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ENGINEERING AND THEORETICAL INVESTIGATIONS ON INTERMEDIATE RATE FILTERS

By

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ISHWAR CHANDRA AGARWAL

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CERTIFICATE

Certified that the work presented in the thesis entitled "Engineering and Theoretical Investigations on Intermediate Rate Filters" prepared by Ishwar Chandra Agarwal has been carried out under my supervision and it has not been submitted elsewhere for a degree.

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SYNOPSIS

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Filtration of municipal water supplies is generally accomplished by employing either rapid or slow sand filters. However, in the context of the needs of developing nations, both rapid and slow sand filters are found to possess many shortcomings. Need, therefore exists of developing a modified filtration system suitable for developing nations.

Such a modified filtration system was considered to be comprised of slow sand filters receiving alum-coagulated influents and operating at rates much higher than conventional slow sand filtration rates. This study was conducted to investigate various engineering and theoretical aspects of the suggested modified filtration system. The entire research was carried out employing either two pilot scale slow sand filters, which were constructed for this study, or various prototype slow sand filters existing at Kanpur Water Works.

The results of the first phase of the study, designed to evaluate the effect of alum coagulation on

influents of slow sand filters indicate that alum coagulation followed by flocculation and sedimentation can be used as pretreatment to the influents of slow sand filters. This finding is at variance with the views expressed by many workers who observe that alum coagulation as pretreatment to the influents of slow sand filters is undesirable because it may result in rapid build up of head loss, may cause occlusion of schmutzdecke with consequent deterioration in biological purification and under certain circumstances of pH changes, aluminium hydroxide may precipitate in lower layers creating build up of head loss in these layers. However, the results of this study clearly show that alum coagulation does not result in rapid build up of head loss and length of filter run remains of the same order of magnitude as in the case of filters using uncoagulated influents. There is no occlusion of schmutzdecke and in fact bacteriological quality of filtered water may improve as a result of alum coagulation. Also there is little precipitation of aluminium hydroxide in the lower layers of filter bed. The little difference between the observed and computed values, using Kozeny-Carman equation applicable for clean and unclogged beds, of head loss in the lower layers of sand bed confirms this.

During second phase of the study, the sand filters receiving alum coagulated influents were successfully operated at rates of filtration of 216, 408, 612 and 1000 $l/m^2/hr$, these rates being higher than the conventional rate of 133 $l/m^2/hr$. Even at the rate of 1000 $l/m^2/hr$, almost all the head loss occured in top layers of filter bed, extending upto a maximum of 6 cm from the top of sand bed, and most of the impurities were removed in these top layers. Sand scraping was found to be effective in removing the deposited impurities and microbial growth and restoring the filter for the next run. This would obviate the necessity of introducing backwashing even at these higher rates of filtration.

Economic analysis was done to arrive at an optimal rate of filtration for both rapid and slow sand filters. A rate of filtration of 625 $l/m^2/hr$ was found to be optimal in the case of slow sand filtration, the overall cost of filtration being Rs. 6 (or 0.8 $) per million litres of water filtered for this rate. In comparison the lowest overall cost of filtration was Rs. 14.90 (or 2.0 $) in the case of rapid sand filtration at a near optimal rate of 8100 $l/m^2/hr$ for such filters. Even after adding the cost of alum, chlorine and power in both the cases, slow sand filtration at a near optimum rate of 612 $l/m^2/hr$ is 51.2% cheaper than rapid sand filtration at corresponding optimal rate viz. 8820 $l/m^2/hr$ in conditions as obtainable at Kanpur Water Works.

Besides these engineering investigations, theoretical analysis of the head loss data at different rates was also performed. It was shown that the total head loss, h, in filters receiving alum coagulated waters and operating at rates varying between 133 and 1000 $1/m^2/hr$ is composed of two distinct components, one being the head loss in top surface layers of filter bed, h_s, and the other being the head loss in the remaining lower layers of bed, h_l. While h_s is time dependent, h_l remains substantially constant with time. The head loss in top sand layers, h_s, progresses exponentially with time or specific deposit. The head loss in the lower layers, h_l, can be predicted from the Kozeny-Carman equation after accounting for the little deposition of suspended matter and consequent microbial growth in lower layers, if any.

A mathematical model as follows is proposed to describe the total head loss at any time in filters receiving alum-coagulated influents and operating in the range of filtration rates of 133 to 1000 $1/m^2/hr$.

$$h = A \left(\frac{\partial h}{\partial l}\right)_o + G\, 10^{kt}$$

Alternatively

$$h = A \left(\frac{\partial h}{\partial l}\right)_o + J\, 10^{m\sigma}$$

However, it may be mentioned that latter equation with respect to specific deposit, σ, is more suitable for higher rates of filtration where the effect of biological growth on head loss build up is rather insignificant.

I. INTRODUCTION

1.1 Importance Of Safe Water Supply

The provision of safe and aesthetically acceptable drinking water to the community is of paramount importance and is an important yardstick of the developmental status of a community. The role of public water supplies, that are bacteriologically unsafe, as vectors of enteric and other water-borne diseases has been established by many incidents and investigators (Sedwick, 1902). The efficacy of water treatment to check water-borne diseases was convincingly demonstrated, among other incidents, by the dramatic results of Altona and Hamburg, Germany during the cholera epidemic of 1892. In this incident, using water from the same source, the Elbe River, Altona, which filtered its water supply, escaped entirely, while Hamburg, which used the water unfiltered, suffered a severe outbreak of disease. Realizing from such incidents the importance of prevention being better than cures treatment of water before its consumption was initiated on a wider scale. Following mainly a more complete coverage of water supply schemes and improvements in their operation and control, the death rate due to typhoid in 78 U.S. cities dropped from 20.54 in 1910 to 0.15 in 1946 per 100,000 (AWWA Manual, 1951). Similar is true of England and Wales where death rate from typhoid per million people dropped from 150 in 1900 to almost zero around 1940's (Manual of British Water

Engineering Practice, 1961).

However, primarily due to lower coverage of people by engineered water supply and sanitation schemes in developing countries, mortality rate due to water-borne diseases is quite high. In Cylon, the mortality rate due to water-borne diseases was 102.8 per 100,000 during 1957 while according to a 1962 estimate, 45% of the urban population was covered by water supply schemes (Miller, 1962; Dieterich and Henderson, 1963). In India the mortality rate due to enteric diseases (typhoid, dysentries etc.) is about 360 per 100,000 where 52% of urban population and more than 55% of rural population were not served by adequate water supply by 1966. (Government of India: Planning Commission, 1966).

1.2 Necessity Of Cheap And Simple Water Treatment System

Realizing the importance of treated water supplies in maintaining better public health, most of the developing countries are paying increasingly greater attention to the treatment of public water supplies. In India, this is evidenced by the exponential rise in fund allotments to water supply schemes, both urban and rural. The successive outlays in first, second, third and fourth five year plans for urban and rural water supply and sanitation schemes are 490, 760, 1,053 and 3,730 millions of rupees respectively. However these outlays are extremely meagre considering that 9,000 millions of rupees are required for supplying safe water to the entire rural population and 10,000 millions of rupees needed for covering entire urban population by water supply and sewerage schemes (Roy, 1973; Siddiqi, 1972).

Because of the imposed constraints of paucity of financial resources coupled with the restricted availability of skilled technical personnel and sophisticated equipment, either new water treatment methods have to be evolved or existing ones modified so that water treatment units can be developed which are cheap, simple and easy to construct, maintain and operate and which do not require either the incorporation of sophisticated and complicated equipment or attendance by skilled technical labour.

1.3 Filtration As The Central Process In Conventional Flow Sheet Of Water Treatment

The primary purpose of according treatment to water is to make it bacteriologically safe, aesthetically acceptable and free from toxic and undesirable chemical constituents.

Though it is possible to remove toxic and undesirable substances from raw waters uncommonly rich in them by giving special treatment, however such special treatment operations are generally quite costly. Therefore, mainly on economical considerations, alternative sources of raw water are selected which can be treated by cheaper methods to acceptable quality levels.

To remove pathogens from water to make it safe, disinfection has been found to be most effective treatment. To give an example, the death rate from typhoid fever dropped from 25 to 4 per 100,000 annually following only chlorination of clear lake water (Fair et al., 1968).

However, one of the major function of conventional water treatment is the removal of suspended and colloidal impurities. Removal of turbidity is essential not only from the view point of aesthetic acceptability but also from the fact that efficient disinfection is not possible in presence of suspended and colloidal impurities which serve as hide-outs for the micro organisms. Also increasing public awareness has resulted in more stringent standard for turbidity in drinking water. Since filtration can remove the turbidity, especially due to colloidal particles, more efficiently and economically, it assumes central importance in the conventional flow sheet of water treatment.

Accepting that filtration is the single most important unit operation in the treatment of municipal water, it becomes desirable that this water treatment operation should achieve the requirements of simplicity and cheapness, along with the essential pre-requirement of good and acceptable filtrate quality.

1.4 Rapid Versus Slow Sand Filtration

Filtration of water, in normal practice has been accomplished employing the following types of filters :

1. Slow Sand Filters
2. Rapid Sand Filters : These may be further sub-divided into two sub-categories:
   2 (a) Gravity Filters
   2 (b) Pressure Filters
3. Diatomaceous Earth Filters

The diatomaceous earth filter was developed primarily for military use during world war II and has since been used to a limited extent for swimming pool waters and small potable supplies (American Water Works Association Manual, 1951). Their employment in India would be further limited due to non-availability of diatomaceous earth deposits in any significant quantities. Rapid filters enclosed in pressure cylindrical vessels and operated under line pressures have usually been installed only in industrial water systems and in the recirculation systems of swimming pools (Fair et al. 1968). Since pressure vessels are quite costly, and because of operating and other difficulties, pressure rapid filters have not been employed for treatment of municipal water supplies.

The remaining two types of filter namely the slow and rapid sand filters, both of the gravity type, have been universally adopted for filtration of municipal water supplies. Several modifications to sand filters including multimedia filters, horizontal filters, upflow, biflow & radial filters etc. have been proposed, but due to various operational problems have not been adopted on any scale. The relevant features of construction, operation and performance of conventional slow and rapid sand filters are compared in Table 1.

As partially apparent from reference to Table 1 the rapid sand filters possess following advantages :

(1) Smaller size; For treatment of $5 \times 10^3 m^3$/day of water, for example, the size of rapid filter will be about 40 m^2 while that of slow filter will be about 1200 m^2.

(2) Ability to remove even high levels of turbidity in conjunction with coagulation and clarification.

(3) Adaptability to fluctuating raw water quality because of conjunctive pretreatment.

(4) Lower initial cost of construction in places where land and labour is expensive and filters have to be covered because of cold climates and other reasons.

Because of first three advantages coupled with overall economy of construction and operation under North American conditions, rapid filters have almost wholly displaced slow filters in North American practice (Fair *et al.*, 1968). However, rapid sand filters have some disadvantages which are the deciding factors in making them unfit for use, especially in the rural and semiurban areas of developing countries. These disadvantages include regular laboratory testing and technical supervision to maintain acceptable filtrate quality, costly operation and complicated hydraulic design due to backwashing, and employment of sophisticated equipment which requires, for their proper working, skilled operation, maintenance and workshop facilities. In as much as the proper infrastructure and auxiliary facilities of laboratory and workshop may not exist and coupled with the restricted availability of financial resources and technical personnel even in some urban water plants of developing countries, the adoption of rapid filters may not be wholly justified and defendable.

The slow sand filters, too, possess a number of dis-advantages like their counterpart though these demerits are of completely different nature.

The decline in the construction of slow sand filters has been attributed to the following disadvantages (Van de Vloed, 1955 and Manual of British Water Engineering Practice, 1961).

(i) initial cost is high;

(ii) large areas of land are required;

(iii) the cleaning process requires a large labour force; and

(iv) they are not suitable for dealing with waters containing high quantities of suspended matter (over 25 mg/l.).

If these disadvantages are analyzed, it becomes clear that first three disadvantages result primarily from adoption of slower rates of filtration. Higher rates of filtration were not employed because it was considered that schmutzdecke, a biological mat formed on top and considered primarily responsible for removal of colloidal particles, was a fragile and delicate layer which could easily be ruptured if water was forced at higher speeds. The fourth disadvantage of inability to cope up with highly turbid influents is due to the absence of any pretreatment. Chemical coagulation, universally adopted before rapid filtration, was not used as it was thought to interfere with the normal functioning of schmutzdecke.

However, with the introduction of microstraining and rapid filtration as pretreatment to slow sand filter influents, together with adoption of higher rates, mechanization in the cleaning process and other improvements in the design and construction of these filters has made slow sand filtration compete favourably with rapid sand filtration on the basis of over all economy of construction and operation in many situations even in advanced countries. Two latest (mid-1972) examples of the above observation are the construction of two treatment plants, one of small capacity of $25x10^3$ m^3/day (to be increased to $.1x10^6$ m^3/day) in Ireland and other of $0.5x10^6$ m^3/day in London, both employing slow sand filtration preceded by rapid filtration. (Journal of Indian Water Works Association, 1972 and 1973).

In tropical and developing countries where the disadvantages of high land costs, expensive labour and troubles caused by frost and reduced biological activity due to extremes of cold may be well within acceptable levels, following advantages of slow sand filtration may make it preferable to rapid sand filtration (Huisman, 1970).

(i) Quality of treated water: No other single process can simultaneously improve to, such an extent, the physical, chemical and bacteriological quality of water.

(ii) Cost of construction : Because of low land costs and cheap labour, the initial cost of construction may also be substantially low as filters do not require any covering.

(iii) Ease of construction : Because the dimensions of slow sand filter are much less critical than those of the rapid sand filter, it is possible to make a greater use of local materials and craftmanship with the minimum of skilled supervision. Design is simpler, no especial pipework, equipment or instruments are required, and a greater lattitude in the screening of media and selection of construction materials can be allowed.

(iv) Cost of operation : Cost almost lies wholly in cleaning the filter beds, done mechanically or manually. If done manually, no special equipment is required.

(v) Ease of operation: operation requires less skill and training and no close supervision is needed.

(vi) Conservation of water : Slow sand filters require only one-tenth quantity of water for washing sand as compared to rapid filters. As wash water reclamation is not practised extensively, this water along with the impurities is flushed to waste.

In view of the merits and demerits of the slow and rapid sand filters as discussed in preceding paragraphs and in view of objectives of filtration being to produce lowest cost water consistent with the maintenance of adequate filtrate quality and acceptable operating conditions, as expressed by Miller in Society of Water Treatment and Examination symposium on filtration, slow sand filters should be the logical choice, especially in the rural and semiurban areas of developing countries, satisfy as they do the objectives of filtration more fully in these areas.

1.5 Need And Identification Of Problem

However, though slow sand filters may be more efficient and economical water purification units in rural and semiurban areas of developing countries, they have found only limited acceptance in urban water supply schemes both of developing and developed nations. Many-a-times the real reasons, especially in developing countries, have been psychological than purely economical and/or technological as expressed in the recent background paper of World Health Organization (Huisman, 1970). It observes that there is evidence that in some parts of world water treatment engineers, fearing to be thought out of date, tend to neglect consideration of slow sand filters when planning new works. Still, some of the disadvantages of slow sand filters are chiefly responsible for their restricted use. In order to assign a major role to these filters in the treatment of municipal water supplies in developing countries in particular and in other parts of the world in general, it is desirable that disadvantages associated with slow sand filters be eliminated or reduced so that they become more competetive and viable units of water purification vis-a-vis rapid sand filters.

As discussed earlier that the disadvantages of high initial cost of construction and requirements of large land areas and of large labour force for cleaning process result primarily from the adoption of lower rates of filtration and the disadvantage of inability to cope up with highly turbid waters stems from the absence of pretreatment except that of sedimentation. Therefore, any study to be conducted with a view to upgrade the status of slow sand filters, should be directed towards (i) searching and evaluating suitable pretreatment unit operation (s) to increase the capacity to treat even highly turbid waters (for example river waters during rainy season), and (ii) increasing their throughput by adoption of enhanced rates of filtration; both being governed by the criteria of not deteriorating the filtrate quality adversely, of not increasing the overall filtration cost to unacceptable and non-competetive levels and of maintaining simplicity of construction, operation and maintenance. It may suffice to mention here that chemical coagulation with alum followed by flocculation and sedimention may be a better choice of pretreatment as compared to others, microstraining or rapid filtration for example, for reasons to be discussed in detail in subsequent chapters.

Also, though a great amount of research data and information is available on various facets of rapid filtration, including development of conceptual models for removal of suspended matter and mathematical models both for build up of head loss and removal of suspended impurities, very few, if not almost non existent, investigations have been carried out to contribute towards the building up of mathematical models in case of slow sand filters. Also need exists of demonstrating the applicability or otherwise of the existing concepts and formulations, developed primarily for rapid sand filtration, to slow sand filtration either directly or after modifications.

1.6 Scope Of The Investigation

As may emerge out from the preceding discussion, the present investigation aims at covering the following subareas in the field of slow sand filtration :

1. Suitability of alum coagulation as pretreatment to the influents of slow sand filters. In as much as alum coagulation, a conventionally unacceptable unit operation of pretreatment, may change the character of schmutzdecke and may result in deterioration in biological purification of water as expressed by Huisman (1970) and Ives (1971) there is a strong case for evaluating the effect of alum coagulation on overall performance of the filtration process incorporating unit operations of alum coagulation, flocculation and sedimentation vis-a-vis the performance of conventional slow sand filtration.
2. Operation of slow sand filters at enhanced rates of filtration to increase their throughput and the evaluation of the impact of increased filtration rates on length of run, overall filtration economics and other relevant factors and parameters.
3. Contribution towards the fundamental knowledge governing the build up of head loss in slow sand filters as the filtration of water progresses.

4. Interpretion of the existing concepts relating to the removal of suspended impurities in the process of filtration in context of slow sand filtration.

It is hoped that the above will not only lead to better understanding but to cheaper and more efficient water filters.

II. LITERATURE REVIEW AND INTERPRETATIONS

2.1 Slow Sand Filtration

2.1.1 Historical Background And Development

Though the concept of separating impurities from water by filtration is as old as civilization, processing of water through built sand beds for treatment of water prior to its supply to a community started as late as early nineteenth century. The introduction of water treatment by means of sand filter beds is due to British enterprise. Following James Peacock's patent of 1791, John Gibb in 1804 built a successful slow sand filter bed at Paisley in conjunction with a settling basin and roughing filter, according to the Manual of British Water Engineering Practice (1961). The next successful filters were constructed in 1827 by Robert Thom at Greenock and this was followed by the construction of improved form of slow sand filter in 1829 at Chelsea based on the observations of James Simpson on a trial filter. Thereafter many units employing slow sand filtration have been built in Britain and many other countries. One of the most recent example of use of slow sand filtration is the commissioning in the middle of 1972 of Coppermills Works in London with a capacity of 0.5×10^6 m^3/day (Jr. Indian Water Works Association, 1973).

Though slow sand filtration takes historical precedence over rapid sand filtration which was adopted for the first time in 1885, comparatively very little research has been done in the area of slow sand filtration. When slow sand filtration proved to be an effective barrier against the transmission of disease by water in the middle and late 19th century its popularity for the treatment of municipal water supplies reached its peak. However, because of the shortcomings of slower rate of filtration and consequently requirement of large land areas, of employment of large labour force for bed cleaning and of the inability to cope up with highly turbid waters, their use declined fast in the 20th century. To reduce the impact of the above shortcomings significant work has been done by Metropoliton Water Board of England, claimed to be the largest authority in Britain using slow sand filters (Manual of British Water Eng. Prac., 1961). Initially, the rates of filtration for slow sand filters acting alone was about 50 $1/m^2/hr$. as given by Ridley in the Society of Water Treatment and Examination (S.W.T.E.) symposium on Filtration in 1967. However, with the introduction of micro straining and roughening rapid filtration as pretreatment, these rates were increased three fold to 150 $1/m^2/hr$ (Ridley, 1967). Further in Coppermills Water Works, London, mentioned earlier also, the rate of filtration adopted is 250 $1/m^2/hr$.

(J.IWWA, 1973). Besides upgrading of filtration rates, a considerable innovation has been done in the cleaning of slow sand filters. There have been basically two approaches; firstly to wash the sand insitu as discussed by Laval (1952) and Burman and Lewin (1961) and secondly mechanization of scraping and washing operations (Lewin, 1961). The advantages and disadvantages of these innovations along with other details have been discussed by Ives (1971) and in the Manual of British Water Engineering Practice (1961). In the latter reference, other developments in the design and constructional features of recently built slow sand filters like the refined underdrainage system, automatic control of inlet and outlet hydraulic arrangements etc. have been detailed. Another trend, indicated by American Water Works Association Manual (1950) has been towards employing shallower depths and coarser sand with a view to cut down sand costs and achieve longer filter runs. Again taking the example of copper mills works, London, the depth of sand and gravel is 69 cm and 7.6 cm as against conventional values of 75-100 cm and 30 cm respectively.

2.1.2 Schmutzdecke and its Role

It is felt that the adoptation of very slow rates for slow sand filters in the nineteenth century was largely due to the misconceptions and the imperfect understanding of the processes taking place, especially the schmutzdecke and its role in the removal of impurities from water. For a long time after the introduction of slow sand filters, their function was thought to be merely the removal of suspended matter by simple straining (Manual of British Water Eng. Prac., 1961). This misconception is understandable as bacteria were unknown and analytical chemistry was not much developed. Consequent upon the realization that most of the particles of suspended matter being filtered were considerably smaller than the interstices between the finest grains of sand in the top layer of the bed, it was explained that the larger particles arrested earlier supported smaller particles and the concept of the "mat of arrested substances" was born which was supposed to contain an arrangement of larger particles supporting smaller particles very much similar in the manner in which smaller pebbles supported the sand in the filter bed.

However with rapid advances in microbiology, physical chemistry and allied disciplines of scientific knowledge, a better understanding about the removal of impurities resulted. T.H.P. Veal (1950) in his book "The Supply of Water" gives the following description.

> "The action that takes place in a sand filter is only partially a straining one, and appears to be mainly due to certain varieties of aerobic bacteria, which work and breed on the surface layers of the material. After a filter bed has been working for a sufficiently

long period a film is formed by these bacteria all over its surface, immediately below which the particles of sand become coated with a bacterial jelly similar in composition to the film. Very fine particles of suspended matter, which could easily pass through the interstices between the particles of sand in a clean bed are caught on this surface film and provide the bacteria there with material for carrying on their work. Organic compounds in solution are also absorbed by these bacteria, and are converted by their digestive processes into simpler compounds of inorganic form.

As the action of a sand filter depends mainly upon the bacterial film on its surface, it is evident that a bed of clean sand will not work efficiently until that film is formed.----"

The first comprehensive and lucid details about the qualitative compositions and role played by the different layers of sand were furnished by Van de Vloed in a report presented at the Third Congress of International Water Supply Association held in London in July 1955. Van de Vloed (1955) attributed the first complete explanation of the purification process in slow sand filters to Kemna (1899). Van de Vloed, laying great emphasis on surface energy of the sand grains, felt that sand available from river etc. looses part of its energy by adsorption of iron, aluminium and manganese but not enough to attain a completely reversed zeta potential. During the first two weeks of filter operation, surface energy becomes decidedly positive. He felt that the upper portion of sand becomes rapidly coated with a layer of reddish brown precipitate which consists of a complex of partly decomposed organic matter (humus) with iron, manganese, aluminium and silica and this favours adsorption of negative organic micelles in colloidal systems. He also noted that the better the adsorption of anions out of solution is the more the carbonic acid formed through fermentation on the surface. Because of adsorption of these anions, after 2-3 weeks a great concentration of inorganic salts mainly phosphates will result. At that moment, according to the light intensity and heat factors, the formation of a surface skin of algae will set in. According to Van de Vloed, this skin has been called "schmutzdecke" by Piefke (1880) and christened as "auto-trophe zone" by Van de Vloed, who described it to be composed of organisms brought to the bed in raw water and of forms native to the superficial layers of the filter, the commonest being sesile algae, diatoms mostly, together with varying proportions of protozoa, a few larger animals such as the larvae of midges (chironomus) and a host of bacteria. Van de Vloed observes that the term dirt layer employed by Piefke is not a very characteristic one, certainly not for secondry slow sand filters as the microscopic examination revealed it to be rich in microorganisms but poor in dirt with perheaps the sole exception of chitinous remains of arthropods and their insolable substances.

Van de Vloed defined the "autotrophe zone" as being the uppermost layers of sand inhabited principally by actively photosynthesizing algae in contrast to the traditional connotation of the word schmutzdecke which was supposed to be a gelatenous bacterial film. He agreed that slow sand filters owe much to this surface skin of algae but clearly mentioned that though autotrophe zone is important, it is not the most efficient part of a slow sand filter. Because recently scraped filters can also remove impurities, though less efficiently, and covered filters in cold climates work satisfactorily. Further as pointed out by Huisman (1970) and Ives (1971) that there exist in Scotland small slow sand filters with upflow and the biologically active zone is at the base of the sand. No schmutzdecke is formed and no matting of algae on the surface takes place, yet they function satisfactorily.

Van de Vloed outlined two effects of the autotrophe zone. (1) mechanical straining removing the particles of suspended matter that are too large to pass through the interstices between the cellulose threads of the chlorophycae and the siliceous bodies of the diatoms, these organisms being smaller than the finest filter sand (2) the synthetic activities of algae result in using up carbon dioxide, the available nitrogen and phosphates from the water and releasing oxygen to water to help in oxidation of suspended and dissolved materials. The thickness of this autotrophe zone seldom exceeds a few millimetres, contrary to previous belief.

Below the autotrophe zone is the "heterotrophe zone", colonized mainly by aerobic bacteria which convert organic material in the water to carbon dioxide, and simple inorganic salts like nitrites and nitrates and phosphates. Pearsall, Gardiner and Greenshields (1946) describe excellently the action of this zone and observe that the numerical balance of the microflora of the filter continually tends towards a dynamic equilibrium with the character of the matter to be digested, with the result that the filtrate of slow sand filter tends to be uniform in character. Piefke mentioned that the thickness of hetrotrophe zone does not exceed 30 cm. Van de Vloed observes that the deepest layer, termed as "Mineral Oxidation Zone" also contributes towards the purification of water by oxidizing the material through contact catalysis on the sand surface.

In the S.W.T.E. 1967 symposium, Ridley (1967) detailed the stages of succession of algae growing in the slow sand filters. During the first seven days, small unicellular chlorophyceae proliferates the supernatant water. In stage II after about five days, diatoms or other algae in the inflow seeded the sand surface, or they may have been left behind from the previous filter run. During stage III after about ten days, there may be proliferation of the larger filamentous algae such as Melosira, Cladophora, Hydrodictyon and Enteromorpha.

Ridley observed that if the unicellular algae dominate the flora throughout the filter run, the head loss rises steadily and even if some species form palmelloid stages (i.e. the gelatenous matrix), the difficulties occur only at the time of cleaning. However, under normal circumstances diatoms replace unicellular algae and these cause excessive head loss as their shape and size are ideal for plugging sand interstices. Sometimes the filamentous matted algae could be overproductive photosynthetically, with the accumulated oxygen bubbles causing the mat to float up from sand, carrying much of the top layers of sand with it. This increased the permeability locally and thus increasing the quantity of water filtered but the resulting in a poor filtrate. Another disadvantage of filamentous algae in the autotrophezone listed by Ridley was the rapid sealing of surface consequent to the death and disintegration of these large algae with attendant taste and odour. Because of a very large length of run, normally ten weeks, and the presence of several tons of living and dead matter, Riddley fears the potential risk of taste and odour in filtrates and the possibility of complexes, for example, muco-poly saccharides. He also observes that even when alive, the filamentous algae add to filtration costs.

Ridley observed that slow sand filter basin is a complex ecosystem with physical conditions comparable to a shallow pond with a permeable floor, where primary production of organic matter by algae is sufficient to support a wide range of bacteria, protozoa, rotifera, crustacea, nematoda, annelida and even insect larvae. Some of the predominant algae proliferating on sand bed were unicellular Vovocales and filamentous algae like Melosira, Ulothrix, Cladophora (Ridley, 1967). In summer-time the filter skin often becomes colonized by filamentous algae which include various Isokontae such as Cladophora, Spirogyra, Rhizoclonium and Hydrodictyon, Heterokontae, usually Tribonema and often the filamentous diatom, Melosira. Sometimes blue-black patches of Oscillatoria also develop (Manual of British Water Eng. Prac., 1961).

Huisman (1970) presented a very clear summary of the mode of action of a slow sand filter in a background paper prepared for World Health Organization. He states that purification of water begins while it awaits for a few hours above sand to begin its passage through bed. During this time larger particles start to settle, smaller ones coalesce and become easy to remove and plank tonic algae are involved in synthetic activities by taking carbondioxide, nitrates, phosphates and other nutrients from water to form cell material and oxygen. On the surface of the sand is the organic layer known as the "schmutzdecke" which is slimy, gelatinous, comprising filamentous algae, diatoms and bacteria. Large particles of mineral and organic matter, living and dead algae, parasites, and a proportion of other impurities are left behind, trapped in the sticky mass, where they are digested and broken down. After the schmutzdecke the water passes through the sand bed taking a few hours and during this time main treatment process takes place. Here some straining takes place, but most of particles like colloids, bacteria and viruses are much smaller than the pore openings. Huisman feels that because of the forces of adsorption (which according to him is a complex phenomenon attributed partly to electrical forces, partly to chemical bonding and partly to molecular activity)centrifugation and sedimentation, the impurities come in contact with grains and are held. Consequently, the sand grains become coated with a sticky layer similar in many respects to the composition of the schmutzdecke, but without the larger particles and algae. This sand coating is a teeming mass of bacteria, bacteriophages, predatory organisms such as rotifers and protozoa, all feeding upon the adsorbed impurities and upon each other. Organic matter is broken down and converted into cell material and inoffensive inorganic materials are carried away in the now mineralized filtrate. This activity of filter declines with depth because of progressively diminishing concentration of impurities.

From a close scrutiny of the preceding paragraphs it clearly emerges that in the beginning the schmutzdecke was understood to be a fragile and delicate layer of bacteria and organic matter and the filtration action was mainly dependent on it. Consequently higher rates of filtration were not employed lest the schmutzdecke may puncture. However, the present day understanding portrays the schmutzdecke primarily as an algal layer, dominated by filamentous algae after sometime from its formation, and plays only a secondary role in the overall purification process. Though its role is important for filters, sometimes it may become an undesirable and nuisance-creating layer by blocking the filters and creating excessive head losses besides degrading the quality of filtrate by giving taste and odour to it.

Before closing this section, it would be interesting to record here some of the observations of Van de Vloed (1955) with regard to coatings on sand grains of Puech-chabal filters, which acted as prefilters to slow sand filters, having 55 cm of sand with an effective size of 0.28 mm and uniformity coefficient of 1.6 operating at velocities of 1-1.5 m/hr. and employing alum coagulated and settled water (average dose of alum 70 ppm). Microscopical examination of sand revealed a very porous dark brown coating. This sand smelt very badly, whilst the taste and odour of the filtered water were good. Boiling the coated sand grains in 5% HCl for two hours resorted the yellow white grains of silica which, in an experimental filter, was completely unable to remove even small quantities of dissolved organic matter and germs. Following the analysis of the reddish hydrochloric liquor, it was found that the sand coating, which was about 10% of the dry and clean sand weight, consisted of nearly 30% of Fe_2O_3, 15% of Al_2O_3, 1 to 8% of MnO_2, 6% of SiO_2 and 2% CO_2, rest being organic matter, water of crystallization, small quantities of calcium etc. He observed that chemical coagulants, if needed, are not so much necessary to form an artificial mat on top of sand bed, but rather to form a positive coating around the silica sand grain. After a ripening period there exists a diffuse jelly-like coat around each grain containing metallic oxides, hydrates and carbonates, organic matter, germs and innumerable oriented water molecules. He agreed with Kemna (1899) that it is the sticky surface of grain which removes impurities but mentioned that the term zooglea should not be given to this formation as it is not formed exclusively by the action of living beings. He felt the presence of MnO_2 important in the diffuse layer as it is a very good catalyst for oxidation.

2.1.3 Pretreatment of Filter Influents

Van de Vloed (1955), Ridley (1967) and Huisman (1970) stressed the fact that slow sand filters are unsuitable for highly turbid waters. Huisman stated that the best results were obtained when the average turbidity was 10 mg/l or less, although short peaks of 100 to 200 mg/l could be accepted. Twort (1963) feels that schmutzdecke is a very delicate membrane which could be easily clogged if influent contained excessive amounts of silt and suspended matter. Also because of reduced amount of light penetrating to the autotrophic zone, photosynthetic activity would be reduced or stopped. Therefore, if incoming water either contains eexcessive amounts of inorganic suspended matter or high concentration of algae, some pretreatment is necessary. To deal with high concentration of algae in raw water containing low turbidities, Ridley described algicidal treatment with copper sulphate and primary rapid sand filtration or micro-straining. However, Huisman (1970) advised against prechlorination or copper sulphate treatment before slow sand filtration as residuals of chlorine or copper could effect the biological activity in filter. On the contrary Bauman et al. (1963) reported favourably using residual chlorine doses averaging 8.8 mg/l, but it appeared that this oxidized the organic and living matter on the sand surface.

In the document of World Health Organization (1970), the pretreatment suggested for rural areas is plain sedimentation with detention period of a few days while the following four processes are recommended for urban and large water supplies :

(i) Plain storage with sedimentation

(ii) Sedimentation with microstraining for algae removal

(iii) Rapid filtration or naturally filtered intakes

(iv) Plain sedimentation and rapid filtration

Agarwal and Agrawal (1973), however, argue that first pretreatment of storing water for long periods may be suitable in some areas, for example impoundage of relatively clean flood water of the Thames River prior to slow sand filtration in London, but may not be possible in majority of the cases. They further indicate that rapid filtration, either alone or with plain sedimentation may be costly and not preferred due to adoption of two types of filter, and consequent complexity and higher costs.

Another method of pretreatment is chemical coagulation which is universally adopted for rapid sand filtration. The use of chemical coagulation before slow sand filtration has generally been prohibited. Most of the British and European authorities including Ives (1971) Huisman (1970), Twort (1963) and Manual of British Water Engineering Practice (1961) and some of American authorities like Babbit, Doland

and Cleasby (1962) caution against the use of coagulation. However, primarily confronted with the problem of treating progressively increasing quantities of highly turbid waters, many water works including those of Kanpur (Agarwal et al, 1973) and Allahabad in India, McMillan slow sand plant (Smith, 1945) and others in U.S.A. and some waterworks in Spain (Van de Vloed, 1955) have been using chemical coagulation before slow sand filtration. The manual of American Water Works Association (1950) also observes that the effectiveness of coagulation process developed for rapid sand filtration has promoted the use of coagulation and sedimentation where slow sand filters are used.

Ives (1971), in one of the most comprehensive and critical review on filtration, observes that coagulation treatment (with alum, for example) before slow sand filtration is undesirable because the alum floc will rapidly seal the sand surface, and occlude the schmutzdecke, with a consequent rapid rise in head loss, and deterioration in biological purification. Also in certain circumstances described by Ives (1957), pH changes in the filter, together with low pH caused by alum dosing, could precipitate aluminium hydroxide in the lower layers of the filter, which gave rise to great difficulty with regard to cleaning. Ives (1957) mentioned that in slow sand filters at Brisbane, England, algal proliferation caused the pH of applied water to rise substantially resulting in precipitation of calcium carbonate and magnesium hydroxide to form deposits on the sand grains. Later during periods of extremely high turbidity, alum coagulant was added and as a result of alum dosing pH of the pretreated water was 5.5 or less and aluminium ions (Al^{+++}) were present in water. The free aluminium ions (Al^{+++}) reacted with $Mg(OH)_2$ on the sand grains to form $Al(OH)_3$ in the depth of the bed, and the resulting flocculant mass caused a rapid rise in the head loss. But Ives (1957) also observes that conditions such as these are rarely encountered in practice and even if it is necessary to feed such heavily coagulant-dosed water to slow sand filters the use of sodium aluminate should obviate the trouble of rapid build up of head loss in the lower layers of sand. However, in view of the apprehensions expressed by Ives (1971), it becomes necessary to evaluate, in thorough details, the effect of chemical coagulation, as a pretreatment, on slow sand filtration before its recommendation to field application.

2.1.4 Modern Trends and Developments

The modern trends and developments in the area of constructional and design aspects of slow sand filters including innovations in the cleaning operation have been indicated to some extent in section 2.1.1. More details are furnished in Huisman's document (1970) wherein he also identifies areas for further research and development.

Important trends in the process of slow sand filtration are indicated below. In the S.W.T.E. symposium, Ridley (1967) noted that artificial turbulence applied to the supernatant water could retard rates of reproduction of algae. In obvious attempts to check excessive growth of algae in schmutzdecke, Huisman (1970) mentioned some experiments in Africa where intermittent filter operation by draining the beds in turn to a few cm below sand surface to allow drying of algae by sun is being tried. In other cases, the beds are being operated unsubmerged, the raw water being sprayed on to a dry surface, with the water level being controlled within the bed. Obviously the usual schmutzdecke is not formed with this method of operation but it is claimed that the additional oxygen absorbed by the sprayed water encourages a different form of biological activity. In Scotland, upward flow biological filters are being tried out for small installations; the advantage being claimed is ability to backwash media by simple manipulation of valves and thus obviating the need for scraping.

Other experiments relate to increasing the colour and odour removing properties of slow sand filters by incorporation of activated charcoal, either as a layer within the bed or as an additive to raw water. Faced with a problem of deeper penetration of suspended matter after adoption of microstrainers for pretreatment, Lynch *et al*. (1965)

successfully employed coating of sand bed by diatomaceous earth @ 0.5 Kg/m^2 by simple "broadcasting" by hand. This "artificial schmutzdecke" not only checked deeper penetration of impurities but also resulted in longer filter runs, 12 weeks compared to earlier 7 weeks even when rate of filtration was increased by 14 percent.

Ridley (1967) also described some experiments using 0.74 m of sand filter column of 0.3-0.4 mm size with uniformity coefficient of 1.0 employing filtration rates of 2.4-2.9 m/hr and having backwashing arrangements. He, however, found filtrate turbidities unsatisfactory but thought further development may be possible. Ives (1971), however being highly critical of this compromise between rapid and slow filtration feels the idea to be too optimistic and naive an approach considering the different mechanisms involved in rapid filtration (physical) and slow filtration (biochemical). In his view, if some means of flocculating the very fine particles of turbid matter were used, possibly cationic polymers, and a rapid chemical oxidation/mineralization could be achieved (for example, with ozone) then such a compromise might work effectively.

Finally both Ridley (1967) and Huisman (1970) recognized that one of the important sub area for further research is the control of algae in autotrophe zone or schmutzdecke. While Huisman called for suitable research to check the over development of Cladophora or blanket weed, Ridley felt that chemical control of algae without interfering with the natural formation of zoogleal film will be the greatest contribution in the area of slow sand filtration. Huisman also refers to covering the filter surface with light matting of hessian or of foam plastic on which the schmutzdecke could form in order to facilitate cleaning and lengthen filter runs.

2.1.5 Mathematical Formulations

More than one and a quarter century elapsed after the introduction of slow sand filters in 1804, before a simple mathematical model was developed explicitly to describe the removal of impurities from water through the process of filtration by Iwasaki (1937). Since Iwasaki's model was based on filtration experiments employing rates of filtration of 0.0035 - 0.012 cm/s and using no flocculent, his results may be viewed as applicable to slow sand filtration. After Iwasaki, and especially after late fifties, a good amount of work has been done towards the mathematical formulation of filtration. Since most of these formulations have been developed for rapid sand filtration as indicated either explicitly or implicitly by their authors, their applicability to slow sand filtration may or may not be valid. However, based either on experimental observations or on theoretical knowledge, any of these mathematical models in their either parent or modified version could be applied to slow sand filtration. Therefore, most of them are presented in following sections.

2.2 Mathematical And Conceptual Models For Water Filtration

2.2.1 Models and Theories of Clarification

Most of the early research in the area of water filtration has been devoted to hydraulic characteristics of filter including head loss, control of rate of flow and backwash etc. Typical of these investigations is the work carried by Hazen (1892) connected with the hydraulic resistance of a clean sand bed and the effects of water temperature changes. T. Iwasaki for the first time in the area of water filtration dealt with the clogging effect and efficiency of removal of impurities.

Iwasaki (1937) first suggested that the change of concentration of suspended particles per unit depth filtering through a column of granular media is proportional to the instantaneous concentration as

$$- \frac{\partial C}{\partial L} = \lambda C \qquad (1)$$

where λ is a filter coefficient and a measure of the efficiency of a filter. λ would depend on various conditions prevailing in filter system at the instant of measurement. Though being empirical in nature, proposed as it was on the basis of experimental observations and intution, a rational basis for this has been provided in the area of aerosol filtration by various investigators including Chen, C.Y. (1956), Friedlander (1958) and Humphery (1955). Based on the assumption of total removal being contributed by individual fibre or grain and introducing the concept of the efficiency of individual grain, η , the following equation can be easily derived

$$\frac{\partial C}{\partial L} = - \frac{3}{2} \frac{(1-f_o)}{d_m} \eta C. \tag{2}$$

In equations (1) and (2) C is the volumetric concentration of material entering a unit volume of filter, L filter depth, f_o porosity of clean bed, dm diameter of grain. The efficiency of individual grain is defined as the certain percentage of the cross-sectional area of the grain confronting the flow from which all particles are removed. In addition to equation (1) which was rigorously experimentally confirmed by Ison (1967) as stated by Ives (1970), Iwasaki proposed another equation based on mass balance.

$$\frac{\partial \sigma}{\partial t} + \frac{v}{(1-f_{\sigma})} \frac{\partial C}{\partial L} = 0 \tag{3}$$

where σ is specific deposit or the volume of suspended matter retained per unit of filter volume, f_{σ} = porosity of deposited material and V superficial filtration velocity.

Iwasaki and later Stein (1940) used a linear equation for λ as

$$\lambda = \lambda_o + c\,\sigma \tag{4}$$

where c is a constant and λ_o is the filter coefficient of clean bed for $\sigma = 0$. Equation (4) suggests a continually increasing filtration efficiency for increasing deposits which is contrary to experience and invalid based on hydrodynamic and geometrical considerations. Ives (1960) proposed a quadratic equation in σ as :

$$\lambda = \lambda_o + c\,\sigma - \frac{\phi\,\sigma^2}{(f_o - \sigma)} \tag{5}$$

Ives and Sholji (1965) related the constants λ_o, c and ϕ to three fundamental parameters namely dm, V and μ and three new constants K_1, K_2 and K_3 to be determined by pilot plant investigation.

Deb (1969) developed a modified mathematical model of filtration of a dilute unisized suspension incorporating all physical variable involved in filtration to describe the space-time variation of concentration of suspension and specific deposit.

$$\frac{DC}{Dt} + \frac{v}{(f_o - \sigma)} \frac{\partial C}{\partial L} + \frac{\partial C}{\partial t} = -\lambda' C \qquad (6)$$

$$v \frac{\partial C}{\partial L} + (1 - f_{\sigma}) \frac{\partial \sigma}{\partial t} + (f_o - \sigma) \frac{\partial C}{\partial t} = 0 \qquad (7)$$

An equation similar to (7) was also developed by Horner (1968). Though Deb considered the local variation of suspension concentration and specific deposit with time, but he found the contribution of terms considering time variation as small relative to other terms. Deb (1970) also presents the numerical solutions of his non-linear partial differential equations using finite difference schemes.

Ives, acknowledging the restrictions of assumptions made in proposing equation (5) pointed out by Fox and Cleasby (1966), formulated a most general equation for λ at the I.W.S.A. 1969 Vienna Congress (Ives, 1971) :

$$\lambda = \lambda_o \left(1 + \frac{b\sigma}{f_o}\right)^y \left(1 - \frac{\sigma}{f_o}\right)^z \left(1 - \frac{\sigma}{\sigma_u}\right)^x \qquad (8)$$

where b is a geometric constant relating to the packing of the filter grains, σ_u is the ultimate or saturation value of specific deposit ($<$ porosity f), and y, z, x are empirical exponents. It was shown by Ives (1971) that by suitable choice of the exponents y, z, and x the previous mathematical models developed by Iwasaki (1937), Ives (1960), Mackrle (1965), Shekhtman (1961), Heertjes and Lerk (1967) and Maroudas (1965) could be explained.

All these theories attempt to analyze the filter performance with only physical parameters evaluated but neglect the very important chemical and electrokinetic parameters. As pointed out by O'Melia and Stumm (1967) they cannot predict the performance of another or same filter using another suspension or without extensive laboratory testing.

In an attempt to introduce a parameter to reflect the chemistry of the filtration system, Yao, Kuan-Mu et al. (1971) proposed the following modification of equation (2).

$$\frac{dC}{dL} = -\frac{3}{2}\,\frac{(1-f_o)}{d_m}\,\alpha\,\eta\,c \qquad (9)$$

They defined α as a collision efficiency factor which reflects the chemistry of the system. Based on practice in coagulation (for example; Swift and Friedlander, 1964) α is defined as a ratio of the number of the contacts which succeed in producing adhesion and the number of collisions which occur between suspended particles and the filter media. Ideally α is equal to 1 in a completely destabilized system. As is clear from the definition that α is a theoretical concept and cannot be directly evaluated by experimentation. In their experiments Yao et al. assumed a value of α equal to 1. It is felt, therefore, that equation (9) also cannot predict filter performance without extensive experimentation like previous filtration theories.

2.2.2 Mechanisms of Clarification

To explain removal of suspended particles from water several mechanisms have been proposed. Some of mechanisms are purely physical mechanisms because they depend only on the physical parameters of the system such as viscosity of water, filtration velocity, size of media and size and density of suspended particles. These mechanisms include straining, sedimentation, interception, inertial impaction, Brownian diffusion and Hydrodynamic action. Hazen (1904) Fair and Geyer (1962), Camp (1964), and Hall (1957) assign most of the removals to mechanical straining along with sedimentation and some inertial impaction. Brownian diffusion as a process of removal was analyzed by Langmuir (1945), Ranz (1951) and Davies (1952).

Several mechanisms have been proposed which may also depend on the chemical parameters like pH, ionic concentration etc. of the suspension. These mechanisms are electrical double layer interaction, Molecular (or Van der Waal's) forces, migration to quiscent areas in the filter interstices, flocculation within filter and various types of chemical bonding. Stanely (1955) reported that hydrous ferric oxide particles are filtered most rapidly in the pH region of their isoelectric point, indicating the significance of electrostatic forces. Cleasby and Bauman (1961) also concluded that electrostatic forces are primarily responsible for the removal of hydrous ferric oxide particles. Davis and Borchardt (1966) report that chemical coagulants are necessary to remove algae & activated carbon and conclude that chemical forces are important and suggest electrostatic effects may be significant. Mackrle and Mackrle (1961) consider the attachment of suspended particles to surface of filter media is controlled by Van der Waal's forces and assume electrostatic effects negligible. Heertjes and Lerk (1967) and Mackrle and Mackrle (1959) proposed mathematical models which assumed equilibrium between Van der Waal's forces and hydrodynamic drag. Hunter and Alexander (1963) studied the flow of clay sols through a silica sand column and concluded that diffusion of the particles into areas of low shear in the porous medium is also an important removal mechanism. Flocculation within filter has been considered a removal mechanism by Fair and Geyer (1961), Camp (1964) and Stanley (1955). Conley and Pitman (1960) using polyacrylamide to the filter influent improved the filtrability of alum floc particles and postulated that the polymers bound the floc particles both to the filter grains and to each other. This polymer is non-ionic, indicating that chemical effects other than those of coulombic origin can be significant. La Mer and Healy (1963), Ruehrwein & Ward (1952), and Michaels (1954) feel that coulombic attraction is often invoked when the suspended particles and filter grains possess opposite charges while specific chemical forces can out weigh electrostatic forces in many cases, for example in the case of adsorption of anionic polymers on negative silica surfaces. Postulated interactions to explain these attachments include ion-exchange, hydrogen bonding and formation of coordinative bonds and linkages (La Mer and Healy, 1963; Michaels & Morelas (1955); Black et al., 1965 and Busch, 1966). Sorption of suspended particles on the surface of filter media has been proposed by Stein (1940), Ives (1961), Camp (1964), O'Melia and Crapps (1964) and Fox and Cleasby (1966).

Agrawal (1966) in his doctoral dissertation has considered the relative importance of all the possible mechanisms proposed by different authors by introducing the concept of dimensionless parameters. These dimensionless parameters for various mechanisms are presented in Table 2. Arguing in a quantitative manner, Agrawal concluded that the removal of micron size particles in a filter is contributed principally by three separate mechanisms each of which can be described by a quantitative parameter. The three mechanisms are : (i) A concentration-dependent electrokinetic mechanism

TABLE 2

DIMENSIONLESS PARAMETERS AND FUNDAMENTAL VARIABLES RELEVANT TO VARIOUS MECHANISMS (AGRAWAL, 1966)

Mechanisms of Removal	Relevant Dimensionless Parameter	Dependence of λ_o on Fundamental Variable						
		d_p	d_m	v	μ	$\rho_e = (\rho_p - \rho_f)$	Conc. of susp.	Nature of susp.
1. Simple Straining	$R = d_p/d_m$	$\infty d_p^{3/2}$	$d_m^{-5/2}$	X^a	X	X	X	X
2. Gravity Settling	$G = \frac{g}{18} \frac{(\rho_p - \rho_f)}{\mu v} d_p^2$	∞d_p^2	d_m^{-1}	v^{-1}	μ^{-1}	ρ_e^1	X	X
3. Interception Alone	R	∞d_p^2	d_m^{-3}	X	X	X	X	X
4. Interception and Gravity	R and G	∞d_p^2	d_m^{-3}	Weak	Weak	Weak	X	X
5. Impaction and Interception	R $I = \frac{\rho_p d_p^2 v}{9 \mu d_m}$	∞d_p^2	d_m^{-3}	Weak	Weak	Weak	X	X
6. Brownian Diffusion	$P_e = \frac{v d_m}{D_{BM}}$	$d_p^{-2/3}$	$d_m^{-5/3}$	$v^{-2/3}$	$\mu^{-2/3}$	X	X	X
7. Migration to Quiescent Areas	-	-	X	X	X	X	Weak	Weak
8. Coagulation in Filter	-	X	X	X	X	X	Weak	Weak

TABLE 2 (CONTINUED)

Mechanisms of Removal	Relevant Dimensionless Parameters	Dependence of λ_o on Fundamental Variable d_p	d_m	v	μ	$\rho_e = (\rho_p - \rho_f)$	Conc. of susp.	Nature of susp.
9. Molecular Forces	$\bar{M} = \frac{.011\,\Lambda}{\mu\, v\, d_m^3}$	$\propto d_p^2$	d_m^{-3}	Weak	Weak	X	X	X
10. Electro-static or Electro-kinetic Forces	$E = \begin{Bmatrix} K_2 \zeta_p^2 C d_m \\ - K_1 \varepsilon \zeta_p \zeta_m d_p \end{Bmatrix} \times \frac{1}{\mu v}$	Weak	Weak	v^{-1}	μ^{-1}	-	Weak	Weak

[a] X = Independent

representing space charge effect due to the charged nature of suspended particles and some what analogous to coagulation within filter bed. The contribution by this mechanism is independent of media characteristics, except porosity and shape, (ii) An electrokinetic mechanism representing the interaction between media and the suspension (iii) A term representing mechanisms other than electrokinetic, primarily simple straining. He also observes that in the absence of flocculation the first two terms are the dominant contributors, while in the filtration of flocculated particles first two terms are the dominant contributors, while in the filtration of flocculated particles first two terms may become small compared compared to third or non-electrokinetic removal term since by optimum flocculation the zeta potential of particles is either reduced to zero or to a very low level making electrokinetic forces very small.

From the survey of the literature cited it becomes clear that both physical and chemical theories show considerable disagreement among themselves. This stems from the fact that most of the investigators assumed that a single mechanism explains the removal of impurities and their systems were tailor-made to highlight the importance of that particular mechanism. However, in a real system, the integrated picture is highly complex and no single mechanism can fully explain the removals. There fore, O'Melia and Stumm (1967) suggested that it is essential to consider particle removal within a filter bed as involving at least two distinct steps; a transport step and an attachment step. The existence of these two separate steps is now well recognized by many workers including Ives (1970, 1971), Ives and Gregory (1967) and Yao, Habibian and O'Melia (1971). First step is of particle transport which is a physical process and is principally affected by those parameters which govern mass transfer. During the first step the particles are transported to the immediate vicinity of the solid-liquid interface presented by the filter (i.e. to a grain of media or to another particle previously retained in the bed). The second step is particle attachment which is a phisco-chemical process and involves the attachment of particle to the surface. According to Ives (1971), the transport mechanisms may be straining, interception, inertia, sedimentation, diffusion, hydrodynamic action and orthokinetic flocculation. They probably all act simultaneously, but the relative importances depend on the characteristics of the particles, the flow and the filter media. Electric double layer interaction, Van der Wall's forces and mutual adsorption are the available attachment mechanisms (Ives, 1971). It may be mentioned, finally, that to describe the particle removal, it is necessary to give importance to both transport and attachment steps. There are no grounds for assuming *a priori* for either transport or attachment to be rate controlling.

2.2.3 Mathematical Models for Head Loss

Since the flow of water through clean bed of porous media at filtration rates normally employed in water filtration is in the laminar range, Darcy's Law can be applied to compute frictional head loss.

$$\frac{h_o}{L} = \frac{1}{K_o} \cdot v \tag{10}$$

where h_o is the initial head loss and K_o the permeability of clean bed. However most of the investigators have preferred the use of Kozeny-Carman equation. The Kozeny Carman equation is based on the concept of hydraulic radius introduced by Schiller (1923). The hydraulic radius r_h can be defined as

$$r_h = \frac{f_o}{s_o\ (1-f_o)} \tag{11}$$

So is the specific surface area and is the ratio of surface area to volume of the grain. The Kozeny-Carman equation is applicable to flow through clean bed and is given below

$$\frac{h_o}{L} = v\ \frac{\mu}{\rho \cdot g}\ K'\ \left(\frac{L_e}{L}\right)^2\ \frac{(1-f_o)^2}{f_o^3}\ S_o^2 \tag{12}$$

The only incalculable factor in this expression is $K' \left(\frac{L_e}{L}\right)^2$ now known as the Kozeny-Carman constant and designated by K_o. For unconsolidated porous media, Carman gives a values of 2.5 for K' and a value of 2.0 for $(Le/L)^2$. This leads to an experimentally determined value for K_o of about 5 by Fair and Hatch (1933).

Since Kozeny-Carman equation (K-C eqn.) is applicable only for flow through clean bed, it cannot be applied to clogged bed resulting from accumulation of suspended material inside the porous matrix. As clearly identified by Sakthivadivel <u>et al</u>. (1972), there are four parameters in Kozeny-Carman equation that will vary with filtration. The variables are :

(i) change in porosity of filter due to clogging

(ii) change in surface area of the matrix grains due to deposition

(iii) change in the tortuosity factor, $(L_e/L)^2$

(iv) change in Carman shape factor, K', which will change due a change in shape of the cross-section.

A generalized equation for head loss during filtration should take account of all the four variables. Based on Kozeny-Carman equation for flow through clean beds, a similar equation can be written for head loss through clogged bed:

$$\frac{h}{L} = v \frac{\mu}{\rho . g} . K. \frac{(1-f)^2}{f^3} . S^2 \tag{13}$$

where h is the head loss through a clogged bed, K is Kozeny-Carman constant for clogged bed, f is the porosity of the clogged bed and S is the specific surface area of deposited grain.

From equations (12) and (13)

$$\frac{h}{h_o} = \frac{K}{K_o} . \frac{S^2}{S_o^2} . \frac{(1-f^2)}{(1-f_o^2)} . (\frac{f_o}{f})^3 \tag{14}$$

If the grain specific surface S_o is replaced by bed specific surface a_o,

$$S_o = \frac{a_o}{(1-f_o)} \tag{15}$$

then equation (14) can be rewritten as

$$\frac{h}{h_o} = \frac{K}{K_o} (\frac{a}{a_o})^2 (\frac{f_o}{f})^3 \tag{16}$$

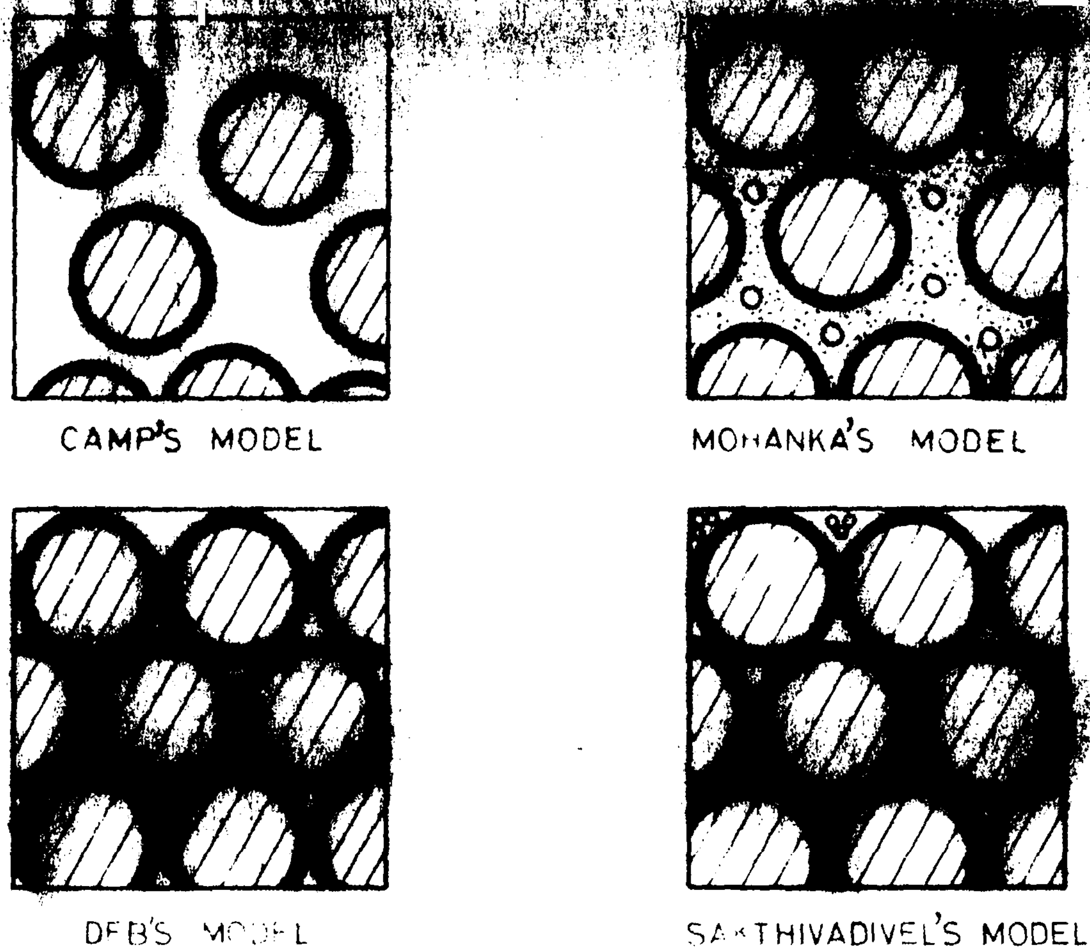
Most of the investigators have based their models on the basis of equation (14) or (16). In order to obtain a relationship either theoretical or empirical or a combination of two, it is necessary to establish the relations between the specific deposit, σ , and the other variables k, a and f. Depending upon the approximations, idealizations and simplified assumptions various workers have produced different models for head loss build up. Broadly viewing, most of the models can be categorized into two distinct categories :

(i) Models which assume uniform coating on media grains

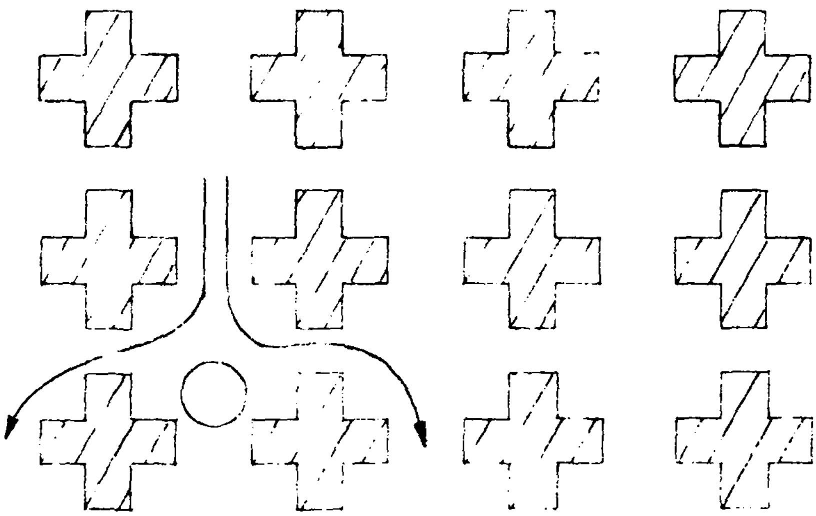
(ii) Models assuming non-uniform coating.

Among the first category can be included the models proposed by Stein (1940), Camp (1964), Deb (1969), Ives (1960) and Sakthivadivel (1972). Camp and Stein both assumed that suspension forms a sheath on each media grain which were assumed to be non-joined spheres. Deb's model was of uniform coating on joined spheres with number of contacts being 8. Ives assumed uniform coating on the surfaces provided by parallel capillary pores of equal length and radius. Saklhivadivel used the "hydraulic radius model".

Realizing that in reality, the porous medium is not composed of pores of uniform size, Herzig et al. (1970) thought the porous medium would be better represented by a combination of caverns connected to each other by longitudinal and transverse constrictious. This model, along with those of Deb, Camp and Sakthivadivels, is pictorially represented in Figure 1. Since in the model of connected caverns, the assumption of uniform coating does not hold, non uniform



DEPOSITION MODELS ASSUMING UNIFORM COATING



HERZIG MODEL OF CAVERNS CONNECTED WITH CONSTRICTIONS

FIGURE 1. PICTORIAL DEPICTION OF DEPOSITION MODELS

clogging was assumed.

After briefly describing some of the model qualitatively, it would be worthwhile to present some mathematical equations. Some of the proposed equations are given in Table 3. A close scrutiny of these equations will reveal that these equations are based on modifications of Kozeny-Carman equations and as such can be considered to originate from equations (14) and (16). It may be interesting to present here further modification of equation (16) by Herzig et al. (1970) They gave decrease in porosity as

$$\frac{f}{f_o} = 1 - \frac{\beta\sigma}{f_o} \tag{17}$$

where β is the inverse of compaction factor of deposited particles. Assuming that K_o does not vary with accumulation of deposit,

$$\frac{h}{h_o} = \left(\frac{a}{a_o}\right)^2 \left(\frac{f_o}{f}\right)^3 \tag{18}$$

They gave then theoretical derivations for change in bed specific surface assuming different models. For example for circular capillaries

$$\frac{a}{a_o} = \left(1 - \frac{\beta\sigma}{f_o}\right)^{1/2} \tag{19}$$

TABLE 3

SUMMARY OF HEAD-LOSS EQUATIONS*

Investigators	Equation Proposed	Remarks
Deb (1969)	$\frac{h}{h_o} = (\frac{S}{S_o})^2 (\frac{K}{K_o})^2 \frac{f_o^3}{(f_o-\sigma)^3}$	The factor $(\frac{S}{S_o})^2 (\frac{K}{K_o})^2$ is designated as $J\psi^2/J_o\psi_o^2$ and is determined experimentally
Deb (1969)	$\frac{h}{h_o} = 1+G(1-10^{-k\sigma}) \left[\frac{f_o}{f_o-\sigma}\right]^3$	Primarily an empirical equation G and K are empirical constants
Mohanka (1969)	$\frac{h}{h_o} = (1+p\frac{\sigma}{f_o})^2 (1-\frac{\sigma}{f_o})^{-1}$	Derived from Mackrle's mathematical model assuming x=y=1. p depends on surface area.
Camp (1964)	$\frac{h}{h_o} = \frac{(1-f_o+\quad)^2}{(1-f_o)^2} \frac{f_o^3}{(f_o-\quad)^3}$ x $\left[1/\sqrt{\frac{\sigma}{3(1-f_o)} \pm 1/4+\frac{\sigma}{3(1-f_o)}} + 1/2\right]$	The ratio (K/K_o) is assumed constant and equal to 1.
Sakthivadivel (1966)	$\frac{h}{h_o} = \frac{(1-f_o+\sigma)^2}{(f_o-\sigma)^3} \frac{f_o^3}{(1-f_o)^2} \frac{1}{\xi^2}$	Only change in porosity considered explicitly. $\xi^2 = (\frac{K}{K_o}) (\frac{S}{S_o})^2$
Shektman (1961)	$\frac{h}{h_o} = 1/(1-\sqrt{1-(f_o-\sigma)/f_o})^3$	$(K/K_o)^2 (S/S_o)^2$ assumed constant and equal to 1 only porosity considered explicitly

* After Sakthivadivel (1972) but modified

and

$$\frac{h}{h_o} = (1 - \frac{\beta\sigma}{f_o})^{-2} \tag{20}$$

and for void between nonjoined spherical grains

$$\frac{a}{a_o} = (1 + \frac{\beta\sigma}{1-f_o})^{2/3} \tag{21}$$

and $$\frac{h}{h_o} = (1 - \frac{\beta\sigma}{f_o})^{-3} \quad (1 + \frac{\beta\sigma}{1-f_o})^{4/3} \tag{22}$$

The equation (21) is similar to one given by Camp who assumed non-joined spherical model. For a general case, Herzig et al. (1970) gave following relationship for uniform coating.

$$\frac{a}{a_o} = \left[1 + \frac{\beta\sigma}{\phi\ (f_o)}\right]^{m_1} \tag{23}$$

where m_1 may vary between 1/2 and 2/3 and $\phi\ (f_o)$ between $- f_o$ and $1-f_o$. Substituting (23) in (16),

$$\frac{h}{h_o} = (1 - \frac{\beta\sigma}{f_o})^{-3} \quad (1 + \frac{\beta\sigma}{\phi\ (f_o)})^{2m_1} \tag{24}$$

Equation (24) considers change in porosity and specific surface but does not consider variation in tortuosity factor or Carman's factor. Depending on the value of m_1 and the type of function $\emptyset$, many equation for h/h_o can be derived. Similar equation has been derived by Horner (1968).

$$\frac{h}{h_o} = \left(1+ \frac{b\sigma}{f_o}\right)^{2y} \left(1- \frac{\sigma}{f_o}\right)^{2z-3} \tag{25}$$

If equations (24) or (25) are expanped neglecting higher powers of σ , it may be written as

$$\frac{h}{h_o} = 1 + \text{constant} \times \frac{\sigma}{f_o} \tag{26}$$

Integration of (26) through all the depth assuming negligible suspension concentration in filtrate yields

$$h = h_o + \text{constant} \times t \tag{27}$$

Equation (27) has been experimentally verified by many observers for rapid sand filtration including Cleasby and Bauman (1962), Eliassen (1941), Ling (1955), Mintz (1960) Heertjes and Lerk (1967) etc.

Finally it would be appropriate to mention here that both Herzig _et al_. (1970) and Sakthivadivel _et al_. (1972)

agree that a universal theory which deals with the problem of head loss with specific deposit does not exist. Most of the theories, which are based on Kozeny-Carman equation, describe fairly well the head loss build up with clogging for the experiments from which they are derived, but they do not allow an accurate prediction for a new situation. Therefore, these theories can, at best, be described as semiempirical.

2.3 Applicability Of Existing Models To Slow Sand Filtration

2.3.1 Models and Theories of Clarification

Since T. Iwasaki was the first to propose a mathematical model for removal of suspended material from water, its applicability to slow sand filtration would become the first logical choice for discussion. As remarked by Ives (1971), Iwasaki's paper which is termed by Ives as the source paper of modern rapid filter theory was an attempt to describe slow sand filtration in mathematical terms. Ives' inference was probably based on two factors, one of employment of filtration rates (0.0035 to 0.012 cm/s) equivalent to those used in slow sand filtration, and two of the absence of coagulation and flocculation. However, Ives in the same paper remarks that Iwasaki's equations were more applicable to physical removal mechanisms as found in rapid filters.

For the sake of discussion, Iwasaki's equations are reproduced below :

$$- \frac{\partial C}{\partial L} = \lambda C \tag{1}$$

$$- \frac{\partial C}{\partial L} = \frac{(1-f_o)}{v} \quad \frac{\partial \sigma}{\partial t} \tag{3}$$

Equation (1) states the amount of suspension material removed in any layer of the filter is proportional to the local concentration of suspension in the flow while equation (3) states that the volume of material removed from suspension in a unit layer during an interval of time is equal to the volume of deposit in the same layer during the same time. Equation (3) is, therefore, an equation of continuity for the suspended solids and is based on mass balance.

In equation (1), λ which is a measure of filter efficiency will obviously depend, among other variables, on various removal mechanisms. Operation of different removal mechanisms in rapid and slow sand filtration, physical and biochemical respectively as asserted by Ives, will not affect the validity of equation (1) though values of λ in the two cases may be different. The validity of equation (1) for slow sand filtration on the basis of experimental results was proved by the findings of Folkman and Wachs (1970) who filtered <u>Chlorella</u> through dune sand. They filtered the algae at filtration rates of 40 to 250 $1/m^2/hr$ corresponding to slow sand filtration rates, through 3 m depth of dune sand. The size analysis of dune sand was not reported but permeabilities were quoted of about 30 m/day as against an approximate value of 50 m /day for a normal slow sand filter, so the dune sand was somewhat finer or less porous than that used in slow sand filters. Therefore, the experiments of Folkman and Wachs approximated to slow sand filtration, although they never mentioned so (Ives, 1971). They reported that

$$\frac{C}{C_o} = B \ \bar{e}^{\lambda L} \tag{28}$$

where B is the initial increase in algae concentration. They had to introduce a factor B as they started taking readings of algae concentration after 3 hours of the start of filtration during which time algae concentration at the inlet increased from its value maintained in the inflow suspension of algae and the initial value of C/C_o noted after 3 hours of start of filtration was always more than unity.

Based on these experimental finding and earlier discussion, it can be said that equation (1) proposed by Iwasaki is equally applicable to slow sand filtration.

The contimuity equation (3) is based on the assumptions, among others, that the density and porosity of the deposited mat and biological and chemical reactions do not cause soluble materials to either accumulate in the deposits or be released from them. All these assumptions may not be completely valid for rapid sand filtration as recognized by many workers. These assumptions, more so those relating to biological and chemical reactions, will not be valid for slow sand filtration and the degree of departure between the idealized and the actual phenomena would be more in case of slow sand filtration than in the case of rapid sand filtration because the level of biological and biochemical activities in rapid sand filtration would be smaller compared to slow sand filtration as a result of backwashing and reduced residential time in the case of former. Because of all these considerations it may be essential to introduce additional term(s) in equation (3) incorporating the removal of soluble impurities and their subsequent possible conversion into cell material (i.e. process of conversion of dissolved solids to suspended solids) and the conversion of suspended impurities into soluble gaseous and liquid end products to adequately describe the mass balance of suspended solids in slow sand filtration. However, in proposing these additional term (s), the tendency of biological life to attain dynamic equilibrium in the twin processes of synthesis and metabolism, especially when the food concentration remains substantially constant in filter influents, should be given proper emphasis.

The applicability of further refinements of equations (1) and (3) as proposed by Deb (1969) and Horner (1968) and portrayed in equations (6) and (7) to slow sand filtration are not being discussed as Deb himself (1969) and Ives (1970) have mentioned that the contribution of extra terms in equations (6) and (7) is negligibly small and therefore for all practical and experimental purposes equations (1) and (3) are equally acceptable.

Ives modification of equation (4) proposed by Iwasaki and Stein to relate λ with σ by introducing an additional negative term was done to incorporate an observation that the efficiency of rapid sand filtration decreased after initial increase with increasing values of σ. This decrease in efficiency has been observed by many investigators (Ives, 1960; Shektman, 1961; Heertjes and Lerk, 1967; and Maroudas, 1965) who gave different relationships between λ and σ. On the contrary, in the area of slow sand filtration it has been generally recognized that the quality of filtrate remains uniform even after long periods. To quote from Manual of British Water Engineering Practice (1961) :

"Despite its disadvantages, the slow sand filter has the merit that, unlike the rapid sand filter, --------- it can give <u>continuous</u> and practically complete clarification of the water for <u>long periods</u>."

The fact of continuous maintenance of uniform filtrate quality over long periods has been stressed by many authors and workers including Huisman (1970), Pearsall, Gardiner and Greenshields (1946) and Twort (1963).

A possible explanation for the difference in the behaviour of two types of filter may be as follows. In formulating the equation (5) or its later and more generalized version, equation (8), Ives assumed (i) spherical grains with uniform deposit, (ii) cylindrical flow path, formed as a result of filling of side spaces by deposition, which narrows with further deposition, and (iii) the increased interstitial velocity due to narrowing pores upto a limiting value which inhibits further deposition. One of the main concept in Ives formulation of equation (8) is the ultimate or limiting specific deposit , σ_u, which causes the velocity to rise to its limiting, inhibiting value. The value of specific deposit at which Deb's modified filter coefficient starts decreasing is 0.16 while the values of σ_u as calculated from the data given by Ives (1963) is about 0.07.

In the case of slow sand filtration, because of the low velocities primarily and because of biological activities secondarily, a stage is never reached in which interstitial velocities become high enough to inhibit further deposition. Or stated differently the value of σ even at the end of run, the typical values being of the order of $3x10^{-3}$, does not become high enough to cause the interstitial velocity to rise to its limiting inhibiting value. Therefore, an equation such as (5) or (8) have to be suitably modified, either by having suitable combinations of various empirical constants and exponents, or by adding additional terms to account for the continually uniform efficiency of purification by slow sand filters.

2.3.2 Mechanisms of Clarification

Since the basic concepts underlying various mechanisms govern water filtration in general, they are equally valid both in the case of rapid and slow sand filters. However, an important point of difference between the two types will be the relative significance of various mechanisms because of the higher velocities of filtration and coarser and more uniformly graded media in the case of rapid filtration. Among the transport mechanisms as considered by Ives (1971), the values λ_o, initial filter coefficient, will be higher for simple straining, gravity settling, interception and Brownian diffusion and probabily for impaction also in the case of slow compared to rapid sand filtration considering d_p, μ, ρ_e to be same in the two cases, indicating that transport mechanisms, at least in the beginning of filtration process, are more significant in slow sand filtration. The relative importance of various attachment mechanisms is under dispute in the case of rapid filtration where considerable work has been done to evaluate their significance. However, in the case of slow sand filteration, practically no work has been done except, probably, the studies of Jorden (1963) and Edward and Monke (1967). Jorden, using sodium meta hexaphosphate to increase negative values of electrophoretic mobility of clay particles, found a linear relationship between percent turbidity removal and electrophoretic mobility. Since he did not determine the charge on schmutzdecke, he postulated two hypotheses : (i) adsorption between oppositely charged clay particles and schmutzdecke assuming the later to be positively charged (ii) Bonding because of sharing of polyvalent cations between clay particles and charged sites on the schmutzdecke, both having same negative charges. Edwards and Monke (1967) in their experiments found the zeta potential of the top inlet to be + 850 mV while the rest of the column had a zeta potential of - 200 mV. Using phenol as a disinfectant, they concluded that accumulation of bacteria and their metabolic products caused the sign reversal on sand particles. This would indicate that electrical double layer interaction may be a significant mechanism of attachment in case of slow sand filters using raw and untreated water

Finally, although because of the complexity of the situation it is difficult to assess the importance of various mechanisms, some observations are made :

(i) In the case of untreated filter effluents, electrostatic forces will be of the repulsive type in the beginning of slow sand filtration process and accordingly double layer interaction cannot be a significant attachment mechanism. However, after the formation of schmutzdecke or attainment of the slimy coating on grains, double layer interaction may be a significant mechanism in light of the observations made by Edwards and Monke (1967).

(ii) Straining will play an increasing role in the removal process with the formation of schmutzdecke because of reduction of pore openings.

(iii) Mutual adsorption and attraction may be significant attachment mechanisms.

(iv) Because of the high uniformity coefficient and lower effective size of media together with low velocities, there may be more dead pockets or areas of low shear zone to which particles may migrate.

2.3.3 Mathematical Models for Head Loss

It is needless to observe that Kozeny-Carman equation is equally valid for slow sand filtration. The applicability of any model described in section 2.2.3 to slow sand filtration will not be discussed for the simple reasons (i) that these models do not describe the real picture even in the case of rapid sand filtration and may be equally valid or unvalid even in rapid filtration (ii) that all the investigators have introduced many empirical constants and exponents which are evaluated experimentally and hence all of these can, at best, be described as semi-empirical and (iii) that because of the formation of schmutzdecke, an entirely different situation exists and the very concept of head loss developing only due to deposit on the surface and in the pores of grain media may be of doubtful validity.

However, since most of the equations are based on modified Kozeny-Carman equation, and can be derived by giving suitable values to exponents and functions in generalized equations (24) or (25), some discussion will follow to point the differences in the two cases. Considering the limit expansions of (24) or (25) as presented in (26), reproduced below :

$$\frac{h}{h_o} = 1 + \text{constant} \times \frac{\sigma}{f_o} \qquad (26)$$

or its following form involving time instead of

$$h = h_o + \text{constant} \times t \qquad (27)$$

it is noted that equations (26) or (27) may not be true in case of slow sand filtration. Folkman and Wachs found that rapid filter theory, modified to allow for an exponential head loss rise, adequately described their results of filtration of *Chlorella* through dune sand which approximated slow sand filtration. Ives (1971) comments that exponential rise in head loss was probably due to growth of *Chlorella* in the logarithmic phase, during the long periods of operation extending upto 300 hours.

It is felt that because of the predominace of algae and bacteria in the schmutzdecke and to some distance below through which majority of the head loss generally occurs and because of the probable growth of these organisms in log arithmic phase, an exponential increase in head with time or an equation of the type

$$\frac{h}{h_o} = 1 + \text{constant} \times e^{t}$$

may more suitably describe the head loss build up in slow sand filters unlike a linear increase in head loss with time or specific deposit in case of rapid sand filters.

2.4 Identification Of Some Research And Developmental Needs In The Area Of Slow Sand Filtration

From the preceding discussion and literature cited, the need for research and development in the following areas of slow sand filtration clearly emerges.

One of the most important research needs in the area of slow sand filtration lies in better understanding of slow sand filtration through development of suitable theoretical and mathematical models of head loss build up and removal of impurities. Though, as rightly expressed by Mintz (1966) and Ives (1970) that due to the complexity of particle and fluid motions in filter pores, and the fact that natural waters vary greatly in quality, development of theoretical and mathematical models to predict exactly filter behaviour will not be possible. However, the role of theory will be to provide a rational experimental procedure, a rational way of analyzing and extrapolating the experimental data and rational development of various parameters of the process, to be determined both theoretically and experimentally. For example, there exists a need, in the slow sand filtration area, for rationally developing a mathematical model for head loss build up in terms of specific deposit and initial porosity (σ and f_0 being two of the most important parameters of filtration process in head loss build up) and other theoretical constants (for example β , the compaction factor). On the basis of rational experimental procedure and analysis of data, the theoretical constants (e.g. β) can be evaluated experimentally.

Another important area of research is the understanding of the role of schmutzdecke. By contributing towards the knowledge relating to the composition of schmutzdecke and consequently to its role, many of the disadvantages associated with schmutzdecke, for example prolific growth of filamentous algae causing excessive head loss, adding to filter cleaning costs, and resulting in taste and odour etc. in filtrate (Ridley, 1967), could be reduced or eliminated.

Similarly a better understanding of the schmutzdecke may lead to the development of suitable pretreatment for the influents of slow sand filters, which is universally acceptable and adoptable (alum coagulation, for example), to remove their inability to cope up with highly turbid waters. It may further lead to the adoption of higher rates of filtration to reduce the requirements of large land areas which result in higher initial cost of construction. Since many authors and investigators have cautioned against adoptation of chemical coagulation and higher rates of filtration without giving theoretical justification or experimental evidence, it becomes imperative to demonstrate the validity or invalidity of their observations on theoretical and/or experimental grounds. In case, their observations are proved valid, alternative pretreatments have to be searched and evaluated.

Finally, there remains much to be achieved in the developmental side of slow sand filters. Based on better understanding of the hydraulics of filtration and other phenomena, many improvements relating to constructional features and cleaning process, which may be suitable and acceptable both in urban and rural surroundings of developing and developed nations, are required to be made.

It is felt that suitable solutions to these research and developmental needs in the area of slow sand filters will be extremely helpful in providing cheap and simple and yet efficient filters to both developing and developed countries.

III. MATERIALS AND METHODS

3.1 Objectives Of The Study

Based on the discussion and literature cited in earlier chapters, the major objectives of the present investigation can now be more clearly defined as follows :

(i) To evaluate the suitability of alum-coagulation as pretreatment for the influents of slow sand filters,

(ii) To evaluate the effect of operating slow sand filters at much higher rates of filtration in comparison to conventional filtration rates, and

(iii) To develop mathematical models to describe the build up of head loss in slow sand filtration.

3.2 Laboratory Studies Versus Field Studies

A great majority of research investigations on various facets of both rapid and slow sand filtration have been conducted employing laboratory-scale experimental units. Laboratory-scale studies may be both essential and desirable, especially when directed towards contributing to the fundamental and theoretical aspects of filtration, as the various parameters can be controlled, altered and evaluated more accurately and conveniently.

In the case of rapid sand filtration, a more complete simulation of field conditions is possible even in laboratory-scale units and the results of the laboratory-scale filters may be indicative of what will happen in a prototype. This is because the physical and chemical quality of influent water is more important than the biological characteristics. Physical and chemical characteristics can be generally more fully simulated by addition of chemicals and natural products (for example alum and clay) to tap water to prepare synthetic influents. However, in case of slow sand filtration where biological and organic character of filter influent may be equally important in comparison to physical and chemical quality, preparation of synthetic filter influent may be difficult, in convenient and unsatisfactory. Also, as laboratory-size units are rather small (10-15 cm. diameter cylinders have usually been employed), they may suffer from boundary effects in as much as the natural development of schmutzdecke is concerned. In view of the desired objective of evaluating the effect of alum coagulation on schmutzdecke and related parameters, it was considered necessary to eliminate the effect of scale and environment on the development of schmutzdecke. Further, as schmutzdecke and sand layers were intended to be examined in detail by various analyses including the volatile matter content, algal mass, identification of various microscopic and macroscopic forms of life etc. etc., the small size unit was considered inadequate to provide enough quantities of schmutzdecke and sand layers without seriously affecting the performance of the filter. Folkman and Wachs (1970) have reported such disturbance caused by removal of large number of sand samples.

To avoid the inherent disadvantages associated with the extrapolation of results obtained on the basis of laboratory-scale studies due to incompleteness of simulation of field conditions and to eliminate other difficulties mentioned above, it was decided to carry out the investigations at Kanpur Water Works employing both prototype and pilot-scale filter units. In support of the use of pilot-filters, the following observation by Camp (1964) may be noted.

> "It is the writer's opinion that pilot plant studies of filtration must be widely used if worthwhile improvements are to be made in filtration".

3.3 Kanpur Water Works

3.3.1 General

Kanpur is the biggest town of Uttar Pradesh and industrial metropolis of Northern India. Situated on the right bank of River Ganga at an altitude of about 135 meters above mean sea level, its climate is tropical with shade temperatures varying from almost 0°C to 48°C during an annual cycle. Its population, according to 1971 census figures, is 12.37 lakhs.

3.3.2 Sources of Raw Water

Kanpur Water Works, presently processing an annual average quantity of 205 million litres of raw water, has two sources of raw water. River Ganga, which used to be the sole source of raw water for Kanpur Water Works till 1920, presently supplies slightly less than two-thirds of total annual water requirements while the Lower Ganges Canal meets the balance. The relative positions of tapping points are indicated in index map (Figure 2). The choosing of Lower Ganges Canal as a supplementary source of raw water has been necessitated because of the recurring problem of shifting of the Ganga's course every year after monsoon floods. The fair weather stream of Ganga which used to hug Bhaironghat pumping station till 1920, now recedes as far as 5-6 Km. away after the monsoons (Figure 2). A supply channel 5½ Km. long has been dug to tap the shifted stream of Ganga to ensure supply of raw water to Bhaironghat pumping station.

Because Lower Ganges Canal is primarily an irrigation canal, not more than one-thirds of total raw water supply is available from Lower Ganges canal even though the Ganga-water is much more turbid; during September 1971 to September 1972 the turbidity of Ganga water varied from 28-2870 jtu while that of Lower Ganges Canal fluctuated between 24-775 jtu (AGARWAL _et al_., 1972).

3.3.3 Flow Sheet of Kanpur Water Works

Designed for a capacity of 22×10^3 m^3/day in 1892 Kanpur Water Works included only settling tanks and slow sand filters. During 1912, multiple filtration was adopted.

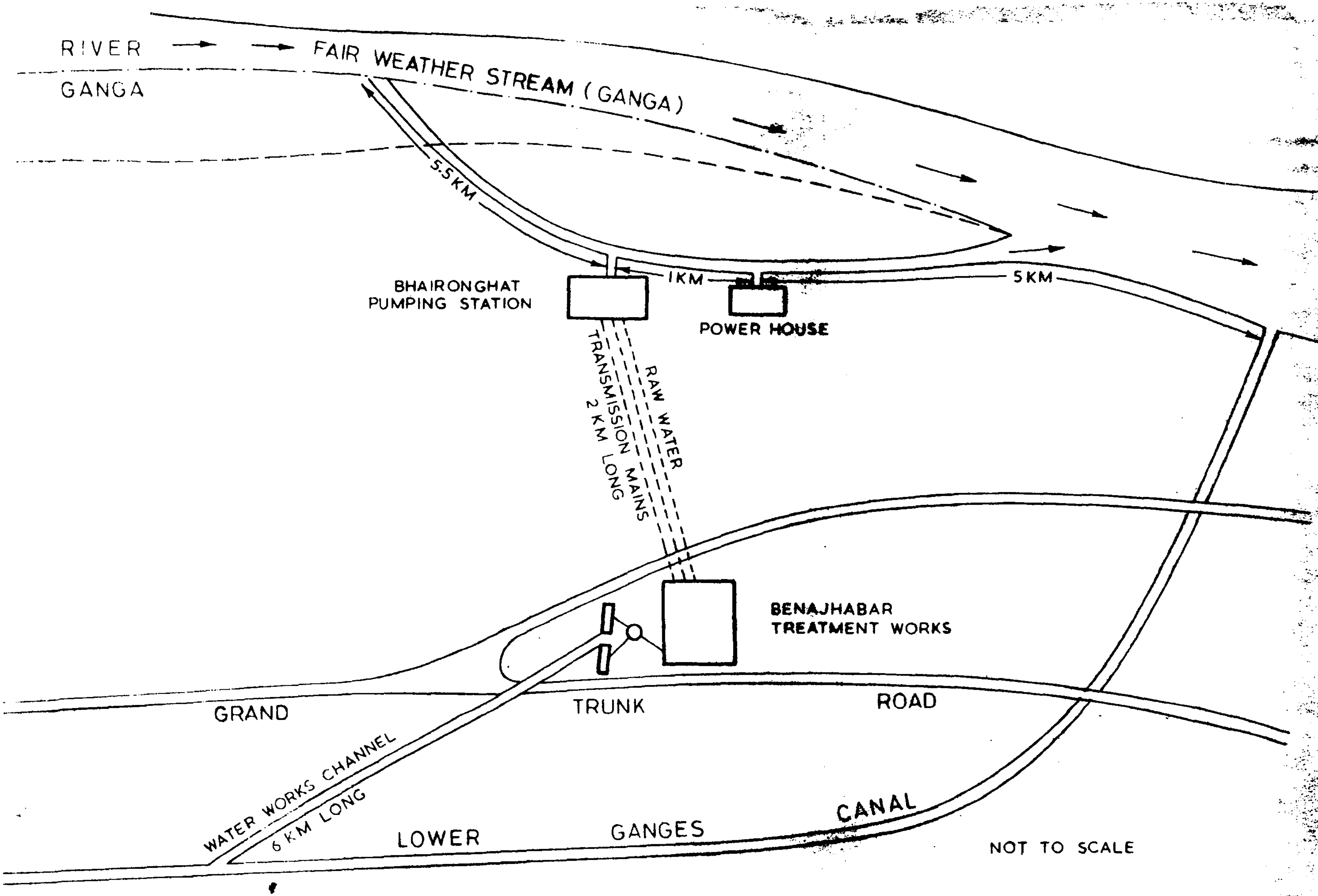


FIGURE 2. INDEX MAP SHOWING RAW WATER SOURCES

Three types of filters namely "Degrosseurs", Peuch-chabal filters (also known as prefilters) and slow sand filters (also designated as final filters) worked in series. A set of six mechanical filters were commissioned in 1939 when the pretreatment of water by coarse filtration was replaced by alum-coagulation followed by flocculation and settling. In 1960, ten rapid sand filters were added. Because of all these alterations and additions, the present flow sheet of Kanpur Water Works is fairly involved. Therefore, an over-simplified flow sheet will first be discussed to offer an integrated picture of the various treatment processes.

The schematic depiction of simplified flow sheet of Kanpur Water Works is presented in Figure 3. The firm arrows indicate the addition of chemicals regularly round the year while dashed arrows indicate addition of chemicals during limited periods only. The treatment given to raw water round the year includes sedimentation, alum-coagulation flocculation, secondary sedimentation to be followed by either slow or rapid sand filtration and post chlorination. However during periods of prolific algae growth on the filter beds, occuring during summer, copper sulphate and chlorine are added to raw Ganga water at the raw water pumping station. Both copper sulphate and chlorine are added in solution form and their doses are 1.5 to 4.5 mg/l and about 1.2 mg/l respectively. Also, during periods of high turbidities

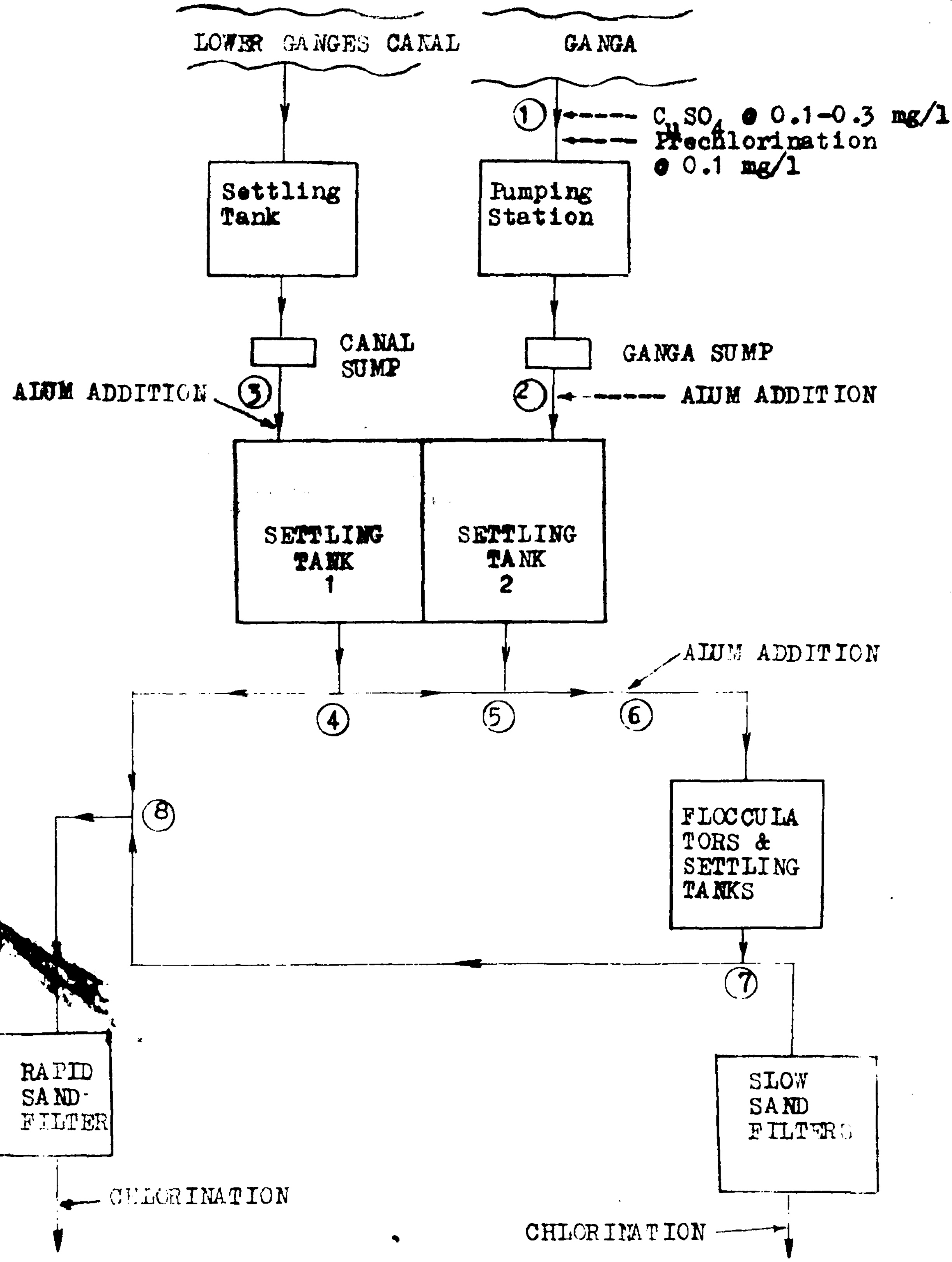


FIGURE 3. FLOW SHEET OF KANPUR WATER WORKS

in rainy season,alum is also added to Ganga water at point 2 in Figure 3.

During normal operation periods the following flow sheet is adopted as depicted in Figure 3. The raw water from Ganga is pumped into the Ganga sump from where it flows into settling tank 2. The raw water from canal, after being settled in pre-settling tanks is pumped and alum is added to it at point 3. This alum-coagulated canal water is settled in settling tank 1. About half of the water coming out of settling tank 1 is diverted at point 4 towards mechanical filters while remaining half is mixed with Ganga water at point 5 coming out of settling tank 2. Alum is added to this mixed water at point 6 . This water is then routed through the units of flocculators and settling tanks. The flocculated and clarified water is divided into two streams at point 7 ; one stream going to slow sand filters while the other going and subsequently meeting the alum coagulated and settled water from point 4 at point 8 . This mixed water, then, goes to rapid sand filters. Therefore, to summarize, while slow sand filters always receive alum-coagulated, flocculated and settled waters rapid sand filters get waters that are a mixture of the above described waters and of alum coagulated waters that have been pre and post settled but have not passed through clariflocculators. The effluents, both from rapid and slow sand filters are chlorinated and stored in underground clear water tanks.

3.3.4 Details of Layout and Units of Kanpur Water Works

The raw water from Ganga is pumped from Bhaironghat Pumping Station by six centrifugal pumps. Raw water is led through four transmission mains, two of 50 cm diameter, one of 60 cm and one of 120 cm diameter, to the treatment works at Benajhabar, 2 km away. Raw water from Lower Ganges Canal first flows into two large settling tanks, popularly known as "Motijheel", and then pumped to the Treatment Works (Figure 4).

The pretreatment units include two settling tanks (104 m x 76 m x 5.3 m); a coagulation unit consisting of a flash mixing device and a flocculation chamber equipped with horizontal shafts with blades rotating on chains; two clarifiers, circular in shape and 30.5 m in diameter, along with two secondary settling tanks provided with baffles and hopper bottom, each having the dimensions of 91.5 m x 30.5 m x 4.05 m. However, during the entire period of study, the flash mixing device and flocculation units were not working.

As mentioned earlier two types of filtration, namely slow sand and rapid sand, are adopted at the Kanpur Water Works. The rapid filters consist of six old filters each of size 7.3 m x 5.5 m, operating at a filtration rate of 4400 $1/m^2/hr$ and ten new units. Out of these ten, four units are of the size 9.7 m x 7.6 m employing a rate of filtration of 4400 $1/m^2/hr$ and the remaining six units with the dimension

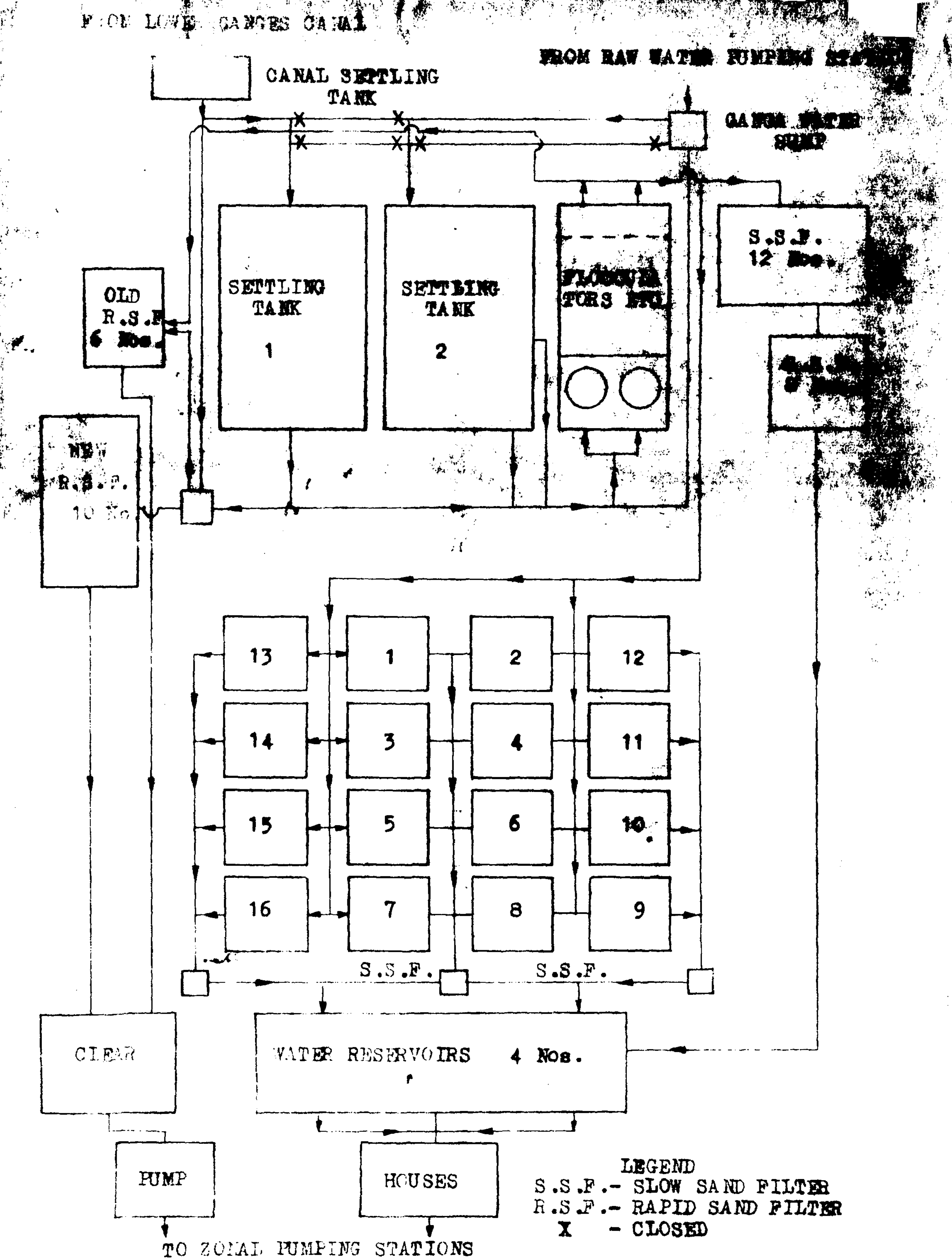


FIGURE 4. SCHEMATIC LAYOUT OF KANPUR TREATMENT WORKS

of 7.6 m x 7.6 m using a rate of filtration of 5880 $1/m^2/hr$.

Slow sand filters, totalling thirtysix, are of two sizes. The twenty units are of smaller size, 49 m x 8.5 m while the sixteen are of bigger dimensions, 61 m x 30.5 m. The first twenty units were originally designed as a part of a multiple filter system along with the Degrosseurs and worked in series with the bigger sand filters. However, now all the filters are operating in parallel with Degrosseurs serving merely as passage way for pretreated water.

All the thirtysix slow sand filters are operated at 133 $l/m^2/hr$ and their media consist of 25 cm of gravel overlaid by varying depths of sand. The under drainage system consists of the main U-shaped concrete channel and laterals formed by open-jointed bricks. These bricks, forming laterals, are underlying a full open jointed layer of bricks covering the complete plan area of filters. All the filters have manually controlled inlet and outlet sluice valves and the filtered water is discharged over the weir into an outlet chamber.

Filtered water from slow as well as rapid sand filters after chlorination at various points is stored in four underground clear water reservoirs. The disinfected water is pumped by a battery of fifteen pumps to twentytwo zonal pumping stations.

3.3.5 Use of Prototype Filters for Study

For conducting a part of these studies and especially for verifying the validity of Pilot-plant data under full-scale operating conditions a number of the slow sand filters as described above, operated with all conditions (except the modification being studied) as normal, were used

3.4 Pilot Scale Filters

3.4.1 Selection of Size

As discussed earlier in section 3.2.1 it was decided to carry out a major part of the study employing pilot-scale slow sand filters. For such pilot filters, a minimum size of 1.83 m x 0.91 m was considered essential to avoid inlet and out let effects and to allow easy and proper construction of piezometric tappings and sampling ports etc.

A bigger filter would have required more quantities of water, would have been costly and more time consuming. For example, the filter unit with the plan dimension of 0.91 x 1.83 m would require 5320 litres of water per day at the rate of 133 $1/m^2/hr$ and 40,000 litres at the rate of 1000 $1/m^2/hr$ and thus two units requiring double these quantities of water at the respective rates. Also it may be mentioned that the two units along with the auxiliaries were built at a cost of more than Rs. 5000 and took more than five months to become ready for operation. Further, it required 1.52 m^3 (54 cft) of sand and 0.38 m^3 (13.5 cft) of gravel. Therefore, in view of the limitations of financial resources, time and labour, the size of 0.91 X 1.83 m was found to be the most suitable.

3.4.2 Constructional Features

All the constructional features of pilot-scale filters were kept similar to those of prototypes. The total depth of filters is 2.6 m with about 0.65 m above the ground level. The middle longitudinal wall is common to both the filters and the other two long walls have piezometric tappings and sampling ports at different depths to record head loss and to collect filter effluents at different depths down the filter. A plexiglass sheet 0.90x0.25 m was fixed with the piezometric tappings and sampling ports being on either side of the sheet. The sheet served as a window to physically see the entire depth of sand bed. To collect the samples from different levels and to install a plank on which glass manometers connected to piezometric tappings were fixed, a recess 1.5 m long 1.05 m wide and 2.5 m deep with three steps was constructed for each filter. There are five openings each for the purposes of filtrate collection and head loss measurements in both the filters, one opening being above the sand bed and four being in the sand bed at various depths. The openings were formed by embedding 1.25 cm diameter and 38 cm long galvanized iron pipe lengths in the long walls of the two filters and all the pipes had iron mesh over the inside end to avoid the chocking of pipes with sand. The distance, centre to centre, between openings is 12, 6, 38 and 55 cm respectively.

The inlet section consisted of a 44x36x93 cm chamber into which 10 cm diameter filter influent pipe discharges and has an open outlet at the top for over flow to maintain constant level in the filter. The influent goes from this chamber into the filter by means of two rectangular openings and is filtered through a sand depth of 90 cm and a gravel depth of 25 cm. The filtered water, after passing through a layer of open-jointed bricks laid for supporting the gravel, is collected by laterals formed by open-jointed bricks. The laterals discharge water into a U-shaped concrete channel in the middle of concrete floor. The water then passes through a 10 cm sluice valve into the outlet chamber which discharges water into rate of flow measuring chamber. After flow measurement, water passes through a water meter and then to a channel. The water meter, 5 cm size, were installed to provide the most reliable method of recording total flow of filtered water. The layout of the pilot-scale filter is given in Figure 5.

3.4.3 Media Specifications

The media consisted of sand of brownish colour, and gravel. The grain size distribution is plotted in Figure 6. The effective size of sand is 0.3 mm and uniformity coefficient

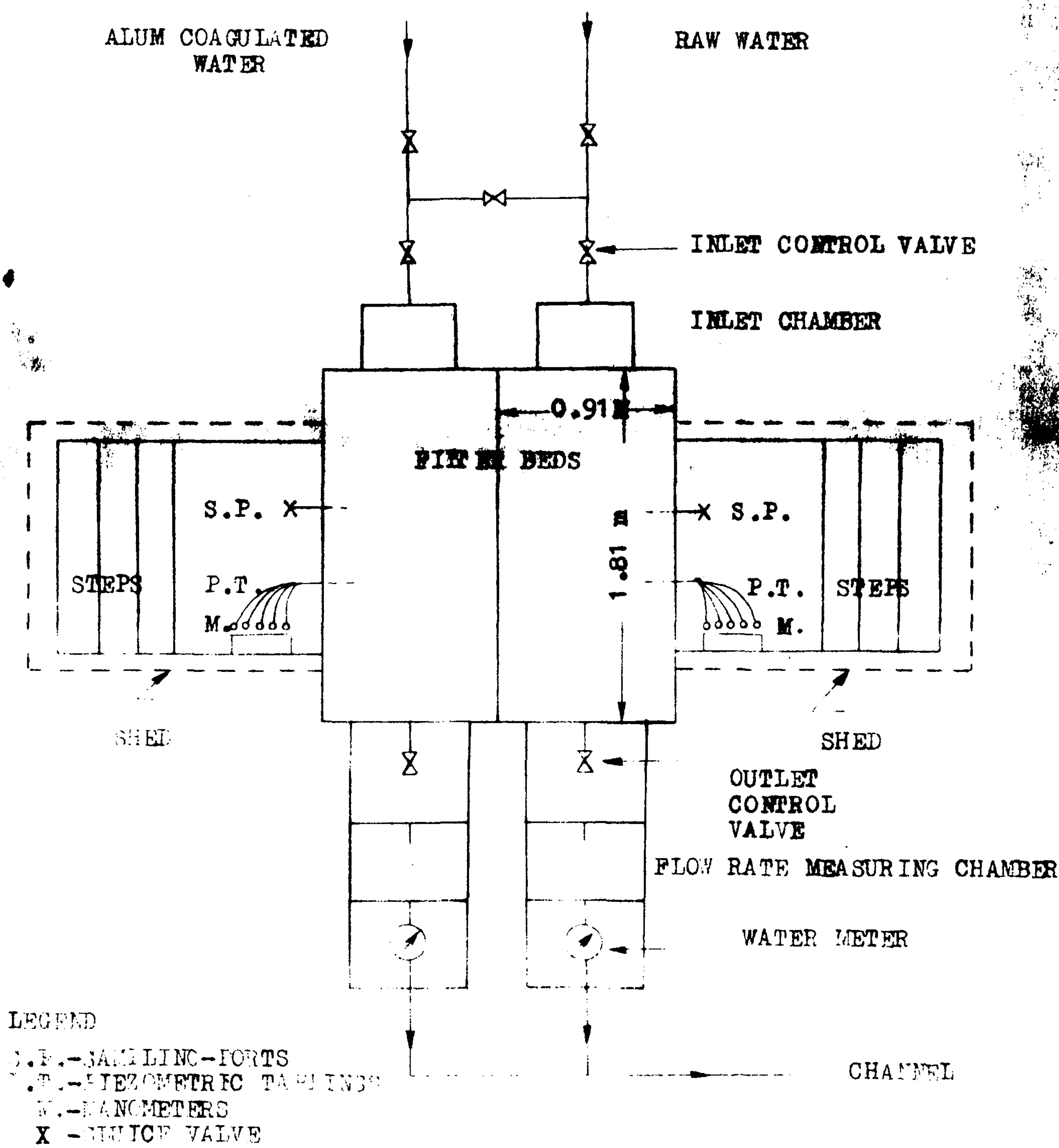


FIGURE 5. LAYOUT PLAN OF PILOT SCALE FILTERS

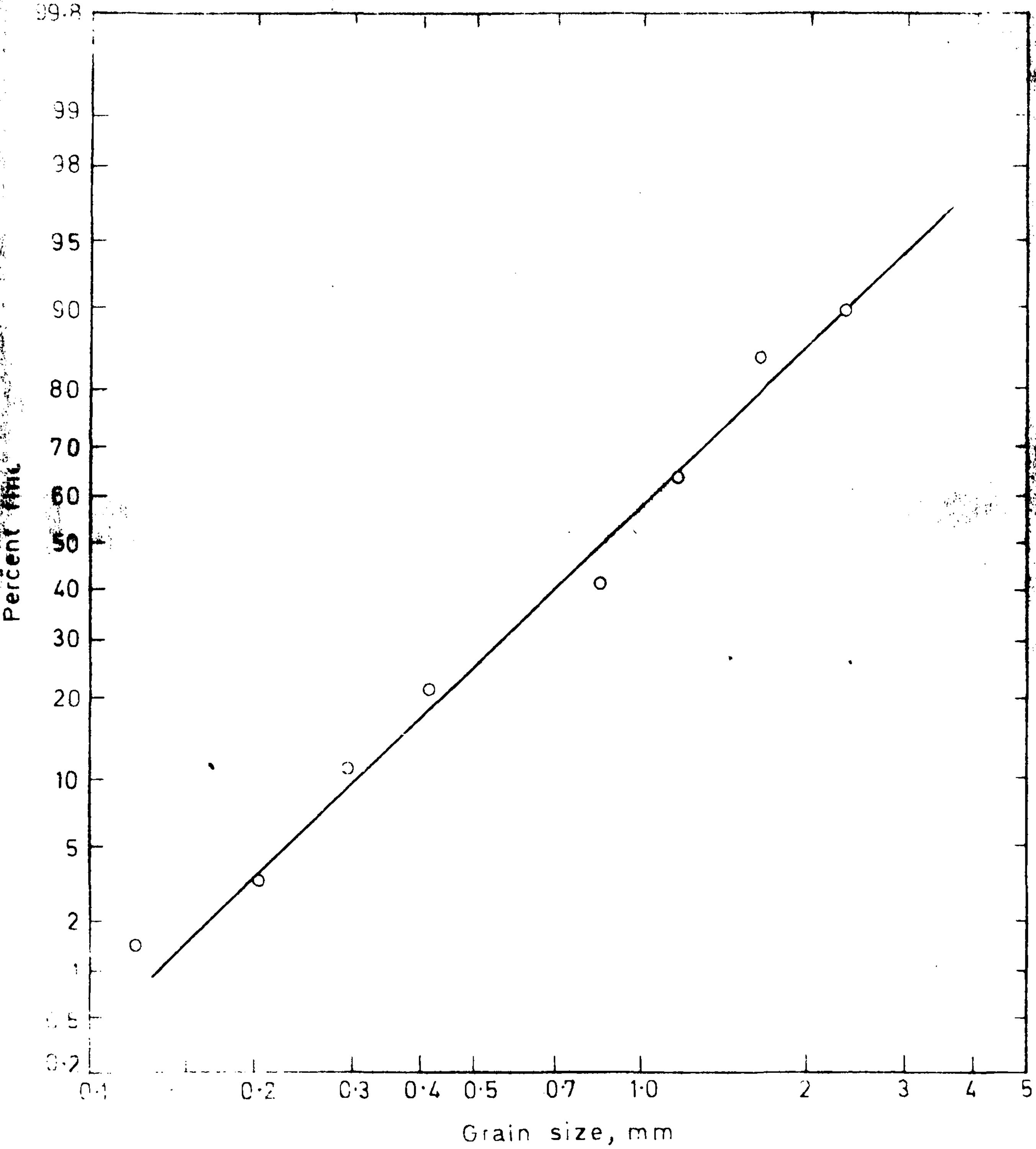


FIGURE 8. GRAIN SIZE DISTRIBUTION OF SAND USED

is 3.5. The total depth of sand was about 90 cm. The gravel was put in 3 layers, top 5 cm of size between 4.8 and 9.6 mm middle 5 cm of size between 9.6 and 25 mm, and bottom 15 cm of size between 25 and 75 mm.

3.4.5 Types and Sources of Filter Influents

The Pilont Plant Studies were carried out using both raw Ganga water and settled, alum-coagulated, flocculated and settled water. Because of the five 10-cm sluice valves, two each on one pipe line bringing filter influents to two filters and one on a cross-pipe fitted in between the two valves on each line, either filter could be fed with either raw water, or pretreated water or a combination of both (Figure 5). However this study was conducted by feeding only one type of water to any one filter and not employing a mixture of the two types of water.

The raw Ganga Water was taken directly from Ganga water sump (Figure 4) through a 70 m long 10 cm diameter cast iron pipe. The coagulated, flocculated and settled water was tapped from a channel feeding water to all the thirtysix slow sand filters. The pipe used in transmitting this water was also 10 cm diameter cast iron pipe, but of a length of about 46 m.

3.5 Operating Procedure and Measurement Techniques

3.5.1 Pilot-Scale Filters

3.5.1.1 Starting a filter

To start the first filter run of both the pilot filters, same sand and gravel were obtained from Kanpur Water Works as were being used for prototype slow sand filters. The sand was washed, at least, five times in batches in churning type of washing machines to ensure freedom from dirt and effective removal of organic matter and possible coatings of calcium and magnesium carbonate etc. The gravel was also washed repetitively in a big container. The washed gravel was then placed in three layers of 15, 5 and 5 cm as per details of the sizes given earlier. To avoid air entrainment in sand layers during filling of a small sand depth of filter was filled with water and sand in small quantities was thrown from a height into the water. This process was continued till the required depth of sand was obtained.

To start the actual filter run, appropriate sluice valves on the two inlet pipes and a cross-pipe were manipulated to introduce the particular type (raw or pretreated) of water into the filter. The filling of water was done fairly slowly to avoid the possible disturbance due to the kinetic energy of water. During filling of water all the ten openings in the longitudinal walls for sample collection and head loss measurement were kept open to expel the entrained air, if any. After water reached the level of overflow channel in the inlet section, outlet sluice valve was opened. After adjustment of outlet sluice valve to give the desired rate of flow, inlet sluice valve was manipulated to maintain a constant level of water in the filter. To facilitate the maintenance of constant level, water was allowed to alwalys trickle from the overflow channel in the inlet chamber. After the desired rate of filtration was obtained, head loss measurements were recorded followed by collection of samples. The details of these operations follow in the succeeding subsections.

3.5.1.2 Measurement and Adjustment of Filtration Rate

A straight mouthpiece, of 7.5 cm asbestos-cement pipe discharged the filtered effluent from the outlet sluice valve chamber, into the flow measuring chamber. During the times of flow measurement the water was collected in a container of suitable capacity. To determine the rate of flow, water was collected for 3 to 6 minutes, depending on the filtration rate and the rate determined by the equation

$$v = \frac{\int_{t_1}^{t_2} dQ\ dt}{(t_2-t_1) \times A} \tag{29}$$

where dQ is the quantity of water collected in time dt and A is the plan area of filter and v is rate of filtration.

To adjust the rate of filtration to the required value, outlet sluice valve was opened to provide, as a first approximation, a rate of flow sufficiently close to its required value. This rate was estimated by collecting sufficiently large quantity of water, of the order of 30 litres, and the time recorded by a stop watch. The process was repeated several times, each time adjusting the opening of outlet sluice valve to get the required rate of flow. After achieving the required rate of flow and "locking" the opening of outlet sluice valve at its desired position, rate of flow was checked three times after an interval of about half to one hour. If found incorrect, the entire process was repeated.

On subsequent days, daily readings of water meters, installed after the rate of flow measurement chamber, were recorded to get the total amount of water filtered in a day. This figure was used to verify the correctness of average rate of filtration using equation (29). Flow measurement by water meter was thought to be the most reliable and yet simple and cheap method and this yielded, after computations, most accurate and reliable values of average rate of filtration.

Whenever the computed rate of flow based on meter readings were found to be different than the required rate of flow, the entire process of adjusting the rate of flow described earlier was repeated.

3.5.1.3 Head Loss Measurement

Head loss measurements were made after adjustment of required rate of filtration and attainment of constant levels in the manometers. To eliminate the erroneous effect on water levels attained in the manometers caused by either air entrainment or sand deposit in the tubes, manometers were disconnected from piezometric openings to allow free flow of water to expel any entrained air or deposited sand. After reconnecting the manometers, the water levels were allowed to reach a steady state to give constant readings.

3.5.1.4 Collection of Samples

Collection of samples from sampling ports was done after recording head loss readings to avoid fluctuations of water levels in the manometers due to flow of water through sampling taps. To remove all the stagnant water in the pipe and to collect representative samples, each tap was opened slightly for about 2 minutes before collecting the samples for laboratory analysis.

Sample volumes, measuring between 200 to 1000 ml were collected either in polyethylene or corning glass bottles, depending upon the analyses to be performed. Glass bottles were generally employed to collect samples for bacteriological analysis, while polyethylene bottles were used to collect samples to determine turbidity, total and soluble aluminium, chlorine demand, chemical oxygen demand and algal content.

3.5.2 Prototype Filters

The filter runs employing prototype filters were generally employed either to confirm results obtained on the basis of pilot-scale filters or to obtain more data relating to the length of filter run etc. at various rates of filtration.

Because of the operational and constructional difficulties, it was not possible to alter the existing arrangements to collect samples and to observe head losses in the various layers of sand bed. Therefore, only influent and effluent samples were collected and total head loss across the entire bed depth measured.

In the absence of rate of flow recorders and head loss gauges, simple but reliable methods were adopted to measure rate of flow and head loss. The rate of flow was computed by measuring the depth of water over a rectangular weir fixed in the outlet section of prototype filters. As an additional check, employed occasionally to determine the rate of flow, the inlet valve was closed and rate of fall of water level determined by noting the fall of water level in a fixed time interval. Head loss was measured as the difference between the water levels of influent and filtered water.

Influent samples were collected from the inlet chamber of prototype filters while filtered water from the outlet section. For collecting filtered water, a man was lowered in the outlet section and a representative sample was collected.

3.6 Schedule Of Filter Runs

In all, 86 filter runs were conducted, 18 on pilot scale filters and 68 on prototype filters. Out of 18 pilot-filter runs, 6 runs were made essentially to examine the suitability of alum-coagulation as pretreatment and the balance 12 runs were made to study the effect of higher rates of filtration on length of run, and over all economics of filtration etc.

To evaluate the suitability of alum-coagulation as pretreatment, 2 runs each were made on pilot scale filters using raw water and alum-coagulated water respectively as filter influents. To find out the average length of filter run under field conditions, 63 runs were made on prototype filters using alum-coagulated influents. All these 67 runs constituted phase I(A) of the experiments. The phase I(B) of the experiments was designed to provide data on head loss build up in particular and on suitability of alum-coagulation in general. The two parallel runs on pilot-scale filters during the phase I(B) were conducted under similar operating conditions, employing media of same characteristics and depth and during periods when raw water turbidity was of the same order as that of alum-coagulated filter influent. All the filters for the I phase of study were operated at the rate of 133 $1/m^2/hr$.

During the phase II of the study, pilot and prototype filters were operated at rates of filtration, higher than

133 $1/m^2$/hr. The higher rates of filtration employed were 216, 408, 612 and 1000 $1/m^2$/hr. Because of the importance of the rate of filtration of 612 $1/m^2$/hr, the economically optimal filtration rate as brought out by the analysis of results, four filter runs on pilot-filters and five filter runs on prototype filters were made at that rate using alum-coagulated water. The pilot-scale filter was also operated once at the rate of 612 $1/m^2$/hr using raw water.

The number of runs and other details of operating filters at higher rates of filtration are included in Table 4 which presents a summary of all the filter runs except the trial and other runs. The other runs include runs which could not be completed due to operating and other difficulties beyond the control of the investigator.

3.7 Analytical Techniques And Equipment

The summary of analytical techniques employed is presented in Table 5.

3.7.1 Filter Influents and Effluents

3.7.1.1 Turbidity

Turbidity of filter influents and effluents was measured using a Hellige Turbidimeter. To cover a wide range of turbidity, standard curves were prepared using stock suspensions of turbidity for five ranges, 0-1.5, 0-4, 0-15, 0-50, 0-150 A.P.H.A. Turbidity units (parts per million SiO_2).

TABLE 4

SUMMARY OF VARIOUS FILTER RUNS

Phase of Experiment	Purpose of Experiment	Rate of Filtration $1/m^2/hr$	Type of Filter Influent	Type of Filter	Number of Runs
I (A)	Evaluation of the suitability of alum coagulated waters	133	1. Raw Water	Pilot-scale	2
			2. Coagulated Water	a. Pilot-scale	2
				b. Prototype	63*
I (B)	Effect of alum-coagulation under similar conditions of operation etc.	133	1. Raw Water	a. Pilot-scale	1
			2. Coagulated Water	b. "	1
II	Effect of operating filters at higher loading rates	216	1. Raw Water	a. Pilot-scale	1
			2. Coagulated Water	b. "	1
		408	Coagulated Water	"	2
		612	1. Raw Water	"	1
			2. Coagulated Water	a. Pilot-scale	4
				b. Prototype	5
		1000	Coagulated Water	Pilot	3

* Concurrent runs of seven prototype filters during September 1971 - September 1972.

TABLE 5

SUMMARY OF ANALYTICAL METHODS

Determination	Method Used	Comment	Reference
For Waters Turbidity	Nephelometric Method	Visual comparison	-
Total & Soluble Aluminium	Colorimetric Method	Eriochrome Cyanine-R Method	Standard Methods(1971)
Chlorine Demand	Colorimetric Method	DPD Method	-do-
Coliform Density	Multiple-Tube Fermentation Technique	Presumptive Test	-do-
Chemical Oxygen Demand	Chemical Oxidation	Dichromate Reflux Method	-do- , Medalia (1951)
Algal Content	Colorimetric Method	Chlorophyll Extract	Creitz & Richards (1955)
pH	Electrometric Method	Glass electrode method	Std.Methods (1971)
For Sand and Schmutzdecke			
Volatile Matter	Gravemetric	-	-do-
Standard Plate Count	Bacteriological	-	-do-
Algal Content	Colorimetric Method	Trichromatic Method for Chlorophyll	-do-
Identification of Biological Life	Visual Comparison	Microscopical Examination	-do-
Weight of Specific Deposit and microbial growth	Gravemetric	Beam Balance Used	-do-
Porosity	-do-	-do-	Taylor(1962)

Two tubes with viewing depths of 50 mm and 20 mm and different combinations of light and dark filters and a rectangular mirror were employed to cover a very wide range of turbidity. Samples having turbidity greater than 100 units were generally diluted with turbidity free water.

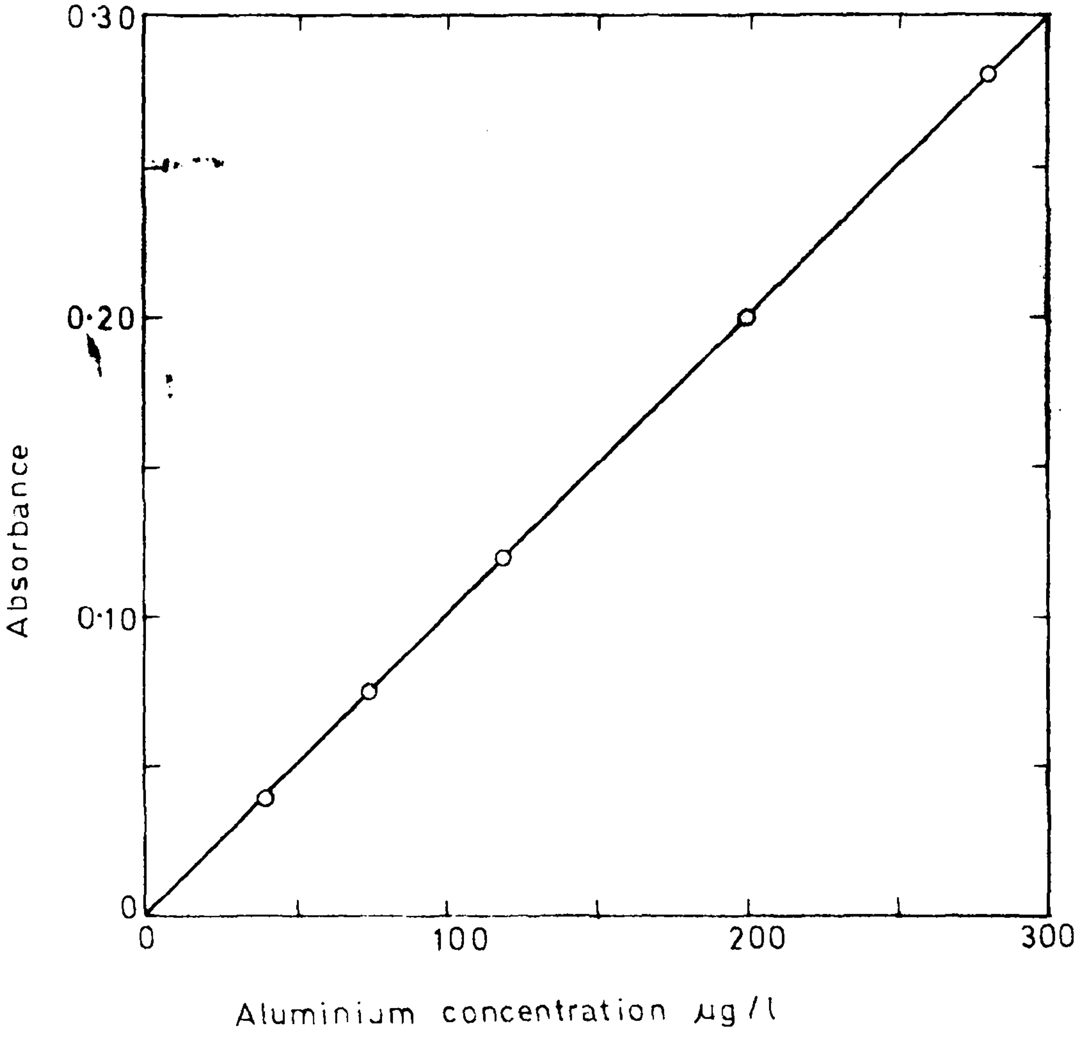
3.7.1.2 Total and Soluble Aluminium

Total aluminium was determined by using Eriochrome Cyanine-R method (Standard Methods, 1971). The complexing dye used was the product of Pfaltz & Bauer, Inc., Flushing, N.Y., U.S.A.

For determination of soluble aluminium in a water sample, the water was filtered on a AA type millipore filter, pore size of 0.80 micron, using a pyrex glass millipore filter assembly. This type of filtration was adopted as it is reported to be the most satisfactory as suggested by Shull(1967). The filtrate was then employed as sample for analysis of soluble aluminium. The standard curve for determination of aluminium concentration is given in Figure 7.

3.7.1.3 Coliform Density

The test for the presence of members of coliform group was carried out using Multiple Tube Fermentation Technique (Presumptive Test) as per the details of Standard Methods (1971). The number of positive findings of coliform group organisms resulting from multiple-portion decimal dilution plantings was recorded as MPN index by referring



[illegible]URE [illegible]. STANDARD CURVE FOR ALUMINIUM
(ERIOCHROME CYANINE R METHOD)

to tables in standard Methods (1971).

3.7.1.4 Chlorine Demand

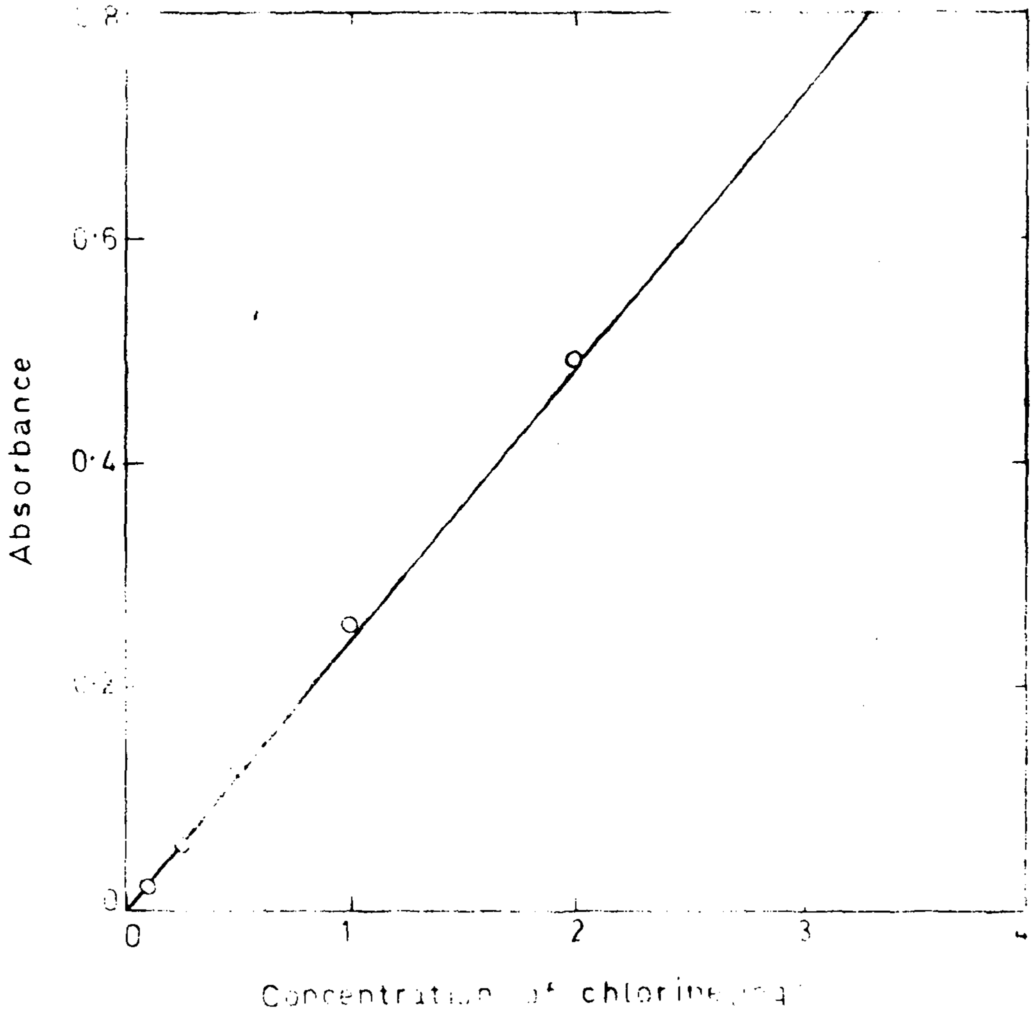
Chlorine demand was found out in the laboratory using a contact period of 30 minutes and an initial dose of 2 mg/l of chlorine. The total available chlorine was determined using N, N'- diethyl - p - phenylenediamine (DPD) as it is considered superior to neutral orthotolidine. The DPD colorimetric method was employed using a Bausch and Lomb Spectrophotometer at a wave length of 540 millimicrons (Standard Methods, 1971). The calibration curve is plotted in Figure 8.

3.7.1.5 Chemical Oxygen Demand

It was determined using dichromate reflux method (International Standards For Drinking-Water, 1963) with the modifications in the normalities of potassium di-chromate and ferrous ammonium sulfate solutions as suggested by Medalia (1951). Instead of using 0.025 N standard potassium di-chromate and 0.01 N ferrous ammonium sulfate, the normalities adopted for the two reagents were 0.01 N and 0.005 N respectively.

3.7.1.6 Algal Content

This was determined indirectly by Chlorophyll extract method as suggested by Richards with Thompson (1952) but modified by Creitz and Richards (1955) employing AA milipore filter.



[illegible] TA[illegible] AV[illegible]ABLE [illegible]
[illegible] METH[illegible]

3.7.1.7 pH Value

The pH was determined by glass electrode method using Beckman pH meter or determined colorimetrically during periods when none of the pH meter was working satisfactorily.

3.7.2 Filter Sand Layers and Schmutzdecke

3.7.2.1 Volatile Matter

The volatile matter content of filter sand and schmutzdecke was found by heating sand or schmutzdecke at 550°C for 15 minutes in a muffle furnace (Standard Methods, 1971).

3.7.2.2 Standard Plate Count

To assess the microbial population residing, the bacterial count, as determined by the standard plate count technique, was done. To find out the plate count in the sand, a known quantity of sand, obtained from sterilized petridishes used for collecting sand from layers of filter bed, was suspended into 100 ml. of sterilized water. Depending on the rough estimate of plate count, serial dilutions were done and 1 or 0.1 ml aliquot was used for plating.

3.7.2.3 Algal Content and Algae Identification

The algal content and the identification of some dominant, common and rare algal genera inhabitating the top sand layers was done. To find out the algal content of the top biological layer, indirect method of chlorophyll extract was employed. The trichromatic method for chlorophyll was used as described in Standard Methods (1971) with the difference that the biological layer was collected in petri-dishes from the known area of filter beds and the whole quantity was put into 100 ml of 90% acetone for 24 hours at 4°C.

Identification of algae was done with the help of a binocular microscope.

3.7.2.4 Identification of Macroscopic and Microscopic Forms of Animal Life Inhabitating Schmutzdecke

Some of the protozoa, rotifers, aquatic diptera worms and higher forms of biological life were identified which inhabitated the top sand layers of both the types of filter using raw water and alum-coagulated water.

3.7.2.5 Weight of Specific Deposit and Microbial Growth

To determine the weight of deposits and microbial growth accumulated in the top sand layers, these layers along with the deposits and growths from a known area of filter bed were scraped. The wet and dry weight of deposits plus microbial growth were determined after separating sand by repeated washings followed by taking weights of wet and dry sand.

3.7.3 Equipment

Except the following four proprietary models, normal equipment and glasswares were used :

(i) Spectrophotometer :

Bausch and Lomb spectronic-20 with a constant voltage transformer was used for all colorimetric methods. For determination of chlorophyll a, b and c, red filter with red sensitive phototube was used to observe readings at wavelengths between 625-960 millimicrons. Blue sensitive phototube with no filter was used in the wavelength range of 340-625 m .

(ii) Hellige Turbidimeter :

A product of Hellige, Inc., Garden City, N.Y. U.S.A., this turbidimeter was used for measurement of turbidity. Stock suspensions supplied by Hellige, Inc. were used to make standard curves.

(iii) Millipore Filter Assembly :

An all pyrex millipore filter assembly was used for filtration of water samples for determination of soluble aluminium and algal content of water samples. AA milipore filter, pore size 0.80 micron, cut in the form of disc of 4 cm diameter was used for filtration.

(iv) pH Meter :

A Beckman Expandomatic pH meter, graduated to 0.01 pH units on the expanded pH scale, was used.

3.8 Computations

(i) Specific Surface Area of Sand Used

For unstratified bed of homogeneously packed sand each component fraction, p_i, of size d_i contributes its share to the total area, the individual area - volume ratios being $6/(\psi\, d_i)$, where ψ is the sphericity (Fair et al., 1968). Assuming uniform sphericity

$$S_o = \frac{6}{\psi} \sum_{i=1}^{n} \frac{p_i}{d_i} \tag{30}$$

The necessary computations to find out $\sum_{i=1}^{n} \frac{p_i}{d_i}$ are given in Table 6. Assuming a value of 0.94 for sphericity for porosity of 0.39 as suggested by Carman (1937), the specific surface area for clean bed is computed from equation (30).

$$S_o = \frac{6}{0.94} \times 16.42 = 104.5$$

This value of S_o has been used in Kozeny-Carman equation.

(ii) Specific Deposit of Suspended Matter in Filter Bed

Generally the specific deposit in the filter was small and arithmatic computations rather than integration were used to compute the deposit.

$$\text{Specific Deposit} = \frac{(\text{Influent Conc.}-\text{Effluent Conc})\text{x Rate of flow x time}}{\text{Density of deposit x volume of bed}}$$

TABLE 6

COMPUTATION OF SPECIFIC SURFACE AREA FOR SAND USED

Size of sand, cmx10^2	Sand smaller than stated size, %	Sand Fraction within adjacent sieve sizes, p_i %	Geometric Mean Diameter, di cm x 10^2	$\frac{p_i}{d_i}$
.75	0			
		1.5	0.97	1.55
1.25	1.5			
		1.7	1.62	1.12
2.1	3.2			
		8.0	2.51	3.20
3.0	11.2			
		10.6	3.55	2.99
4.2	21.8			
		20.2	5.96	3.40
8.5	42.0			
		22.4	10.10	2.21
12.0	64.4			
		20.3	14.20	1.43
16.8	84.7			
		5.0	20.02	0.25
24.0	89.7			
		10.3	38.55	0.27
62.5	100.0			
SUM		100.0		16.42

The units adopted were conc. in mg/l, Rate of flow in litres/day, time in days and volume of bed in litres. The density of deposit was assumed to be 1.0. Consequently the specific deposit expression can be written as follows:

$$\sigma = 10^{-6} \times \frac{\text{(Influent Conc.-Effluent Conc.) x Rate of flow x Time}}{\text{Volume of bed in litres}}$$

IV RESULTS AND DISCUSSIONS - I

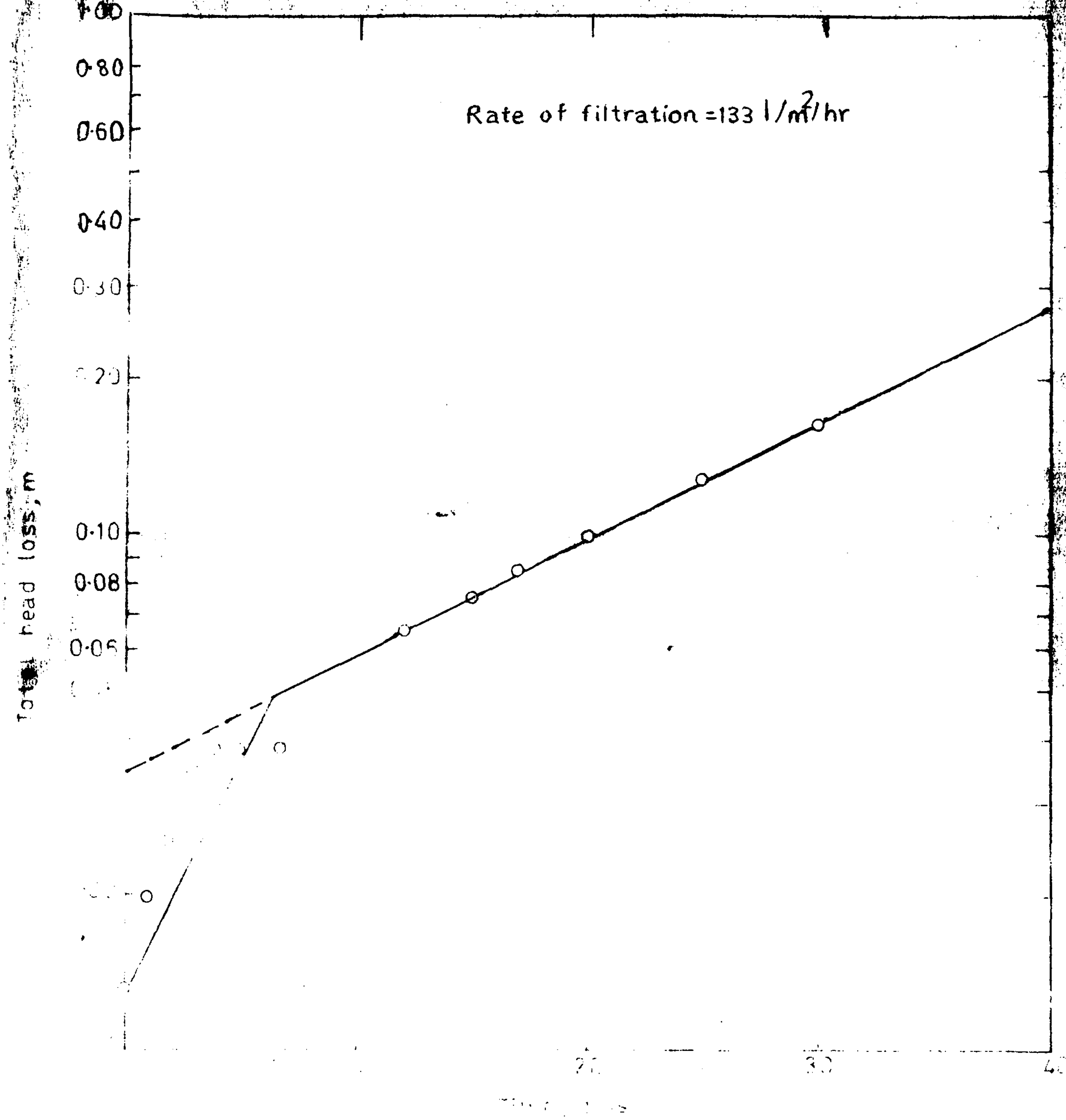
STUDIES ON SUITABILITY OF ALUM-COAGULATION AS PRETREATMENT FOR THE INFLUENTS OF SLOW SAND FILTERS

As has been mentioned earlier in section 2.1.3, alum coagulation has been considered undesirable as pretreatment before slow sand filtration as it may adversely affect the length of filter run, interfere with the functioning of schmutzdecke and under certain circumstances of pH changes, aluminium hydroxide may precipitate in the lower layers of sand.

4.1 Effect Of Alum-Coagulation On Head Loss Progression And Length Of Filter-Run

4.1.1 Pilot Filter with Alum Coagulated Influent

To observe whether alum-coagulation resulted in rapid build up of head loss due to sealing up of sand surface by alum flocs, pilot-scale filters were run at a conventional rate of filtration of 133 $1/m^2/hr$ using alum-coagulated water as filter influent. The head loss progression in one of the filters is plotted in Figure 9. The selection of log scale for the ordinate of total head loss was made following the plotting of total head loss build up against time which yielded an exponential type of curve (Agarwal and Agrawal, 1973). It is observed that the plot consists of two straight lines.


Rate of filtration =133 l/m²/hr
Total head loss, m
1·00
0·80
0·60
0·40
0·30
0·20
0·10
0·08
0·06
20
30
40

The rate of head loss build up is less in the initial phases and more in the latter phases of the filter run. The lower rate of head loss build up may be attributed to the surface deposition of positively charged alum flocs on negatively charged sand particles, and relatively low biological growth. However, in the later phases, deposition of impurities and biological growth causes the formation of a surface mat, resulting in a greater rate of head loss.

As observed from Figure 9 that the head loss even after 30 days of operation is 0.16 m compared to 0.90 m at which the run of a slow sand filter is generally terminated. The length of filter run to achieve a head loss value of 0.9 m can be determined either graphically by extending the later portion of straight line to cross the 0.9 m head loss line or arithmatically by substitution of the value of 0.9 m for head loss in the equation of later portion of straight line of Figure 9. The equation is :

$$\log h = \log (0.035) + \frac{\log (0.2) - \log (0.1)}{34-20.5}\, t$$

$$\log h = \bar{2}.5441 + 2.23 \times 10^{-2}\, t$$

From the equation for h = 0.9 m, t = 63.2 days. Therefore the length of run of pilot-scale filter using alum-coagulated water would be of the order of 60 days at a filtration rate of 133 $1/m^2/hr$ and at a terminal head loss of 0.9 m. Though as a first approximation, still to get an idea of the length of filter run at corresponding rates of filtration in the absence of any pretreatment, data of Fair, *et. al*. (1968) is reproduced below. Fair *et al*. report the length of run varying between 20 to <u>30</u> to 60 days at filtration rates ranging from 42 to <u>125</u> to 417 $1/m^2/hr$ for terminal head loss of about 1m for untreated filter influents, the underlined values being most common. On comparison it is found that the length of run of about 60 days for pilot filters at a rate of filtration of 133 $1/m^2/hr$ is more than twice in comparison to most common length of run of 30 days at a filtration rate of 125 $1/m^2/hr$ for prototype filters employing generally untreated filter influents.

As a second approximation and with a view to compare the length of run in the case of pretreatment being alum-coagulation with length of run obtained for filters using influents which have been pretreated using other unit operations like rapid filtration and microstraining, Table 7 presents data based on Ridley's (1967) works data. Ridley's data is for three water works all using different pretreatments for the influents of slow sand filter. A comparison between data in Table 7 and results of pilot-scale studies reveals that the length of run varying between 30 to 50 days, with no pretreatment to filter influents, is substantially low vis-a-vis the length of run of about 60 days for alum-coagulated filter influents even at 2-3 times increased rates of filtration when pretreatment is either rapid sand filtration or micro-straining, the length of filter run varies between 32 to 48 days compared to 60 days (rounded value) with alum coagulation as the pretreatment, the rates of filtration being roughly of the same order in the two cases.

4.1.2 Prototype Filters with Alum-Coagulated Water

As the length of filter run varies greatly with the season, it may be argued that the comparison of results obtained only from two runs on pilot filter with the average results obtained from scores of runs made on prototype filters may not

TABLE 7

SUMMARY OF RATES OF FILTRATION USED AND LENGTH OF FILTER RUN OBTAINED (RIDLEY, 1967)

Name of Water Works	Type of Pretreatment	Rate of filtration (slow sand filtration) $1/m^2/hr$	Length of Run days
Hanworth Road Works	None	44 to 59	30 to 50
Kempton Park	Rapid Sand Filtration	132 to 152	40 to 44
Ashford Common	Microstraining	132 to 137	32 to 48

be completely valid. To overcome this objection, Table 8 presents data on lengths of run of sixty three runs made on prototype filters at Kanpur Water Works using alum-coagulated waters as filter influents and operating at an average rate of filtration of 133 $1/m^2/hr$. Seven prototype filters were selected on the basis of varying depths of sand media and the period covered by the runs was more than one year (September 1971 to September 1972).

From Table 8 it is observed that the length of a filter run ranged between 13-105 days. The very low value of a filter run of the order of 15 days is primarily due to excessive algal growth on the sand surface during summer months. During summer months, both because of heavy concentration of algae in raw water and because of excessive growth on the sand surface, there is a thick mat of filamentous and filter clogging algae formed on the bed causing rapid build up of head loss and resulting in a very short filter run. However, including even these greatly reduced filter runs, it is found that the average run for various filters ranges between 33 to 50.2 with an overall average of 42.4 days. The head loss progression for an average length of run is also depicted in Figure 10.

This length of run of about 42 days at the rate of filtration of 133 $1/m^2/hr$ compares favourably with the length of filter run of about 40 to 44 days at filtration rates of 132 to 152 $1/m^2/hr$ with rapid filtration as pretreatment

TABLE 8

LENGTH OF FILTER RUNS FOR SEVEN PROTOTYPE FILTERS

Sl. No.	Number of Filter Run	Length of a Filter Run, Days						
		Filter 1	Filter 2	Filter 3	Filter 4	Filter 5	Filter 6	Filter 7
1	I	58	60	36	24	46	60	45
2	II	46	39	20	34	32	29	36
3	III	45	23	38	46	26	26	29
4	IV	37	29	26	26	38	34	20
5	V	25	24	33	34	36	36	59
6	VI	48	61	37	31	19	27	74
7	VII	38	28	13	32	49	48	60
8	VIII	105	104	21	13	63	105	46
9	IX			25	29	65		
10	X			25	76			
11	XI			69	32			
Arithmatic Mean		50.2	46	33	34.3	41.6	45.6	46.1

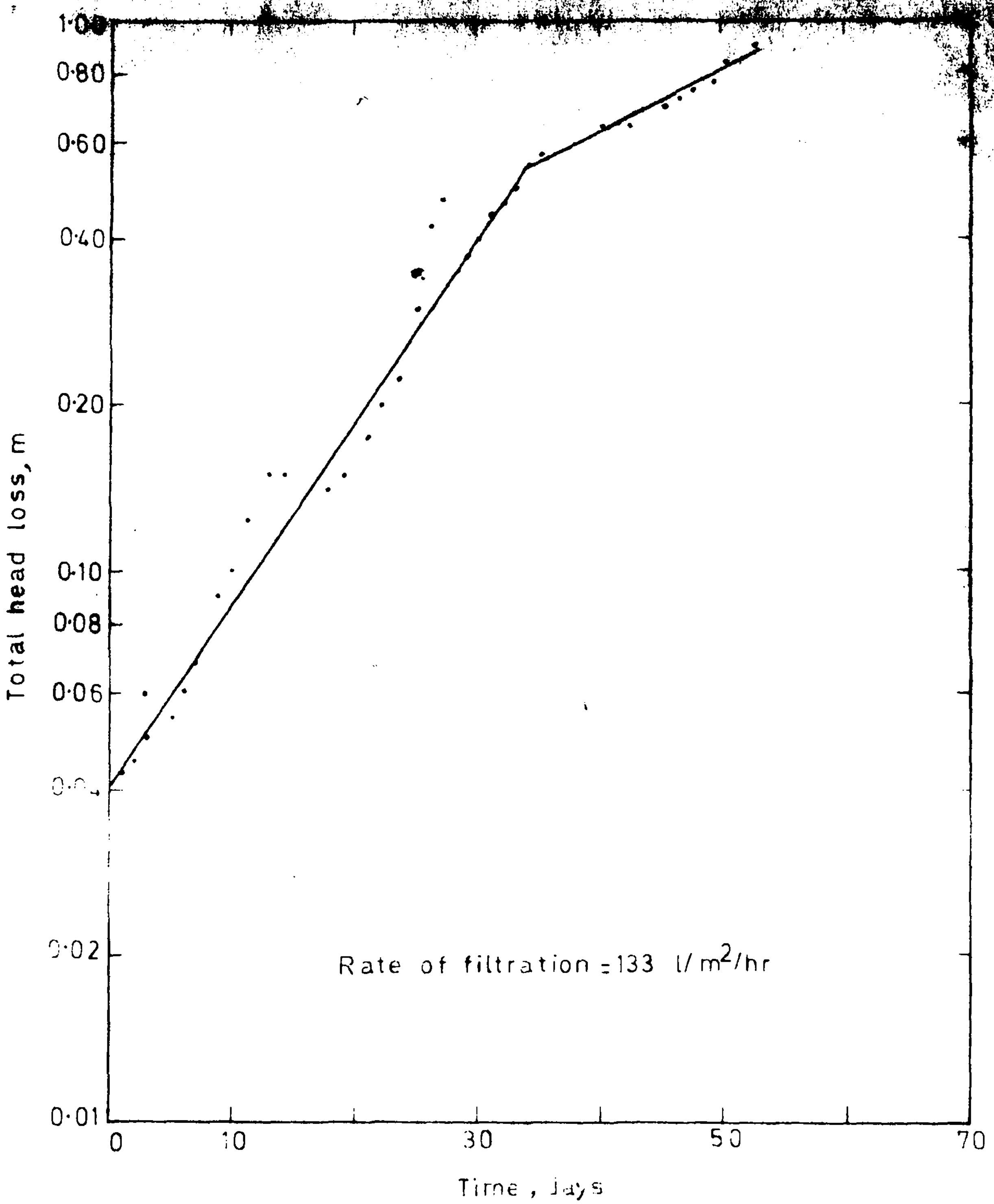


FIGURE 1[illegible]. HEAD LOSS DEVELOPMENT IN A PROTOTYPE FILTER USING ALUM-FLOCCULATED WATER AS INFLUENT and with 32-48 days at filtration rates of 132 to 137 $1/m^2/hr$ using microstraining as pretreatment.

4.1.3 Parallel runs with Coagulated and Uncoagulated Water

It must be mentioned that even the preceding comparison may be a little incomplete because of different operating conditions, constructional features and media specifications. Since the sole objective of this part of study is to evaluate the effect of alum-coagulation on head loss build up, it is essential to maintain similar operating and other conditions. Therefore, to eliminate the effect of external factors and to compare and analyze the results of filter runs rationally, fundamental aspects relating to head loss shall be considered.

From Kozeny-Carman equation, which is reproduced in the following line

$$\frac{h_o}{L} = v \ \frac{\nu}{g} \cdot K_o \ \frac{(1-f_o)^2}{f_o^{\,3}} \cdot S_o^{\,2}$$

it is observed that initial head loss depends on velocity of filtration, temperature through its effect on kinematic viscosity ν , initial porosity and specific surface area and through them on particle size, size distribution and shape and depth of filter media. As filtration proceeds, K_o, f_o and S_o all change as a result of accumulation of suspended impurities in the grain matrix. Because of the changed physical, chemical and even biological character of raw water as a result of pretreatment, the magnitudes of changes in the parameters, K_o, f_o and S_o, may be different in different cases of filter influents being untreated or pretreated by various unit operations. However, because of the complexity of situation and present state of knowledge, it is not possible to precisely know the changing values of the Kozeny-Carman constant and specific surface area and to some extent of porosity either analytically or experimentally.

To overcome all these difficulties, it is essential to consider the overall response of the filter bed in respect of head loss build up due to changes brought about by alum coagulation in the chemical, biological and physical qualities of raw water instead of considering the individual changes in porosity, specific surface area and Kozeny Carman constant. To keep the effect of external factors, other than that due to alum coagulation, on the over all response of filter relating to head loss progression, it is necessary to keep process parameters namely v , ν , f_o and S_o and L substantially same.

Therefore, in light of the preceding discussions, two parallel filter runs on pilot scale filters employing the same rate of filtration, same sand and gravel media of equal depth in both the filters were conducted. One filter received raw water as influent and the other alum-coagulated, flocculated and settled water as influent. Since head loss progression also varies with concentration of suspended matter in the influent, a suitable period of time was chosen during which the turbidity of raw water remained of the same order of magnitude as that of the alum-coagulated influent. The turbidities of raw and pretreated influent could not be maintained exactly the same in the field due to operational difficulties. This disparity in turbidity values was taken care of by plotting head loss build up against specific deposit rather than against time. The plot of head loss build up with specific deposit for the two types of filter influents is shown in Figure 11.

It will be observed that the slopes of the two lines are substantially equal, though head loss progression in the case of raw water as influent is slightly more. Also, the initial head loss in both the cases was the same and around .02 m indicating that initial over all response of the filters with respect to head loss was same because of almost identical conditions. Incidentally it may be mentioned that in the earlier phases of run in both cases, points depart from the straight line plot in both cases. This may be, as indicated earlier, due to the absence of higher forms of biological life.

From the Figure 11 and the preceding discussion it is obvious that alum-coagulation has little,if any, effect on head loss build up vis-a-vis, the absence of this pretreatment when other factors are kept same in the two cases.

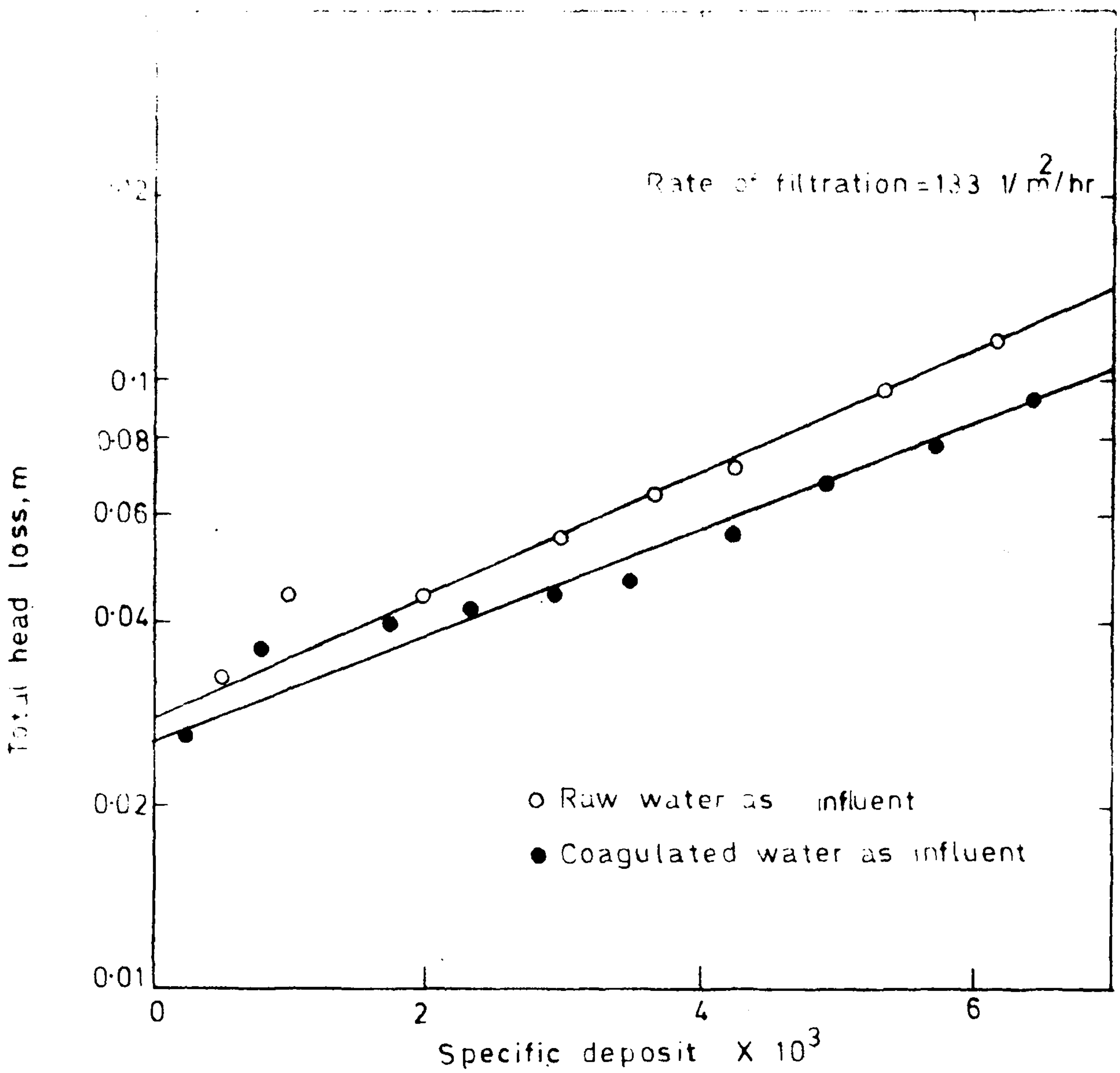


FIGURE 11. HEAD LOSS PROGRESSION WITH SPECIFIC DEPOSIT FOR PILOT FILTERS USING UNTREATED AND ALUM-COAGULATED INFLUENTS

4.2 Effect Of Alum Coagulation On Schmutzdecke And Lower Sand Layers

Large number of samples of schmutzdecke and lower sand layers removed from pilot and prototype filters were analyzed. Table 9 presents comparative results of these analyses for the two types of influents, namely raw water and alum coagulated water.

4.2.1 Algal Content

The algal content of schmutzdecke in both the cases varies between 830 to 1040 mg/m^2 and 860 to 1030 mg/m^2 respectively indicating that alum-coagulation does not retard the growth of algae on sand bed. As schmutzdecke is primarily an algal layer and its main roles are to provide enough oxygen to water to keep it well aerated and to provide straining action through the interlocked cellulose threads and siliceous bodies of algae and diatoms (Van de Vloed, 1955), algal content of this layer becomes important. Because growth of algae depends on the season and the number of days the filter has been in operation besides many other factors, samples were collected from the filters which had been operated for and during the same period of time. Care was also taken to keep the effect of other factors same as far as possible.

4.2.2 Bacterial Growth

The standard plate count in both the cases is of the same order of magnitude. The range of 10^4-10^6 has been given

TABLE 9

COMPARATIVE RESULTS OF ANALYSES PERFORMED ON SCHMUTZDECKE AND LOWER SAND LAYERS

Type of Analysis	Schmutzdecke or Lower Sand Layers removed from filter using as influent	
	Raw Water	Alum-coagulated Water
Schmutzdecke		
Algal Content, mg/m^2	830-1040	860-1030
Standard Plate Count nos./gm of Wet Weight	10^4-10^6	10^4-10^6
Volatile Matter, %	5.6-8.8	17-17.3
Thickness, mm	3-8	3-8
Time of ripening, days		
(i) for first run with washed sand	5-8	2-3
(ii) for subsequent runs	1-2	0-1
Weight of deposits and microbial growth in top sand layers, kg/m^2		
(i) Wet weight	10.7	6
(ii) Dry weight	4.5	3.2
Lower Sand Layers		
Volatile Matter %		
(i) 0-2 cm	1.1-1.4	2.3-2.9
(ii) 4-15 cm	.8-1.3	1.0-1.4
(iii) 25-30 cm	1-1.3	1.0-1.4
Standard Plate Count, nos/gm of wet weight 5 cm - 25 cm	10^4-10^5	10^4-10^5

to indicate that during winters, plate counts were of the order of 10^4 colonies per gm. of wet sand while during summers the values around 10^6 were more common. However during the same season, plate counts in both cases were of the same order of magnitude. It clearly indicates that alum-coagulation does not result in reduction of bacterial population inhabitating the schmutzdecke.

4.2.3 Volatile Matter in Schmutzdecke

The volatile matter content of schmutzdecke developing on sand bed of filter using raw water as influent is about half or less than the corresponding value in case of alum-coagulated influents. This is possibly due to the greater amount of clay and silt being present in the schmutzdecke of filters using raw water as influent. In comparison, as alum-coagulation followed by flocculation and settling can remove most of the clay and silt from raw water, the schmutzdecke developing in filters receiving these waters may have less inorganic matter as observed.

4.2.4 Thickness

The thickness of layer of deposits and microbial growth measured from the top of sand surface varied between 3 and 8 mm in the two cases, depending primarily on the thickness of algal mat developing therein.

4.2.5 Time of Ripening

The time of ripening of schmutzdecke was arbitrarily defined as time taken by the schmutzdecke to develop to an extent that it removed 50% or more of the turbidity of influent water filtering through it. Removal of turbidity was considered to be a better parameter to judge the performance of schmutzdecke as the efficiency of the filter itself is evaluated in terms of turbidity removed rather than bacterial removal since the advent of compulsory practice of post disinfection. It is observed that alum-coagulation results in early maturing of schmutzdecke both for the first and subsequent runs after recouping the filter with washed sand. This may be attributed to following reason. Because of the opposite charges on alum-floc particles and sand and because of the low velocity of filtration, alum-floc particles tend to be deposited on the surface of sand rather than penetrating deeper into bed. As more and more deposits accumulate on surface, the efficiency of removal of turbidity by the top sand layers improves resulting in quick ripening of schmutzdecke. However, whatever may be the reason of early ripening of schmutzdecke, it can be concluded that alum-coagulation is beneficial in making the top sand layers efficient quickly in the removal of turbidity.

4.2.6 Weight of Deposits and Growth in Top Layers

The weight of deposits and microbial growth was determined to give a comparative idea of the weight of the material to be removed in the cleaning process. This is of value where manual labour is employed in scraping filters. The wet as well as dry weight of deposits and micrøbial growth in the case of filters using raw water as influent is more than for filters using alum-coagulated influent. This may partially be due to the higher clay and silt content of schmutzdecke formed from filtration of raw water than that from alum coagulated water and density of clay and silt particles being greater than those of alum flocs.

4.2.7 Hetrotrophe Zone

The lower sand layers were also analyzed for volatile matter and standard plate count. It will be observed from the values furnished in Table 9 that alum coagulation does not affect, adversely, the microbial population inhabitating the slimy and gelatenous layers on sand particles. Since Van de Vloed (1955) has outlined the significant role played by the hetrotrophe zone, which extends down to 30 cm from the top sand surface, in bettering the quality of water, it was considered desirable to find out the bacterial population and the volatile matter both in the presence and absence of alum-coagulation to study its effect on the hetrotrophe zone.

4.2.8 Microscopic Examination

Microscopic examination of the several samples of schmutzdecke scraped from the two types of filter receiving raw and alum-coagulated waters was also undertaken to identify algal and macroscopic and microscopic forms of animals. Though it is very difficult and unjustified to predict the effect of alum coagulation on various species of microbial life, some general observations may, however, be made on the basis of the results. A detailed examination covering several samples of schmutzdecke obtained from filter receiving alum-coagulated water revealed the following types of algae.

Abundant or More Common Varities : Spirogyra, Hydrodictyon, Cladophora, Synedra, Melosira, Lyngbya, Oedogoniun

Less Common Varities : Oscillatoria, Fragilaria, Navicula, Nitzschia

These are similar to those found in filters with conventional pretreatment.

Similarly several samples of schmutzdecke were examined microscopically to identify the various forms of animal life. Some of the phyla or classes of organisms observed in both types of schmutzdecke are named as follows : Zooflagellates including colourless flagellate protozoa, Amoeba, ciliates Rotifers, Platyhelminthes, Nemato-morpha, Nematoda, Annelida (segmented worms), Aquatic Diptera including Larve of the midge and Rhagionidae and Amphibia (tadpole etc.).

Because of the presence of such a wide spectrum of animal life, it can be assumed that a dynamic equalibrium exists in both the types of filters. The presence of higher forms of life is beneficial as it keeps the head loss from building to excessive values.

4.3 Effect Of Alum-Coagulation On Filtrate Quality

To observe whether the quality of filtrate deteriorated as a result of alum coagulation, regular analyses were conducted on filtrates from both the types of pilot filters receiving raw and alum-coagulated waters. The filtrates from different prototype filters using alum-coagulated water as influent were extensively and regularly examined for turbidity and coliform density. The typical results of these analyses are presented in Table 10.

4.3.1 Effluent Turbidity

The main parameter of filtrate quality is effluent turbidity. Though the turbidity of raw water varied between 5-3000 units, the effluents from both types of filters were always turbidity free and sparkling except in one run. In this filter run using raw water as influent whose turbidity increased from 400 to 1800, the effluent turbidity varied between 0.06 to 1 units. It may be mentioned here that as a result of alum-coagulation, flocculation and settling the turbidity of raw waters which varied between 5 to 3000 units was reduced to less than 50 units and filter effluents were always turbidity free. This clearly demonstrates that alum-coagulation in conjunction with flocculation and settling allows the slow sand filters to successfully treat even very highly

TABLE 10

FILTRATE QUALITY PARAMETERS FOR FILTERS EMPLOYING RAW WATER AND ALUM-COAGULATED WATER AS INFLUENTS

Filtrate Quality Parameter	"Raw Water" Filter[1]		"Alum-Coagulated Water"[2] Filters	
	Influent	Effluent	Influent	Effluent
Turbidity, A.P.H.A Turbidity Units	5-75	Nil	3-15	Nil
	400-1800	.06-1	10-50	Nil
M.P.N. of E. Coli per 100 ml.	150 to 16,100	Nil-161	23-3500	Nil to 21
Chlorine demand, mg/l (One typical value)	0.95	0.49	0.81	0.43
COD, mg/l	0.2-5.1	-	0.2-2.1	Nil-1.0
Algal Content, mg/l	-	-	0.5-2.0	Nil

1 Filter, pilot-scale, using raw water as influent

2 Filters, both pilot and prototypes, using alum-coagulated flocculated and settled water as influent.

turbid waters.

4.3.2 Bateriological Quality

The reduction in coliform density for raw water filters varied generally between 90 to 100 % while that for alum-coagulated water filters ranged most of the time close to 95-100%. In pilot-scale filters the efficiency of coliform reduction was sometimes poor in both cases, especially with first runs with washed sand possibly due to the absence of fully developed schmutzdecke and hetrotrophe zone. But the prototype filters, generally, produced filter effluents that were almost free from coliform group of bacteria. Thus it seemed that adoption of alum-coagulation as pretreatment helped in producing filter effluents that are almost bacteria-free. A similar observation, that slow sand filters receiving alum-coagulated influents deliver effluents completely free of coliform bacteria, has been made by the national rapporteur of Spain in International Water Supply Association (Van de Vloed, 1955).

4.3.3 Organic Matter in Effluent

The chlorine demand of filter effluents in the two cases were also of the same order provided influents also had nearly same chlorine demand. The low values of COD and undetectable concentration of algae tend to confirm the view that alum coagulation as pretreatment for slow sand filter influents does not adversely affect the filtrate quality.

4.4 Precipitation Of Aluminium Hydroxide In Lower Sand Layers

As described earlier in detail in section 2.1.3, in certain circumstances of pH changes in the filter together with low pH (Below 5.5) caused by heavy alum dosing, free aluminium ions (Al^{+++}) which are present at those low pH's could react with magnesium hydroxide, deposited earlier on sand grains to precipitate aluminium hydroxide. The $Al(OH)_3$ precipitated in lower layers of sand would cause build up of head loss in these layers and scraping of filter would not be helpful.

As build up of head loss in lower layers of sand is undesirable, it is necessary to investigate whether alum hydroxide precipitates in lower layers in the normal range of pH changes and, if it does precipitate, does it precipitate in enough quantities to cause appreciable build up of head loss ? To observe whether aluminium hydroxide precipitated in lower sand layers, some theoretical aspects of aqueous chemistry of aluminium followed by results are presented.

It is known from solubility product principle and formation of polynuclear hydroxo complexes that in the region of pH less than 4, the hydrated trivalent aluminium species is the most active ion species, while in the region of pH between 4 and 6 one or more of the polynuclear multivalent cations such as $Al_6(OH)_{15}^{3+}$, $Al_8(OH)_{20}^{4+}$, $Al_7(OH)_{17}^{4+}$ and

$Al_{13}(OH)_{34}^{5+}$ may be predominant species. In the region of pH 6 to 8, $Al(OH)_3$ is the dominant species and $Al(OH)_4^-$, the aluminate ion, is believed to predominate above pH 8 (Black and Chen, 1967). Therefore, if the pH of the water is between 6 and 8, the presence of aluminium ions is ruled out and there is no possibility of Al^{3+} reacting with $Mg(OH)_2$ to form $Al(OH)_3$.

To check this, regular monitoring of pH of filter influents and effluents from different depths of filter bed showed that pH never decreased below 7 even when the dose of alum was of the order of 60 mg/l. Therefore, the precipitation of $Al(OH)_3$ in lower layers of sand due to reaction of Al^{+++} with $Mg(OH)_2$ is not possible, at least in the conditions as obtainable at Kanpur Water Works.

However due to pH changes in the filter, there is interconversion between various polynuclear hydroxo complexes and $Al(OH)_3$ (both dissolved and precipitated) since the solubility of $Al(OH)_3$ and the formation of polynuclear hydroxo complexes very much depends on pH. In slow sand filtration because of photosynthetic activity of algae (absorbing CO_2 to increase pH) metabolic activity of bacterial and other biological life (releasing CO_2 to decrease pH) and because of dilution effect, there is no fixed pattern of pH change. Because of all these factors, it may not be always possible to conclude, based on the changing profile of different forms (total, soluble and precipitated) of aluminium, that $Al(OH)_3$

does not precipitate in the lower layers. Table 11 presents the variation of different forms of aluminium in effluents withdrawn from different depths along the bed.

TABLE 11

VARIATION OF DIFFERENT FORMS OF ALUMINIUM IN EFFLUENTS FROM VARIOUS DEPTHS ALONG BED

Depth from top of bed, cm	Total Aluminium Conc. mg/l	Soluble Aluminium Conc.,mg/l	Calculated con. of precipitated Aluminium, mg/l
Above sand bed	0.23	0.10	0.13
3	0.18	0.16	0.02
9	0.18	0.15	0.03
47	0.17	0.15	0.02
102	0.16	0.14	0.02

It is observed from Table 11 that concentration of total aluminium is decreasing continuously indicating the removal of alum flocs by different sand layers. The removal of alum flocs is understandably maximum by the top sand layers including schmutzdecke. The soluble aluminium concentration rises immediately below the schmutzdecke. This is because more aluminium has gone into solution because of the increase in pH as a result of photosynthetic activities of algal mass in schmutzdecke. However in hetrotrophe zone or in lower layers of sand below schmutzdecke, pH decreases as a result of metabolic activities of primarily bacterial population and therefore the concentration of soluble aluminium remains either constant or decreases very slightly. However, it may be assumed that there is little precipitation of $Al(OH)_3$ in lower layers of sand.

A more suitable way of establishing that significant quantities of $Al(OH)_3$ are not deposited in the lower layers of sand would be to show that head loss in the lower strata of sand in the filter remains substantially constant with passage of time. Therefore, the head loss development in the lower layers of filters was also recorded in filters using alum-coagulated water as influent. Some typical values of observed head loss at the rate of filtration of 133 $1/m^2/hr$ are given in Table 12 which also gives computed values of head loss based on Kozeny-Carman equation applicable for clean and unclogged porous media.

The closeness of observed and calculated values in the lower sand layers confirms that little or no precipitation of aluminium hydroxide occurred in the lower layers of sand.

TABLE 12

OBSERVED AND COMPUTED VALUES OF HEAD LOSS

Location of layer cm below top of bed From	To	Depth of the sand layers, cm	Head Loss, cm Calculated from K-C equation	Observed after 0 days	15 days	30 days
0	3	3	.03	0.2	6.3	14.7
3	9	6	.06	0.2	0.2	0.3
9	47	38	0.37	0.4	0.5	0.5
47	90	43	0.42	0.5	0.5	0.5
	TOTAL	90	0.88	1.3	7.5	16.0

Therefore, from the results and discussions presented in this chapter, it can be concluded that the apprehensions expressed by various investigators against the use of alum-coagulation as pretreatment to the influents of slow sand filters are not justified, at least in the conditions as obtainable at Kanpur Water Works.

V RESULTS AND DISCUSSIONS - II

OPERATING SLOW SAND FILTERS AT HIGHER RATES OF FILTRATION

Based on the conclusion drawn from the results and discussion presented in Chapter IV that slow sand filters can be operated successfully with alum coagulated water without adverse effect on the length of filter run and the quality of the effluent, it was considered possible to adopt rates of filtration, much higher than conventional because of the substantially uniform quality and lower turbidity (about 10 units) of alum-coagulated filter influents.

To explore the above, the pilot scale filters were run at rates of 216, 408, 612 and 1000 $1/m^2/hr$. Later prototype filters were also operated at 612 $1/m^2/hr$. The results observed are discussed below.

5.1 Effect Of Higher Rates Of Filtration On Filter Run

Based on an earlier preliminary study extending for over an year (1970-71) on over-loading of slow sand filters, Agarwal, et al.(1972) found that a 23 percent increase in rate of filtration from 133 to 164 $1/m^2/hr$ resulted in only 8% decrease in the length of filter run.

The results obtained by operating slow sand filters both pilot and prototype at higher filtration rates are presented in Table 13. The lengths of filter run at different rates of filtration are averages of several runs both on pilot and prototype filters except in case of 216 $l/m^2/hr$.

TABLE 13

EFFECT OF HIGHER RATES OF FILTRATION ON FILTER RUN

Rate of Filtration $l/m^2/hr$	Average Length of Filter Run days	Ratio between this rate of filtration and 133 $l/m^2/hr$	Ratio between this length of run and 40 days
133	40	1	1
216	39*	-	-
408	15	3.1	$\frac{1}{2.7}$
612	10	4.6	$\frac{1}{4}$
1000	3.5	7.5	$\frac{1}{11.4}$

* Based only on one filter run on pilot filter.

In this singular case the length of run of 39 days based only on one filter run was considered as an unrepresentative value.

Assuming the rate of filtration of 133 $l/m^2/hr$ and the length of filter run of 40 days as the bases, it is observed that increases of 3.1 and 4.6 fold in rates of filtration decreased the length of filter runs to 1/2.7th and 1/4th of the basis value of 40 days. These ratios between rates of filtration and lengths of run are 7.5 and 1/11.4th respectively at the rate of filtration of 1000 l/m^2/hr.

5.2 Effect Of Higher Rates Of Filtration On Yield

Though increasing the rate of filtration decreases the length of filter run it may still result in filtering more quantities of water per filter run at increased rates of filtration. Therefore Table 14 presents data on the quantities of water filtered at various rates of filtration.

TABLE 14

YIELD OF FILTER AT DIFFERENT FILTRATION RATES

Rate of filtration l/m^2/hr.	Length of filter run days	Quantity of water filtered per run l/m^2	No. of days lost in scraping and recoupment per run	Effective Yield	
				in l/m^2/hr.	as percent ratio of filtration rate
133	40	128×10^3	3.5	122	92
408	15	147×10^3	3.5	332	81
612	10	147×10^3	3.5	454	74
1000	3.5	84×10^3	3.5	500	50

It is observed from Table 14 that although the length of run became one fourth from 40 to 10 days, increasing the rate of filtration from 133 to 612 $1/m^2/hr$ has resulted in about 15% increase in quantity of water filtered per run from $128x10^3$ to $147x10^3$ $1/m^2$. But a further increase in rate of filtration to 1000 $1/m^2/hr$ decreases the length of run to an extent that there is a net decrease in the quantity of water filtered per unit area. This indicates that filtration rates higher than a certain optimum value will not increase the output of the filter and in fact may decrease the quantity of water filtered per unit surface area as evidenced from Table 14.

Also, as a result of the reduced length of filter run, the number of scrapings of the sand surface becomes more per year and hence recoupment of filter media has also to be done more often. To account for this, number of days lost in scraping and recoupment of filters with washed media calculated for one filter run was added to lengths to filter run at various rates of filtration. In calculating number of days lost in sand scraping and recoupment, average value of 3.5 days, as applicable to full size units (size 30.5x61 m in case of Kanpur Water Works), have been adopted. This value of 3.5 days has been added to the length of filter run to get an effective length of run. The effective length of run is used in computing the effective yield of filter at different rates of filtration. It is observed that increase in the rate of filtration increases the effective yield of filter per unit area per unit time.

But the effective yield as a percentage of the corresponding rate of filtration decreases. This observation also illustrates that increase in rate of filtration after a certain optimum value does not result in a concommitant increase in the effective yield of filter.

Since one of the primary aims of increasing the rate of filtration is to reduce the land area requirements of slow sand filters, it will be desirable to review the discussion in the preceding two paragraphs from the consideration of land area. It is obvious that increase in filtration rate will result in reducing the land area requirements. Table 15 presents the data relating to land area requirements at different rates of filtration to filter one million litre of water per day.

TABLE 15

LAND AREA REQUIREMENTS AT DIFFERENT RATES OF FILTRATION

Rate of filtration $1/m^2/hr$	Effective yield of filter $1/m^2/hr$	Assumed Quantity of water to be filtered 1/day	Land Area required m^2	Percent reduction in land area required
133	122	10^6	340	0
408	332	10^6	126	63
612	454	10^6	92	73
1000	500	10^6	83	85

It is observed that increasing the rate of filtration from 133 to 1000 $1/m^2/hr$ reduces the land area required for filtering 10^6 1/day of water from 340 to only 88 m^2. It will also be observed that a 4.6 fold increase in rate of filtration from 133 to 612 $1/m^2/hr$ resulted in about one fourth decrease in land area required from 340 to 92 m^2. However a further increase of 63% in rate of filtration from 612 to 1000 $1/m^2/hr$ decrease the land area requirement by only 9.8% from 92 to 83.

5.3 Economics Of Higher Rates Of Filtration

Though an increase in rate of filtration above a certain value (around 612 $1/m^2/hr$) may not result in a corresponding decrease in land area required, it may still be argued that there is atleast some decrease. To arrive at a rational optimum rate of filtration, a complete econimic analysis is essential. To compare the cost of filtration at different rates not only in case of slow sand filtration but also that of rapid sand filtration, cost analysis was done for both types of filters at four rates of filtration in each case.

5.3.1 Basis of Cost Analysis

To compare the costs for the two types of filter, cost per million litres (m.1.) of water treated has been taken as the basis. The cost per m.l. of water treated is calculated by dividing the annual cost per filter by its effective annual yield. The total cost of filtration per filter per annum, C_T, is given by

$$C_T = C.r + C.d + C.m + O \tag{31}$$

where C is the capital cost per filter; r, d and m are the annual rates of interest, depreciation and maintenance and repairs. O is the annual operational cost per filter unit and includes the cost of sand backwashing/scraping, recoupment and overhauling of filters and the cost of man power involved in actual operation and supervision of filter.

Table 16 presents the rates of land power, water, alum and chlorine along with the annual rates of interest, depreciation, maintenance and repairs adopted for calculations to arrive at cost of filtration per unit per annum for rapid and slow sand filters.

Table 17 provides the cost data for both types of filter in accordance with equation (31) to obtain cost of filtration per filter unit per annum for prototype filters at Kanpur Water Works. All the costs are arrived on the basis of current prices (late 1972).

5.3.1.1 Cost Of Filtration at Conventional Rates

Table 18 includes sample calculations for actual quantity of water filtered per unit per annum for the rates of filtration adopted for rapid and slow sand filters at

TABLE 16

VARIOUS RATES EMPLOYED FOR COMPUTATIONS RELATING TO COST-ANALYSIS

Sl. No.	Item	Rate	Comments
1.	Land Cost	Rs. 47.60 per square meter	Rate of compensation paid by Kanpur Municipal Corporation for acquired land in the heart of the city
2.	Interest on Capital	7% per annum (p.a.)	Normal rate of interest charged by LIC & State Govt. on loans to corporations.
3.	Depreciation		
	(i)	2% p.a.	For civil works of slow sand filters with design life of 50 years
	(ii)	3% p.a.	For civil works of rapid sand filters with design life of 33.33 years
	(iii)	4% p.a.	For mechanical equipment used in rapid sand filters
4.	Maintenance & Repairs		
	(i)	1% p.a.	For slow sand filters
	(ii)	2% p.a.	For rapid sand filters
5.	Power	Rs. 0.10 per unit	Rates at which Kanpur Water Works purchases/ sells the particular item.
6.	Water	Rs. 0.16 per 1000 l	
7.	Alum	Rs. 280 per ton	
8.	Chlorine	Rs. 1.50 per Kg.	

TABLE 17

COST DATA ON SLOW AND RAPID SAND FILTERS

Sl. No.	Item	Slow Sand Filter (size 30.5x61m) Capital Cost			Rapid Sand Filters (size 7.6x7.6m) Capital Cost		
		Capital Cost	Annual cost on interest, depreciation and maintenance etc.	Annual operational Cost	Capital Cost	Annual cost on interest, depreciation and maintenance etc.	Annual Operational Cost
		Rupees	Rupees	Rupees	Rupees	Rupees	Rupees
1.	Land	88,880	6,222	-	2,890	202	-
2.	Civil Works	1,92,949	19,099	-	1,26,000	15,120	-
3.	Mechanical Equipment	1,000	110	-	1,08,000	14,040	-
4.	Sand						
	(i) Scraping/Backwashing*		-	1,600	-	-	11,316
	(ii) Recoupment	-	-	1,280	-	-	2,175
5.	Operation	-	-	1,333	-	-	2,940
6.	Supervision	-	-	100	-	-	655
	TOTAL	2,82,829	25,431	4,313	2,36,890	29,362	17,086

* Includes cost of scraping and carting to sand washing machine in case of slow sand filter and cost of power on pumping air and water and cost of backwash water (2 percent of filtered quantity) for rapid sand filter .

TABLE 18

SAMPLE DATA ON QUANTITY OF WATER FILTERED AND COST OF FILTRATION FOR SLOW AND RAPID SAND FILTERS

Sl. No.	Item	Type of Filter	
		Slow (Conventional Rates)	Rapid (Conventional Rates)
1.	Quantity of Water Filtered		
	(i) No. of days lost in scraping/backwashing and overhauling, days/year/filter	32	14.4
	(ii) Total quantity of water filtered, m.l./year/filter	1980 (@ 133 $1/m^2/hr$)	2863 (@ 5880 $1/m^2/hr$)
2.	Cost of Filtration		
	(i) Rupees	29,744	46,448
	(ii) Rupees/m.l.of water	15.02	16.23
3.	Other Costs, all in Rs./m.l.of water		
	(i) Alum (av.dose=35 mg/l)	9.80	9.80
	(ii) Chlorine	3.00 (av.dose=2mg/l)	3.75 (av.dose=2.5 mg/l)
	(iii) Extra power cost on lifting the water to av. value of head loss	0.13	0.39
4.	Total Cost of Filtration		
	(i) Without alum-coagulation	18.15	-
	(ii) With alum-coagulation	27.95	30.17

Kanpur Water Works. Data on cost of filtration, cost of alum, cost of chlorine at annual average doses of 2.5 and 2.0 mg/l for rapid and slow sand filters respectively, and cost on power required to lift the water to a head equal to the average head loss in filtration.

5.3.1.2 Cost of Filtration at Higher Rates

Cost of filtration is an inverse function of the effective yield of the filter which is a function of the rate of filtration. Therefore, changes in filtration rates will affect cost of filtration very markedly. Though cost of power and of post chlorination will also be affected by changes in rate of filtration because of changed head loss characteristics and filter effluent quality, such variations may be relatively small and for the present study it will be assumed that values given for these items in Table 18 remain constant at higher rates of filtration. Higher rates of filtration reduce length of filter run as shown in Table 13. In terms of cost, increasing the

rate of filtration will not affect the cost on interest, will affect the cost on depreciation and maintenance to some extent, which may sometimes be negligible, and will increase the operational cost quite markedly. The overall cost of filtration decreases initially to attain an optimum value and then increases continuously with increase in filtration rate.

5.3.2 Cost of Filtration at Different Rates for Rapid and Slow Sand Filters

Based on data presented in Tables 17 and 18, costs of filtration at four different rates of filtration in both the cases of rapid and slow sand filters were computed and are furnished in Table 19 and 20.

TABLE 19

SUMMARY OF COST DATA FOR RAPID SAND FILTRATION

Rate of Filtration $1/m^2/hr$	Length of filter run, days	Cost per million litres of water in Rupees: Filtration						
		Cost on interest etc.*	Operational cost	Total cost	Power	Alum	Chlorine	Total
5,000	1.56	11.85	4.46	16.31	0.39	9.80	3.75	30.25
5,880	1.00	10.25	5.98	16.23	0.39	9.80	3.75	30.17
8,820	0.52	7.06	7.95	15.01	0.39	9.80	3.75	28.95
11,760	0.31	5.85	11.06	16.91	0.39	9.80	3.75	30.85

* Includes cost on interest, depreciation and maintenance and repairs.

The total cost of water treated is nearly same for rates of filtration of 5000 and 5880 $1/m^2/hr$. The first rate is the standard rate of filtration generally recommended for rapid sand filtration while a value of 5,880 is currently being adopted at Kanpur Water Works. The rate of 8,820 $1/m^2/hr$ is presently being favoured and adopted by modern water works with excellent facilities of quality control. The last rate of 11,760 $1/m^2/hr$ was included in Table 19 to show that increased rate of filtration may result in increased cost of water treated besides creating other problems of better quality control and operational problems.

TABLE 20

SUMMARY OF COST DATA FOR SLOW SAND FILTERS

Rate of filtration $1/m^2/hr$		Length of run days	Cost per million litres of Water, Rupees Filtration						
			Interest etc.	Operational	Total	Power	Alum	Chlorine	Total
Conventional	133	40	12.85	2.17	15.02	.13	-	3.00	18.15
	133	40	12.85	2.17	15.02	.13	9.80	3.00	27.95
Alum Coag	408	15	5.15	2.18	7.33	.13	9.80	3.00	20.26
Influents	612	10	3.71	2.52	6.23	.13	9.80	3.00	19.16
	1000	3.5	4.27	5.23	9.50	.13	9.80	3.00	22.43

A comparison of data presented in Table 19 and 20 reveals that overall cost of filtration employing slow sand filtration is always less than the cost for rapid sand filtration for conditions as obtainable at Kanpur Water Works. If the cost of treatment for the two types of filter is compared for Kanpur Water Works, slow sand filtration costing Rs. 27.95 has only slight edge over rapid sand filtration costing Rs. 30.17.

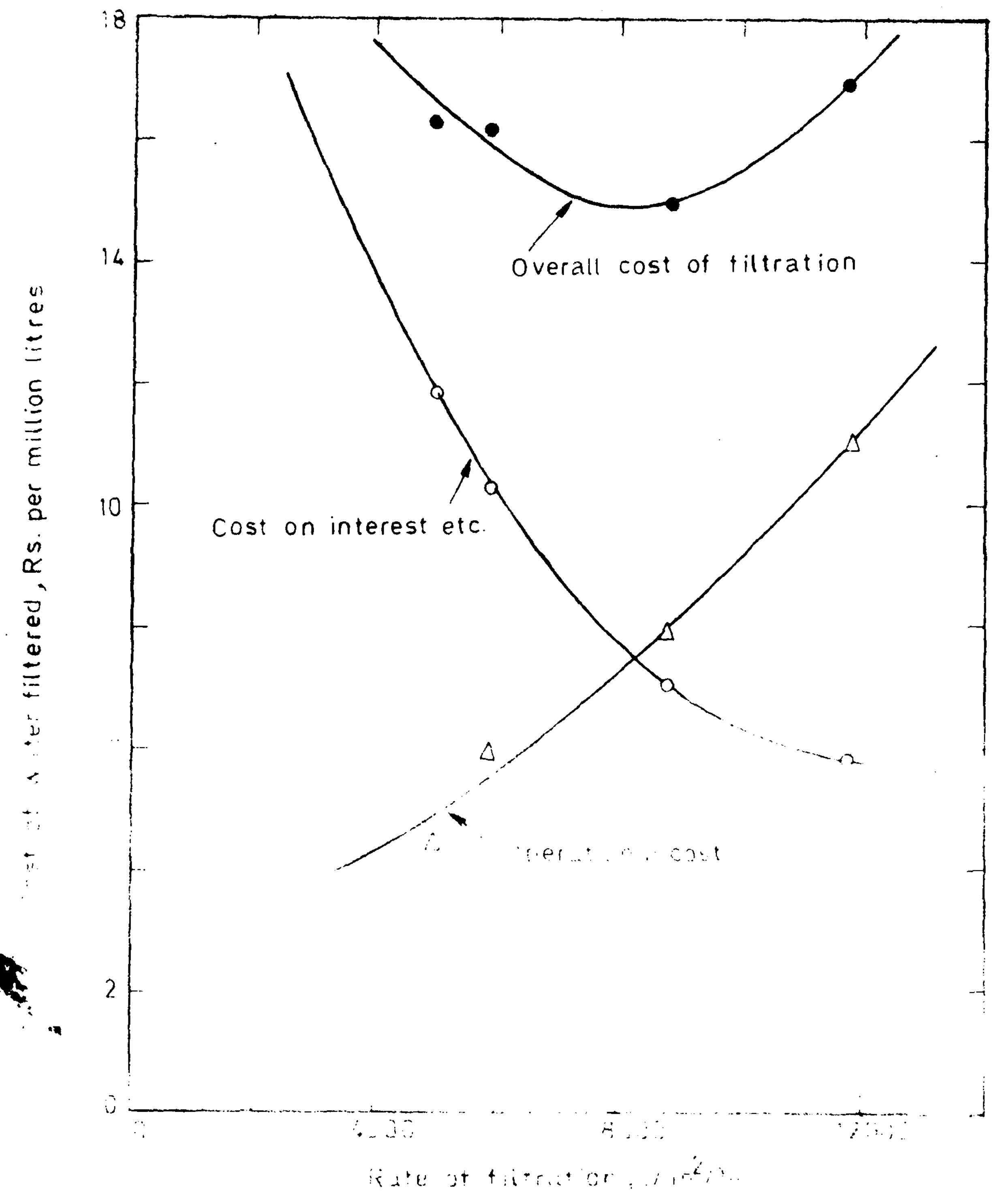
A comparison of conventional slow sand filtration (i.e. using raw water as influent without pretreatment of alum-coagulation) at the rate of 133 $1/m^2/hr$ with rapid sand filtration at conventional rate of 5000 $1/m^2/hr$ shows that the former is much cheaper at Rs. 18.15 than the latter at Rs. 30.25. It will be more suitable to compare the rates of filtration of 612 and 8,820 $1/m^2/hr$ for slow and rapid sand filters respectively. This is because these rates of filtration are higher above the conventional rates and the cost of filtration at these rates is least in both the Tables 19 and 20. It will be seen that at optimum conditions of operation slow sand filtration at a rate of 612 $1/m^2/hr$ is 51.2% cheaper than the optimal rapid sand filtration at a rate of 8,820 $1/m^2/hr$. Based on a saving of 51.2% the actual amount of money saved for a filtration capacity of $100x10^6$ 1/day would amount to Rs. 35,750 per annum or Rs. 1,790,000 for the design life of 50 years for slow sand filters.

To find out the optimal rate of filtration both for rapid and slow sand filters, the cost response curves for both the cases are presented in Figures 12 and 13. It is observed that over all cost of filtration achieves a minimal value of Rs. 14.90/m.l. of water filtered around a rate of 8100 1/m^2/hr for rapid sand filters and a minimum value of Rs. 6/m.l. of water filtered for slow sand filters at a rate of 625 1/m^2/hr. Therefore, rates of 8100 and 625 1/m^2/hr can be assumed to be the optimal rate of filtration for rapid and slow sand filters respectively in conditions as obtainable at Kanpur Water Works.

5.4 Distribution Of Removals And Head Loss Along Depth At Higher Rates Of Filtration

Though higher rates of filtration will be highly desirable as they substantially reduce the land requirements and cost of treatment, but it should be ensured that the removal of impurities and hence build up of head loss is confined to top layers of sand. If the impurities travel deep down in the bed because of increased rate of filtration, the quality of the effluent will deteriorate and head losses in the lower layers of bed will progressively increase and scraping will not be effective in removing the impurities to restore the filter.

To check the distribution of removals and head loss, the variation of filtrate turbidities and head loss build up



GURE ... COST RE... FOR RAP...
SAND FILTER

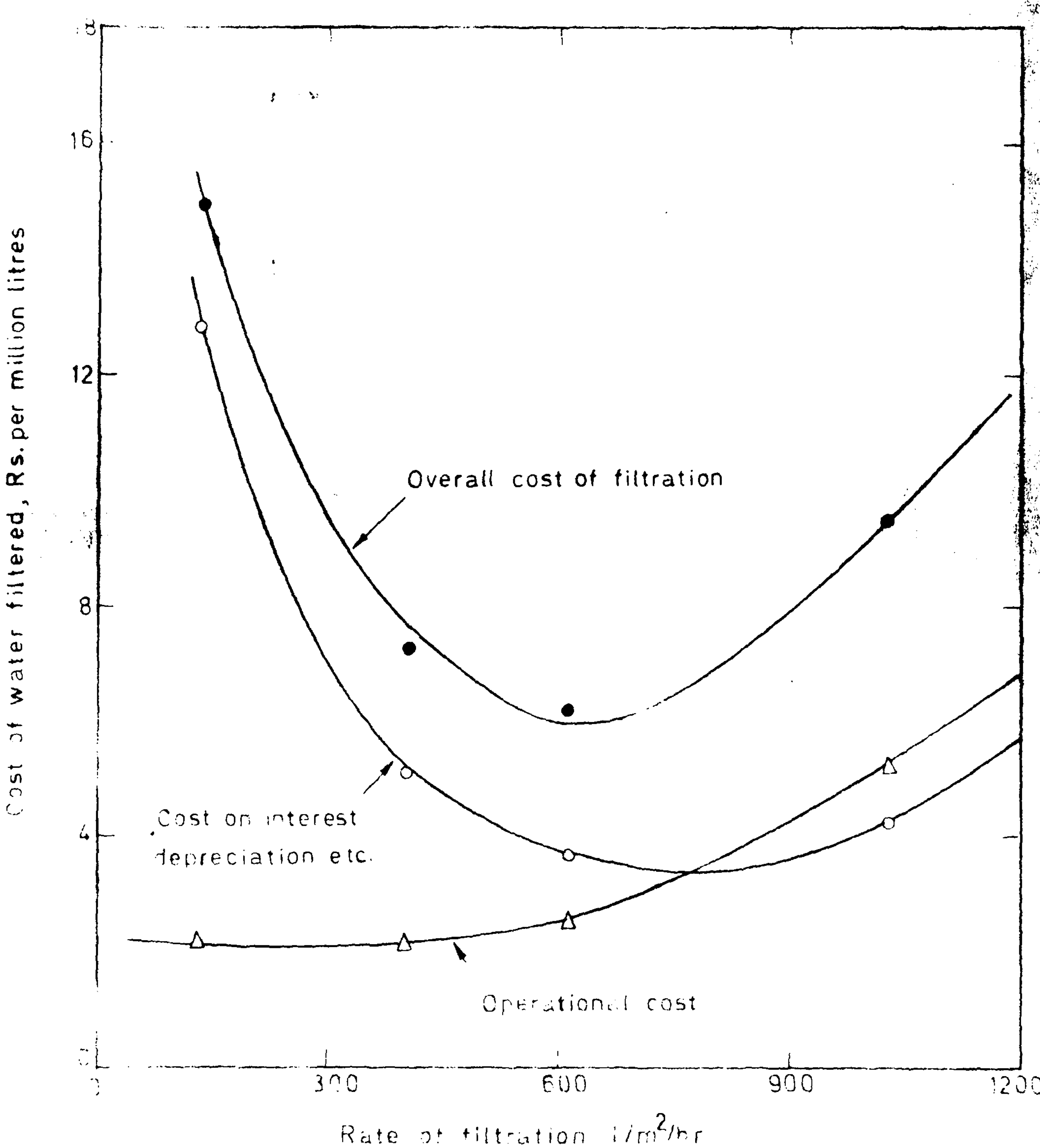


FIGURE 13. COST RESPONSE CURVES FOR SLOW SAND
FILTERS

along the depth of bed is plotted in Figures 14 and 15 for four rates of filtration, namely 133, 216, 612 and 1000 1/m^2 hr. The nature of curves confirm that with alum coagulation pretreatment, as proposed by the investigator, almost all the removals are distributed in the top 10 cm. or so and little impurities penetrate deeper in the bed even at the highest flow rates. From Figure 14 it is observed that the effluent contained little or no turbidity almost from the start of the filter run at all the rates of filtration. Determination of nonfiltrable residue also confirmed that filter effluents contained little suspended solids.

The pattern of the head loss curves in Figure 15 indicates that majority of head loss occured in the top 5 cms or so. The head loss in the lower layers remained constant as evidenced by the substantially parallel portions of head loss curves for lower layers and head loss in lower layers was close to the values computed from Kozeny-Carman equation for clean and unclogged bed of porous media. Thus from Figures 14 and 15 it is established that little impurities travelled deeper into the bed even at the highest rate of filtration till the last day of filter run.

5.5 Effect Of Higher Rate Of Filtration On Filtrate Quality And Schmutzdecke

Partly because of the importance of rate of filtration of 612 1/m^2/ hr being close to the optimum value, and partly

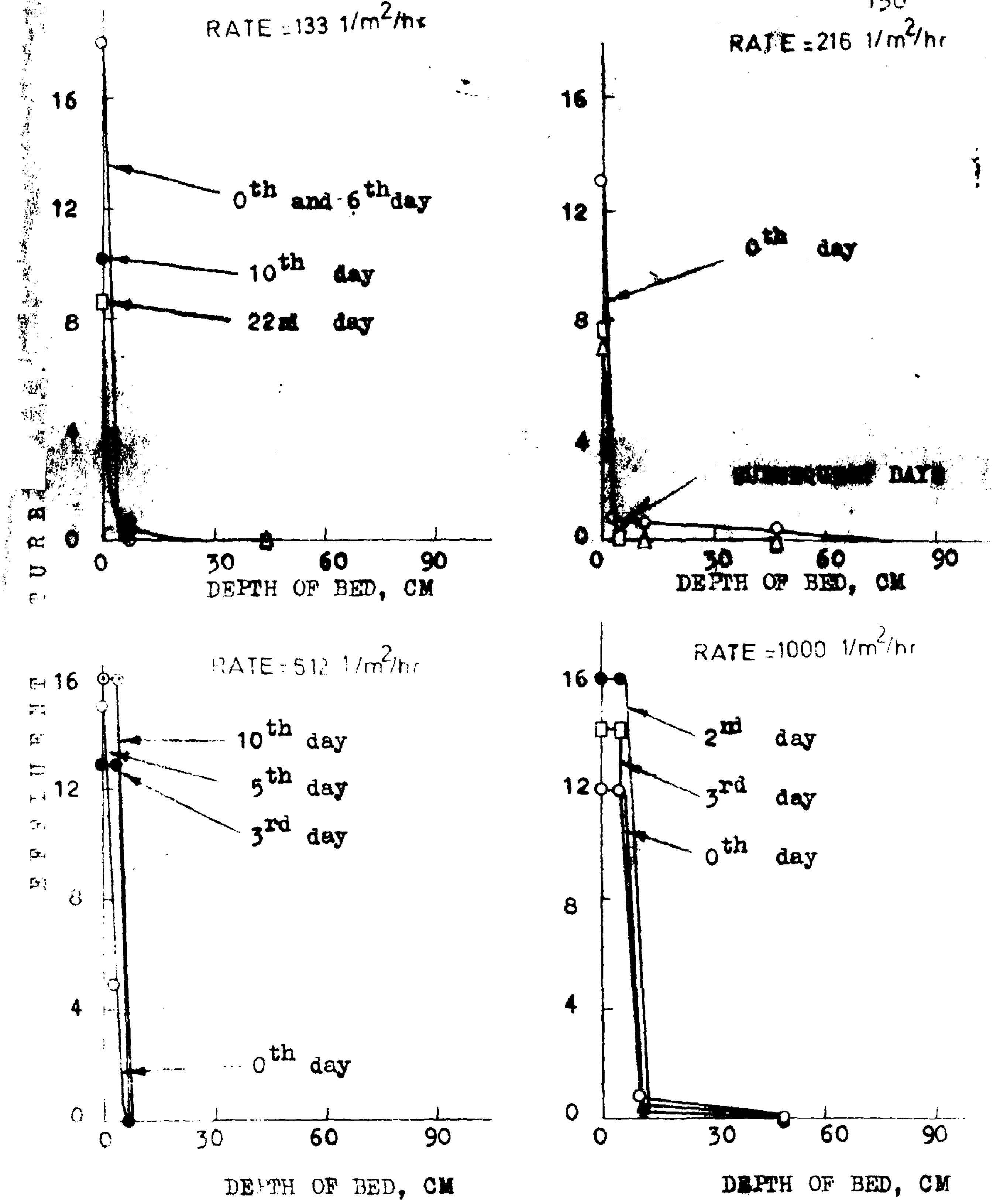


FIGURE 14. VARIATION OF EFFLUENT TURBIDITY WITH DEPTH AT FOUR RATES OF FILTRATION

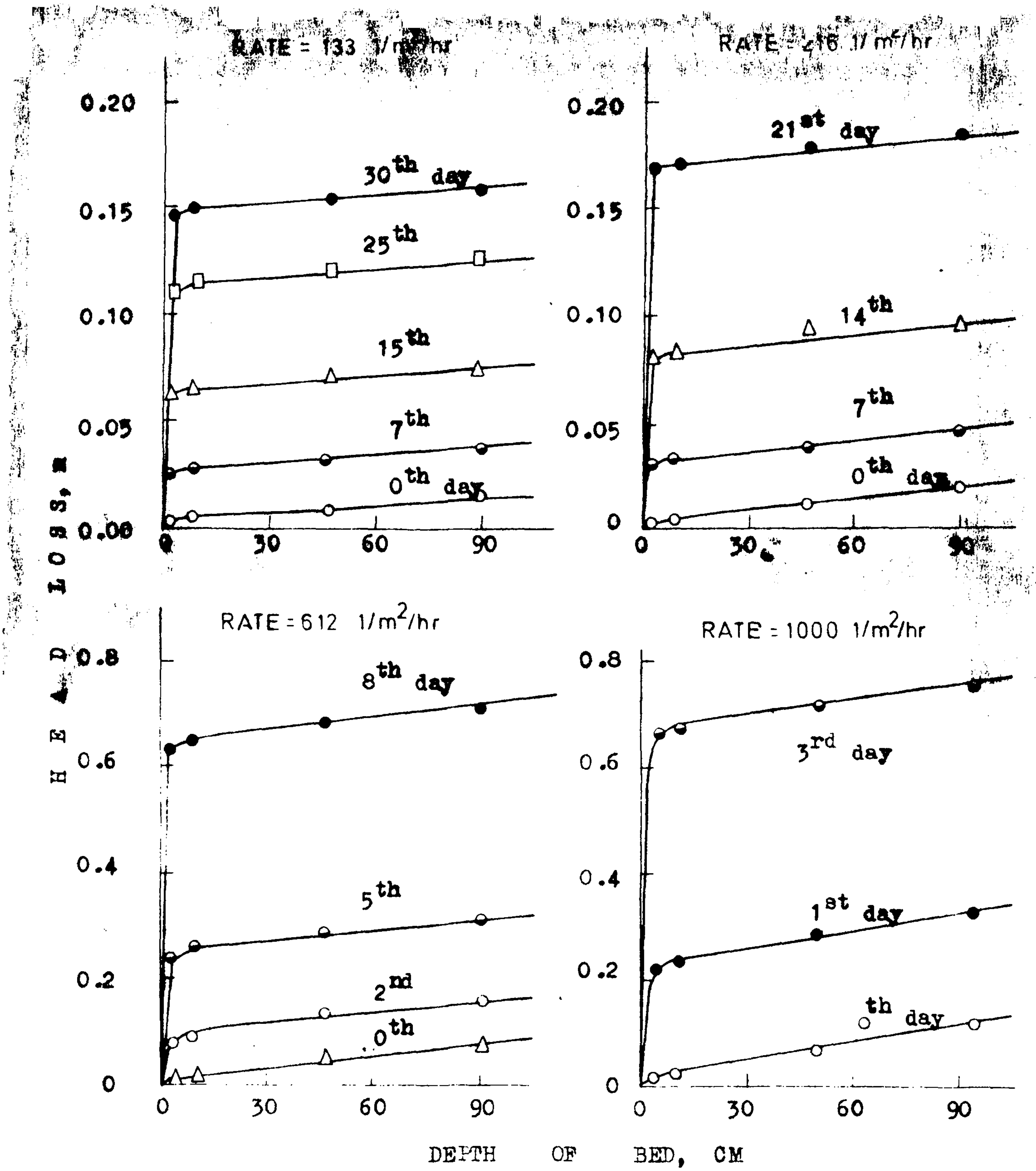


FIGURE 15. PATTERN OF HEAD LOSS DISTRIBUTION AT FOUR RATES OF FILTRATION

with a view to present the typical and representative trends, the effect of only this rate of filtration on filtrate quality is discussed. The results of various analyses performed on influents and effluents of filters operating at a rate of 612 $1/m^2/hr$ are presented in Table 21.

TABLE 21

PERFORMANCE OF FILTER AT A FILTRATION RATE OF 612 $1/m^2/hr$

Quality Parameter	Influent	Effluent
Turbidity, APHA units	7-15	Nil
M.P.N. of E.Coli, per 100 ml.	930-3500	35-92
COD, mg/l	4.8	1
Algal content, mg/l	0.9	Traces
Non-filterable residue, mg/l	5.22	Traces

Comparison of results of Table 21 with those at the rate of filtration of 133 $1/m^2/hr$ in Table 10 shows that except in coliform density, the effluent at the higher rate is similar in quality to that at lower rate of 133 $1/m^2/hr$. Because of the compulsory practice of post disinfection, even a higher coliform density of about 90 per 100 ml (against upto 35 at conventional rates) can be accepted, especially when effluents from rapid sand filters having an M.P.N. of E. Coli of 161 and greater are found entirely acceptable at Kanpur Water Works.

As a result of several analyses of schmutzdecke scraped from filters operating at 612 $1/m^2/hr$, it was found that average volatile matter and algal content of schmutzdecke was of the order of 2% and 120 mg/m^2 respectively. These values compare poorly with the corresponding values of about 17% and 900 mg/m^2 respectively. This is primarily due to little or no growth of filamentous algal at higher rate of filtration because of highly reduced length of run of only 10 days. This observation is based on physical and microscopical examination of schmutzdecke and is supported by the findings of Ridley (1967) who reported that filamentous algae grow effectively only after 10 days of filter operation. Little growth of filamentous algae at the filtration rate of 612 $1/m^2/hr$ may be an added advantage because as observed by Ridley (1967) filamentous algae increase the cost of cleaning substantially when they grow proliferously due to longer filtration runs. Another advantage of short run of 10 days at the rate of 612 $1/m^2/hr$ is that there is little possibility of disintegration of algae and attendant taste and odour problems in the effluent (Ridley 1967, Van de Vloed, 1955).

5.6 Operating Prototype Filters At Higher Rates Of Filtration

Encouraged by the results obtained by operating pilot filters at higher rates of filtration, prototype filters were also run at higher filtration rates. Only the results obtained at the filtration rate of 612 $1/m^2/hr$ are presented for discussion.

The average length of run was found about ten days, the same value as in the case of pilot scale filters.

The turbidity of the effluent was always nil except for a brief period in the beginning of the first run when prototype filters were first operated at higher rate of 612 $1/m^2/hr$, the effluent was more turbid than the influent for a period of about one hour. This is because adoption of higher velocity of filtration resulted in resuspension of impurities retained earlier in the sand matrix. But after this initial period of about 1 hr. effluents became sparkingly clear with turbidity values being zero.

The coliform density of effluent was, most of the time, close to 35/100 ml though higher values were also obtained occasionally. However, the coliform density of the effluent from these filters was considerably less than that of the effluents from rapid sand filters.

The chemical oxygen demand and chlorine demand were also low and of the same order of magnitude as of the effluents from prototype slow sand filters operating at a rate of $133/1/m^2/hr$. The algal content and the suspended solids in the effluent at the rate of 612 $1/m^2/hr$ were in undetectable concentration.

All these results confirm that there is practically no difference in the quality of effluents obtained from pilot and prototype filters operating at the rate of 612 $1/m^2/hr$.

5.7 Advantages Of Operating Slow Sand Filters At Higher Rates Of Filtration

From the results presented in this chapter, it is observed that the following advantages result from operating slow sand filters at rates of filtration much higher than the conventional rates.

(i) Reduction in land area requirement : By operating filters at 612 $1/m^2/hr$, the land area required is only one-fourth than that at conventional rate.

(ii) Increase in quantity of water filtered : The effective yield of the filter per unit area per unit time increases 3.7 times by increasing the rate from 133 to 612 $1/m^2/hr$.

(iii) Over all economy in cost of water treated : The cost per million litres of water treated is Rs. 19.16 at near optimal rate of slow sand filtration compared to Rs. 28.95 for near optimal rate of rapid sand filtration, indicating an over all saving of about 51%. It may be mentioned that treatment cost for slow sand filtration using uncoagulated influents is Rs. 18.15. This leads to the observation that adoption of higher rates of filtration more or less offsets the additional cost incurred on alum-coagulation . This advantage is in addition to the very important advantages of filtering much more water per unit area per unit time even when the turbidity of raw water fluctuates within an extremely wide range which is unacceptable in case of conventional slow sand filters.

(iv) Effluent quality : The effluent quality at higher rate of filtration is not adversely affected. Though the coliform density may be somewhat higher at higher rates, it is still much less than that of the effluents of rapid sand filters.

(v) No need of backwashing : Since almost all the removals and head loss occur in top few cm of sand bed even at the highest rate tried, the scraping effectively restores the filter. Therefore, there is essentially no need of backwashing the filters. This is a very important advantage as backwashing requires sophisticated equipment, complicates the hydraulic design of filters and is comparatively very costly.

(vi) Absence of taste and odour : As a result of reduced length of run at higher filtration rates, there is little possibility of disintegration of algae and consequently no taste and odour are detected in the effluents. On the contrary, the problem of taste and odour in effluents has been reported at conventional filtration rates.

(vii) Reduction in growth of filamentous algae : Again, as a result of reduced length of run of 10 days, there is little time for excessive growth of filamentous algae. As a result, scraping and cleaning of the sand is more easy and quick and may be more advantageous economically also .

It can be concluded that as a result of adopting alum-coagulation as pretreatment, the slow sand filters can be operated at much higher rates of filtration and can cope up with even highly turbid waters. Adoption of higher rates of filtration greatly reduces the land area requirements and thereby resulting in much lower cost of construction. Therefore, by adopting the proposed pretreatment of alum coagulation to slow sand filter influents and by operating these filters at filtration rates much higher than conventional, most of the disadvantages of slow sand filters can be eliminated or reduced to a very large extent.

VI. RESULTS AND DISCUSSIONS - III

DEVELOPMENT OF MATHEMATICAL MODEL FOR BUILDUP OF HEAD LOSS

6.1 Variation Of Head Loss With Depth

The variation of head loss progression with depth has been depicted earlier in Figure 15. Examination of various plots at different rates of filtration reveals that the total head loss at any time can be split into two distinct components. First is the head loss in the top sand layers, extending upto about 6 cms or less from the top of sand bed and second is the head loss in the remaining depth of the bed. Mathematically, it can be expressed as follows :

$$h = h_s + h_l \qquad (32)$$

where h is the total head loss at any time and h_s and h_l are the head losses in the top surface layers and the remaining depth of sand bed.

It is observed from Figure 15 that while h_t is time dependant, h_l remains sustantially constant with time. The head loss in top layers, h_s, is a function of the nature and concentration of the suspended particles removed and microbial population developing, as also of media characteristics and velocity of filtration. The head loss in lower layers is dependent on all the variables in the Kozeny-Carman equation namely Kozeny-Carman constant, specific surface area, porosity and kinematic viscosity. In addition, the little accumulation of suspended matter and biological growth, if any, may also affect the head loss in lower sand layers.

The headloss in the lower layers can be written as :

$$h_1 = (\frac{\partial h}{\partial l}) . l \qquad (33)$$

where $\frac{\partial H}{\partial l}$ is the head loss gradient in the lower layers of sand. On the basis of results plotted in Figure 15, it was concluded that head loss in lower layers of sand did not differ significantly from the values computed from Kozeny-Carman equation. Any increase in the observed values over the computed values may be attributed to the little accumulation of suspended impurities and consequent microbial growth in the lower layers of sand. Therefore, in a general way :

$$\frac{\partial h}{\partial l} = (\frac{\partial h}{\partial l})_o \times A \qquad (34)$$

where $(\frac{\partial h}{\partial l})_o$ is the head loss gradient for clean and unclogged bed and A is constant to reflect the accumulation and consequent growth in the lower layers. The value of A may be equal to or greater than unity. Based on computation from Kozeny-Carman equation and Figure 15, the average values of A at four different filtration rates of 133, 216, 612 and 1000 $l/m^2/hr$

are 1.2, 1.01, 1.84 and 1.44 respectively.

6.2 Head Loss Progression In Top Sand Layers

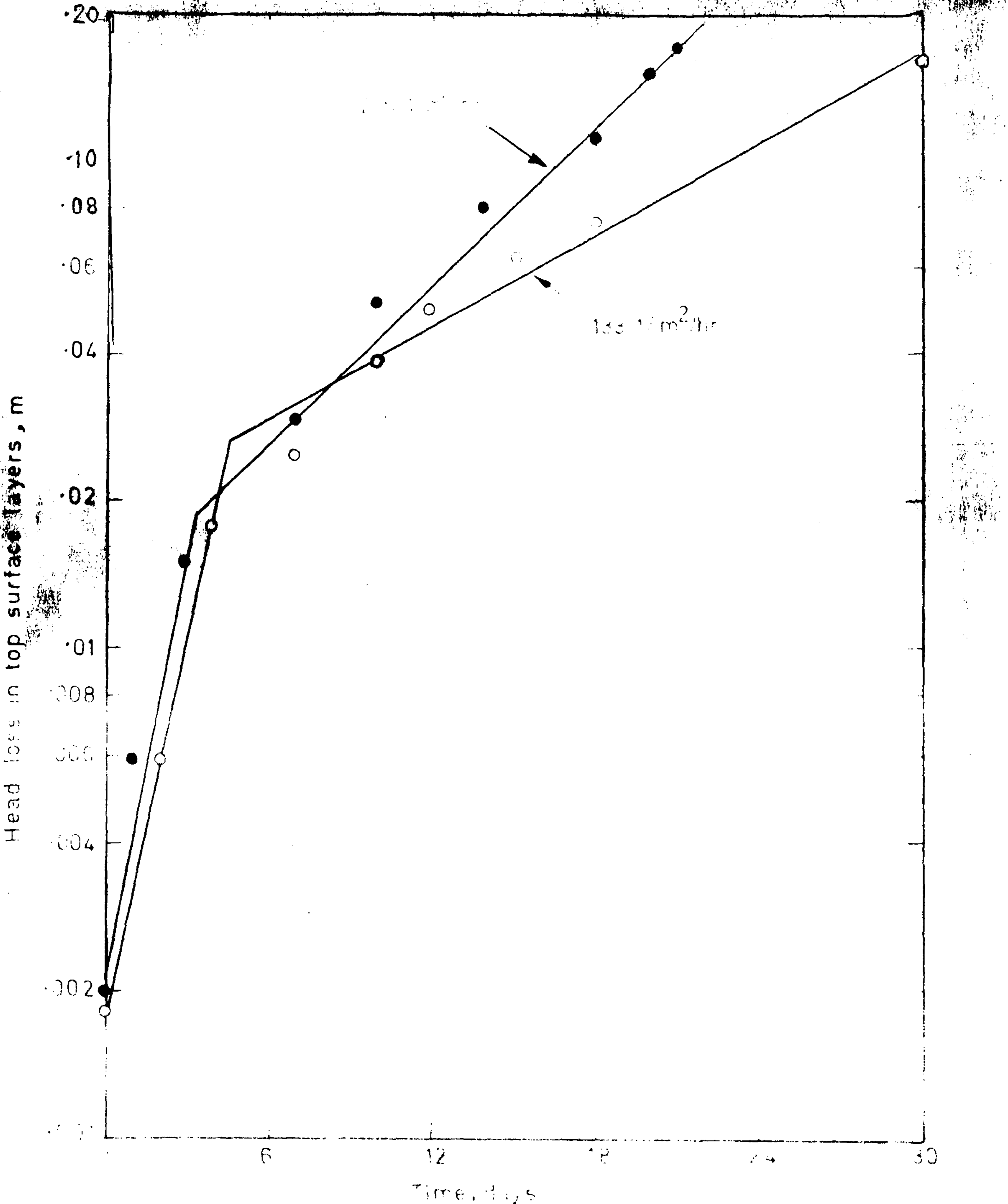
To investigate the progression of head loss in top surface layers with time, h_s for different rates of filtration is plotted in Figure 16 and 17. It is observed that the head loss progression in top sand layers plots as a series of two straight lines at different rates of filtration.

The first line is steeper than the second line at all the rates of filtration as seen from Figures 16 and 17. The steepness of the first line compared to second one should not be interpreted to mean that the rate of head loss build up in the earlier phases of the run is more than that in the later phases. On the contrary, the reverse is true as evidenced from the results of Table 22.

TABLE 22

RATE OF HEAD LOSS BUILD UP DURING EARLY AND LATER PHASES OF A FILTER RUN AT DIFFERENT FILTRATION RATES

Rate of Filtration	Rate of Head loss build up x 10^3, m/day	
$1/m^2/hr$	During early phase	During latter phase
133	2.9	3.2
216	4.3	6.7
612	26.0	86.3
1000	200.0	212.5



·20
·10
·08
·06
·04
·02
·01
Head loss in top surface layers, m
6
12
30
Time, days

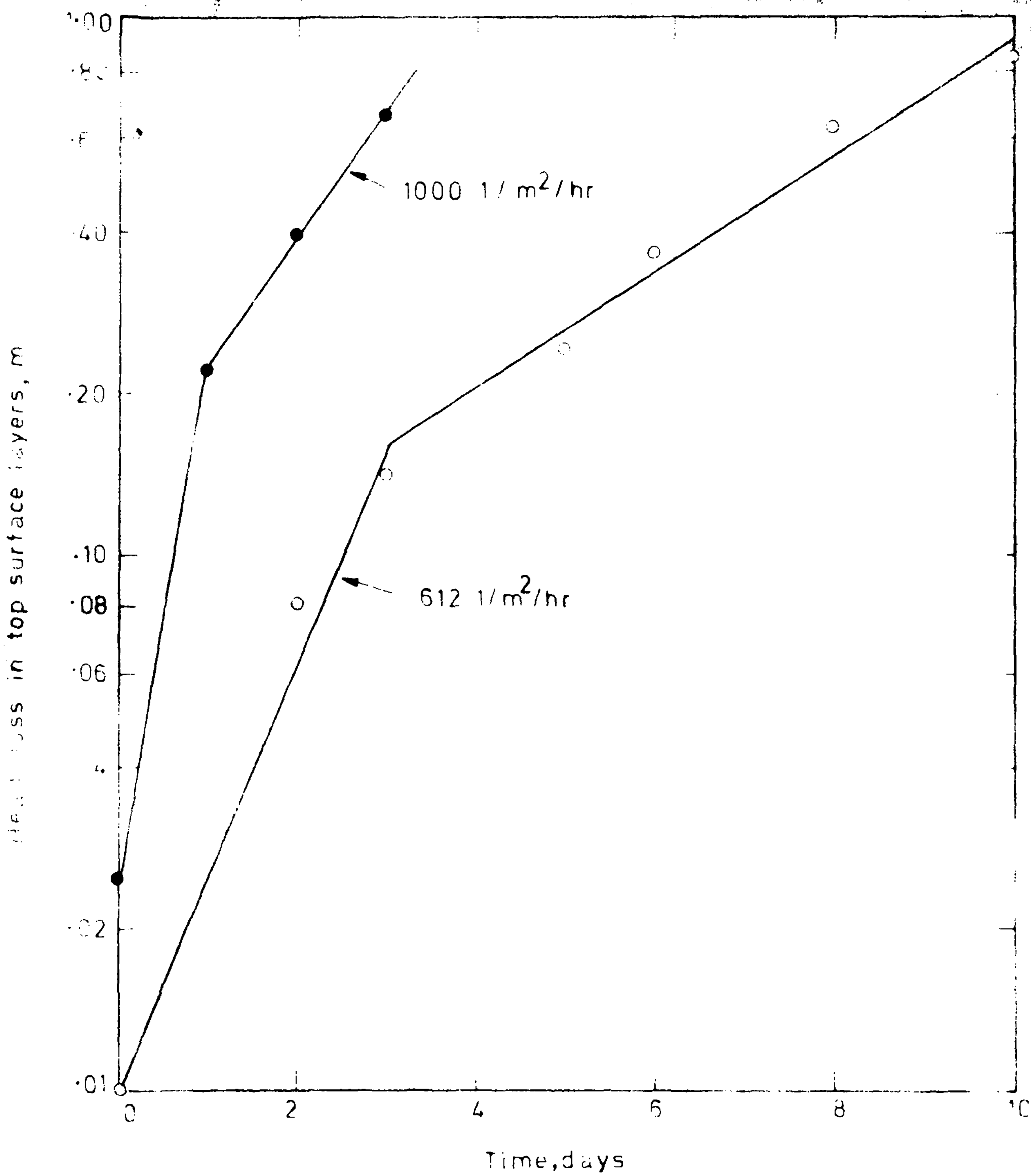


FIGURE 17. HEAD LOSS PROGRESSION IN TOP SURFACE LAYERS AT DIFFERENT RATES OF FILTRATION

From Table 22 it is clear that the rate of head loss build up is more in the later phases of the run than in the earlier phases. This can be explained as following. In the earlier phases of the run, the deposition of the positively charged floc particles occurs on the surface of the negatively charged sand particles. During later phases, as the run proceeds the constriction of the pore channels results. There is also the development of algae and bacteria in the top sand layers. This not only causes the constriction of pore channels because of slimy and gelatinous coating on the sand but also results in the development of a surface mat. Because of these two phenomena, the rate of build up of head loss is more in the latter phases of the run.

Irrespective of the reasons of the different rate of head loss build up in top sand layers, a mathematical equation of the following type can be proposed to describe the progression of head loss in these top surface layers.

$$h_s = G.\ 10^{kt} \qquad (35)$$

where G and k are empirical constants which assume atleast two values for each of the rates of filtration tried. Thus based on experimental data it is observed that head loss progression in top sand layers increases exponentially with respect to time.

Since turbidity of influent may vary from day to day, an alternative parameter than time may be specific deposit. Figure 18 and 19 show the head loss progression in top sand layers with specific deposit. The nature of curves is very much similar to the nature of curves in Figure 16 and 17. Therefore, a similar exponential equation can be proposed.

$$h_s = J \ 10^{m\sigma} \tag{36}$$

Again J and m are empirical constants and may have more than one value for each rate of filtration.

The values of empirical constants G, k and J, m employed in proposed equations (35) and (36) can be determined experimentally by making pilot filter runs. The values of these four empirical constants have been computed from Figures 16 to 19 and are presented in Table 23.

It is observed from Table 23 that the values of m at the rates of 133 and 216 $1/m^2/hr$, both during initial and final phases of the filter run show an erratic trend. This is to be expected as at low rates of filtration, there is more time available for algae and bacteria etc. to grow abundantly. Therefore, the build up of head loss is not only due to the deposition of removed impurities but the growth of biological life also contributes significantly at lower rates of filtration. As such the head loss build up

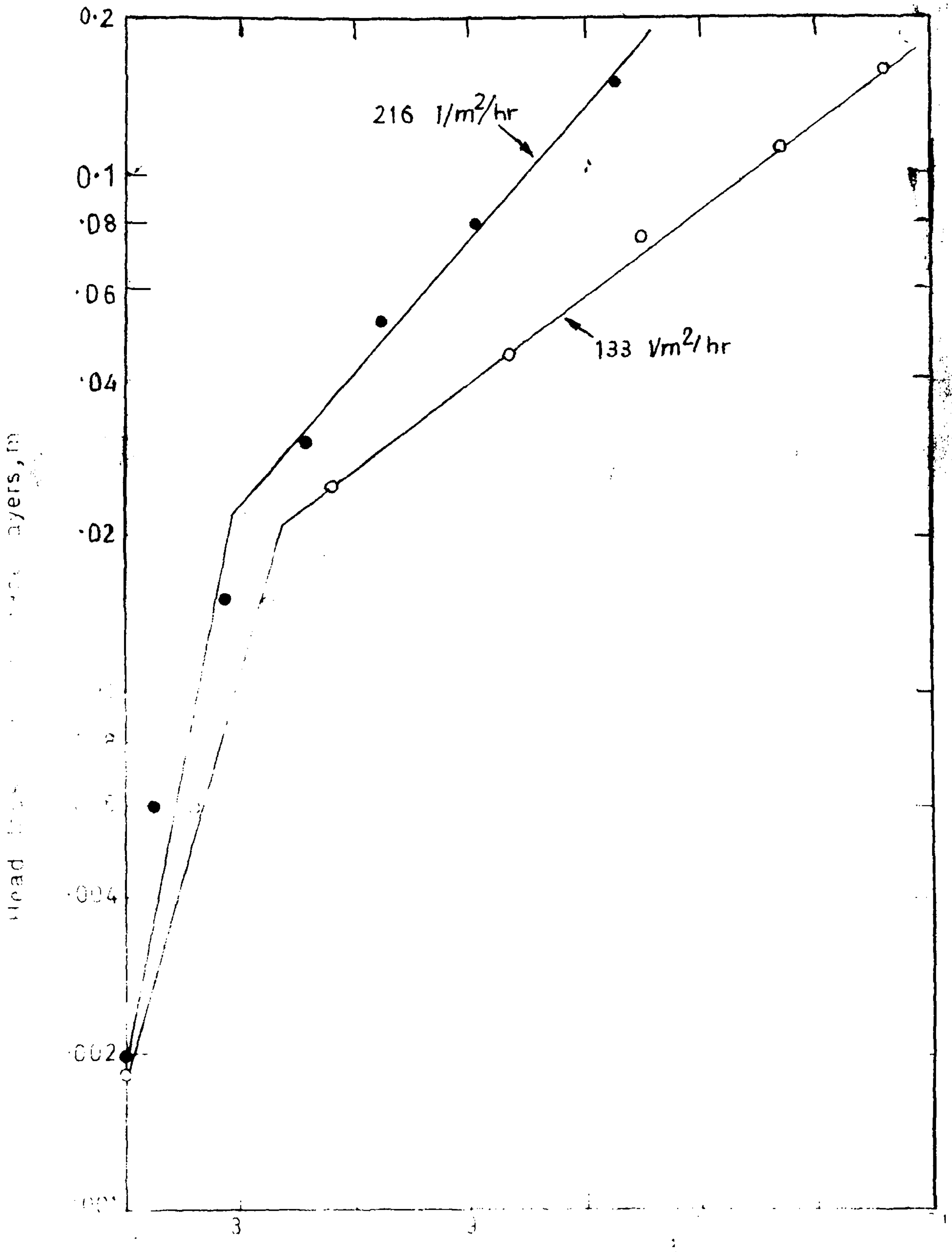
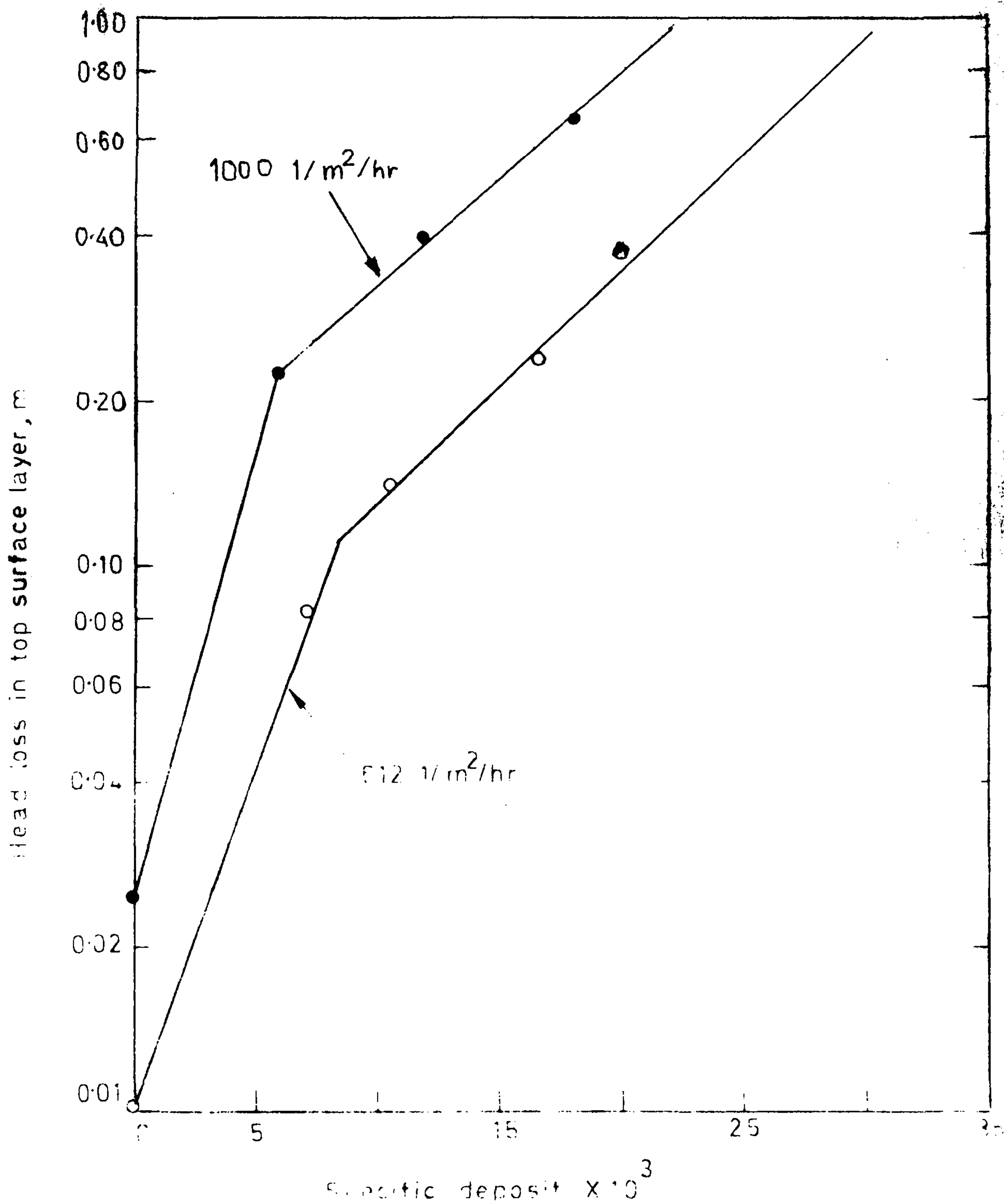


FIGURE ... HEAD ... TOP SURFACE
LAYERS WITH SPECIFIC DEPOSIT AT ...
... RATES



1·00
0·80
0·60
0·40
0·20
0·10
0·08
0·06
0·04
0·02
0·01
5
15
25
1000 1/m²/hr
Head loss in top surface layer, m
Specific deposit X 10³

at lower rates of filtration may not be suitably described in terms of the parameter of specific deposit alone. However at higher rates of filtration specific deposit may be a good parameter for describing the head loss build up.

TABLE 23

VALUES OF VARIOUS EMPIRICAL CONSTANTS USED IN HEAD LOSS MODELS

Rate of filtration $1/m^2/hr$	VALUES OF EMPIRICAL CONSTANTS							
	During initial phases of run				During later phases of run			
	G cm	$kx10^2$ day^{-1}	J cm	$mx10^{-2}$	G cm	$kx10^2$ day^{-1}	J cm	$mx10^{-2}$
133	.18	25	.18	2.6	1.9	3.2	1.2	.57
216	.2	29	.2	3.7	1.3	5.5	1.2	.85
612	1	40	1	1.2	7.5	10.7	4.7	.43
1000	2.5	95	2.5	1.8	13.0	24.0	13.0	.50

6.3 Mathematic Model Involving Depth And Time Or Specific Deposit

To describe the head loss at any particular instant for the entire depth of filter bed , equations (35) and (36) can be substituted in equation (32) to yield.

$$h = G\ 10^{kt} + A.\ \left(\frac{\partial h}{\partial l}\right) L \qquad (37)$$

If it is desired to know total head loss for known values of specific deposit, the following equation can be used.

$$h = J\ 10^{m\sigma} + A.\ \left(\frac{\partial h}{\partial l}\right) L \qquad (38)$$

The various exmpirical constants can be determined experimentally by making filter runs.

To summarize, equations (37) and (38) express the total head loss at any time as the sum of two components, h_s and h_l. The head loss in top surface layers, h_s, can be expressed either in terms of time or specific deposit. Though time may be a better parameter at all rates of filtration attempted in this study, specific deposit may or may not be a suitable parameter to cover the entire range of filtration rates tried. The build up of head loss in top sand layers may not be suitably expressed in terms of specific deposit alone at the lower rates of filtration employed in conventional slow sand filtration where biological growth may also contribute significantly to the build up of head loss.

The head loss in the remaining bottom layers, h_l, can be given by Kozeny-Carman equation but modified by multiplication with an empirical constant to account for the little change in head loss due to deposition of suspended matter and growth of biological life, if any.

Therefore, it can be concluded that a mathematical model describing the build up of head loss both in conventional slow sand filters and in filters operating at rates much higher than conventional rates (i.e. at rates intermediate between the conventional rates of slow and rapid sand filtration) has been developed.

VII. ENGINEERING SIGNIFICANCE OF THE STUDY

7.1 Aims Of The Study

The primary aims of any engineering study are to satisfy the existing need within the frame work of available and acceptable technology of that region and provide solutions that are economical in over-all cost and yet deliver the products that meet the prescribed quality standards. To achieve the above mentioned aims, this study was conducted to evolve a modified filtration system which is simple to construct, operate and maintain and which can process the raw water, of widely fluctuating quality, to produce treated water in combition with other pretreatment processes (e.g. chlorination) at lower cost of filtration with acceptable filtrate quality.

The conventially acceptable and universally adopted filtration units are the slow and the rapid sand filter, both of which possess many shortcomings. The main disadvantages of rapid sand filters include the use of sophisticated equipment, complicated hydraulic design and costly operation, all due to the process of backwashing to remove the suspended impurities retained in the sand matrix. The principal disadvantages of slow sand filters are inability to cope up with highly turbid raw waters in absence of any pretreatment high initial cost of construction and requirement of large land areas and a large labour force due to adoption of low filtration rates.

The need, therefore, exists of developing a modified filtration system which is acceptable especially to the semiurban and rural areas of developing countries as well as to the urban areas of these countries. Such a system should, therefore, eliminate the process of backwashing as adopted in rapid sand filters, and should also eliminate or reduce the above mentioned disadvantages of slow sand filters. This study aimed at developing such a modified filtration system, which while attempting to remove the disadvantages of both rapid and slow sand filters as discussed above, can deliver processed water of acceptable quality at lower over-all cost of treatment.

7.2 Achievements Of The Study

7.2.1 Development of Intermediate Rate Filters

The results of this study clearly indicate the following :

(i) The conventional slow sand filters, can be successfully operated with alum coagulated influents without any adverse effect on filtrate quality and length of filter run.

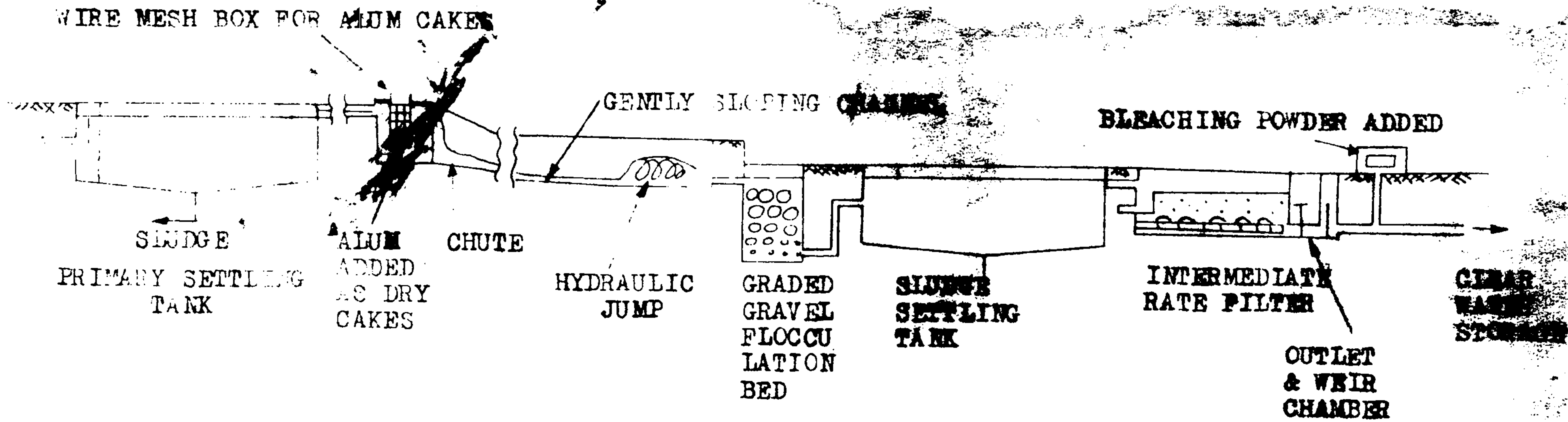
(ii) As a result of alum-coagulated influents, the slow sand filters can be operated at rates of filtration much higher than the conventional rates even when the turbidity of raw water varies over a very wide range.

(iii) As almost all of the head loss occurs in top few cm of sand bed where most of the impurities are arrested, scraping of top sand layers is effective in removing the impurities to restore the filter. Therefore, there is no necessity of backwashing the filter for cleaning the sand for next filter run.

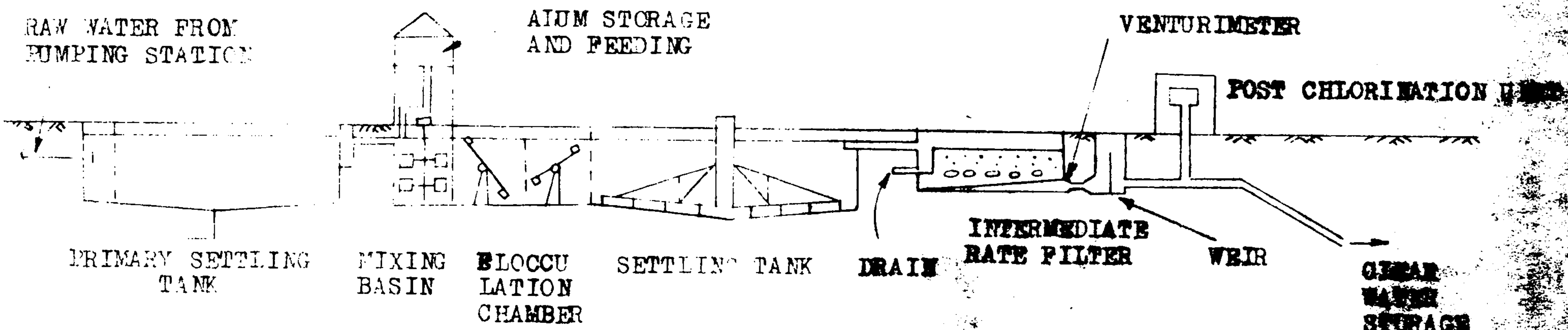
From the preceding observations, it is obvious that a modified filtration system has been developed which differs from both the rapid and slow sand filtration. The modified filtration differs from slow sand filtration in that it has alum-coagulation as pretreatment for filter influents and adopts much higher rates of filtration. While the modified filtration is different from rapid filtration in that the former does not use backwashing for cleaning operation. It would, therefore, be improper to call it either slow or rapid sand filtration. Because the modified filtration uses intermediate rates of filtration, being about five times the conventional slow sand filtration rate and about one-eighth of standard rapid filtration rate, the developed filtration system would be named "INTERMEDIATE RATE FILTRATION" and the units adopting this filtration "INTERMEDIATE RATE FILTERS".

7.2.2 Flow Sheet Of Treatment Using Intermediate Rate Filters

The flow sheets of water treatment employing intermediate rate filters for semi-urban and rural areas of developing countries and for urban areas are sketched in Figure 20. For urban areas, the units for alum-coagulation,



DIAGRAMATIC SECTION OF LAYOUT FOR SEMIURBAN & RURAL AREAS



DIAGRAMMATIC SECTION OF LAYOUT FOR [illegible] AREAS

FIGURE 20. SUGGESTED FLOWSHEETS FOR WATER TREATMENT

flocculation and settling may employ the mechanical equipment like flash-mixing devices, flocculating paddles and scrapers or plows attached to rotating arms for continuous sludge removal.

However, use of such sophisticated equipment for pretreatment of the influents of intermediate rate filters would be neither acceptable nor desirable for semiurban and rural areas of developing nation. To avoid the use of these sophisticated equipment and yet to effectively reduce the turbidity of raw waters to acceptable levels by alum coagulation, alternative devices are suggested for effective mixing, flocculation and setting. It is suggested that alum, in the form of dry cakes can be added into the channel of raw water. For effective mixing of alum with water, a chute followed by a channel can be constructed to create hydraulic jump for producing turbulence. Effective flocculation can be achieved by using baffled channels or a flocculation chamber which is filled with graded gravel from top to bottom as suggested by Kardile (1973) or similar devices in place of mechanical mixers. The settling tanks may be rectangular with hopper bottoms to drain the sludge under hydrostatic pressure. The disinfection may be achieved by adding bleaching powder solution to the filtered water to avoid the use of chlorine cylinders which may not be available in semiurban and rural areas.

7.2.3 Advantages Of Intermediate Rate Filters Over Slow Sand Filters

The first major advantage of intermediate rate filters over slow sand filters is that the former can cope up with even highly turbid waters due to the adoption of alum-coagulation as pretreatment for the filter influents in contrast to the slow filters. Alum coagulation was chosen as pre-treatment in preference to the other unit operations namely coarse rapid filtration and microstraining for the following reasons.

(i) Alum coagulation is universally adopted for pretreating the influents of rapid sand filters and has been found to be efficient in effectively pretreating the raw waters of even widely changing characteristics.

(ii) Coarse rapid filtration would be unacceptable because it has the disadvantage of employing backwashing for cleaning the media. Backwashing has been considered unacceptable for reasons to be discussed in Section 7.2.4.

(iii) Microstraining is only suited to treat waters highly rich in algal matter and cannot cope up with even moderately turbid raw waters.

The second major advantage of intermediate rate filters over slow sand filters is that they can be operated at much higher rates of filtration. At a filtration rate of 612 $1/m^2/hr$ intermediate rate filters would occupy only about one-fourth the area required by slow sand filters operating at conventional rate of 133 $1/m^2/hr$. A big reduction in land area requirements would also substantially reduce the initial cost of construction and the cost of media (sand and gravel) and thus the principal disadvantages of conventional slow sand filters would be reduced to a large extent.

Other advantages of intermediate rate filters result from reduced length of run. In slow sand filters, because of relative large length of filter run, there is many-a-times prolific growth of filamentous algae adding very substantially to filter cleaning costs (Ridley, 1967). Also a large length of run often causes disintegration of algae resulting in taste and odour in the filter effluents (Van de Vloed, 1955; Ridley, 1967). The intermediate rate filters would have little growth of filamentous algae and practically no problems of taste and odour in the effluents because of low length of filter run. It was observed that the algae content of schmutzdecke in the filters at the rate of 612 $1/m^2/hr$ is only one seventh to that at the rate of 133 $1/m^2/hr$.

However, there may be some operational and managerial problems in the case of intermediate rate filters as a result of reduced length of run. The number of scrapings would increase four fold and consequently more frequent recoupment of intermediate rate filters would be required in comparison to slow sand filters. Also more sand may have to be washed per scraping because of increased depth of scraping. But because of the absence of filamentous algae in case of intermediate rate filters, the sand would be more easily and quickly scraped and washed.

7.2.4 Advantages of Intermediate Rate Filters over rapid Sand Filters

The most significant advantage of intermediate rate filters over rapid sand filters is that the former would not require backwashing as practised in the case of latter. As the intermediate rate filters are recommended especially for rural and semiurban areas of developing nations, this is of great engineering significance as backwashing makes the hydraulic design of filter complicated, requires sophisticated equipment like air compressors and is relatively very costly. From Table 17 it is observed that sand scraping costs about Rs. 1600 per filter per annum while sand backwashing costs Rs. 11,316 per filter per annum when the effective yield of the slow sand filter is about 70% than that of the rapid sand filter (Table 18).

Another disadvantage of backwashing relates to the organic quality of filter effluent. Van de Vloed (1955) has stressed the role of microbial life in the hetrotrophe zone, which extends upto 30 cm deeper into the sand bed, in significantly improving the quality of filtering water by extracting from it both suspended and soluble organic matter and thus helping to produce good quality filtrates. However, in rapid sand filters majority of the microbial population gets washed out of bed as a result of backwashing.

Therefore, both as a result of the presence of hetrotrophe zone and due to more residence-time available for water being filtered, the quality of the effluents of the intermediate rate filters is better compared to that of the rapid sand filters as observed from the results of this study.

7.2.5 Overall Economic Advantages of Intermediate Rate Filters

Results of economic analysis indicate that the operation of intermediate rate filters at rate of about 625 $1/m^2/hr$ would result in the lowest of overall cost of treatment in comparison to slow and rapid filters operating at conventional rates in conditions as obtainable at Kanpur Water Works. It was found that the cost of treatment of water for intermediate rate filters at a filtration rate of 612 $1/m^2/hr$ is 51.2% cheaper than that for rapid sand filtration at near optimal rate of 8820 $1/m^2/hr$.

7.3 Areas Where Intermediate Rate Filters Can Be Successfully Adopted

From the above discussion, it is obvious that intermediate rate filters are simple, cheap and easy to construct, operate and maintain. Therefore these filters would be highly suitable for semiurban and rural areas of developing and developed countries. They will be, in general suitable for urban areas of developing nations. The intermediate rate filtration will be found especially useful for urban water works where both rapid and slow sand filters exist and which are situated in the heart of the urban area. Pressed with the growing demand for treated water for the increasing population and industrial growth and not finding enough space to expand the works, these plants can adopt intermediate rate filtration to increase the output of existing slow sand filters by more than four times with some modifications in filter accessories and thus obviating the need for constructing new water plants at different location.

It is hoped that the adoption of intermediate rate filters would go a long way in providing safe and aesthetically acceptable drinking water at lower cost to the millions of people especially of the developing countries.

VIII. CONCLUSIONS

Based on the findings of engineering and theoretical investigations, the following conclusions may be drawn :

(i) Alum-coagulation followed by flocculation and sedimentation can be used as pretreatment to the influents of slow sand filter without adversely affecting the overall performance. The length of filter run is not reduced and the quality of the filter effluent does not deteriorate.

(ii) The adoption of alum-coagulation would make slow sand filters suitable to treat even highly turbid waters by proper control of the unit operations of alum-coagulation, flocculation and sedimentation. Even when the turbidity of raw water was about 3500 jtu slow sand filters receiving alum-coagulated influents produced effluents of almost zero turbidity.

(iii) As a result of alum-coagulation, slow sand filters can be successfully operated at filtration rates much higher than conventional. Four filtration rates upto 1000 $1/m^2/hr$, being higher than the conventional rate of 133 $1/m^2/hr$ were tried. Filtrate quality at all the four rates attempted was acceptable.

(iv) Primarily because of the adoption of intermediate rates of filtration (i.e. intermediate between the conventional slow and rapid sand filtration) and because of other reasons detailed in section 7.2.1, the filters using alum-coagulated influents but retaining almost all the constructional and operational features of slow sand filters have been named "Intermediate Rate Filters".

(v) Because of the adoption of higher rates of filtration, intermediate rate filters would require much less land areas and capital investment on construction (including the costs of land and filter media) would be substantially reduced. At a rate of 612 $1/m^2/hr$ these filters would require only one fourth of the land area required by slow sand filters operating at a rate of 133 $1/m^2/hr$ to filter the same quantity of water.

(vi) Intermediate rate filters operating at an optimal rate of 625 $1/m^2/hr$ could filter water at a cost of filtration of Rs. 6.00 per million litres of water compared to the cost of filtration of Rs. 14.90 for rapid sand filters operating at optimal filtration rate of 8100 $1/m^2/hr$ in conditions as obtainable at Kanpur Water Works.

(vii) Including the cost of chemicals (alum and chlorine) and power, the cost of treatment of water employing intermediate rate filtration at a rate of 612 $1/m^2/hr$ is 51.2% cheaper compared to the cost of treatment employing rapid sand filtration at a rate of 8820 $1/m^2/hr$.

(viii) Because of the reduced length of filter run, intermediate rate filters are likely to be free from the problems of taste and odour in effluents as against effluents of slow sand filters which have been reported to possess taste and odour as a result of very long length of filter run resulting in disintegration of algae and formation of organic complexes.

(ix) The reduced length of run of about 10 days at a filtration rate of 612 $1/m^2/hr$ would also help in keeping the growth of filamentous algae to very low levels. This is of advantage as these algae add substantially to sand scraping and washing costs.

(x) The intermediate rate filters, besides possessing all the above mentioned advantages, would not require the use of imported and sophisticated equipment. Being simple and economical to construct, operate and maintain, they are highly suitable for semiurban and rural areas being labour intensive rather than equipment-intensive.

They are also of great value to urban water works where slow sand filters and units of alum coagulation, latter having been constructed for use in conjunction with rapid filtration, exist. With some modifications, the existing slow sand filters can be operated at much higher rates to deliver more water per unit area per unit time.

(xi) A mathematical model to describe the head loss in both slow and intermediate rate filters has been developed. The total head loss in these filters is the sum of two components: one is the head loss in top sand layers, h_s, and other is the head loss in remaining lower layers, h_l. While h_s is time variant, h_l remains substantially constant with time.

(xii) Head Loss in top sand layers of intermediate rate filters can be described by an exponential function of time or specific deposit. In case of slow sand filters an exponential increase with time may be more suitable. Head loss in lower layers for both types of filter can be predicted by Kozeny Carman equation modified to account for little accumulation of deposits and microbial growth, if any, in these layers of filter bed.

IX. SUGGESTIONS FOR FUTURE WORK

(i) There is a need for developing simpler and efficient units of alum-coagulation and flocculation which may be adoptable in the semiurban and rural areas of developing nations. The suggested units of alum mixing and flocculation namely a chute followed by a gently sloping channel to create hydraulic jump and graded gravel flocculation bed, should be evaluated, preferably on the basis of pilot scale studies.

(ii) In view of the substantial savings achieved by adoption of intermediate rate filtration as indicated by this study, more economic analyses of similar nature should be carried out in conditions similar as well as different from those as obtainable at Kanpur Water Works.

(iii) In the present study, rate of filtration was the only variable considered in the economic analysis. However, more variables like depth of filter media etc. should be introduced and a more comprehensive economic analysis should be done.

(iv) Theoretical studies relating to dominant mechanisms of clarification and mathematical models governing the removal of suspended impurities should be carried out to get a better insight into the working of intermediate rate filters and to improve their functioning.

(v) Simple and cheap mechanical devices to scrape the filters should be developed. Since in the case of intermediate rate filters the growth of filamentous algae is very small and the sand can be easily scraped, it should be possible to develop simple and manually operated device (s) for scraping which require (s) less energy and time.

(vi) Because of the lower rates of filtration and other factors in comparison to rapid sand filters, the intermediate rate filters, in conjunction with alum coagulation, flocculation and settling may be more efficient in removing viruses also as they are in removing bacteria. This aspect, should be investigated as it may be quite significant.

BIBLIOGRAPHY

1. Agrawal, G.D. (1966) Electrokinetic Phenomena in Water Filtration. Ph.D. Thesis, Univ. Of California, Berkeley.

2. Agarwal, I.C., Agrawal, G.D. and Misra, Y.D. (1972) Overloading Of Slow Sand Filters. Symp. On Water Treatment, Distribution and Management, Nagpur, India, 30.

3. Agarwal, I.C., Agrawal, G.D. and Misra, Y.D. (1973) Management and Financing Structure Of Kanpur Water Works. Seminar on Financing and Management of Water Supply and Sew. Works. Jan. 1973, Trivandrum.

4. Agarwal, I.C., Agrawal, G.D. and Misra, Y.D. (1973) Cost Analysis of Slow and Rapid Sand Filters. J. Indian Water Works Ass., 5, 64.

5. Agarwal, I.C. and Agrawal, G.D. Operating Slow Sand Filters with Alum Coagulated Water. Symp. on Env. Pollution Jan. 1973, Nagpur, India.

6. Am. Water Works Ass. Manual (1951) Water Quality and Treatment. 2nd ed., Am. Water Works Ass., Inc. New York.

7. Babbit, H.E., Doland, J.J., and Cleasby, J.L. (1962) Water Supply Engineering. 6th ed., Mc Graw-Hill, New York.

8. Bauman, E.R., Willrich, T.L., and Ludwig, D.D. (1963) For a Purer Water Supply Consider Prechlorination. Agr. Eng., 44, 138.

9. Black, A.P., and Chen, C.L. (1967) Electrokinetic Behaviour of Aluminium Species in Dilute Dispersed Kaolinite Systems. J. Am. Water Wrosk Ass. 59,1173.

10. Black, A.P., Birkner, F.B., and Morgan J.J. (1965) Destabilization of Dilute Clay Suspensions with Labeled Polymers. J. Am. Water Works Ass., 57,1547.

11. Burman, N.P. and Lewin, J. (1961) Microbiological and Operational Investigation of Relative Effects of Skimming and in-situ Sand Washing on Two Experimental slow sand Filters. J. Ins. Water Eng., 15,355.

12. Busch, P.L. (1966) Chemical Intractions in the Aggregation of Bacterial. Ph.D. Thesis, Harvard Univ. Cambridge, Mass.

13. Camp, T.R. (1964) Theory of Water Filtration. J. San. Eng. Div., Proc. Am. Soc. Civ. Engrs., 90, SA4,1.

14. Carman, P.C.(1937) Fluid Flow through Granular Beds. Trans. Inst. Chem. Engrs., 15,150.

15. Chen, C.Y.(1956) Filtration of Aerosols by Fibrous Media, Chem. Rev., 50,1161.

16. Cleasby, J.L. and Bauman, E.R. (1961) Selection of Sand Filtration Rates. J. Am. Water Works Ass., 54,579.

17. Conley, W.R. and Pitman, R.W. (1960) Innovations in Water Clarification. J., Am. Water Works Ass., 52, 1319.

18. Creitz, G.I. and Richards, F.A. (1955) The Estimation and Characterization of Plankton Populations by Pigment Analysis. J. Mar. Res., 14,211.

19. Davies, C.N. and Alyward, M. (1951) The Trajectories of Heavy, Solid Particles in a Two-Dimensional Jet of Ideal Fluid Impinging Normally upon a Plate. Proc. Phycs. Soc., 64B, 889.

20. Davis, E. and Borchardt, J.A. (1966) Sand Filtration of Particulate Matter. J. San. Eng. Div., Proc. Am. Soc. Civ. Engrs., 92, SA5, 47.

21. Deb, A.K. (1969) Theory of Sand Filtration. J. San. Eng. Div., Proc. Am. Soc. Civ. Engrs., 95, SA3, 399.

22. Deb, A.K. (1970) Numerical Solution of Filtration Equations. J. San. Eng. Div., Proc. Am. Soc. Civ. Engrs., SA2, 195.

23. Dieterich, B.H. and Henderson, J.M. (1963) Urban Water Supply Needs in Seventy Five Developing Countries, W.H.O., Publ. Health Papers, 23,80.

24. Edward, D.M. and Monke, E.J. (1967) Electrokinetics and slow sand Filtration. J. Am. Water Works Ass.,59, 1310.

25. Eliassen, R. (1941) Clogging of Rapid Sand Filter. J.Am. Water Works Ass., 33, 926.

26. Fair, G.M., Geyer J.C. (1961) Elements of Water Supply and Waste-Water Disposal, John Wiley & Sons, New York.

27. Fair, G.M., Geyer, J.C., and Okum, D.A. (1968) Water and Wastewater Engineering. Vol. 2. John Wiley & Sons, New York.

28. Fair, G.M., and Hatch, L.P. (1933) Fundamental Factors Governing the Streamline Flow of Water through sand. J. Am. Water Works Ass., 25, 1951.

29. Folkman, Y. and Wachs, A.M. (1970) Filtration of Chlorella through Dune Sand. J. San. Eng. Div., Proc. Am. Soc. Civ. Engrs., 96, SA3, 675.

30. Fox, D.M. and Cleasby, J.L. (1966) Experimental Evaluation of Sand Filtration Theory. J. San. Eng. Div.,Proc. Am. Soc. Civ. Engrs., 92,SA5, 61.

31. Friedlander, S.K. (1958) Theory of Aerosols Filtration. Ind. Eng. Chem., 50, 1161.

32. Government of India : Planning Commission (1966) Fourth Five-Year Plan - A Draft Outline.

33. Hazen, A. (1892) Some Physical Properties of Sands and Gravels. 24th Annual Report, Massachusetts State Board of Health.

34. Hazen, A. (1904) On Sedimentation. Trans. Am. Soc. Civ. Engrs., 53, 63.

35. Hall, W.A. (1957) An Analysis of Sand Filtration. J. Am. Soc. Civ. Engrs., 83, SA3, Paper 1276.

36. Heertzes, P.M., Lerk, C.F. (1967) The functioning of Deep Bed Filters. Trans. Inst. Chem. Eng. 45, T 129.

37. Herzig, J.P., Leclerc, D.M., and LeGoff, P. (1970) Flow of Suspension Through Porous Media- Application to Deep Filtration. Ind. Eng. Chem., Vol. 62, No. 5, 8.

38. Horner, R.M.W. (1968) Water Clarification and Aquifer Recharge, Ph.D. Thesis, Univ. of London.

39. Humphery, A.E. and Gadon, E.C. Jr. (1955) Air Sterilisation by Fibrous Media. Ind. Eng. Chem., 47,924.

40. Hunter, R.J. and Alexander, A.E. (1963) Surface Properties and Flow Behaviour of Kaolinite. J. Colloid Sci., 18, 846.

41. Huisman, L. (1970) Biological or Slow Sand Filters, Background Paper, W.H.O., WHO/CWS/RD/70.1.

42. Ison, C.R. (1967) Dilute Suspensions in Filtration, Ph.D. Thesis, Univ. of London.

43. Ives, K.J. (1957) Algae and Water Supplies, Physical Removal of Algae, Water Wastes Eng., 61, 432.

44. Ives, K.J. (1960) Rational Design of Filters. Proc. Inst. Civ. Engrs., 16,189.

45. Ives, K.J. (1960) Filtration through a Porous Septum : A Theoretical Consideration of Boucher's Law. Proc. Inst. Civ. Engrs., 17,333.

46. Ives, K.J. (1961) New Concepts in Filtration. Water & Water Eng., 65, 307,341, 385.

47. Ives, K.J. (1961) Filtration Using Radioactive Algae. J. San. Eng. Div., Proc. Soc. Civ. Engrs., 87, SA3,

48. Ives, K.J. (1963) Simplified Rational Analysis of Filter Behaviour. Proc. Inst. Civ. Engrs., 25, 345.

49. Ives, K.J. and Sholji, I. (1965) Research on Variables Affecting Filtration. J. San. Eng. Div., Proc. Am. Soc. Civ. Engrs., 91, SA4, 1.

50. Ives, K.J. and Gregory J. (1967) Basic Concepts of Filtration. Proc. Soc. Water Treat. Exam. 16,147.

51. Ives, K.J. (1970) Advances in Deep-Bed Filtration. Trans. Inst. Chem. Engrs., 48, T94.

52. Ives, K.J. (1970) The Role of Theory in the Advancement of Rapid Filtration Technology. 2nd Conf. on Water Quality & Tech., Budapest, 5148.

53. Ives, K.J. (1971) Filtration of Water and Wastewater. CRC Critical Reviews in Env. Control, 293.

54. Iwasaki, T. (1937) Some Notes on Sand Filtration. J. Am. Water Works Ass., 29, 1591.

55. Jorden, R.M. (1963) Electrophoretic Studies of Filtration. J. Am. Water Works Ass., 55, 771.

56. Journal Indian Water Wrosk Ass. (1972), Vol. 4, No. 2, 124.

57. Journal Indian Water Works Ass. (1973), Vol. 5, No.1,97.

58. Kemna, A. (1899) The Biology of Sand Filtration. Trans. Inst. Water Engrs., 4, 40, 157.

59. Lamer, V.K. and Healey T.W. (1963) Adsorption-Flocculation Reactions of Macromolecules at the Solid-Liquid Interface. Rev. Pure & Applied Chem., 13, 112.

60. Langmuir, I. (1945) The Collected Works of Irving Langmuir.

61. Laval, M. (1952) A Method of Washing Filter Sand. J. Inst. Water Engrs., 6, 155.

62. Lewin, J. (1961) Mechanization of Slow Sand and Secondary Filter Bed Cleaning. Trans. Inst. Water Engrs., 15,15.

63. Ling, J.T. (1955) A Study of Filtration through Uniform Sand Filters, Proc. Am. Soc. Civ. Engrs., 81,1.

64. Lynch, W.O., Baker, C.R. and Haberer, J.H. (1965) Experiences with Microstraining at Illion. J. Am. Water Works Ass., 57, 1422.

65. Mackrle, V. and Mackrle, S. (1961) Adhesion in Filters, J. San. Eng. Div. Proc. Am. Soc. Civ. Engrs., 87,17.

66. Mackrle, V., Dracka, O. and Svec, J. (1965) Hydrodynamics of the disposal of low level liquid radioactive Wastes in Soil, Int. At. Energy Agency, Contract Report 98.

67. Maroudas, A. and Eisenklam, P. (1965) Clarification of Suspensions : A Study of Particle Deposition in Granular Media. Chem. Eng. Sci., 20, 867.

68. Medalia, A.I. (1951) Test for Traces of Organic Matter in Water. Anal. Chem., 23, 1318.

69. Michaels, A.S. (1954) Aggregation of Suspensions by Polyelectrolytes. Ind. Eng. Chem., 46, 1485.

70. Michaels, A.S. and Morelas, O. (1955) Polyelectrolyte Adsorption by Kaolinite. Ind. Eng. Chem., 47, 1801.

71. Mintz, D.M. (1966) Modern Theory of Filtration. Int. Water Supply Ass., Congr., London.

72. Miller, A.P. (1962) Water and Man's Health. Agency for Int Development, Washington, D.C., 24.

73. Mohanka, S.S. (1969) Theory of Multi Layer Filtration. J. San. Eng. Div., Proc. Am. Sco. Civ. Engrs., 95, SA6, 1079.

74. O'Melia, C.R. and Crapps, D.K. (1964) Some Chemical Aspects of Rapid Sand Filtration. J. Am. Water Works Ass., 56, 1326.

75. O'Melia, C.R. and Stumm, W. (1967) Theory of Water Filtration. J. Am. Water Works Ass., 59, 1393.

76. Pearsall, W.H., Gardiner, A.C. and Greenshields, F. (1946) Freshwater Biology and Water Supply in Britain, Sci. Pub. No. 11 of the Freshwater Biological Association.

77. Ranz, W.E. and Wong J.B. (1952) Impaction of Dust and Smoke Particles on Surface and Body Collectors. Ind. Eng. Chem., 44, 6, 1371.

78. Richards, F.A. with Thompson, T.G. (1952) The Estimation and Characterization of Plankton Populations by Pigment Analysis. J. Mar. Res., 11 (2), 156.

79. Ridley J.E. (1967) Experience in the Use of Slow Sand Filtration, Double Sand Filtration and Microstraining. Proc. Soc. Water Treat. Exam., 16, 170.

80. Roy, A.K. (1973) Rural Water Supply In India. Symp. Env. Pollut., 61.

81. Ruehrwein, R.A. and Ward, D.W. (1952) Mechanism of Clay Aggregation by Polyelectrolytes. Soil Sci., 73, 425.

82. Sakthivadivel, R., Thanikachalam, V., and Seetharaman, S. (1972) Head Loss Theories in Filtration. J. Am. Water Works Ass., 64(4) 233.

83. Schiller, L.Z. (1923) Augen., Math. Mech., 3,2.

84. Sedgwick, W.T. (1902) Principles of Sanitary Science and the Public Health. Macmillan, New York, 170.

85. Shekhtman, Yu. M. (1961) Filtration of Suspensions of Low Concentration, Publishing House of the U.S.S.R., Academy of Sci. Moscow.

86. Swift, D.L. and Friedlander, S.K. (1964) J. Colloid Sci, 19, 621.

87. Smith, J.C. (1945) Uprating McMillan Slow Sand Plant, J. Am. Water Works Ass., 37, 269.

88. Shull, K.E. and Guthan, G.R. (1967) Rapid Modified Eriochrome Cyanine - R Method for Determination of Aluminium in Water. J. Am. Water Works Ass., 59, 1456.

89. Siddiqi, R.H. (1972) Personal communication

90. Skeat, W.O., ed. (1961) Manual of British Water Engineering Practice, 3rd ed., Inst. Water Engrs., Heffer & Sons, Cambridge, England.

91. Standard Methods for the Examination of Water and Wastewater (1971). 13th ed., APHA : AWWA: WPCF, Am. Public Health Ass. 1740 Broadway, New York, N.Y. 10019.

92. Stanely, D.R. (1955) Sand Filtration Studied with Radiotracers. J. San. Eng. Div., Proc. Soc. Civ. Engrs., 81, 592.

93. Stein, P.C. (1940) A Study of the Theory of Rapid Filtration of Water Through Sand. Ph.D. Thesis, MIT, Cambridge Mass.

94. Taylor, D.W. (1962) Fundamentals of Soil Mechanics. Chap. 3. Asia Publishing House, India.

95. Twort, A.C. (1963) A Textbook of Water Supply. Am. Elsewier Publishing Co. Inc., New York.

96. Van de Vloed, A., (1955) Comparison between Slow Sand and rapid filters. Report, I.W.S.A. Congress, London.

97. Veal, T.H.P. (1950) The Supply of Water. 2nd ed., Chapman & Hall Ltd.

98. World Health Organization (1963) International Standards for Drinking Water, 2nd ed., Geneva, Switzerland.

99. Yao, K.M., Habibian, M.T. and O'Melia, C.R. (1971) Water and Wastewater Filtration : Concepts and Applications, Env. Sci. Tech., 5, 1105.