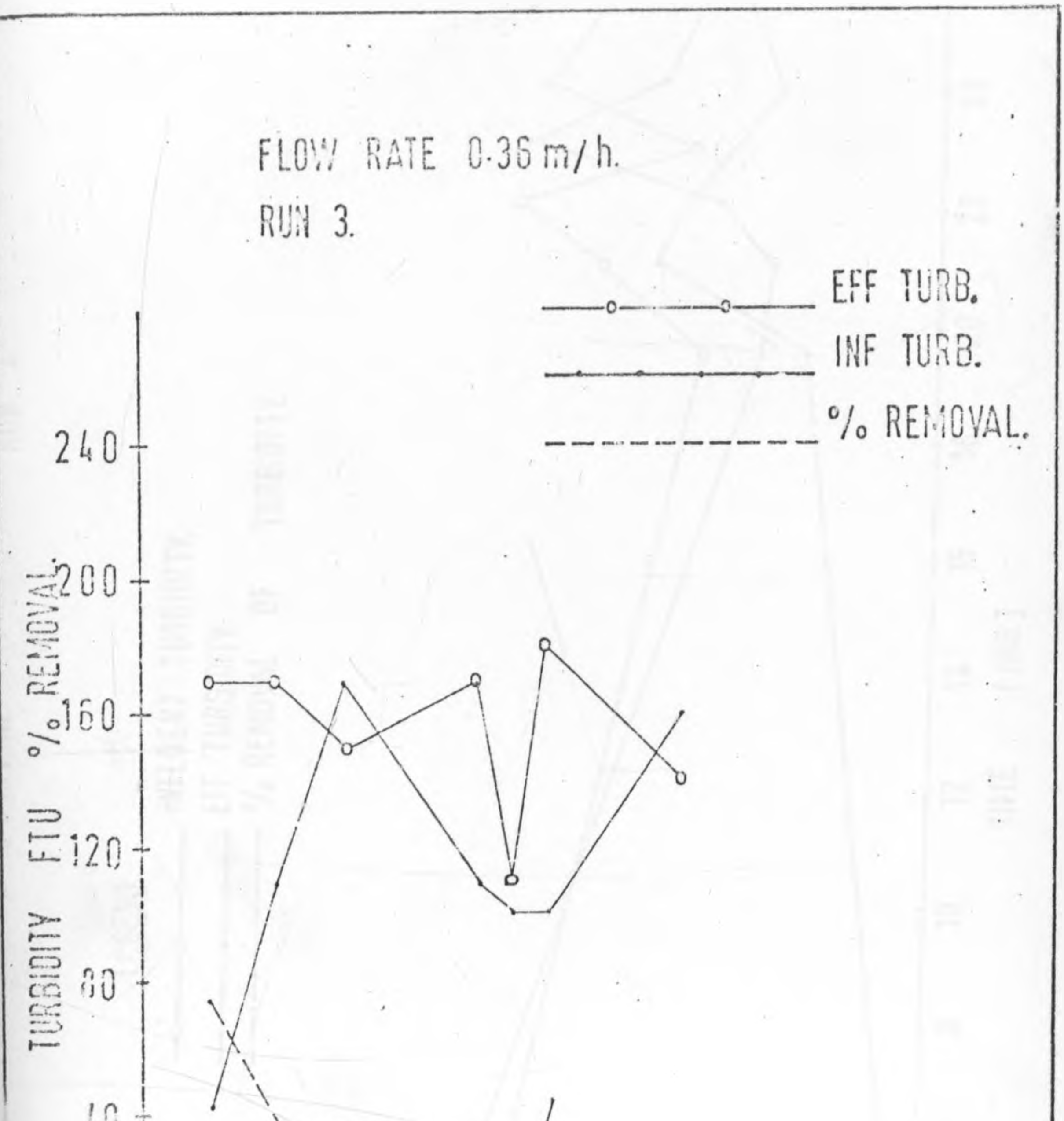
The readings of the Colour and pH variations are tabulated and are found in the appendix. No graphical presentations were made for these parameters.

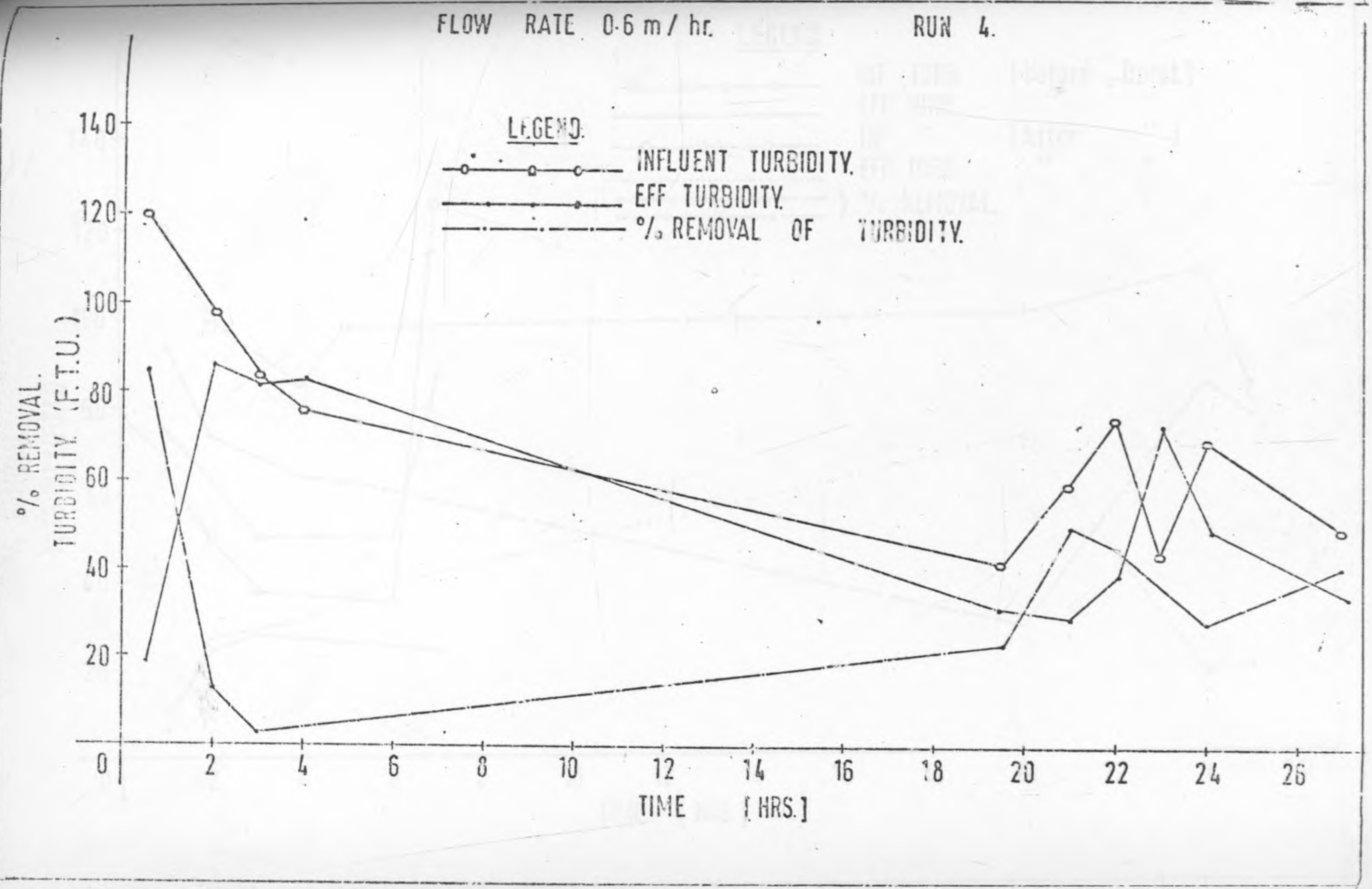
The colour variations and removal is seen to follow nearly the same pattern as that of turbidity removal.

6.4 GRAPHS OF TURBIDITY VARIATIONS

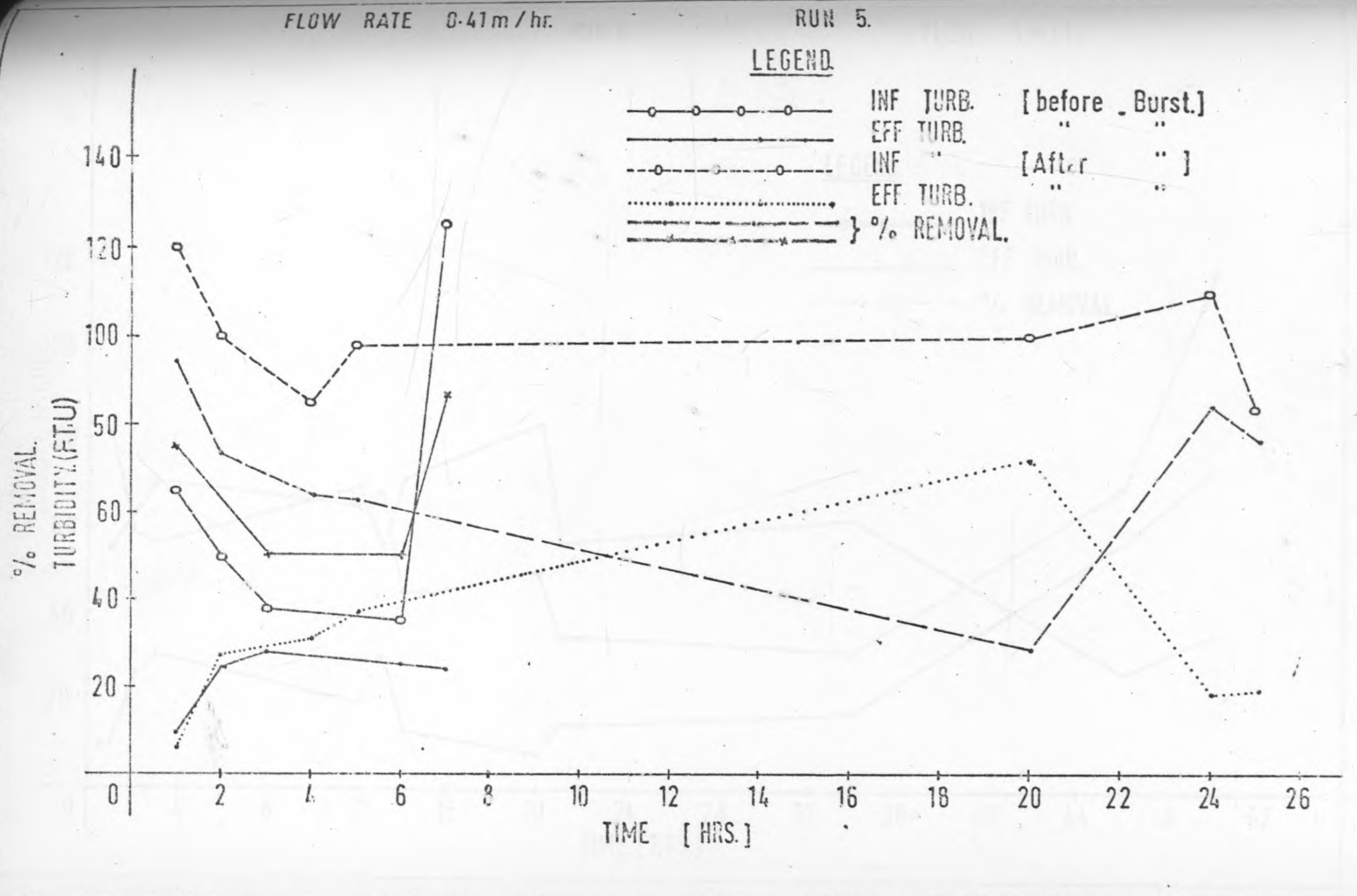

RUN No. I FLOWRATE 0.96 M/hr
LEGEND
% REMOVAL
INF TURB
EFF TURB
% REMOVAL TURBIDITY (F T U)
70
60
50
40
30
20
10
0
2
4
6
8
10
TIME (HRS)

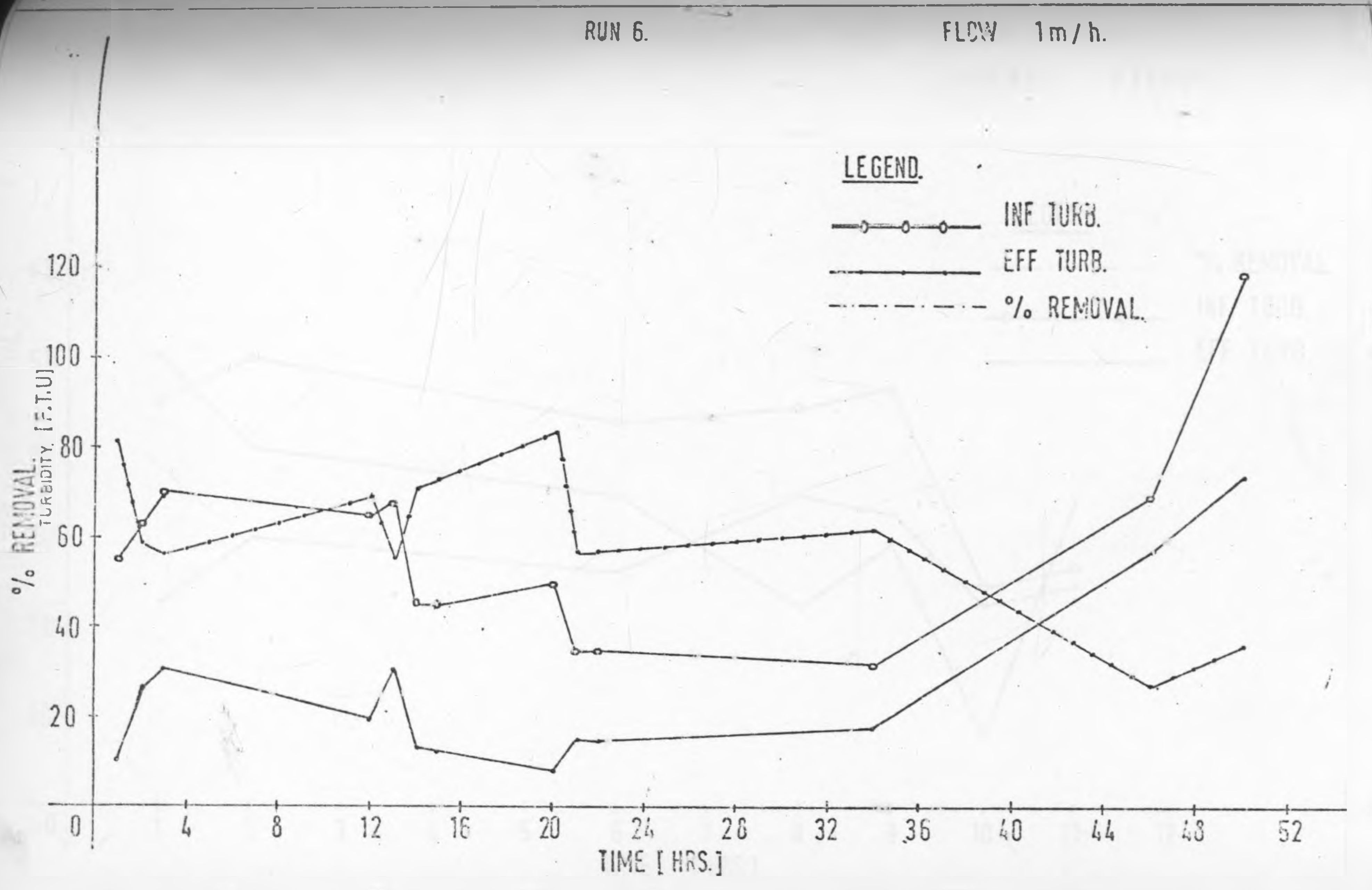

RUN 2.
FLOWRATE 0·52m/l.
LEGEND
INF TURB.
EFF TURB.
% REMOVAL
% REMOVAL. TURB. (F.T.U).
140
120
100
80
60
40
20
0
2
4
6
8
10
TIME (HRS).

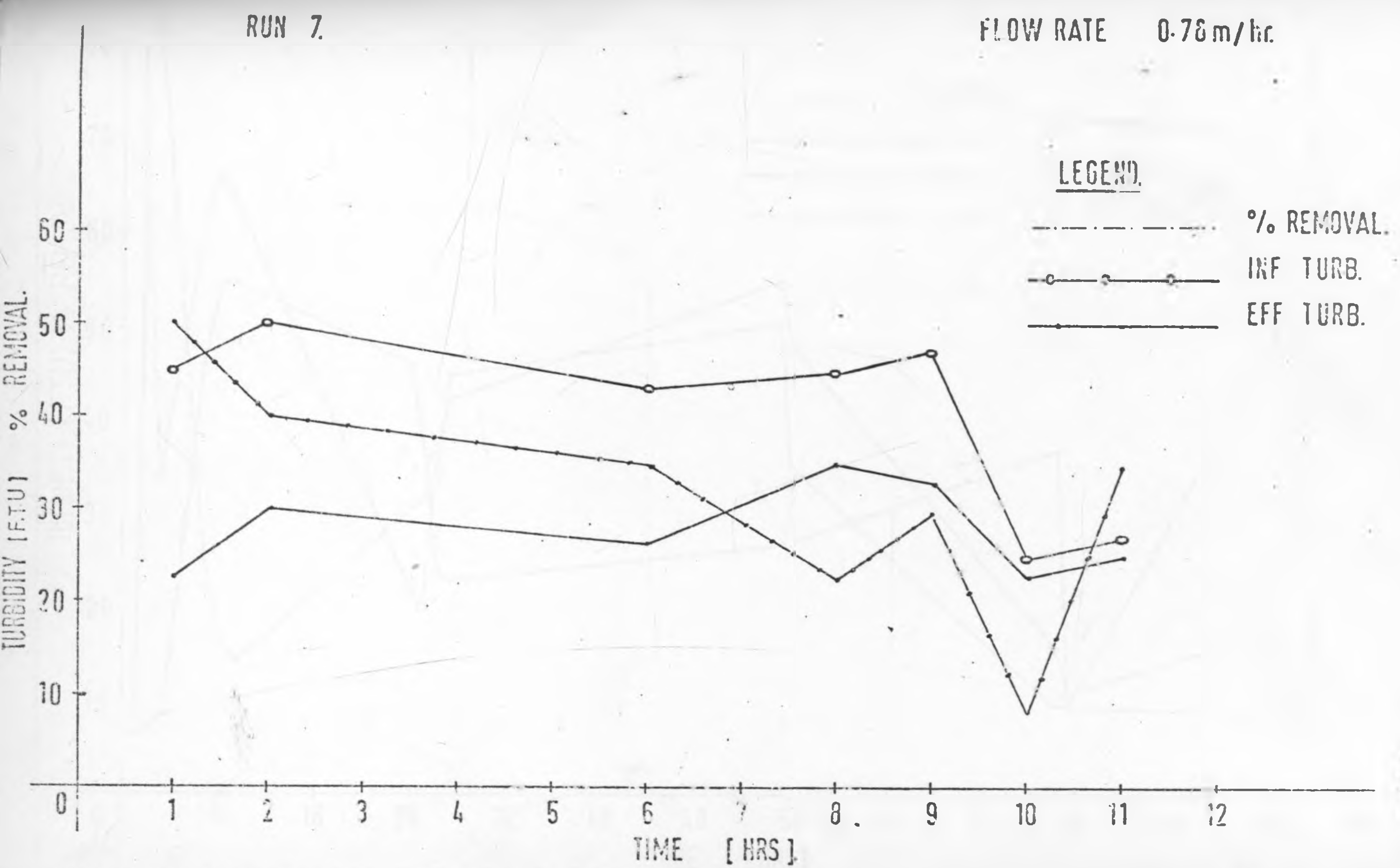

FLOW RATE 0·36 m/h.
RUN 3.
EFF TURB.
INF TURB.
% REMOVAL.
TURBIDITY FTU % REMOVAL
240
200
160
120
80
40
0
2
4
6
8
10
12
TIME [HRS.]

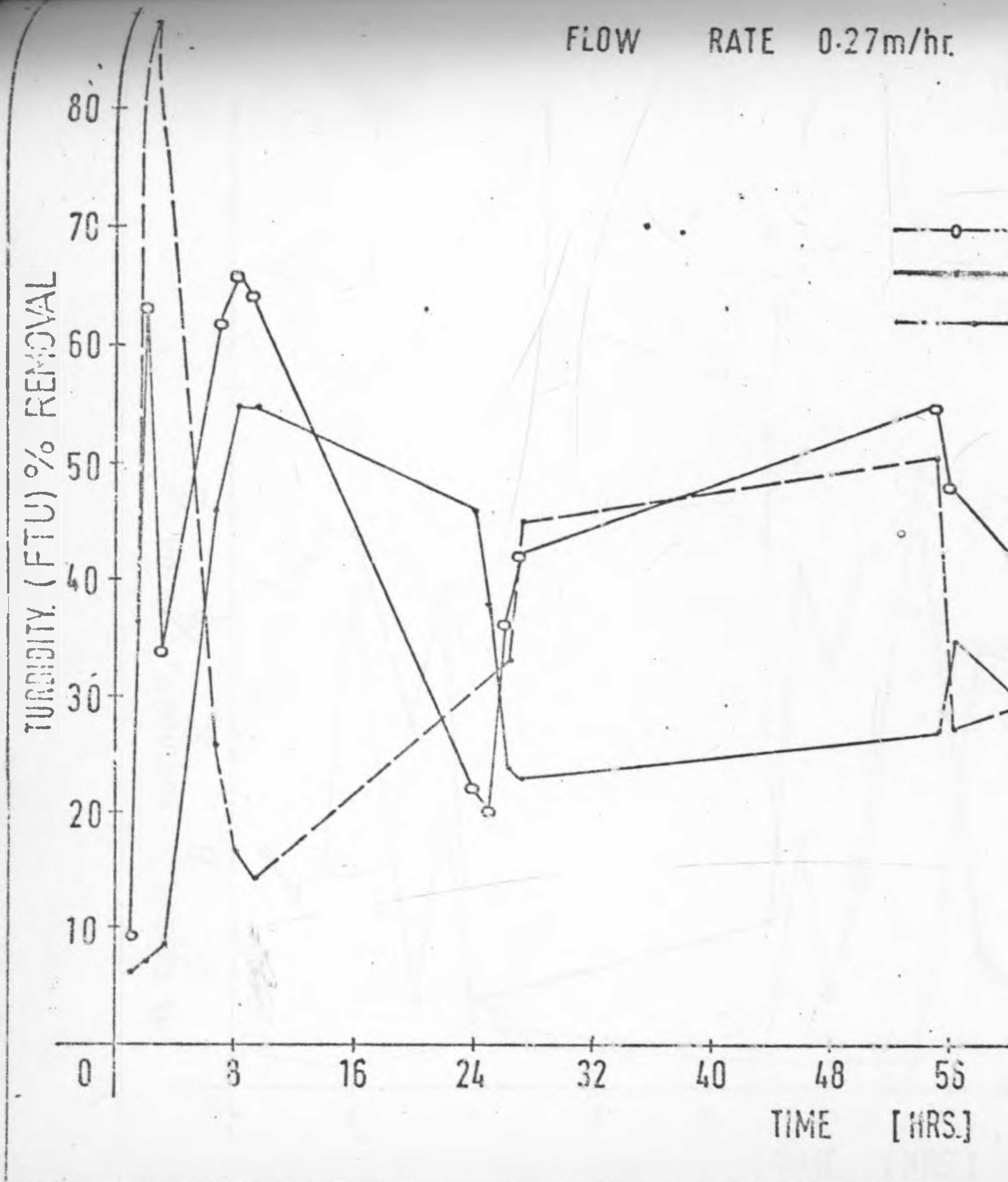


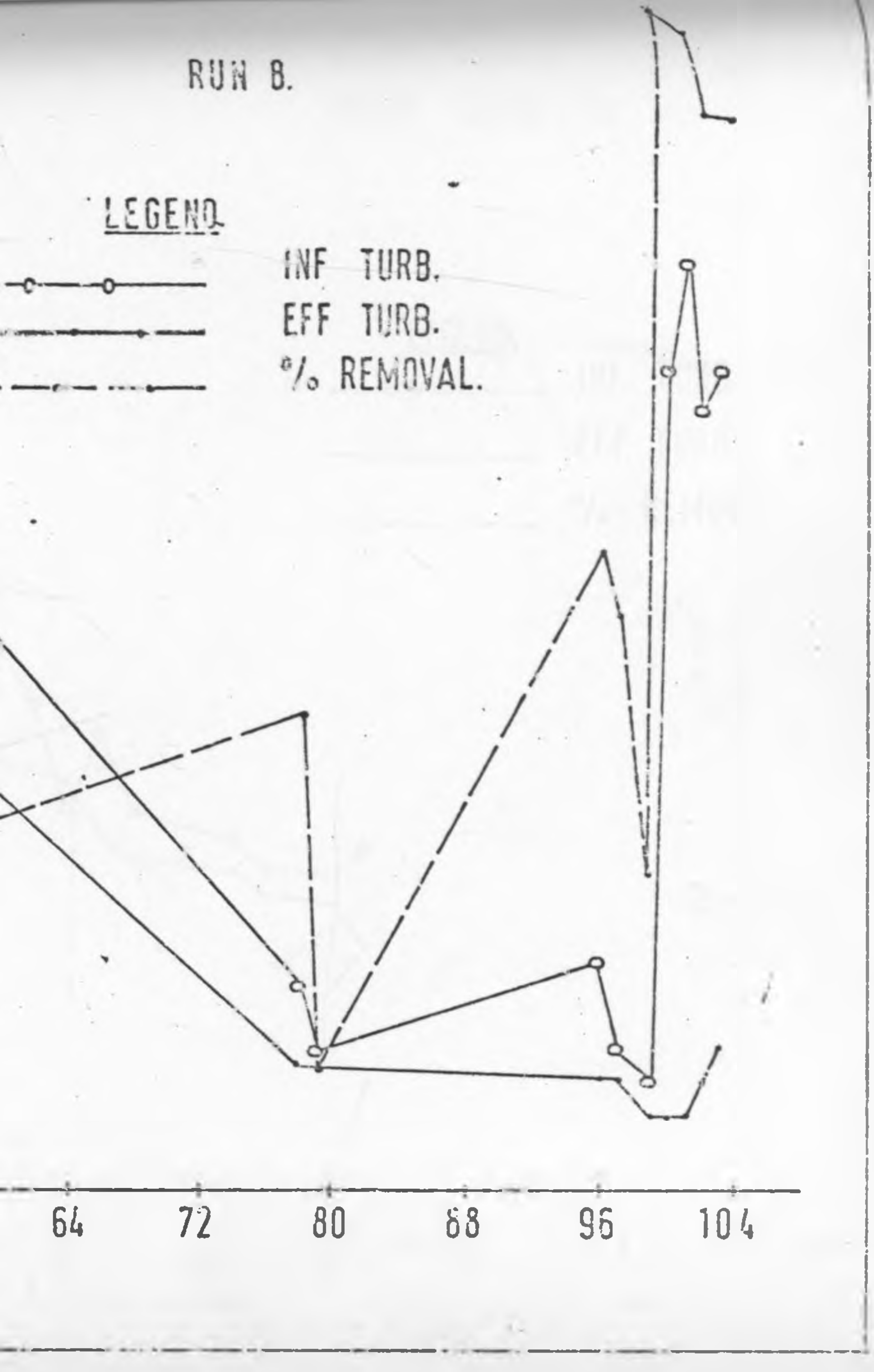
FLOW RATE 0·6 m / hr.
RUN 4.
LEGEND.
INFLUENT TURBIDITY.
EFF TURBIDITY.
% REMOVAL OF TURBIDITY.
% REMOVAL.
TURBIDITY (F.T.U.)
140
120
100
80
60
40
20
0
2
4
6
8
10
12
14
16
18
20
22
24
26
TIME [HRS.]


FLOW RATE 0·41m/hr.
RUN 5.
LEGEND
INF TURB. [before Burst.]
EFF TURB. " "
INF " [After "]
EFF TURB. " "
} % REMOVAL.
% REMOVAL.
TURBIDITY (FTU)
140
120
100
50
60
40
20
0
2
4
6
8
10
12
14
16
18
20
22
24
26
TIME [HRS.]

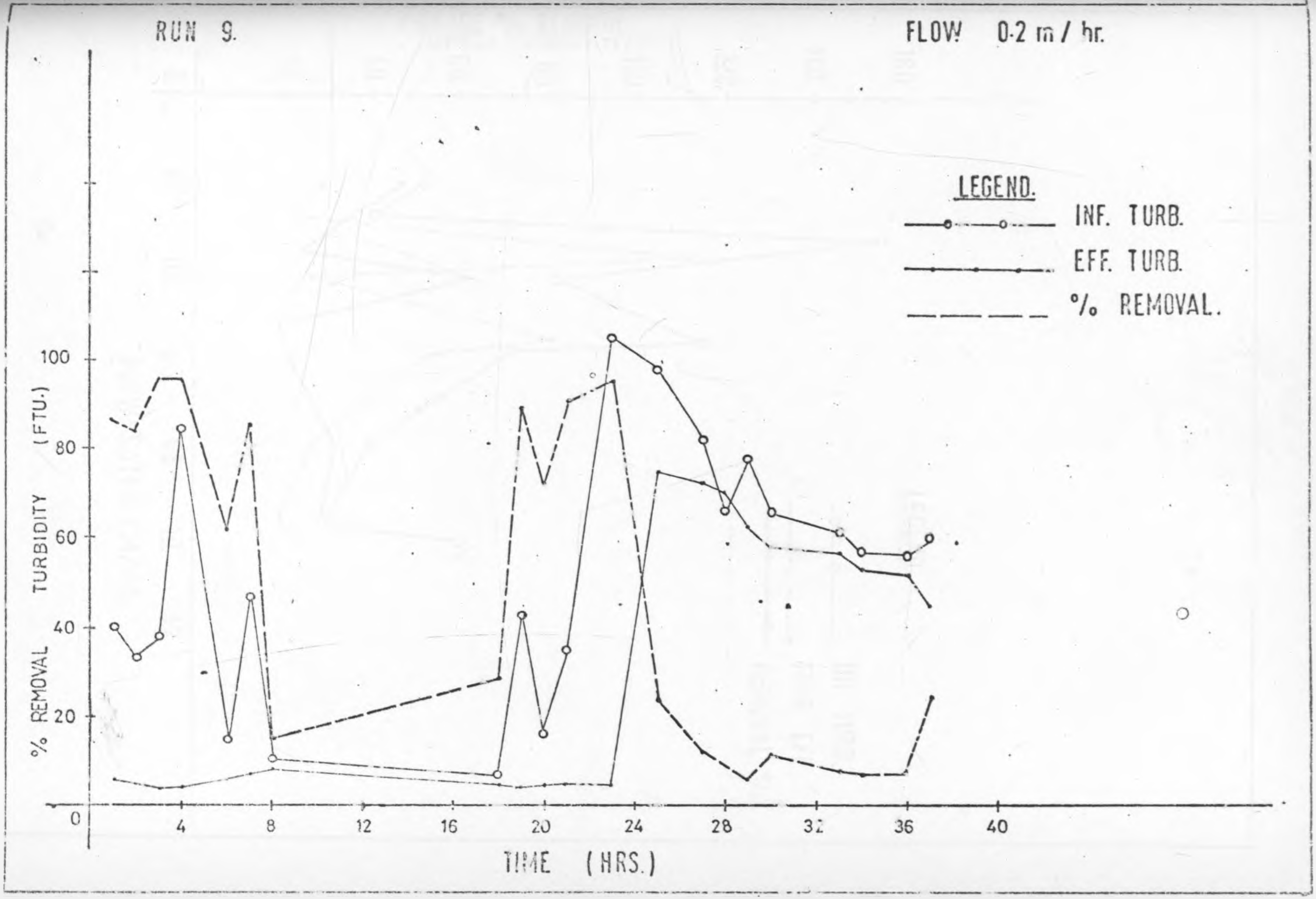

RUN 6.
FLOW 1m/h.
LEGEND.
INF TURB.
EFF TURB.
% REMOVAL.
% REMOVAL.
TURBIDITY [F.T.U]
120
100
80
60
40
20
0
4
8
12
16
20
24
28
32
36
40
44
48
52
TIME [HRS.]


RUN 7.
FLOW RATE 0·78 m/hr.
LEGEND.
% REMOVAL.
INF TURB.
EFF TURB.
TURBIDITY [F.T.U] % REMOVAL.
60
50
40
30
20
10
0
1
2
3
4
5
6
7
8
9
10
11
12
TIME [HRS].

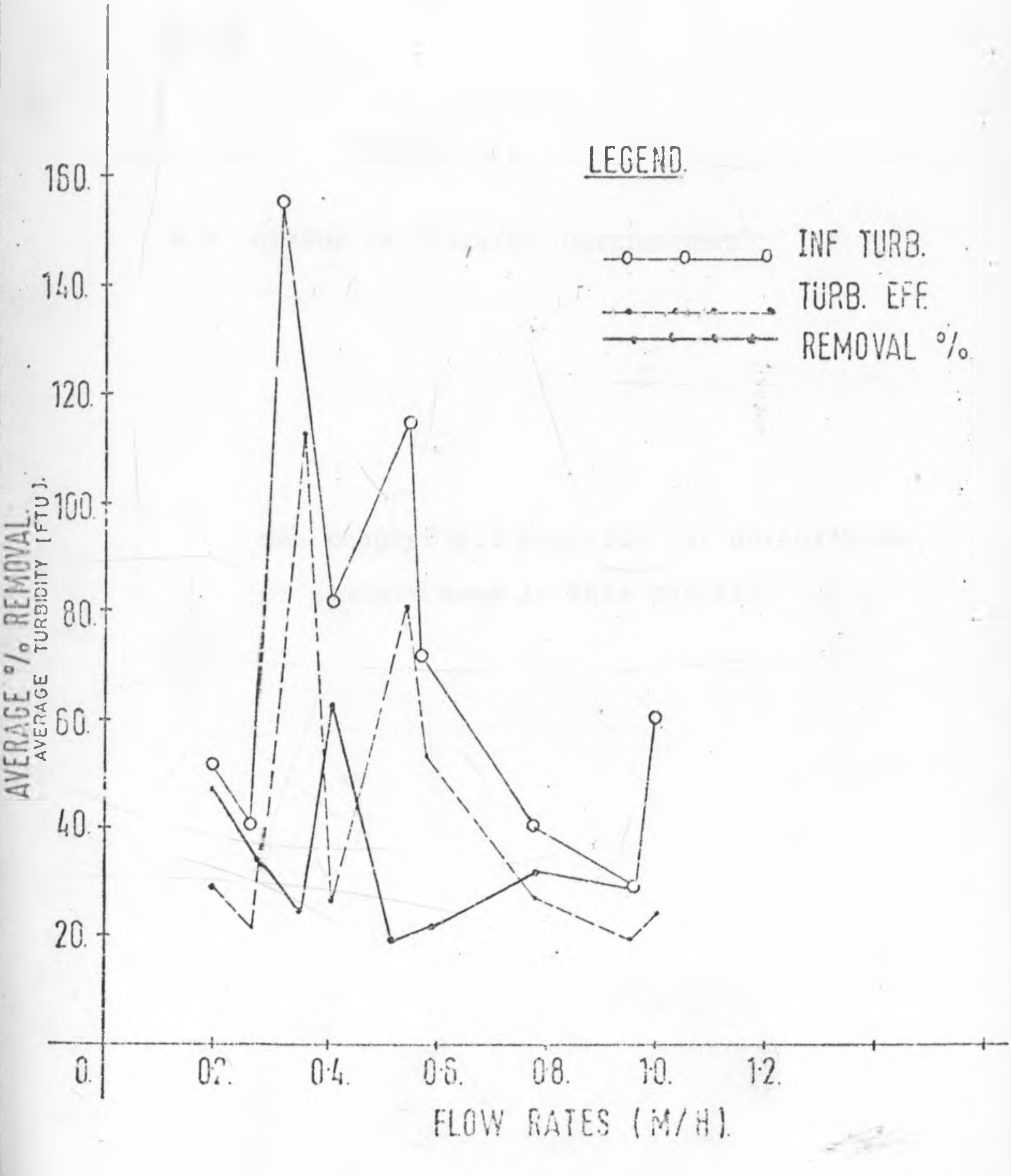

FLOW RATE 0·27m/hr.
TURBIDITY (FTU) % REMOVAL
80
70
60
50
40
30
20
10
0
8
16
24
32
40
48
56
TIME [HRS.]



RUN 8.
LEGEND
INF TURB.
EFF TURB.
% REMOVAL.
64
72
80
88
96
104

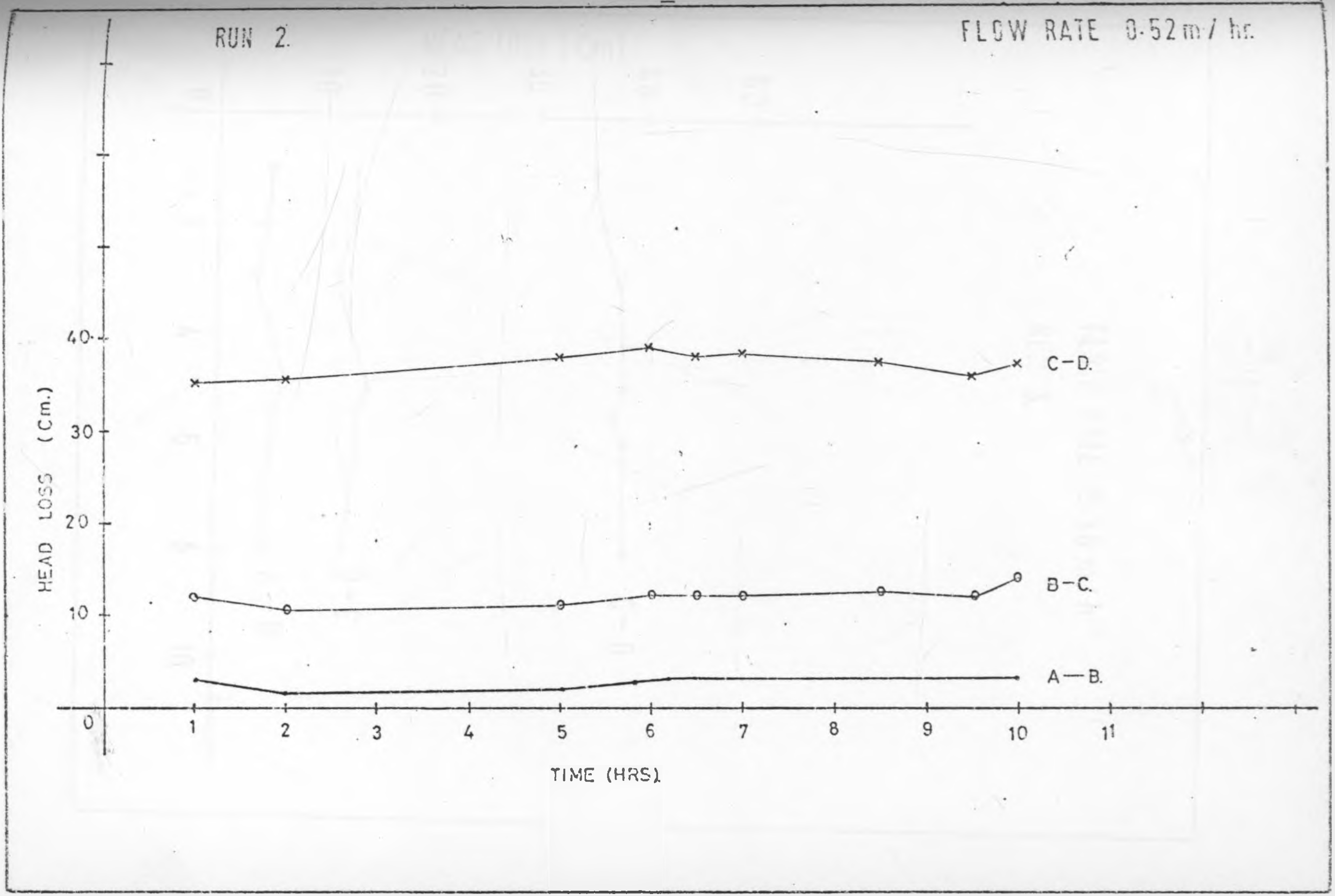


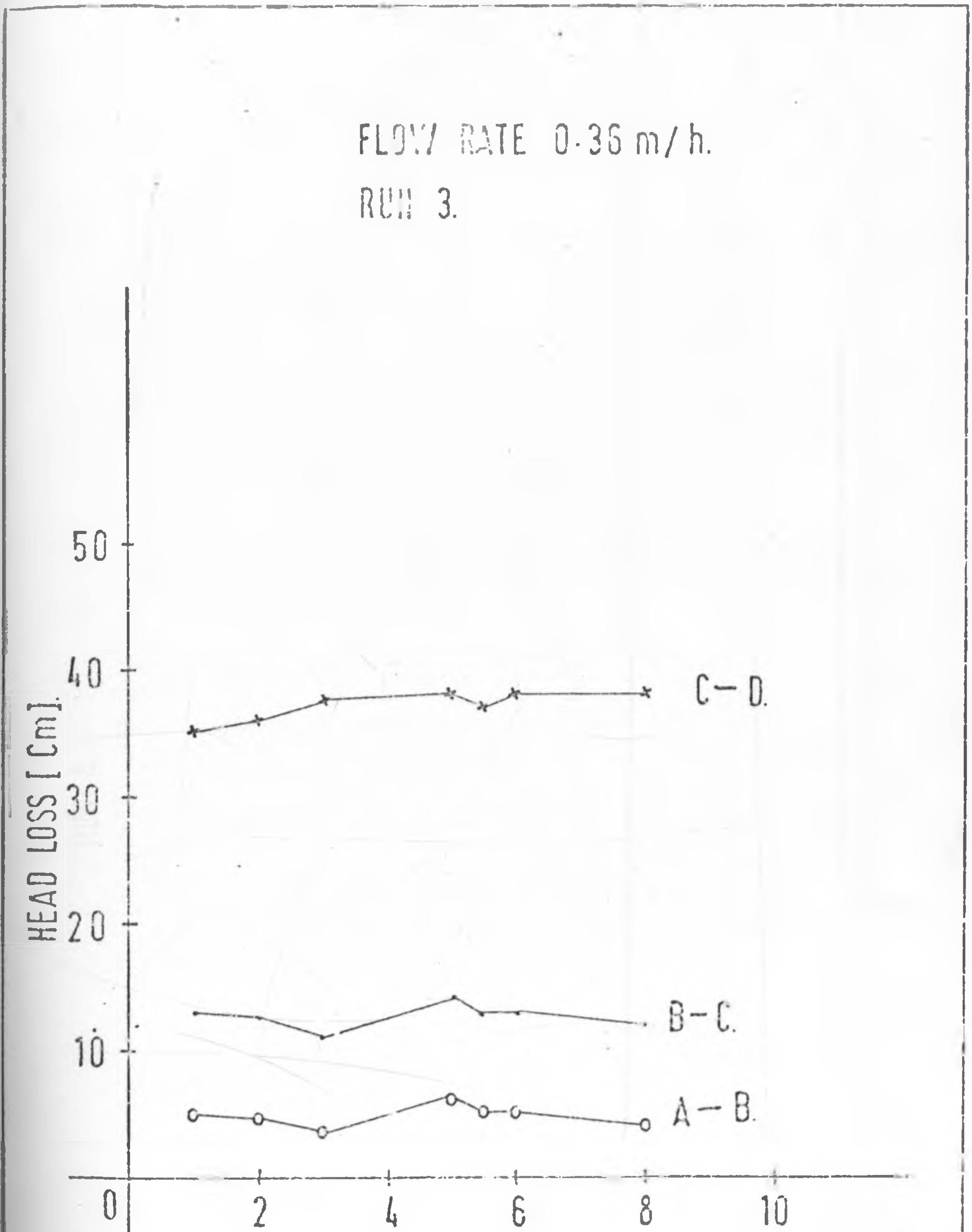
RUN 9.
FLOW 0·2 m / hr.
LEGEND.
INF. TURB.
EFF. TURB.
% REMOVAL.
% REMOVAL TURBIDITY (FTU.)
100
80
60
40
20
0
4
8
12
16
20
24
28
32
36
40
TIME (HRS.)

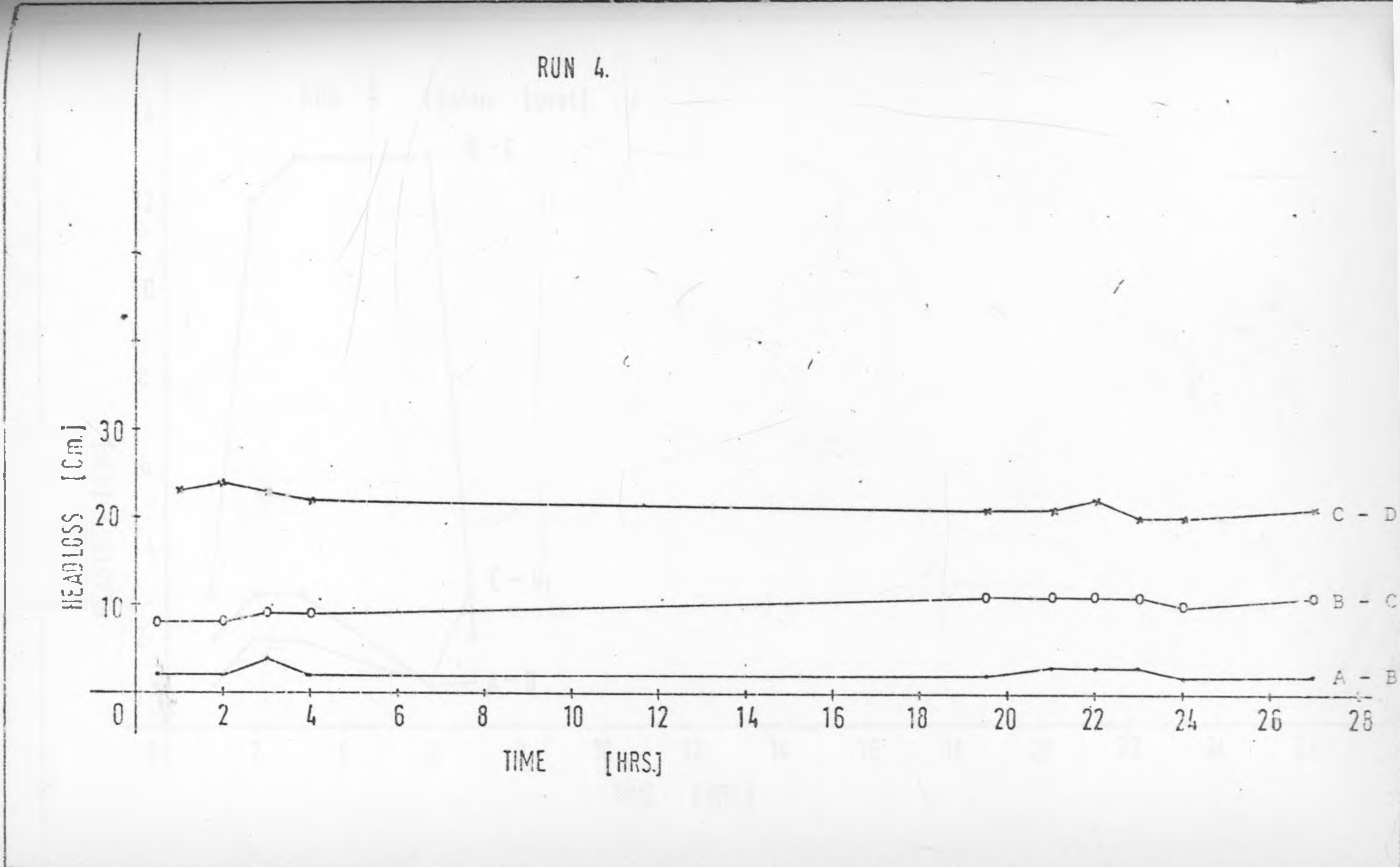

LEGEND.
INF TURB.
TURB. EFF.
REMOVAL %
AVERAGE % REMOVAL.
AVERAGE TURBIDITY (FTU).
160.
140.
120.
100.
80.
60.
40.
20.
0.
0.2.
0.4.
0.6.
0.8.
1.0.
1.2.
FLOW RATES (M/H).

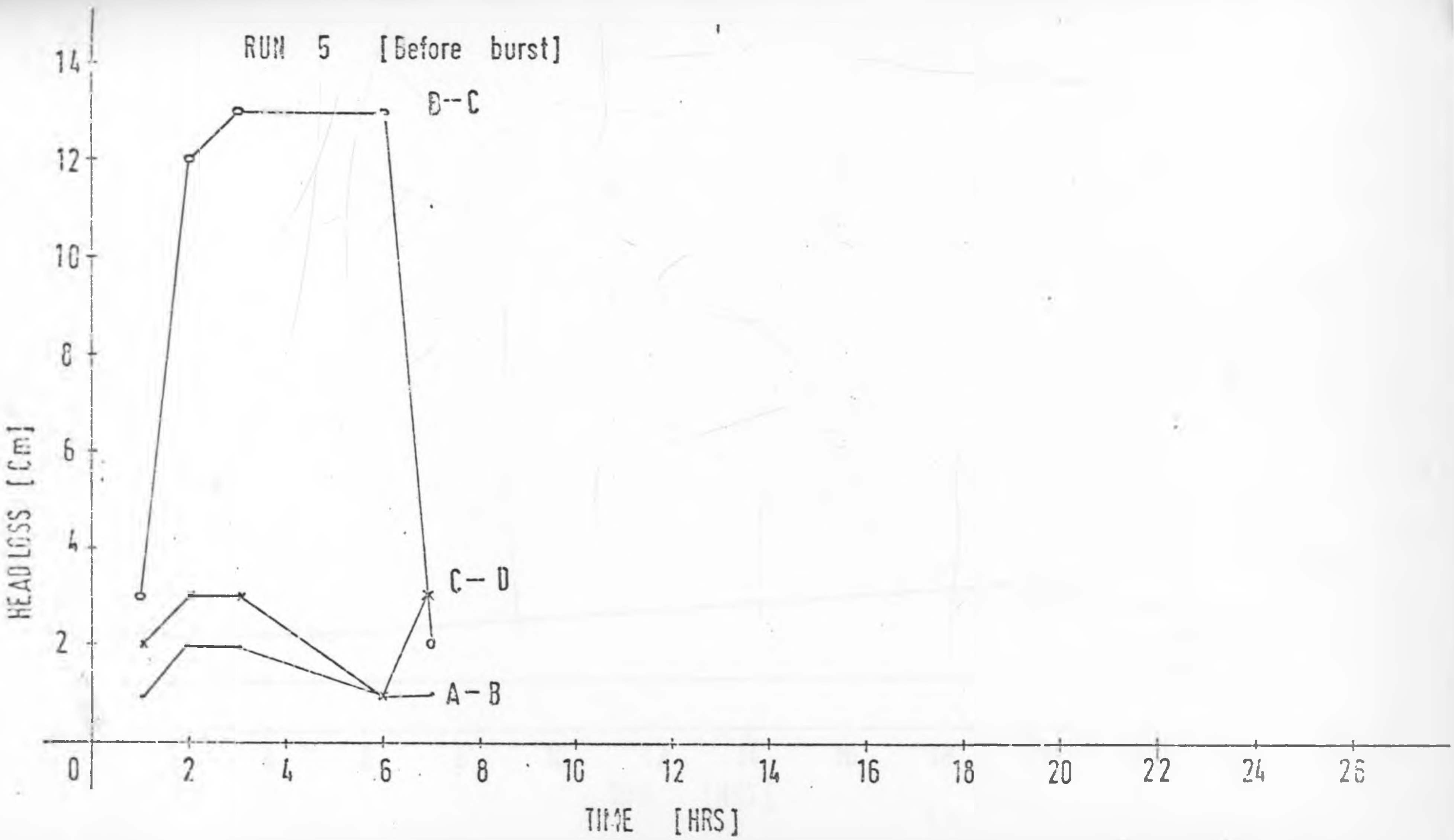
6.5 GRAPHS OF HEADLOSS DEVELOPMENT

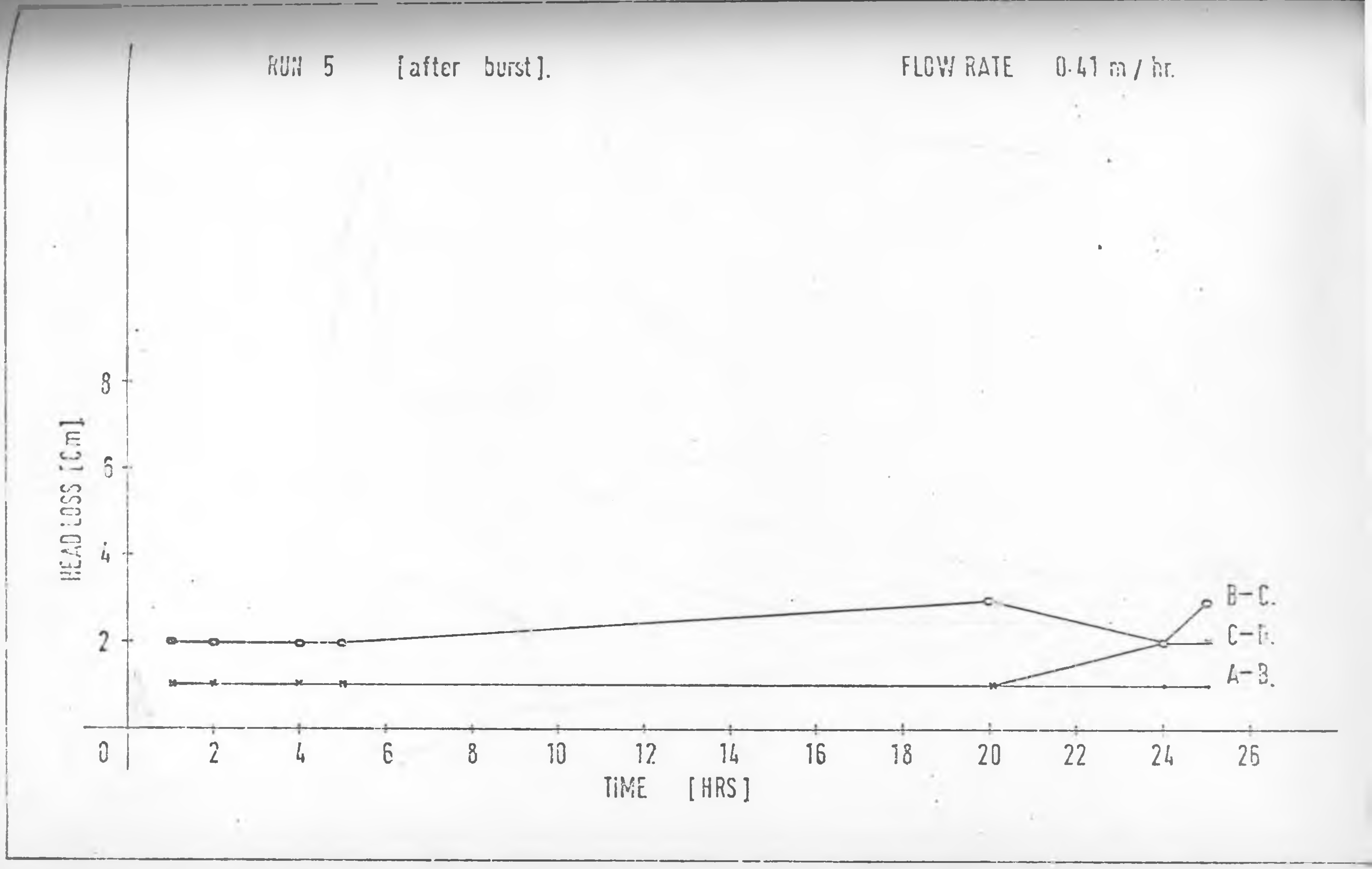
See chapter 6.2 page 106 for definitions of symbols used in this chapter.

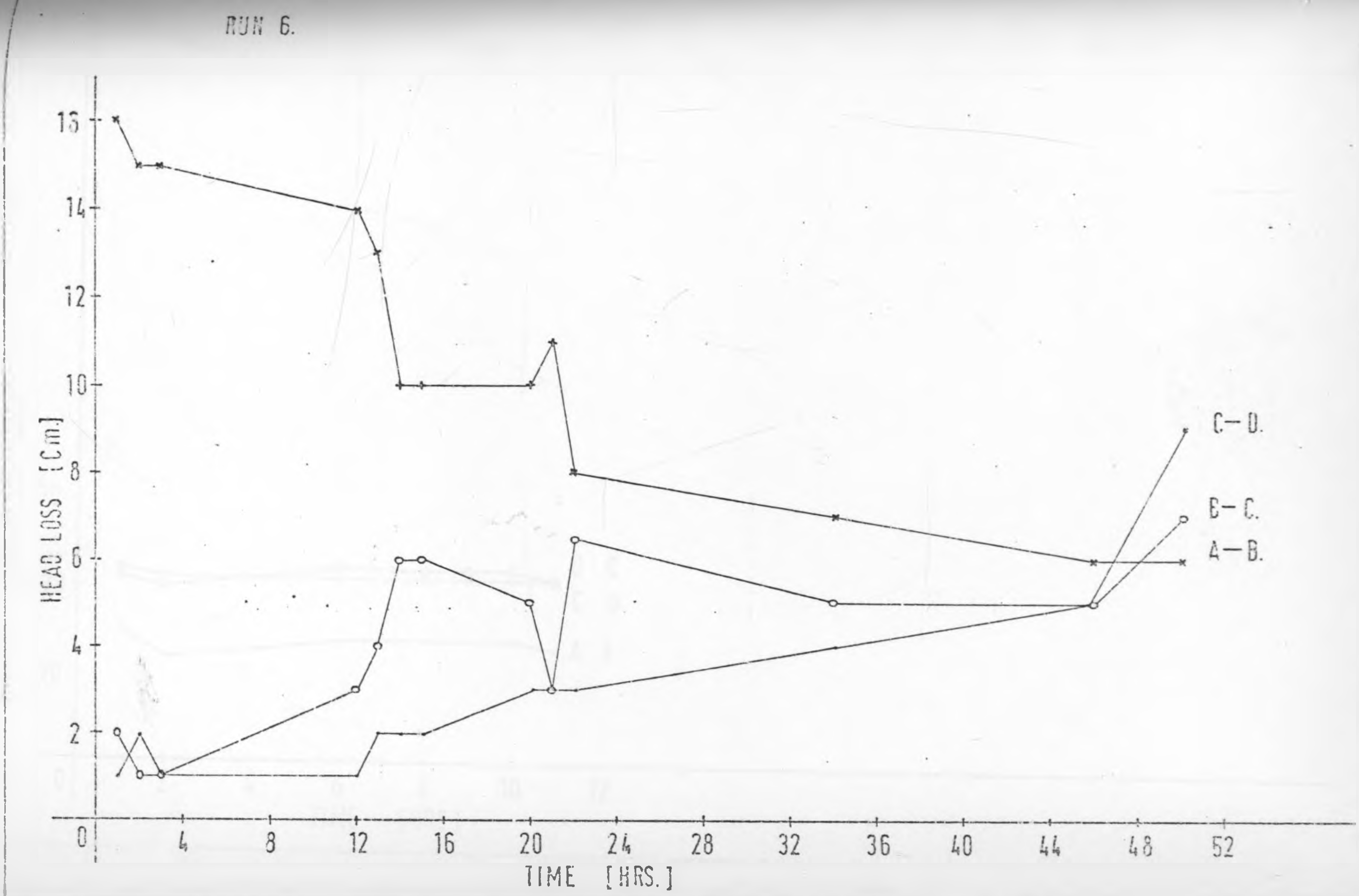

RUN 2.
FLOW RATE 0.52 m/hr.
HEAD LOSS (Cm.)
40
30
20
10
0
1
2
3
4
5
6
7
8
9
10
11
TIME (HRS)
C-D.
B-C.
A—B.

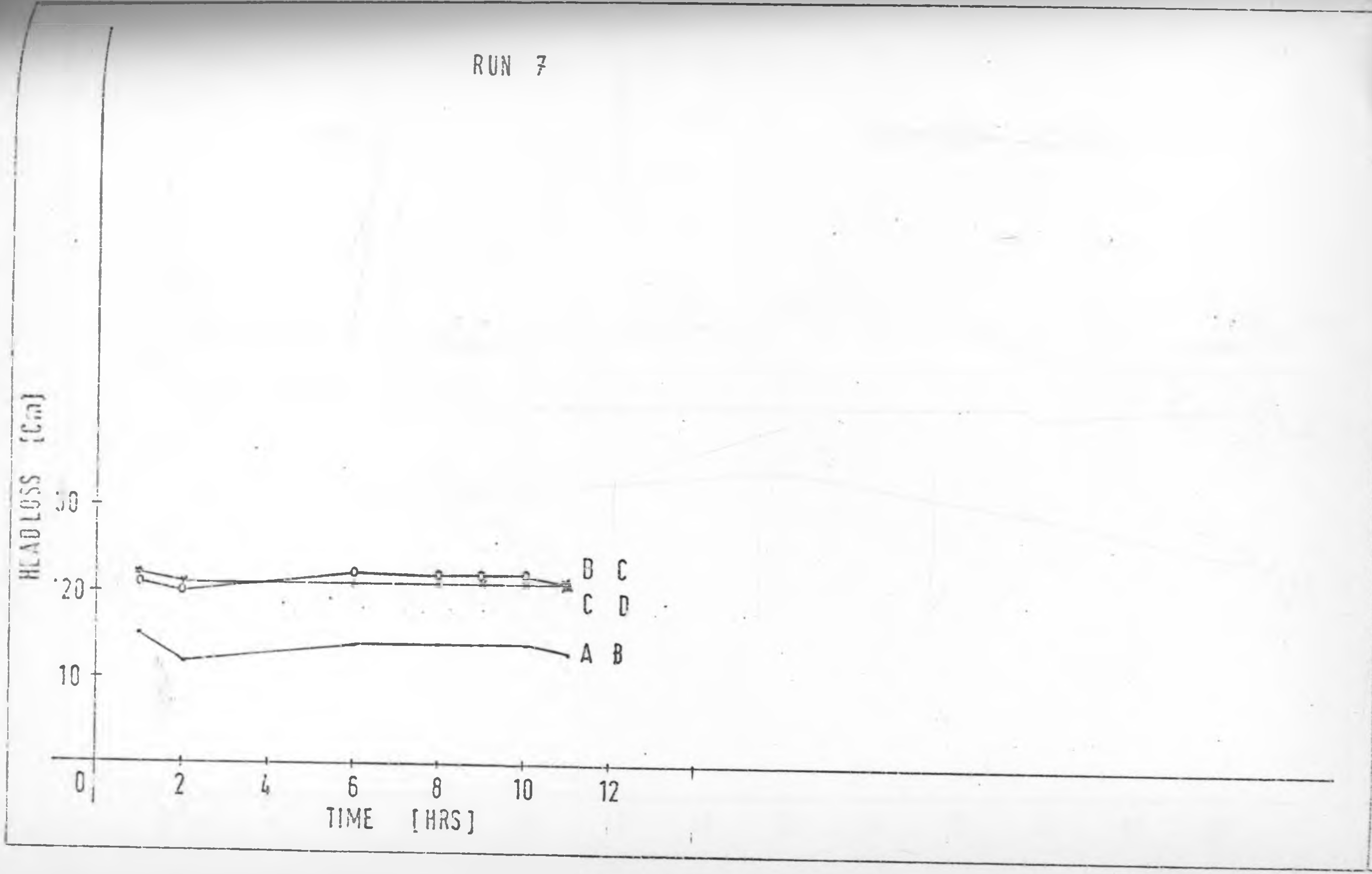

FLOW RATE 0·36 m/h.
RUN 3.
HEAD LOSS [Cm].
50
40
30
20
10
0
2
4
6
8
10
C — D.
B — C.
A — B.

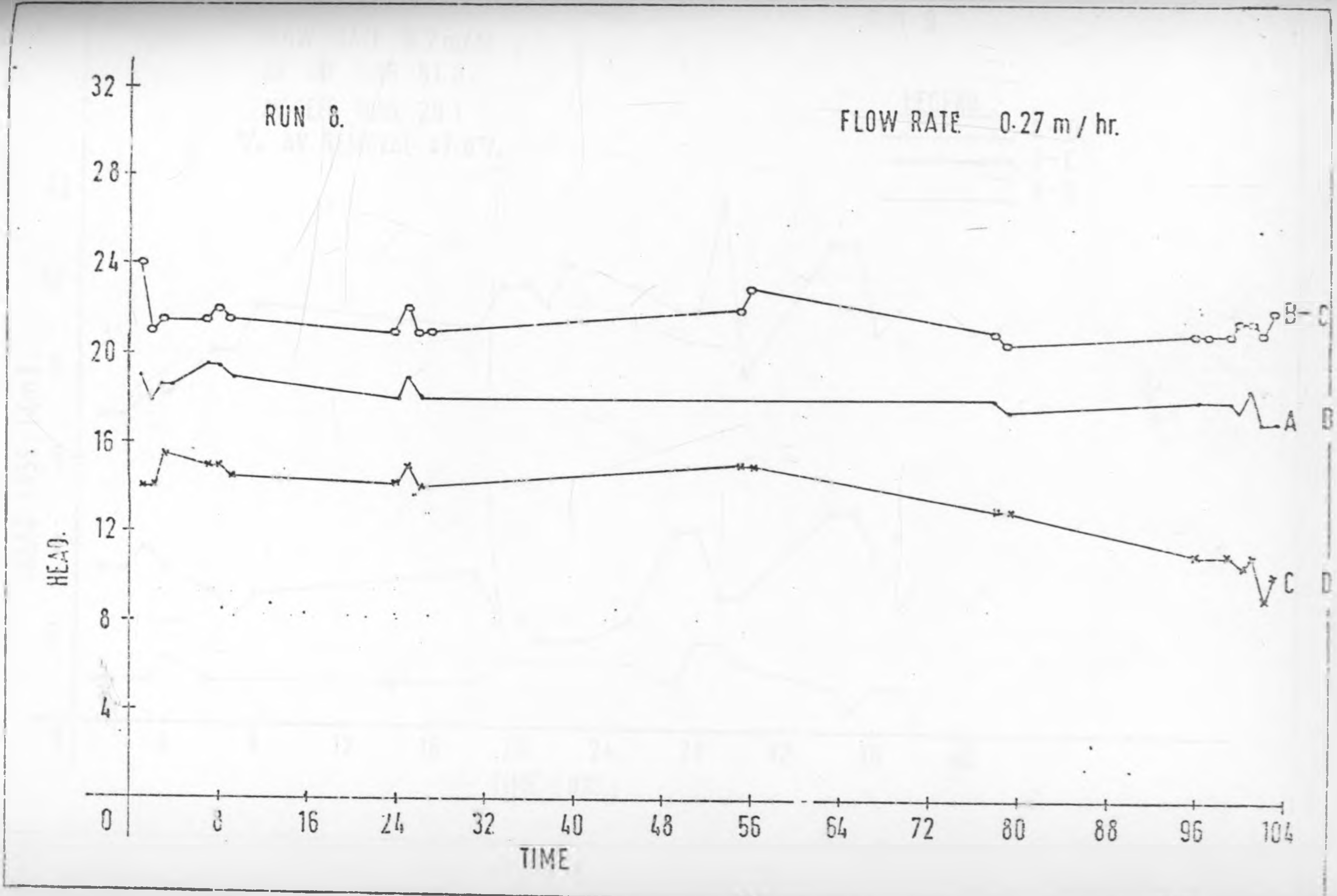

RUN 4.
HEADLOSS [Cm.]
30
20
10
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
TIME [HRS.]
C - D
B - C
A - B

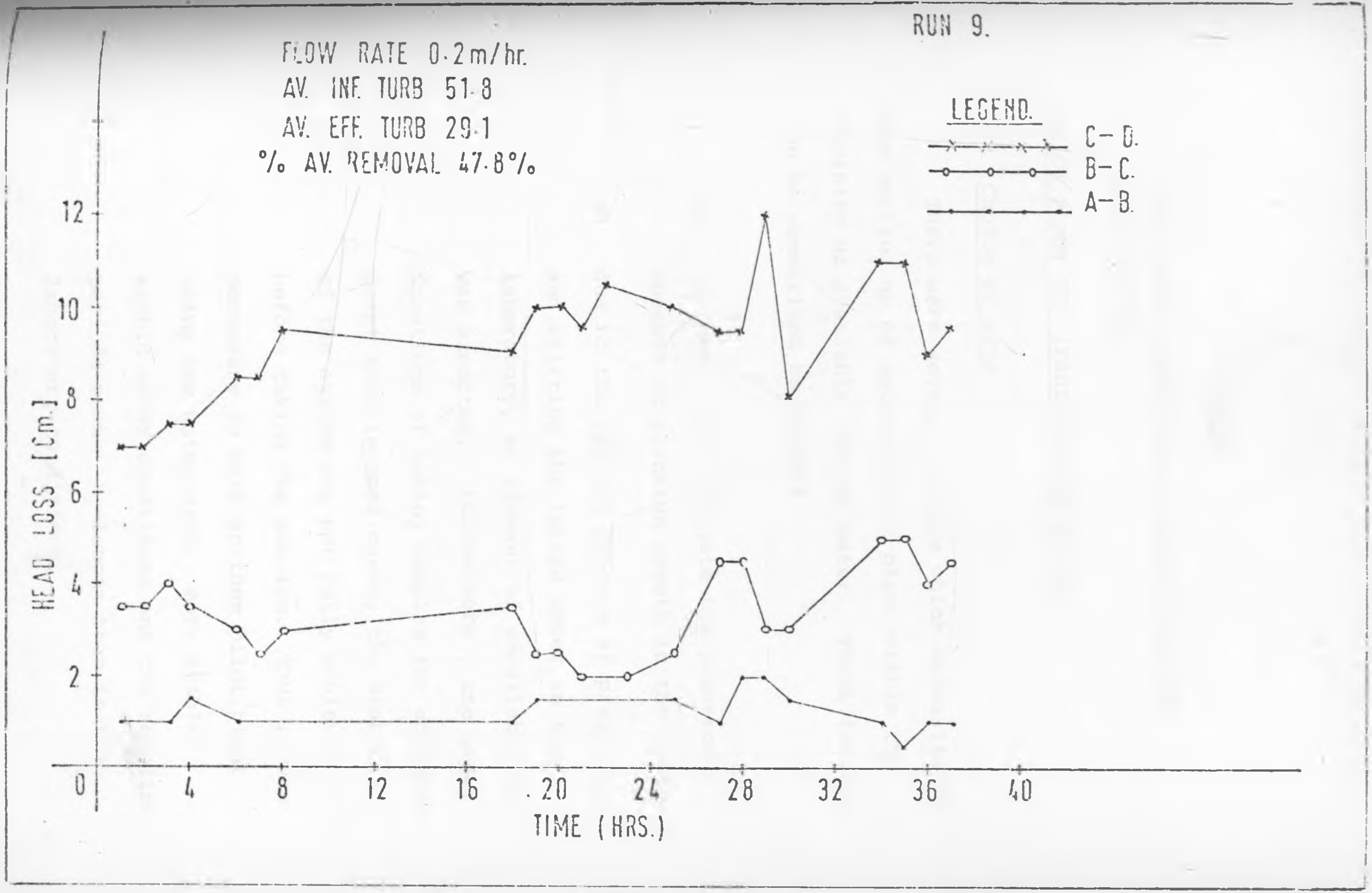

RUN 5 [Before burst]
B--C
C--D
A--B
HEADLOSS [Cm]
TIME [HRS]
0
2
4
6
8
10
12
14
16
18
20
22
24
26


RUN 5 [after burst].
FLOW RATE 0.41 m / hr.
HEAD LOSS [Cm]
8
6
4
2
0
2
4
6
8
10
12
14
16
18
20
22
24
26
TIME [HRS]
B−C.
C−D.
A−B.


RUN 6.
16
14
12
10
8
6
4
2
0
HEAD LOSS [Cm]
4
8
12
16
20
24
28
32
36
40
44
48
52
TIME [HRS.]
C– D.
B– C.
A– B.


RUN 7
HEAD LOSS [Cm]
30
20
10
0
2
4
6
8
10
12
TIME [HRS]
B C
C D
A B


RUN 8.
FLOW RATE 0.27 m / hr.
HEAD.
TIME
B-C
A
C
32
28
24
20
16
12
8
4
0
8
16
24
32
40
48
56
64
72
80
88
96
104


RUN 9.
FLOW RATE 0·2m/hr.
AV. INF. TURB 51·8
AV. EFF. TURB 29·1
% AV. REMOVAL 47·8%
LEGEND.
C– D.
B– C.
A– B.
HEAD LOSS [Cm.]
0
2
4
6
8
10
12
4
8
12
16
20
24
28
32
36
40
TIME (HRS.)

CHAPTER 7

THE DESIGN, OPERATION & LOCATION OF THE PILOT PLANT.

PILOT PLANT II (KABETE WATER WORKS)

7.1 Choice of site

There were several factors which necessitated the setting up of another pilot plant within the vicinity of available natural water. These factors can be summarised as follows:-

a) the need to investigate the presence/ absence of plankton growth in the filter

b) due to the way and process of preparing and stirring the turbid water in the laboratory, an element of unreliability was expected. Furthermore , the short durations of taking samples for analysis meant that in most cases, the stability of the system was not fully achieved before taking the samples. Thus it was necessary to have another pilot plant using raw water with more stable turbid water conditions and the sampling periods much longer than in the laboratory conditions.

The choice of the site was made having the following points in mind:-

a) distance from the department

b) availability of transport facilities to the site

c) the ease of bringing samples for analysis to the Environmental Health Engineering laboratory.

Kabete water works, run by the city council of Nairobi for water treatment and distribution was chosen. The site is about 10 km away from the department. The treatment works handles 24,600 m^3/day of water which is supplied from Ruiru dam and Kikuyu springs. The raw water from Kikuyu springs is a small percentage of the total volume of water handled and has relatively low turbidity. But the raw water from Ruiru Dam has appreciavely high turbidity which requires addition treatment of coagulation and sedimentation before being filtered and disinfected. Although the turbidity of this water varies with small magnitude, it was hoped that with the coming of the rains, an appreciable variation would be possible during the course of the experiment. Unfortunately, the rains came later than expected and the turbidities of the water was generally very low.

7.2 The arrangement of the pilot plant

The layout for the system is shown in fig. 7A. One of the steel pipes of diameter 405 mm, bringing raw water from Ruiru Dam was tapped and used as the source of raw water for the investigation. An off take 19mm diam GI pipe was connected to the steel pipe and the filter box. The filter was fabricated from galvanized iron sheets of gauge 16 with dimensions 2500 mm x 1250 mm. The height of the filter was 2500 mm and the diameter was 400 mm. The filter bottom had aconical shaped inlet and above this was a perforated galvanized iron plate which supported the filter media and also acted as a distributor plate releasing inlet water uniformly over the filter bed. A 20mm diam. hose pipe was provided to drain off the filtrate (see plate 7F). To facilitate the placing of the microscopic slides inside the filter, two cut-out windows were made on the filter.

7.3 Filter box contents

The filter box contained the following facilities and filter media:-

(i) A perforated supporting plate (which has been described earlier).

(ii) 100 mm ¾" aggregate size
50 mm ⅜" aggregate size.

(iii) 1000 mm depth of filter sand of effective size 0.22 mm and the uniformly coefficient of 6.46. It was the same sand used for laboratory investigations.

(iv) Microscopic slides placed inside the windows. There were however some filter runs in which slides were not inside the filter.

7.4 Filter Control Devices, Filtration and Sampling Procedures

The first filter run was started by opening all the valves along the 19 mm diam. GI pipe which was conveying the raw water into the filter. Valve V2 was closed while valves VI and V3 (see fig. 7A page 136) were fully opened. Because the pipeline had been in existence for over two years, it was possible that the pipes were rusted and hence the first influent raw water carried lot of dirt from the pipe line. This indeed was the case when the first influent water was drained to waste by opening valve V3 and closing valve V2. After sometimes, valve V2 was opened gradually and valve V3 was closed completely. Valve V4 was fully opened and valve V5 was fully closed. The flow rate was then determined by use of a graduated cylinder. A stop watch was used for keeping the time. An interval of 15 seconds was again maintained. To obtain the desired filter rate, valve VI was manipulated by either closing or opening until the right filter rate is obtained. Valve V2 was always fully opened and was never used to adjust the flow rates. To convert the flow rates in $cm^3/15$ seconds to m/hr, calculations were done as illustrated by the following using a flow rate of 200 $cm^3/15$ seconds as an example.

Diameter of the filter plant = 0.4 m

cross-sectional area Ac = $\frac{\Pi D^2}{4}$

$$= \frac{\Pi \times 0.4^2}{4} \quad m^2$$

Volume of water per hour = $(200 \times 4 \times 60) \times 10^{-6} m^3$

$$\text{Flow rate per hour} = \left[\frac{(200 \times 4 \times 60) \times 10^{-6}}{\Pi \times \frac{0.4^2}{4}} \right] \text{m/hr}$$

$$= \underline{\underline{0.38 \text{ m/hr}}}$$

When the correct flow rate was obtained, valve V4 was closed and valve V5 was opened so that the filtrate could be drained away by the use of the hose pipe.

During sampling, the effluent sample was first taken. Valve V5 was closed while valve V4 was opened. The sample was then taken using the collecting graduated cylinder,and a stop watch used for measuring time. In this way, both the flow rate and the filtrate sample couldbe obtained at the same time. Having obtained the filtrate sample, the raw water sample was taken by first closing valve V2 and opening valve V3. The raw water sample was then taken and valve V3 was immediately closed again and valve V2 fully opened.

If flow rate adjustment was required valve VI was manipulated until the right flow rate was obtained. The above procedures were followed every time sampling was to be done.

When placing the microscopic slides, the covers of the windows were removed and the slides inserted inside. The covers were then replaced and and bolted firmly so that there was no serious leakage of water. Immediately after starting a filter run, there was an appreciable leakage observed. But as the filtration process continued, leakages stopped due to fine sand filling the openings. Slides were always removed after draining off the filter completely. The draining of the filter was done overnight so that in the following day, the slides were removed. The slides were then placed inside glass bottles containing moist cotton wool and taken to the University laboratory for investigation. The raw water and filtrate samples were also carried to the University laboratory in plastic bottles for analysis.

For backwashing the filter, upward flushing was only applied. After the completion of a filter run, the flow rate was increased and the wash water drained to waste. After fifteen to twenty minutes, backwashing was stopped and the flow rate adjusted, for the next filter run. No samples of backwash water was taken for analysis. No headloss measurements were made during this experiment.

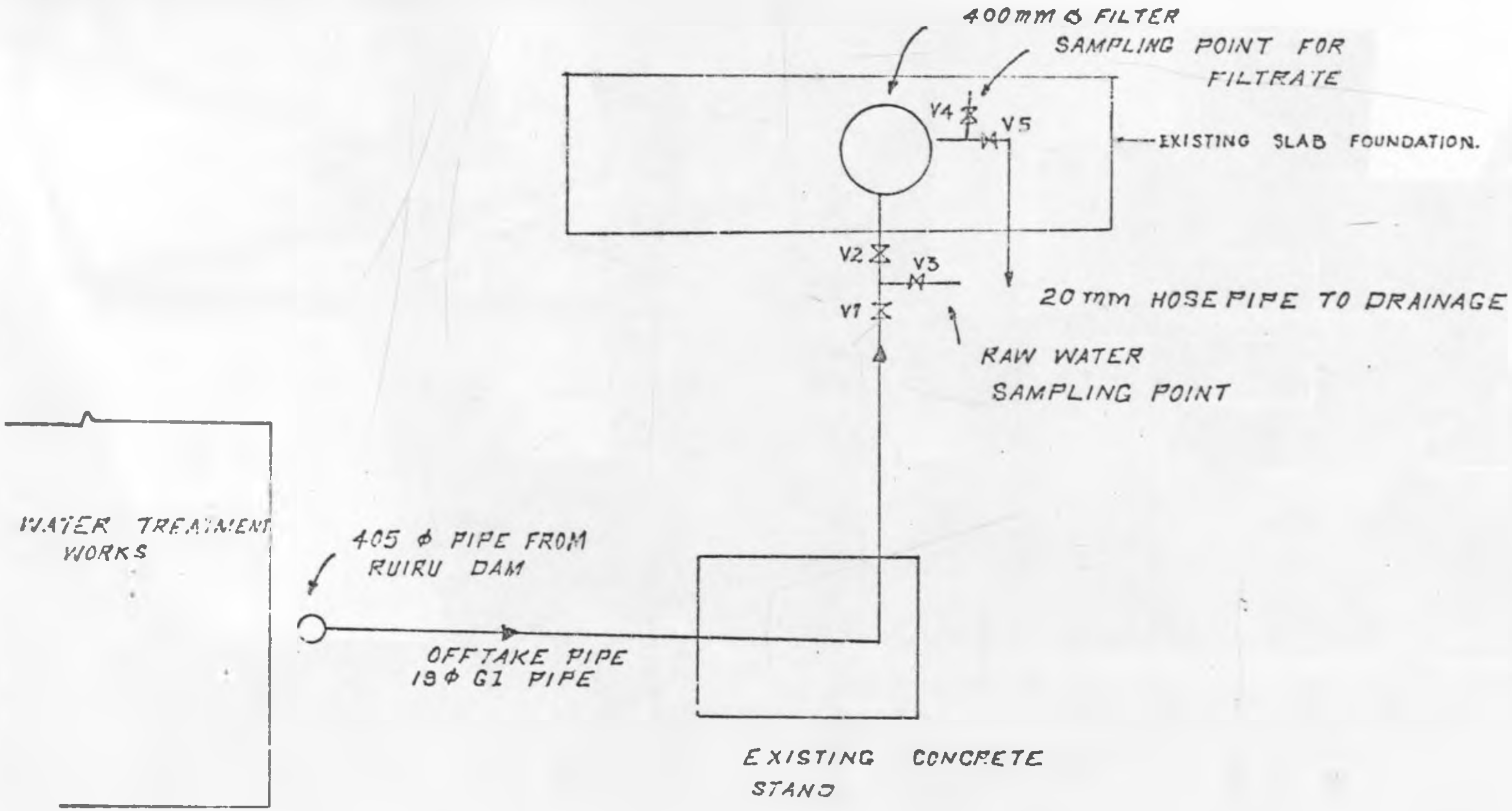


FIG. 7A LAYOUT OF THE PILOT PLANT

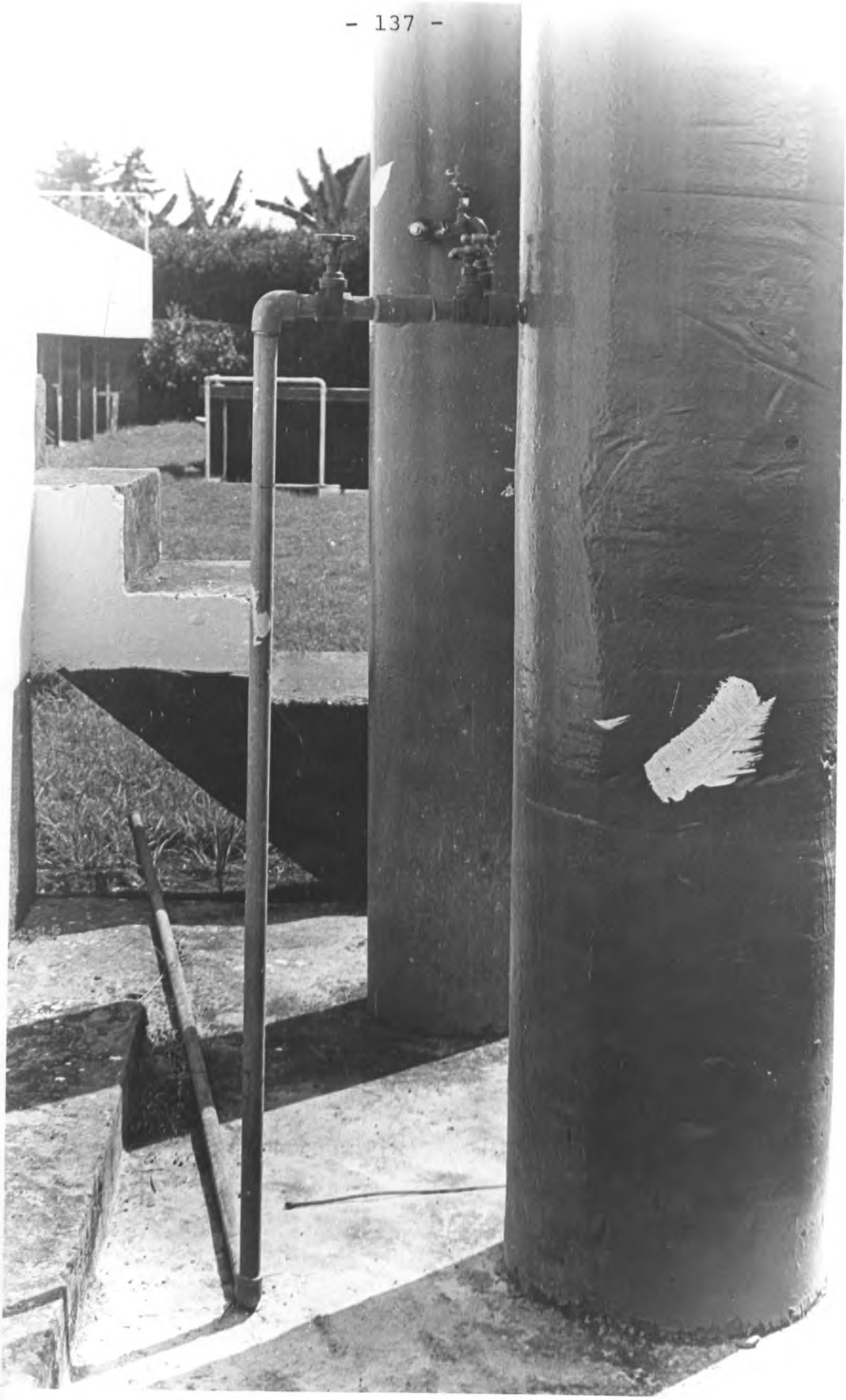


Plate 7B Detail of raw water off-take

Plate 7C Filter in position

Plate 7D Details of cut-out windows

7.5 Test Procedures

All analyses were done in the Environmental Health Engineering laboratory by transporting the samples. The analyses carried out were on pH, Colour, Turbidity, Iron Content and the Microscopic investigations. The procedures for carrying out pH, colour and turbidity analysis have been described in chapter 5.6.4. Iron content tests were carried out as follows: 5 ml of the solution under test was placed into a stopper funnel acidified with 1 ml of dilute hydrochloric acid, two drops of potassium permanganate solution was added and the content mixed. 5 ml of ammonium thiocynate solution and 10 ml of equal volumes of amyl acetate and amyl alcoholic solvent were added and the contents mixed thoroughly by shaking vigorously. The lower (aqueous) layer was decanted and the upper layer transferred into a 10 ml comparator test-tube and placed in the right hand compartment of the Lovibond comparator. A blank test was prepared using distilled water and placed in the left hand side of the Lovibond comparator. The colours were then matched and the iron content read on the disc.

The microscopic investigations were carried out by trying to identify the deposit on the slides through a microscope.

7.6 Results of Tests

The pilot plant was set up in early February and the filter commissioned on 18/2/80. The first samples were taken on 19/2/80. The experiment was eventually stopped on 25/4/80. In between this period, four filter runs were made. Filter run I had a duration of 17 days, filter run 2 duration of 12 days, filter run 3 a duration of 10 days and finally filter run 4 had a duration of 12 days. The results of the tests carried out are tabulated in Table 7.1 - 7.4.

7.6.1 Turbidity and Colour Removals

For the four filter runs, the rate of flow ranged from 0.36 to 0.44 M/hr. The turbidities for raw water were generally low within the range of 0.8 FTU to 13 FTU. It was suspected that the high values of turbidities were due to the rusted pipes conveying the raw water to the filter. The influent turbidities were always higher than the effluent turbidities. There was however one exception during the last sampling of run I. But this may have been due to the fact that the valves of the inlet pipe were manipulated to obtain the desired flow rate before taking samples. The graphical illustrations for the turbidity variations and removals are illustrated in figures 7.1 to 7.4.1. No graphical illustrations for the colour variations and removals were made.

7.6.2 Iron and pH Variations

There was no major significant variation of pH during the filter runs. The differences also between the raw water and filtrate pH values are insignificant. The same can be said of the iron content for raw and filtered water. No graphical illustrations were made of these parameters. The results are tabulated in tables 7.1 to 7.4.

7.6.3 Microscopic Examination

In principle, it is generally found that during slow sand filtration process, a thin film of slimy layer known as schmutzdecke is formed on the top surface of the filter sand. To be found also are the presence of macro as well as microorganisms in the filter.

Due to the above observations about a slow sand filter, it was felt that a microscopic investigation should be undertaken to determine the presence or absence of these organisms in an upward flow filter having a fine filter sand media usually meant for slow sand filters and using low flow rates in the range 0.1 - 1.0 m/hr. Two windows were made on the filter wall for insertion of microscopic slides into the filter.

During the investigation, slides were inserted inside the filter during filter run No. 2 and No. 3. Four slides were inserted inside the lower window during Run 2. No slides were inserted inside the upper window.

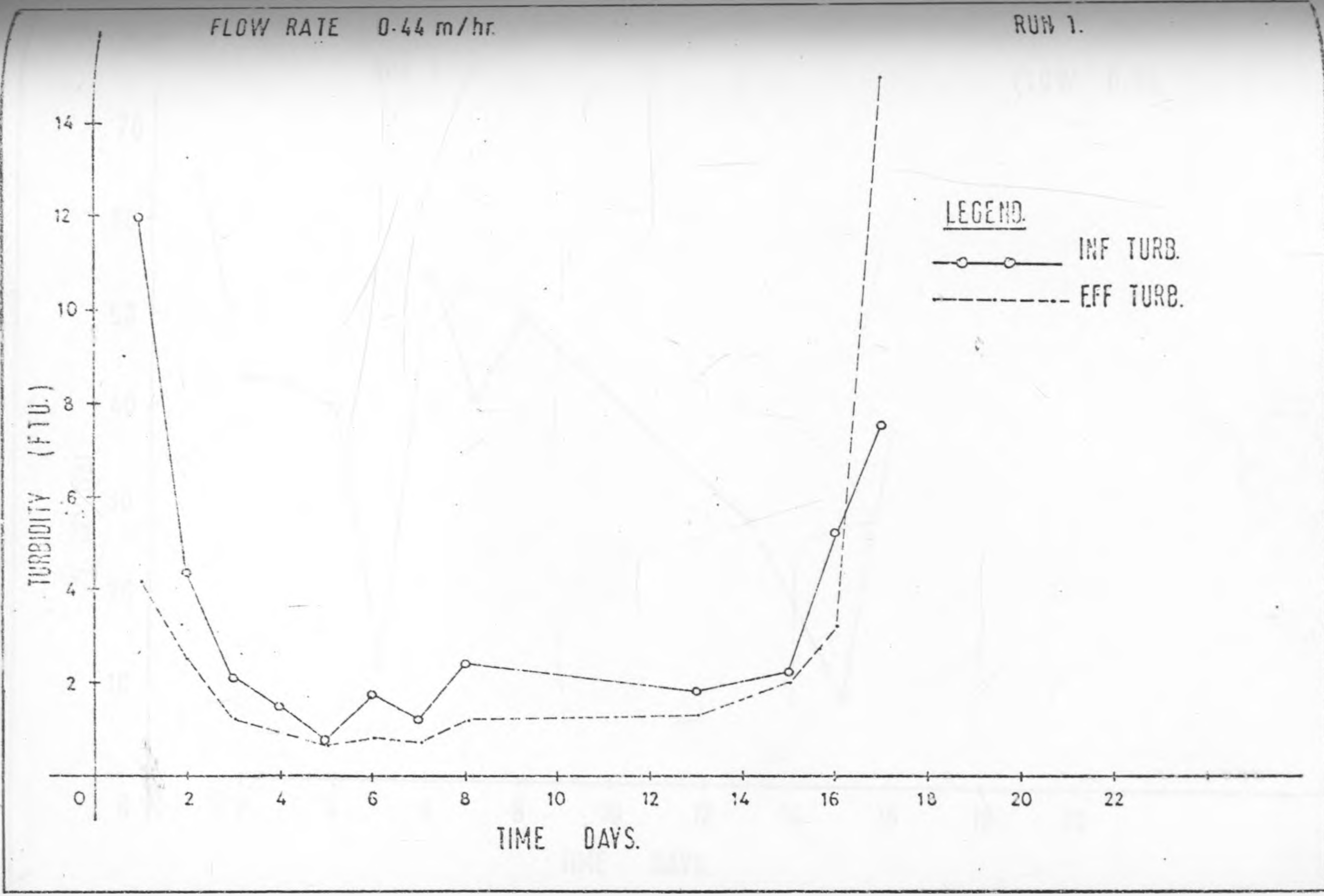
After a few days, it was noticed that algae had grown inside the filter around the lower window. This was thought to have been due to the sun shining directly on the window during some times of the day. The window was then covered with a black paper and the experiment carried on.

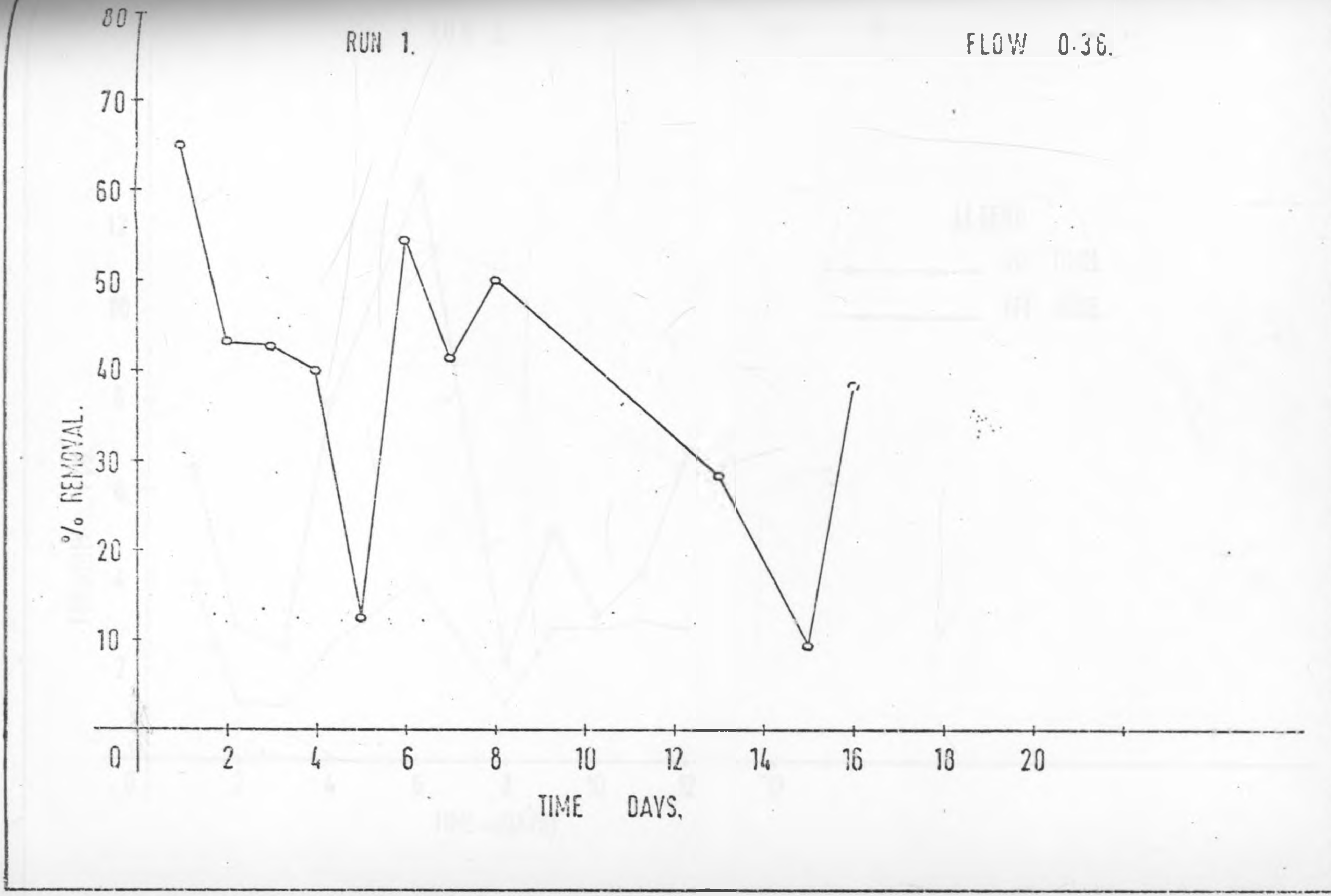
When the slides were removed, it was found that they were heavily covered with fine sand particles. A microscopic examination of the slides revealed particles of sand and film of water with some traces of green algae. No distinct form of life was observed.

Three slides were inserted inside the lower window and further three slides were inserted inside the upper window for Run No.3. The widows were covered to avoid direct sunshine. When the slides were removed after the filter run, they were once again found to be heavily covered with fine particles of sand and when the slides were placed on a microscope and examined, it was observed that the slides, were covered with sand particles and a film of water Bacteriological examinations of the sand sample on the slides was, however, not carried out.

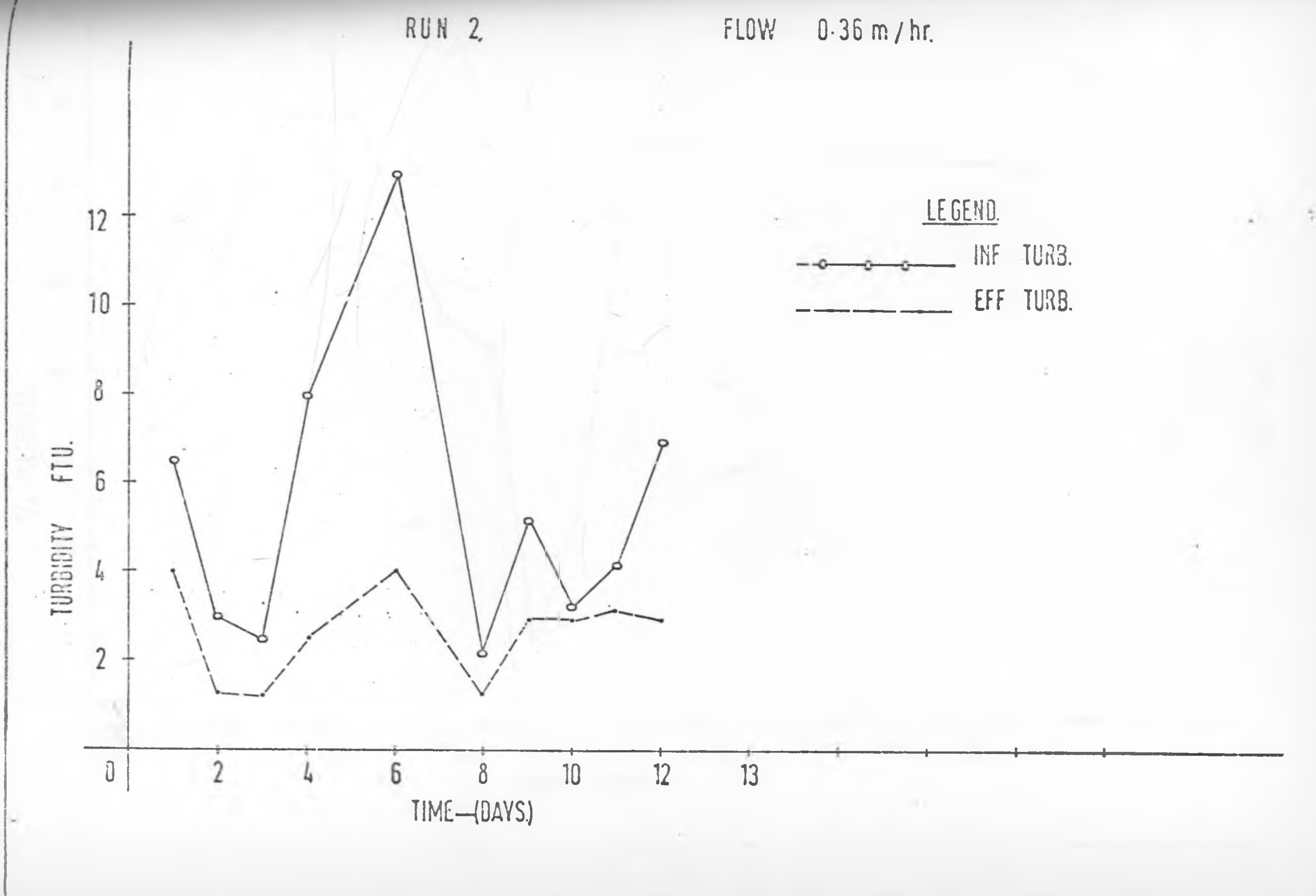
When the microscopic investigation could not yield useful information, the placing of slides inside the filter was stopped for the fourth filter run and the other parameters were monitored.

7.7 GRAPHS OF TURBIDITY VARIATIONS

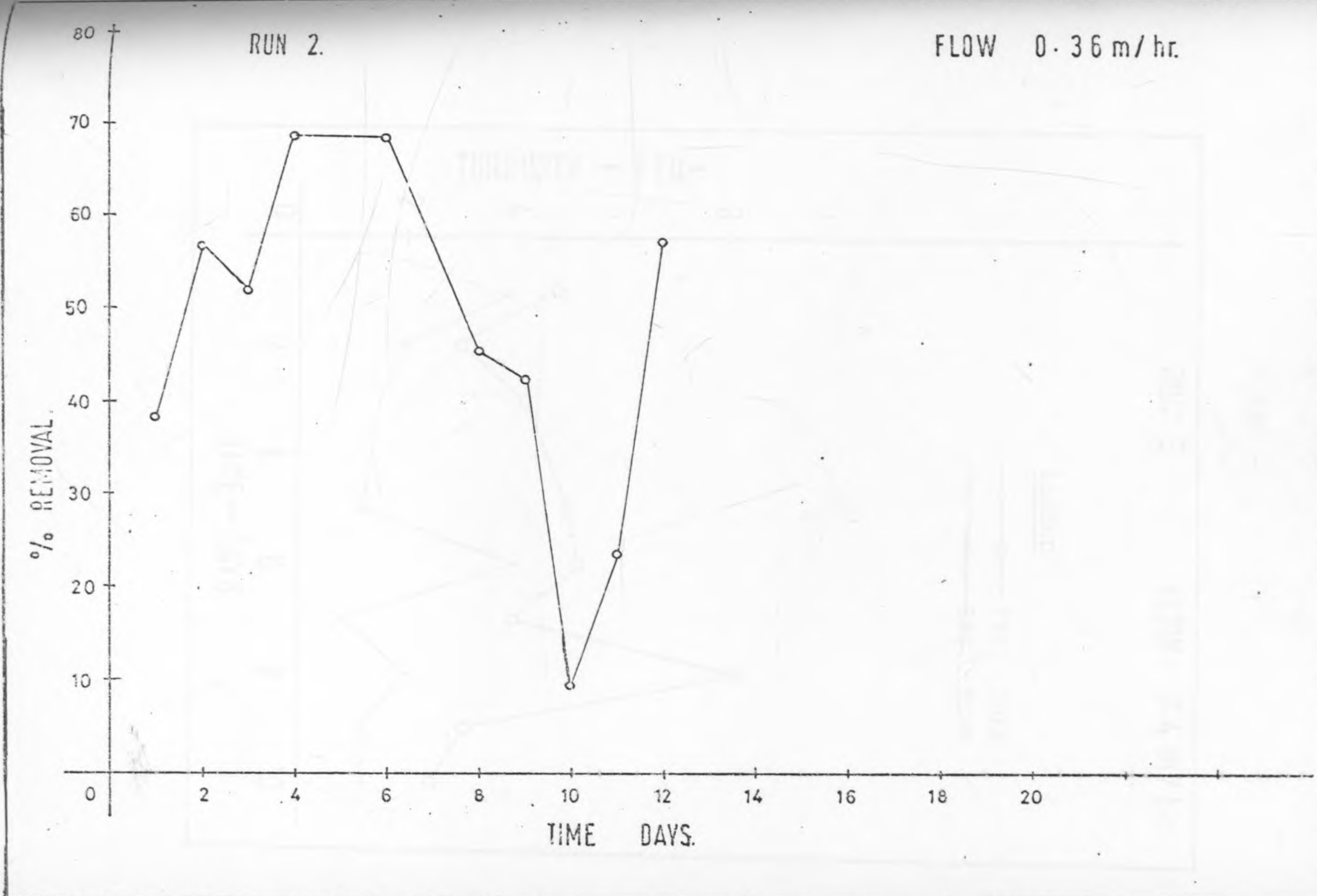

FLOW RATE 0·44 m/hr.
RUN 1.
LEGEND.
INF TURB.
EFF TURB.
TURBIDITY (FTU.)
14
12
10
8
6
4
2
0
2
4
6
8
10
12
14
16
18
20
22
TIME DAYS.

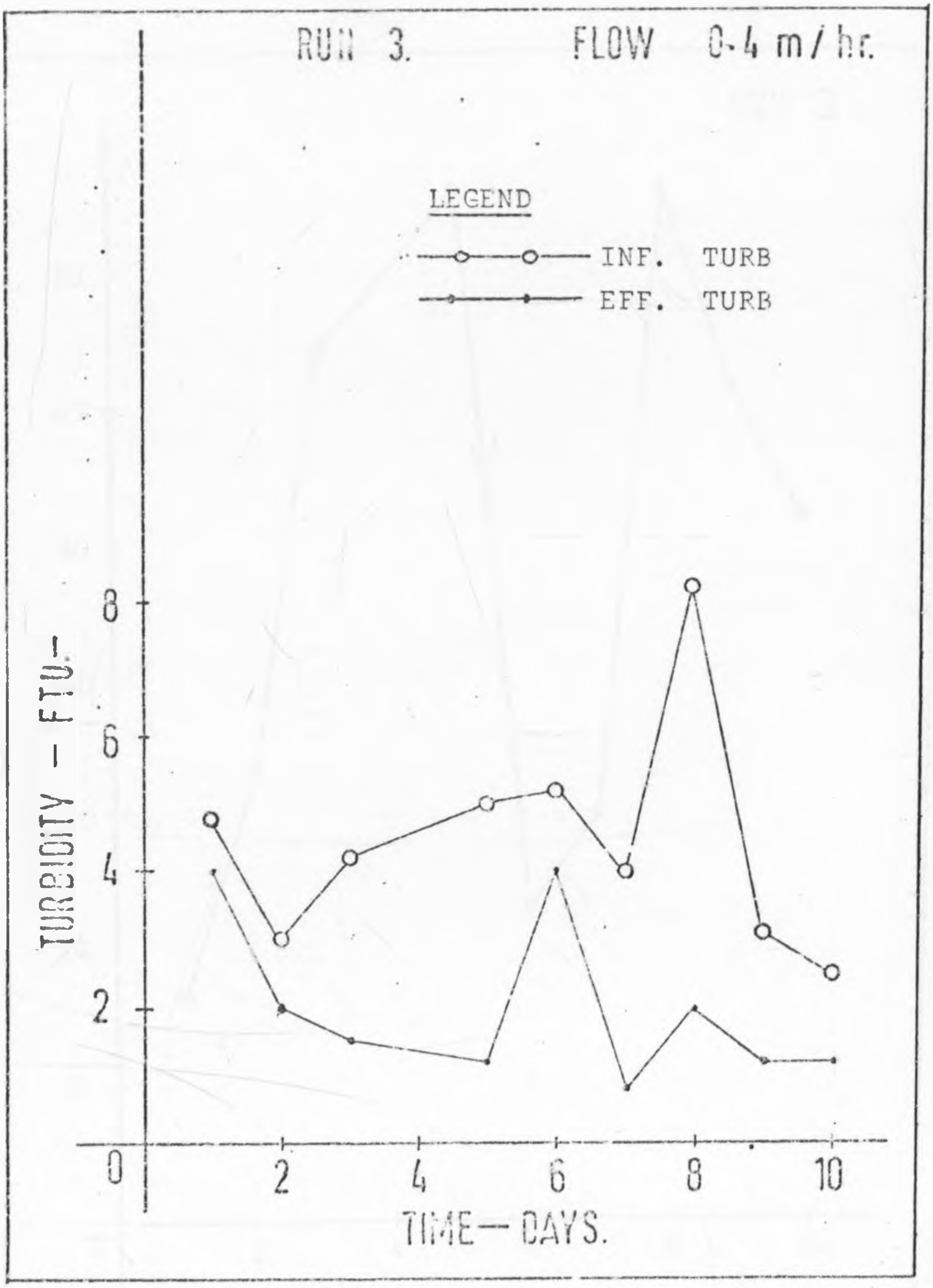


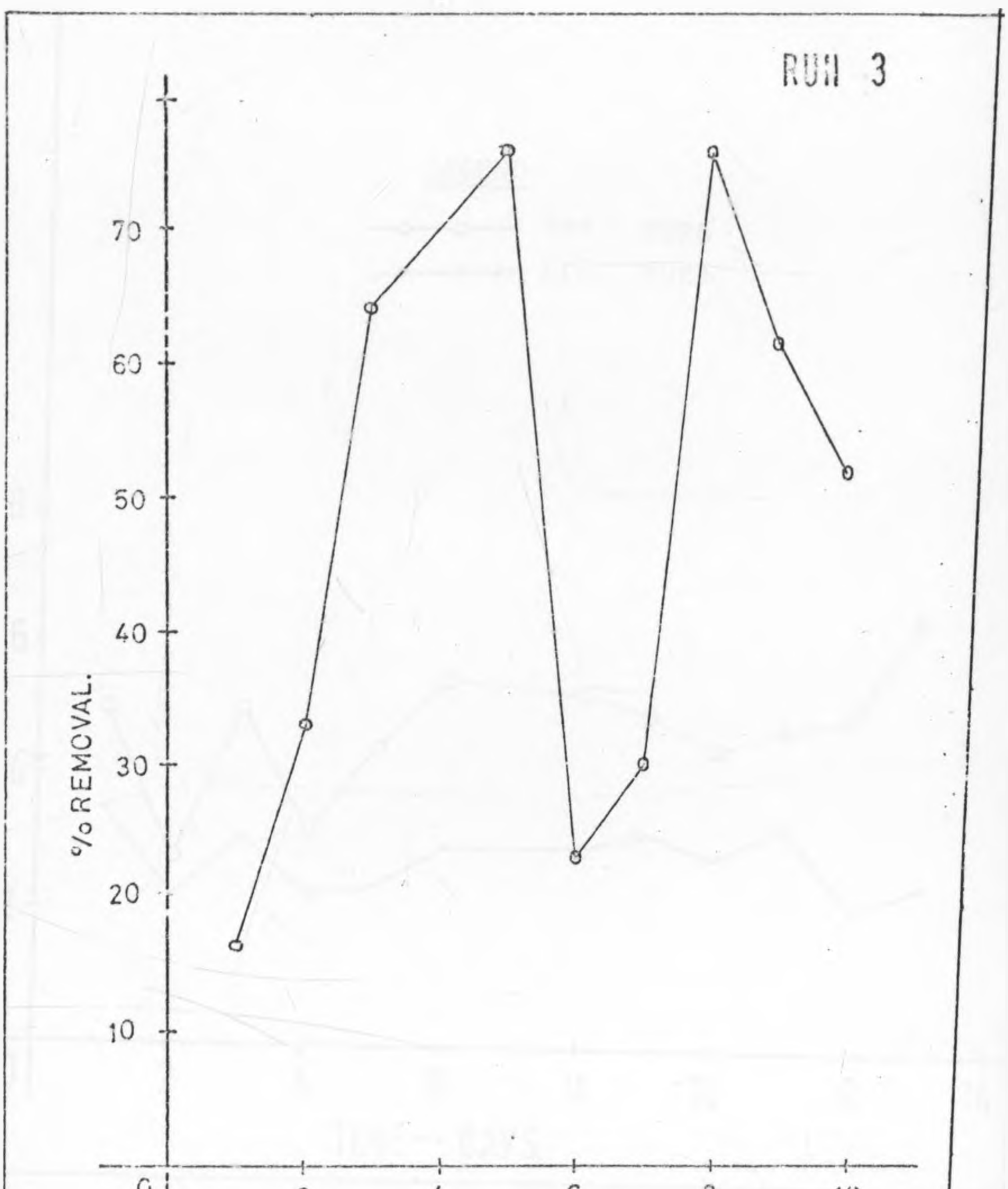
RUN 1.
FLOW 0·36.
% REMOVAL.
TIME DAYS.
0
2
4
6
8
10
12
14
16
18
20
10
20
30
40
50
60
70
80

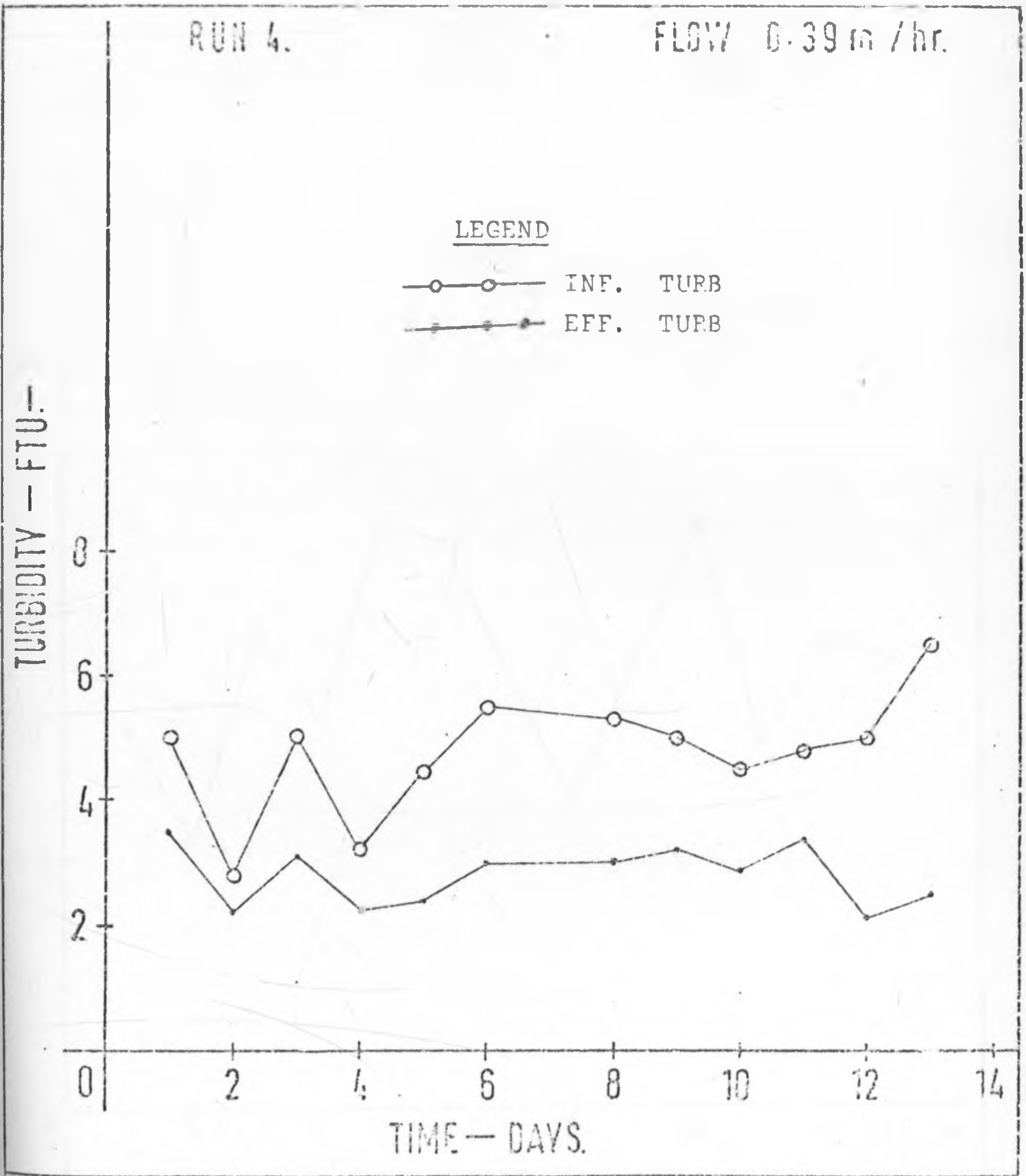


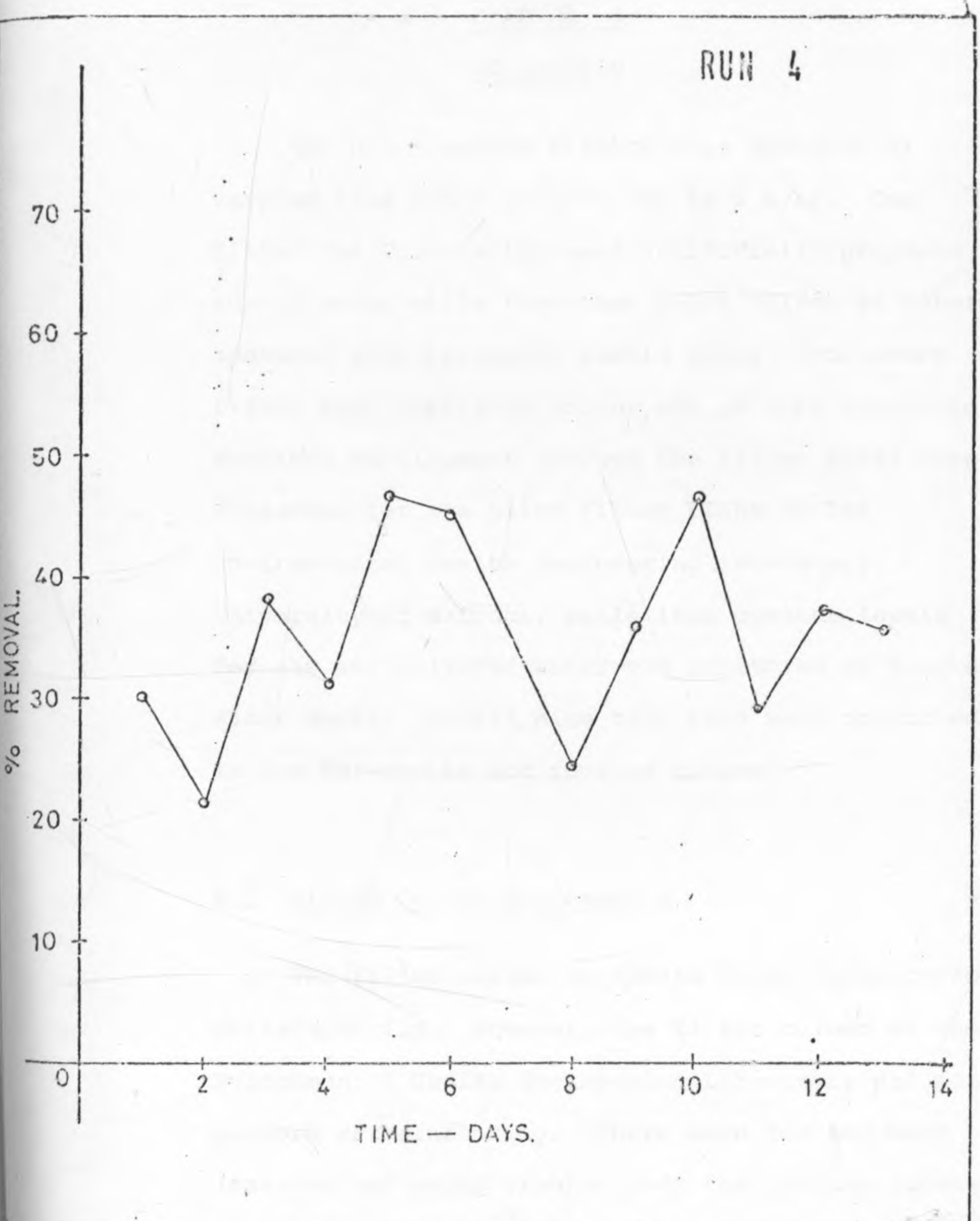
RUN 2.
FLOW 0·36 m / hr.
LEGEND.
INF TURB.
EFF TURB.
TURBIDITY FTU.
12
10
8
6
4
2
0
2
4
6
8
10
12
13
TIME—(DAYS.)


RUN 2.
FLOW 0·36 m/hr.
% REMOVAL.
TIME DAYS.
0
10
20
30
40
50
60
70
80
2
4
6
8
10
12
14
16
18
20


RUN 3. FLOW 0.4 m/hr.
LEGEND
INF. TURB
EFF. TURB
TURBIDITY - FTU-
8
6
4
2
0
2
4
6
8
10
TIME — DAYS.


RUN 3
% REMOVAL.
70
60
50
40
30
20
10
0
2
4
6
8
10
TIME — DAYS.


RUN 4.
FLOW 0·39 m /hr.
LEGEND
INF. TURB
EFF. TURB
TURBIDITY — FTU.
0
2
4
6
8
10
12
14
TIME — DAYS.


RUN 4
% REMOVAL.
70
60
50
40
30
20
10
0
2
4
6
8
10
12
14
TIME — DAYS.

CHAPTER 8

DISCUSSION

The pilot upflow filters were operated at varying flow rates of 0.2 m/hr to 1 m/hr. One filter (at University) used artificially prepared turbid water while the other pilot filter at Kabete operated with naturally turbid water. For every filter run, turbidity, colour and pH were monitored. Headloss development through the filter media were monitored for the pilot filter plant at the Environmental Health Engineering laboratory, University of Nairobi, while iron content levels for raw and filtered waterwere monitored at Kabete Water Works. In all, nine test runs were conducted in the University and four at Kabete.

8.1 Filter Column Performance

The filter column at Kabete Water Works performed satisfactorily. However, the filter column at the Evironmental Health Engineering Laboratory did not perform satisfactorily. There were far too many leakages and major cracks along the perspex joints. Each time a crack appeared it had to be sealed, so that the filtration operation had to be stopped. This resulted in some short filter runs. One major crack occured during the backwashing process after Run I. This necessitated the use of the whole perspex column for filtration and an outlet for the filtrate was made on the perspex which was connected to the drain pipe. A crack also developed during Run 2 after ten hours of filter operation. The filtration was stopped and the crack sealed off. A third major crack developed during Run 5 after 7 hours of filter operation. The filter run was stopped and the crack sealed off. The same flow rate was resumed after the sealing of the crack. Other minor cracks and leakages also occured, some of which are tabulated in table 5. The major cause of the perspex failure along the joints could be attributed to the use of chloroform. The use of chloroform in joining the perspex edges seemed not to have been strong enough to withstand the pressure exerted by the pump and the weight of the filter media. Araldite was used for sealing off the cracks and proved to be more effective than chloroform.

No attempt was made to roughen the filter walls. As a result, short circuiting became a potential problem. Because the pilot plant at the University had a rectangular cross-sectional shape, the effect of short circuiting was more pronounced than the performance of the pilot filter at Kabete water works which was circular in shape.

8.2 Filter Media.

The sand of Effective size 0.22 mm and the uniformity coefficient of 2.46 was found to be too fine and tended to be clogged easily with water of high turbidity of the range used in the laboratory. The clogging was found to be within the lower depth of the filter. This resulted in short filter runs. It was also found that when cleaning the filter media at higher flow rates, the fine sand media could easily be washed away. Thus for effective cleaning of the filter, upward backwashing at slightly higher flow rates than during filtration process, were to be applied for long periods.

For example after run 3, it was found that the filter was heavily clogged. The filter was then flushed downwards using the water above the sand surface. Then water was pumped at higher rate through the filter media and the dirty water drained off through the effluent pipe. After 15 minutes, the pump was stopped and downward flushing was done. When all the water had drained off, raw water was pumped upwards. The process was repeated three times. This took over an hour which is too long compared to a period of 10 to 20 minutes spent in backwashing upward flow filters generally. To wash properly the gravel layer, water was pumped at maximum pump speed into the filter and let to rise upto the level of gravel/sand layer. Then it was drained off while the pump is still on. This system had the effect of scouring vigorously the fine sand and muddy clogging in the gravel media and the filter bottom and then washing and draining away this dirt. The same process was applied after Run 8 when the lower depth of the filter media was found to be clogged.

Due to the relatively low turbidity of raw water at Kabete water works, clogging of the filter bed was not considered a potential problem, hence no clogging effects were investigated.

8.3 Turbidity Removal

Of all the parameters monitored during this investigation, turbidity removal and headloss development in the filter need special attention. A relatively high turbidity of upto 160 FTU was artificially prepared in the laboratory and the removal efficiency observed with filter age for various filtration rates. It was found that the method of stirring and releasing the turbid water was not satisfactory and resulted in exaggerated turbidity values.

On occassions when the turbid water tank was stirred, it was noticed that there was an increase in influent turbidity into the upflow filter which eventually resulted in an increase in the effluent turbidity. Because of the relatively low flow rates used and short periods between sampling times, the effect of the high turbidity into the influent water was two fold. First, immediately after the stirring operation, when the samples were taken, the removal percentages were noticed to be extremely high in the region of 80 FTU - 90 FTU.

As the filter run progressed and the soil suspension settled in the turbid water tank, the effluent turbidity tended to be higher than influent turbidity resulting in negative percentage removals. This eventually tended to lower the overall average percentage removals for each filter run.

Thus for some filter runs, the negative values of removals can therefore be misleading unless these are considered on a wider perspective.

For graphical representations negative values of turbidity removals were not plotted. No negative values of percentage removal of turbidity were recorded during the performance of the second filter at Kabete water works. This could be attributed to the more stable influent turbidity conditions and the long sampling periods.

When negative values of percentage removals are ignored, the average removal efficiencies for all the flow rates are raised to a minimum of over 30 FTU. The maximum removal efficiency recorded was 67% for flow rate 0.41 m/hr. The removal efficiency recorded for the second filter at Kabete was in the range 40 - 60 FTU.

In other parts of the world, various researchers have achieved turbidity removals of around 50% when no pretreatment is provided. Fox and Priest (11) obtained an average of 50% turbidity reduction without coagulant dosing. He used a relatively coarser sand than the sand used by this author. It was hoped that with finer sand, the turbidity reduction would be increased. Generally, this was not the case and it was apparent that turbidity reduction was low.

For most of the filter runs, there was a general trend of decreasing turbidity removals with filter age.

8.4 Headloss Development

Most test runs showed gradual and slow headlosses with time. These agreed with the observations of Ives (18) et al about the headloss development of upflow filters with time. But a few observations need to be made concerning headloss development with respect to increasing flow rates and the effect of clogging of the lower filter media.

It was generally noted that the headloss through the gravel media was small while it was increasing with the depth of the sand media.

However, there were some recorded cases when the headloss through gravel media was high and even greater than for sand media. This was apparent for Runs 3,7, and 8. Clogging of the gravel layer due to inadequte filter cleaning could have contributed to this. Mud accumulated within the lower depth of gravel and sand. After thorough cleaning of the filter bed, low headloss through the gravel media and a gradually increasing headloss along the sand media were recorded. This effect could be seen by comparing the headloss patterns of filter runs 8 and 9. The headloss pattern of run 8 showed high headloss through the the gravel layer of upto 19 cm. No clear pattern of headloss with time was observed. Whereas the headloss patterns of run 9 are more noticeable with small headloss accross the gravel layer and higher values for sand media, a gradually increasing headloss with time is observed.

An unsual headloss pattern is observed for filter run 6. The flow rate was 1 m/hr. The initial headloss across the gravel layer was substantiallyhigh. However, a decreasing headloss across the gravel with time was observed while for the sand media, there was a gradual increase.

This could be explained as follows:
At the start of the filter run, the gravel layer was clogged with mud. The effect of filtration washed the mud deeper into the filter media and possibly retained within the sand media. Meanwhile as more mud is removed from the gravel layer the headloss decreased. It is possible to assume that over a long period of time if filtration was to be carried on, the headloss across the gravel would

eventually increase as the sand layer got clogged up. The relatively high flow rate of 1 m/hr made it possible to wash up the mud within the gravel layer and be retained by the sand media.

This is generally in conformity with the known theory of filtration that with increasing filtration rate , greater depth penetration of particulate matter is achieved through the filter depth.

CHAPTER 9

CONCLUSIONS

A brief summary of the literature review on filtration and filters in particular has been made. Generally it has been observed from past experiences that there are many operational problems with the conventional rapid and slow sand filters. This has lead to the introduction of upward flow filtration, amongst other innovations. It has also been observed for many years that upflow filters have many advantages over the downflow filters. An upflow filter can retain a great deal of suspended matter within its filter media which can easily be washed out during backwashing. Successful applications of upward flow filtration for portable water production, treatment of sewage and industrial effluents have been done in many parts of the world. Turbidity has also been identified as one of the causes of filter failures, particularly for slow sand filters in Kenya. There is therefore a need for research into effective turbidity removal using cheap and less sophisticated methods. Upward flow filters have been known to offer many operational advantages over the conventional downflow filters in terms of greater turbidity reductions, higher filtration rates , more compact plant achievement and lower-maintenance costs.

The conclussions below are based on the results obtained from the performance of two upward flow pilot filters investigated at the University and Kabete water works respectively.

(1) A filter sand media having an effective size of 0.22 mm and a coefficient of uniformity of 2.46 was observed to be too fine for use in relatively high turbid water. Consequently, clogging of the filter was observed to be a potential problem.

(2) Because of the size of the filter media, proper backwashing of the bed was not possible. At relatively low flow rates of just above 1 m/hr, fluidization of the filter bed was observed and the fine filter sand was easily washed to drainage. Downward flushing was not effective for filter cleaning.

(3) Turbidity reduction was in the order of 30% - 60% for the filter plant at the University while a more consistant turbidity reduction of 40% - 50% was observed for the filter at Kabete waterworks.

(4) With proper filter backwashing, a gradually increasing headloss development was observed over different depths of filter media.

(5) Hourly sampling periodsof the raw and filtered waterwere found to be too short for steady filter conditions to be fully achieved.

(6) The method of stirring the turbid waterin the tank to keep the soil particles in suspension was not satisfactory. Also the method of injecting turbidity into the raw water system was unsatisfactory because the pipe tended to be clogged so that constant checking on the valve was done to prevent rusting.

(7) The perspex filter box performed poorly in general. The walls were too smooth and short circuiting was a potential problem. The frequent failure of the perspex joints and numerous water leakages contributed to many short filter runs.

(8) The operation of the pilot filter at Kabete has demonstrated that an upward flow filter can easily be operated to cater for filtration of water for a community by using gravitational head. No power requirement is needed to pump raw water through the filter bed if an intake source is located on higher ground level and the treatment plant is located at down stream.

CHAPTER 10

RECOMMENDATIONS

The following recommendations are made in respect to the investigations carried out.

(1) The conclusions outlined in chapter 9 are based on short term investigation. The investigation was done within a period of less than four months. It is therefore fair to say that to arrive at firmer conclusions long term period of investigation is necessary.

(2) A more reliable form of continuous stirring system of the turbid water should be devised in future research. This would ensure a more systematic form of creating raw water turbidity.

(3) In future research work, perspex walls should be roughened to control the effect of short circuiting if a filter of perspex material is used.

(4) More research should be made to determine the best methods of cleaning the gravel and lower sand media of upward flow filters. It was observed that without proper cleaning, the lower filter media tended to clog up easily, hence reducing the filter efficiency.

(5) Further research into the performance of a coarser sand media into high water turbidity reduction should be done. A combination of different sand and gravel sizes should be looked into.

(6) The use of upflow filters in series with slow sand filters should be investigated in the field. The attractiveness of such a system lies is the fact that sand would be the main media to be used. The use of chemicals would not be undertaken and it is possible that the whole system would prove to be cheap such that frequent failures of slow sand filters due to high turbidity might be remove .

(7) During the cause of this investigation, no conclusive observation could be made on the microscopic examination of the presence or absence of Plankton life in the filter. It is therefore recommended that more research under more controlled conditions should be made on this particular aspect.

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APPENDIX

READINGS AND ANALYSIS OF DATA

PILOT PLANT: UNIVERSITY OF NAIROBI RUN 1

Flow Rate: ·9.6 m/hr.

Date 10/2/80

Time: 9.00 a.m.

Time Interval (Hrs)	TURBIDITY (FTU)		COLOUR °H		PH		Flow Rate cm^3/15 secs.	% Removal Turbidity	% Removal Colour
	INF	EFF	INF	EFF	INF	EFF			
½	38	12	160	80	7.6	7.9	250	68.4	50
1	12	12	70	75	8.3	8.4	250	0	-7.1
2	27	20	150	100	7.8	7.4	250	25.9	33.3
4	30	22	150	120	7.5	7.4	245	26.7	20
5	10	8	70	50	7.4	7.4	250	20	28.6
7	43	20	250	200	7.7	7.6	250	53.5	20.0
8	40	28	300	250	7.6	7.7	250	30	16.7
9	33.5	30	200	150	7.8	7.7	250	10.4	25
A RAGE	29.2	19	168.8	128.1	7.7	7.7	249.4	29.4	23.3

Date 10/3/80 Flow Rate RUN 2 PILOT PLANT: UNIVERSITY OF NAIROBI
Time 10.00 a.m. 0.52 m/hr

Time Interval HRS	Turbidity FTU INF	Turbidity FTU EFF	Colour °H INF	Colour °H EFF	PH INF	PH EFF	Flow Rate cm^3/15secs	Manometer Readings A	B	C	D
1	58	24	100	50	7.3	7.3	185	223	220	232	197
2	150	25	400	50	7.8	7.7	185	224	222.5	233	197.5
5	150	100	400	350	7.4	7.4	195	226	224	236	197
6	140	100	500	250	7.8	7.8	190	227	224	236	197
6.5	145	110	500	250	7.8	7.8	195	227	224	236	190
7	160	125	500	200	7.8	7.8	200	228	225	237	198.5
8.5	120	100	300	150	7.4	7.4	200	226	223	235.5	198
9.5	40	80	200	150	7.8	7.8	190	225	222	234	198
10	68	64	200	150	7.7	7.7	170	222	219	233	196
AVERAGE	114.6	80.9	344.4	177.8	7.6	7.6	190				

ANALYSIS RUN 2

Time Interval	% Removal Turbidity	% Removal Colour	Headloss Overstated filter Lengths		
			A-B	B-C	C-D
1	58.6	50	3	12	35
2	83.3	87.5	1.5	10.5	35.5
5	33.3	12.5	2	11	35.5
6	28.6	50	3	12	39
6.5	24.1	50	3	12	38
7	21.9	60	3	12	38.5
8.5	16.7	50	3	12.5	37.5
9.5	-100	25	3	12	36
10	5.9	25	3	14	37
AVERAGE	19.2	45.6	2.7	12	36.9

Date 21/3/80 Flow Rate 0.36 Time 10.00 a.m. RUN 3

Time Interval	Turbidity FTU		Colour °H		PH		Flow Rate	Manometer Readings			
HRS	INF	EFF	INF	EFF	INF	EFF	cm^3/15secs	A	B	C	D
1	170	43	500	150	7.4	7.4	120	221	234	229	199
2	170	110	500	400	7.6	7.6	130	222	239.5	230	194
3	150	170	500	500	7.4	7.4	125	226	237	233.5	196
5	170	110	750	350	7.4	7.4	140	226	240	234	196
5.5	110	100	750	250	7.3	7.3	130	227	240	235	197
6	180	100	750	250	7.3	7.3	130	227	240	235	197
8	140	160	500	400	7.8	7.8	130	227	239	235	197
AVERAGE	155.7	113.3	607.1	328.6	7.5	7.5	129.3				

ANALYSIS RUN 3

TIME Interval (HRS)	% Removal Turbidity	% Removal Colour	Headloss through stated Filter lengths		
			A-B	B-C	C-D
1	74.7	70	13	5	35
2	35.3	20	12.5	4.5	36
3	-13.3	0	11	3.5	37.5
5	35.3	53.3	14	6	38
5.5	9.1	66.7	13	5	37
6	44.4	66.7	13	5	38
8	-14.3	20	12	4	38
Average	24.5	42.4	12.6	4.7	37.1

PILOT PLANT: UNIVERSITY OF NAIROBI — Flow rate — RUN 4

DATE 23/3/80 — Time 3.30 p.m. — 0.6 m/hr

Time Interval (HRS)	Turbidity INF	Turbidity EFF	Colour (^oH) INF	Colour (^oH) EFF	PH INF	PH EFF	Flow Rate cm^3/15secs	Manometer Readings CM A	B	C	D
½	120	19	200	100	7.4	7.4	225	234	232	240	217
2	98	86	500	400	7.5	7.4	210	236	234	242	218
3	84	82	300	300	7.4	7.3	215	234	230	239	216
4	76	83	350	300	7.4	7.4	210	232	230	239	217
19.5	42	32	200	150	7.4	7.3	200	230	228	239	217
21	60	30	400	50	7.5	7.5	210	233	230	241	220
22	75	40	400	250	7.5	7.6	210	235	232	243	221
23	44	74	200	300	7.5	7.5	220	233	230	240	220
24	70	50	300	200	7.4	7.4	220	232	230	240	220
27	60	35	200	150	7.6	7.6	225	232	230	241	220
AVERAGE	72.9	53.1	305	220	7.5	7.4	214.5				

ANALYSIS RUN 4.

Time Interval (HRS.)	% Removal Turbidity	% Removal Colour	Headloss over stated filter lengths		
			A-B	B-C	C-D
½	84.2	50	2	8	23
2	12.2	20	2	8	24
3	2.4	0	4	9	23
4	-9.2	14.3	2	9	22
19.5	23.8	25	2	11	21
21	50	87.5	3	11	21
22	46.7	37.5	3	11	22
23	-68.2	-50	3	11	20
24	28.6	33.3	2	10	20
27	41.7	25	2	11	21
AVERAGE	21.2	24.3	2.5	9.9	21.7

RUN 5

Flow Rate 0.41 m/hr. Time 9.00 a.m. DATE: 27.3.80

Time Interval	Turbidity FTU		Colour ^oH		PH		Flow Rate	Manometer Readings			
Hrs.	INF	EFF	INF	EFF	INF	EFF	cm^3/15secs	A	B	C	D
1	65	10	200	50	7.5	7.5	150	198	199	196	194
2	50	25	200	100	7.5	7.5	150	212	210	198	195
3	38	28	100	50	7.5	7.5	140	212	210	197	194
6	35	25	100	50	7.5	7.5	150	211	210	197	194
7	125	24	300	40	7.6	7.6	150	197	196	194	193
AVERAGE	62.6	22.4	180	58	7.5	7.5	148				

ANALYSIS RUN 5

Time Interval Hrs.	% Removal Turbidity	% Removal Colour	Headloss over stated filter lengths		
			A-B	B-C	C-D
1	54.6	75	1	3	2
2	50	50	2	12	3
3	26.3	50	2	13	3
6	28.6	50	1	2	1
7	80.8	86.6	1	13	3
	48.1	62.3	1.4	8.6	2.4

Date 4/4/80 Flow Rate 0.41 m/hr Time 4.30 p.m. RUN 5

Time Interval (hrs.)	Turbidity FTU INF	EFF	Colour °H INF	EFF	PH INF	EFF	Flow rate cm^3/15secs	Manometer Readings A	B	C	D
1	120	6	400	20	7.4	7.4	150	200	199	197	196
2	100	27	400	120	7.4	7.4	150	200	199	197	196
4	85	31	300	100	7.6	7.6	145	199	198	196	195
5	98	37	300	75	7.7	7.7	140	199	198	196	195
20	100	72	250	100	7.5	7.5	120	199	198	195	194
24	110	18	500	50	7.5	7.5	120	199	198	195	194
25	84	19	500	60	7.6	7.6	140	200	199	196	194
AV.	99.6	30	378.6	75	7.5	7.5	137.9				

ANALYSIS RUN 5

Time Interval (Hrs)	% Removal Turbidity	% Removal Colour	Headloss over stated filter lengths		
			A-B	B-C	C-D
1	95	95	1	2	1
2	73	70	1	2	1
4	63.5	66.7	1	2	1
5	62.2	75	1	2	1
20	28	60	1	3	1
24	83.6	90	1	2	2
25	77.4	88	1	3	2
Average	67	77.8	1	2.3	1.3

Flow Rate 1 m/hr
Time of starting 9.30 a.m.

RUN 6
Date 13/4/80

Time (Hrs)	Flow rate cm^3/15sec.	Turbidity FTU		Colour ^oH		PH		MANOMETER READINGS (cm)			
		INF	EFF	INF	EFF	INF	EFF	A	B	C	D
1	390	35	10	150	300	7.4	7.2	230	229	227	211
2	370	63	26	300	75	7.6	7.5	229	227	226	211
3	350	70	30	250	100	7.5	7.5	228	227	226	211
12	360	65	20	300	50	7.4	7.5	229	228	225	211
13	350	67.5	30	200	100	7.6	7.5	231	229	225	212
14	349	45	13	200	70	7.4	7.4	230	228	222	212
15	350	45	12	150	50	7.5	7.5	231	229	223	213
20	360	50	8	200	50	7.7	7.7	232	229	224	214
21	350	35	15	150	40	7.4	7.3	234	231	228	217
22	355	35	15	80	60	7.6	7.6	234.5	231.5	225	217
34	350	32	12	160	50	7.75	7.75	234	230	225	218
46	340	80	58	300	300	7.5	7.5	232	227	222	216
50	325	120	75	400	300	7.5	7.5	235	226	219	213
AV.	353.77	58.65	24.92	213.46	118.85	7.53	7.5				

ANALYSIS Flow 1 m/hr. RUN 6

TIME (HRS)	% REMOVAL TURB.	% REMOVAL COLOUR	HEADLOSS OVER STATED FILTER LENGTHS		
			A-B (cm)	B-C (cm)	C-D (cm)
1					
1	81.8	80	1	2	16
2	58.7	75	2	1	15
3	57.1	60	1	1	15
12	69.2	83.3	1	3	14
13	55.6	50	2	4	13
14	71.1	65	2	6	10
15	73.3	66.7	2	6	10
20	84.0	75	3	5	10
21	57.1	73.3	3	3	11
22	57.1	25	3	6.5	8
34	62.5	68.8	4	5	7
46	27.5	0	5	5	6
50	37.5	25	9	7	6
Average	60.19	57.5	2.92	4.19	10.85

Date 18.4.80
Time 9.00 a.m.

Flow rate
0.78 m/hr

PILOT PLANT
UNIVERSITY OF NAIROBI

RUN 7

Time Interval HRS	Turbidity FTU INF	EFF	Colour ^{o}H INF	EFF	PH INF	EFF	Flow rate cm^3/15secs	Manometer Readings cm A	B	C	D
1	45	22.5	200	100	7.5	7.5	285	224	239	218	196
2	50	30	250	300	7.5	7.6	280	225	237	217	196
6	43	26	300	200	7.5	7.5	280	225	239	217	196
8	45	35	300	300	7.5	7.5	285	225	239	217	196
9	47	33	300	240	7.5	7.5	285	226	240	218	197
10	25	23	150	100	7.5	7.6	280	226	240	218	197
11	27	20	100	75	7.5	7.5	285	226	239	218	197
Average	40.3	27.1	228.6	187.9	7.5	7.5	282.9				

ANALYSIS

Time Interval (Hrs)	% Removal Turbidity	% Removal Colour	Loss of Head through stated Filter lengths		
			A-B	B-C	C-D
1	50	50	15	21	22
2	40	-16.7	12	20	21
6	39.5	33.3	14	22	21
8	22.2	0	14	22	21
9	29.8	20	14	22	21
10	8	33.3	14	22	21
11	35	25	13	21	21
Average	32.1	20.7	13.7	21.4	21.1

Flow Rate 0. 27 m/hr Time 9.00 a.m. Date 24/4/80 RUN 8.

Time Interval (Hrs)	Turbidity FTU INF	Turbidity FTU EFF	Colour °H INF	Colour °H EFF	PH INF	PH EFF	Flow Rate cm^3/15secs	Manometer Readings (cm) A	B	C	D
1	9.5	6	40	30	7.6	7.6	100	194	213	191	177
2	63	7	200	50	7.5	7.6	100	194	212	191	177
3	34	8	250	20	7.3	7.4	100	194.5	213	191	176
7	62	46	350	300	7.5	7.6	110	194	213.5	191.5	176
8	66	55	400	250	7.5	7.4	100	193	212.5	191	176
9	64	55	500	400	7.5	7.6	100	194	213	191	176.5
24	22	46	150	300	7.4	7.4	90	193	211	190	176
25	20	38	150	200	7.3	7.4	100	194	213	191	176
26	36	24	300	200	7.3	7.4	100	194	212	191	177
27	42	23	450	400	7.8	7.7	100	197	215	194	177
55	55	27	400	300	7.3	7.4	120	198	216	194	179
56	48	35	400	300	7.3	7.4	120	198	216	193	178
78	16	10	70	50	7.45	7.4	100	199	217	196	183
79	11	10	50	30	7.35	7.35	100	199	216.5	196	183
96	18	9	70	40	7.3	7.4	85	194	212	191	180
97	11	9	50	30	7.25	7.35	90	194	212	191	180
99	9	6	40	30	7.4	7.4	90	195	213	192	181
100	64	6	300	20	7.25	7.4	100	196	213.5	192	181.5
101	72	6	300	30	7.3	7.3	110	196	214.5	193	182
102	61	9	300	30	7.4	7.5	110	195	212	191	182
103	64	11	300	50	7.3	7.3	110	198	215	193	183
Average	40.4	21.2	241.4	148.1	7.4	7.4	101.7				

ANALYSIS RUN 8

Flow rate 0.27 m/hr

Time Interval (Hrs)	% Removal Turbidity	% Removal Colour	Headloss through stated filter Lengths		
			A-B	B-C	C-D
1	36.8	25	19	24	14
2	88.9	75	18	21	14
3	76.5	92	18.5	21.5	15.5
7	25.8	14.3	19.5	22	15
8	16.7	37.5	19.5	21.5	15
9	14.1	20	19	22	14.5
24	-109.4	-100	18	21	14
25	-90	-33.3	19	22	15
26	33.3	33.3	18	21	14
27	45.2	11.1	18	21	17
55	50.9	25	18	22	15
56	27.1	25	18	23	15
78	37.5	28.6	18	21	13
79	9.1	40	17.5	20.5	13
96	50	42.9	18	21	11
97	18.2	40	18	21	11
99	33.3	25	18	21	11
100	90.6	93.3	17.5	21.5	10.5
101	91.7	90	18.5	21.5	11
102	85.2	83.3	17	21	9
103	82.8	83.3	17	22	10
Average	34.0	35.8	18.2	21.5	13.2

Flow Rate 0.2 m/hr   Time 3 p.m.   Date 24/5/80   RUN 9

Time Interval HRS	TURBIDITY		COLOUR		PH		Flow Rate cm^3/15sec.	Manometer Readings			
	INF	EFF	INF	EFF	INF	EFF		A	B	C	D
1	40	5.6	200	30	7.6	7.6	70	193	192	195.5	198.5
2	33	5.0	300	20	7.5	7.5	65	193	192	195.5	188.5
3	38	3.9	350	20	7.5	7.5	65	193	192	196	188.5
4	85	4	400	30	7.55	7.5	65	194	192.5	196	188.5
6	15	5.8	200	40	7.5	7.6	80	195	194	197	188.5
7	47	7	350	50	7.6	7.5	85	196	195	197.5	189
8	10	8.5	150	40	7.4	7.5	85	196.5	195.5	198.5	189
18	7	5	250	40	7.45	7.4	80	196.5	195.5	198	189
19	42	4.8	300	40	7.35	7.4	80	198.5	197	199.5	189.5
20	16.0	4.5	250	50	7.4	7.35	70	198.5	197	199.5	189.5
21	35	5	300	50	7.5	7.4	80	199	197.5	199.5	190
23	105	4.8	500	60	7.5	7.3	75	199.5	198	200	189.5
25	98	75	500	250	7.5	7.3	50	200	198.5	200	190
27	82	72	400	250	7.4	7.3	60	196	195	199.5	190
28	66	70	400	300	7.35	7.4	75	196	194	198.5	189
29	68	62	500	300	7.5	7.35	65	195	193	196	188
30	62.8	58	450	300	7.6	7.5	80	195	193.5	196.5	188.5
33	61.5	57	400	400	7.4	7.4	70	193	192	197	188
34	57	53	300	250	7.5	7.5	60	193.5	193	197	188
36	56	52	300	250	7.45	7.5	65	193.5	192.5	196.5	187.5
37	60	48	400	300	7.5	7.6	60	193	192	196.5	187
verage	52	29.1	342.9	146.2	7.48	7.44	70.7				

ANALYSIS RUN 9

Time Interval (HRS)	% Removal TURBIDITY	% Removal COLOUR	Headloss over stated filter lengths (cm)		
			A-B	B-C	C-D
1	86	85	1	3.5	7
2	84.8	93.3	1	3.5	7
3	95.4	94.3	1	4	7.5
4	95.3	92.5	1.5	3.5	7.5
6	61.3	80	1	3	8.5
7	85.1	85.7	1	2.5	8.5
8	15	73.3	1	3	9.5
18	28.6	84	1	3.5	9
19	88.6	90	1.5	2.5	10
20	71.9	80	1.5	2.5	10
21	65.7	83.3	1.5	2.0	9.5
23	95.4	88	1.5	2	10.5
25	23.5	50	1.5	2.5	10
27	12.2	37.5	1.0	4.5	9.5
28	-6.1	25	2	4.5	9.5
29	5.9	40	2	3	12
30	11.9	33.3	1.5	3	8
33	7.3	0	1	5	11
34	7.0	16.7	0.5	5	11
36	7.1	16.7	1.0	4	9.0
37	25	25	1.0	4.5	9.5
Average	47.0	60.6	1.2	3.4	9.2

AVERAGES

m^3/hr Flow Rate	(FTU) Average INF. TURB.	(FTU) AV. EFF TURB
0.96	29.2	19.
0.55	114.6	80.9
0.36	155.7	113.3
0.58	72.9	53.1
0.41	81.1	26.2
1.0	58.7	24.9
0.78	40.3	27.1
0.27	40.4	21.2
0.2	51.8	29.1

KABETE PILOT PLANT

Flow Rate 0.44 m/hr. RUN I

DATE	TURBIDITY (FTU)		COLOUR (°H)		IRON CONTENT mg/25 cc.		PH		TURB REMOVAL %	COL. REM. %	FLOW RATE cm^3/15 sec.
	INF	EFF	INF	EFF	INF	EFF	INF	EFF			
19.2.80	12	4.2	80	20	0.003	0.001	6.6	6.5	65	75	375
20.2.80	4.4	2.5	40	20	0.003	0.002	7.2	7.2	43.18	50	225
21.2.80	2.1	1.2	30	15	0.002	0.002	7.4	7.35	42.86	50	230
22.2.80	1.5	0.9	15	10	0.002	0.002	7.3	7.35	40	33.33	240
23.2.80	0.8	0.7	20	10	0.002	0.002	6.7	6.8	12.5	50	235
24.2.80	1.75	0.8	20	10	0.002	0.002	7.1	7.4	54.29	50	215
25.2.80	1.2	0.7	15	10	0.002	0.002	7.4	7.3	41.67	33.33	230
26.2.80	2.4	1.2	30	10	0.002	0.001	7.5	7.6	50	66.67	230
3.3.80	1.8	1.3	15	10	0.003	0.002	7.1	7.1	27.78	33.33	225
5.3.80	2.2	2.0	20	10	0.003	0.002	7.3	7.4	9.1	50	230
6.3.80	5.2	3.2	23	15	0.002	0.002	7.4	7.3	38.46	34.78	230
7.3.80	7.5	15	25	30	0.002	0.002	7.8	7.8	-50	-20	235
AVERAGE	3.57	2.81	27.75	14.17	0.003	0.002	7.2	7.3	31.24	42.20	241.67

Table 7.1

Kabete PilotPlant Flow Rate 0.36 m/hr RUN 2.

DATE	TURBIDITY (FTU)		COLOUR °H		PH		IRON mg/25 cc.		FLOW cm^3/15 sec.	% REMOVAL TURB.	% REMOVAL COL.
	INF	EFF	INF	EFF	INF	EFF	INF	EFF			
19.3.80	6.5	4.0	30	12.5	7.3	7.4	0.003	0.002	190	38.46	58.33
20.3.80	3	1.3	20	15	7.5	7.5	0.002	0.002	195	56.67	25
21.3.80	2.5	1.2	15	10	7.3	7.3	0.0025	0.002	195	52	33.33
22.3.80	8	2.5	75	25	7.7	7.6	0.002	0.003	190	68.75	66.67
24.3.80	13	4.1	150	50	7.8	7.5	0.002	0.002	190	68.46	66.67
26.3.80	2.2	1.2	15	10	7.1	7.4	0.003	0.002	190	45.45	33.33
27.3.80	5.2	3	50	20	7.2	7.2	0.002	0.002	200	42.31	60
28.3.80	3.3	3.0	15	7.5	7.3	7.3	0.0025	0.002	195	9.1	50
29.3.80	4.2	3.2	30	10	7.5	7.4	0.002	0.002	195.5	23.8	66.67
30.3.80	7	3.0	30	20	7.3	7.3	0.0125	0.002	190	57.14	33.33
VERAGE	5.49	2.65	43	18	7.4	7.4	0.003	0.002	193.1	56.21	49.33

Table 7.2

Kabete Pilot Plant | Flow 0.4 m/hr. | RUN 3

DATE	TURBIDITY (FTU) INF	EFF	COLOUR (^oH) INF	EFF	PH INF	EFF	IRON mg/25 cc. INF	EFF	FLOW cm^3/15 sec.	% Removal TURB.	% Removal COL.
2.4.80	4.8	4.0	30	15	7.3	7.3	0.002	0.002	225	16.67	50
3.4.80	3	2.0	20	10	6.8	7.1	0.002	0.002	200	33.33	50
4.4.80	4.2	1.5	35	10	7.4	7.4	0.002	0.002	215	64.49	71.42
6.4.80	5	1.2	15	10	7.5	7.4	0.0125	0.002	220	76	33.33
7.4.80	5.2	4.0	30	25	7.3	7.1	0.003	0.002	200	23.08	16.67
8.4.80	4.0	2.8	15	10	7.2	7.2	0.002	0.002	230	30	33.33
9.4.80	8.2	2.0	80	15	7.3	7.6	0.002	0.0125	215	75.61	81.25
10.4.80	3.1	1.2	35	10	7.5	7.5	0.003	0.002	220	61.29	71.42
11.4.80	2.5	1.2	30	10	7.5	7.4	0.0125	0.002	200	52	66.67
AVERAGE	4.4	2.20	32.2	12.8	7.3	7.3	0.005	0.003	213.9	48	52.7

Table 7.3

Kabete Pilot Plant — Flow Rate 0.39 m/hr. — RUN 4

DATE	TURBIDITY (FTU)		COLOUR (^{o}H)		PH		IRON $mg/25cm^3$		FLOW $cm^3/15sec.$	% Removal TURB.	% Removal COL.
	INF	EFF	INF	EFF	INF	EFF	INF	EFF			
13.4.80	5	3.5	30	7.5	7.3	7.3	0.003	0.002	225	30	75
14.4.80	2.8	2.2	20	5	7.5	7.4	0.025	0.002	215	21.42	75
15.4.80	5	3.1	10	5.5	7.3	7.2	0.002	0.002	200	38	45
16.4.80	3.2	2.2	15	5	7.1	7.3	0.003	0.003	215	31.25	66.67
17.4.80	4.5	2.4	20	10	7.6	7.5	0.002	0.002	185	46.67	50
18.4.80	5.5	3.0	30	5	7.5	7.5	0.003	0.0125	200	45.45	83.33
20.4.80	5.3	4	25	10	7.6	7.4	0.002	0.002	200	24.5	60
21.4.80	5	3.2	20	5	7.2	7.3	0.002	0.025	215	36	60
22.4.80	4.5	2.4	20	10	7.3	7.4	0.0125	0.002	230	46.67	50
23.4.80	4.8	3.4	10	7.5	7.3	7.3	0.002	0.002	235	29.17	25
24.4.80	5	2.1	20	10	7.7	7.6	0.002	0.002	190	58	50
25.4.80	6.5	2.5	30	25	7.2	7.3	0.002	0.002	215	61.5	16.67
AVERAGE	4.8	2.8	20.8	8.8	7.4	7.4	0.005	0.005	210.4	39.1	54.7

Table 7.4

After Run 2

Flush Water Analysis

Time Interval (Mins.)	Turbidity FTU	Colour ^{o}H	PH
3	120	500	7.5
6	115	450	7.5
9	77	450	7.6
12	48	400	7.5
15	33	150	7.5
18	27	100	7.5
21	25	75	7.5
24	23	40	7.4
27	36	60	7.5
30	27	100	7.6

Flush Water Analysis

I

Time Interval (Minutes)	Turbidity FTU	Colour ^{o}H	PH
0	180	500	7.4
3	170	500	7.5
6	170	500	7.4
9	160	400	7.5
12	150	450	7.5
15	155	300	7.5
18	150	800	7.5
21	150	600	7.5
24	150	750	7.5
27	155	500	7.5
30	155	250	7.5

After Run 3

II

Time Interval (Mins.)	Turbidity FTU	Colour ^oH	PH
0	80	300	7.5
3	160	600	7.4
6	80	200	7.4
9	210	500	7.4
12	170	400	7.4
15	2500	500	7.4

RESULTS OF FLUSHINGS (DOWNWARDS)

I

Time (Mins.)	TURB. FTU	COL. ^{o}H
0	160	1600
3	155	750
6	165	500
9	135	300
12	148	400
15	125	400
18	98	300
21		

II

Time (Min)	TURB. FTU	COL ^{o}H
0	56	150
3	77.5	300
6	47.5	400
9	53	200
12	70	200
15	71	300

RUN 6

III

Time (Min)	TURB. FTU	COL. °H
0	36	200
3	35	250
6	62	300
9	53	200
12	50	150
15	48	300
18	49	200
21	47	300

FLUSH WATER ANALYSIS

I

Time Interval (Hrs)	Turbidity FTU	Colour ^{o}H	PH
0	8.5	150	7.65
3	31	300	7.7
6	46	300	7.6
9	63	500	7.6
12	67.5	500	7.65
15	65	500	7.5
18	69	600	7.4
21	70	600	7.3

AFTER RUN 7

II

Time Interval (Hrs)	Turbidity FTU	Colour ^{o}H	PH
0	4.5	40	7.5
3	16	150	7.5
6	16	150	7.55
9	34	150	7.6
12	31	200	8
15	37.5	250	7.45
18	38	250	7.7
21	42	300	7.6
24	45	325	7.8