AREA DE ABASTECIMIENTO Y REMOCION DE AGUA

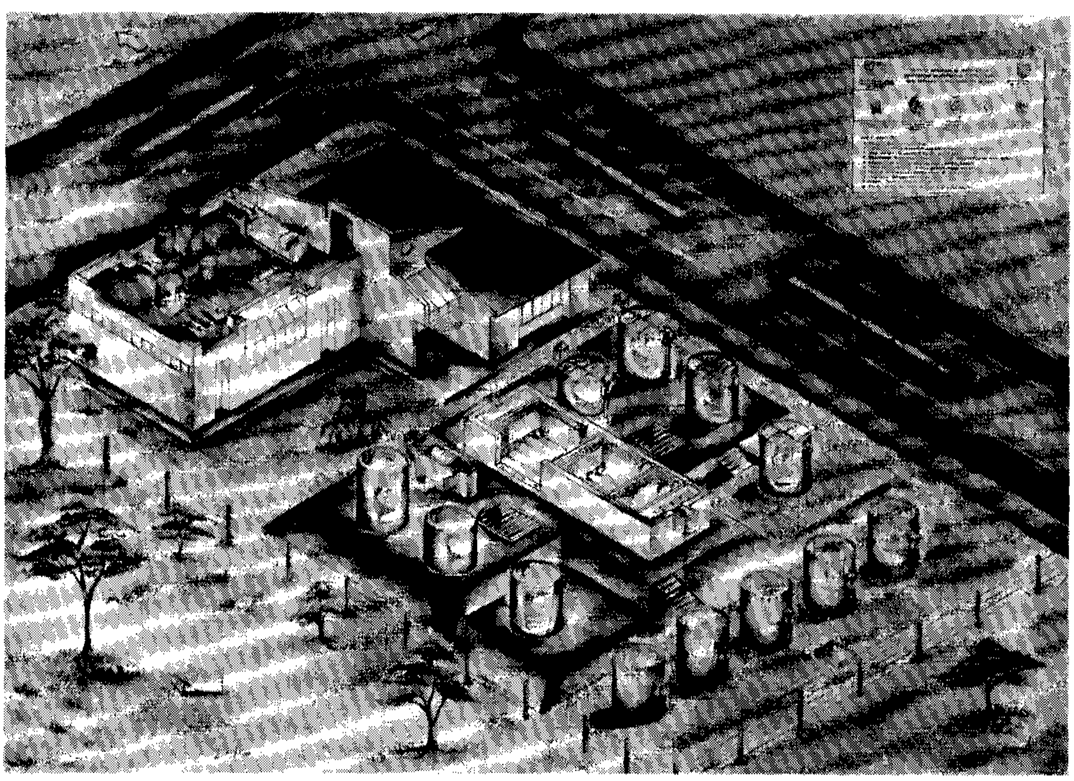
UNIVERSIDAD DEL VALLE - FACULTAD DE INGENIERA CALI - COLUMBIA

IRC INTERNATIONAL WATER AND SANITATION CENTRE
WHO collaborating centre

THE HAGUE, THE NETHERLANDS

Pre-treatment Methods for Community Water Supply

An overview of techniques and present experience



Pre-treatment
Research and Demonstration
Project

IRC, INTERNATIONAL WATER AND SANITATION CENTRE

IRC is concerned with knowledge generation and transfer, and technical information exchange for water supply and sanitation improvement in developing countries. The emphasis is on innovative approaches to prevailing problems. The target groups are management and technical staff concerned with planning implementation and use of water supply and sanitation facilities in rural and urban fringe areas.

The centre works together with partners in developing countries, United Nations organizations, bilateral donors and non-governmental organizations. Its multidisciplinary staff provides support through development and demonstration projects, evaluation and advice, training and education, publications, and general information exchange.

Activities integrate technical and non-technical issues in water supply and sanitation and include community participation particularly the role of women, hygiene education, appropriate technology, operation and maintenance, community-based financial management, and development of technical information exchange.

IRC is an independent, non-profit organization. It is supported by and linked with the Netherlands Government, UNDP, UNICEF, the World Bank, and WHO for whom it acts as a Collaborating Centre for Community Water Supply and Sanitation.

For further information

IRC
P.O. Box 93190
2509 AD The Hague
The Netherlands
Telephone: + 31-70-814911
Telex: 33296 irc nl
Cable: Worldwater, The Hague

PRE-TREATMENT METHODS FOR COMMUNITY WATER SUPPLY

An overview of techniques and present experience

compiled by

J.E.M. Smet and J.T. Visscher

with contributions from

A. Castilla, C. Engels, G. Galvis, L. Huisman, K.J. Ives,
T.S.A. Mbwette, M. Pardón, W. Poggenburg, E. Quiroga,
J.P. Rajapakse, J.E.M. Smet, M. Wegelin, D. Wheeler and H. Wolters

IRC International Water and Sanitation Centre

The Hague, The Netherlands

February, 1989

This publication is part of the "Research and Demonstration Project on Simple Pre-treatment Methods", which is financially supported by the Netherlands Ministry for Development Co-operation Section for Research and Technology. Responsibility for the contents rests solely with the authors of the respective chapters; the publication does not constitute an endorsement by the Ministry for Development Co-operation.

TABLE OF CONTENTS

CONTRIBUTORS

A. Castilla	Area de Abastecimiento y Remoción de Agua, Universidad del Valle, Cali, Colombia
C. Engels	Stadtwerke Düsseldorf AG, Federal Republic of Germany
G. Galvis	Area de Abastecimiento y Remoción de Agua, Universidad del Valle, Cali, Colombia
Professor L. Huisman	formerly: Delft University of Technology, Department of Civil Engineering, Division of Sanitary Engineering, Delft, The Netherlands
Professor K.J. Ives	Department of Civil and Municipal Engineering, University College London, UK
T.S.A. Mbwette	Imperial College, London, UK
M. Pardón	Environmental Health Unit, The Robens Institute, University of Surrey, UK
W. Poggenburg	Stadtwerke Düsseldorf AG, Federal Republic of Germany
E. Quiroga	Area de Abastecimiento y Remoción de Agua, Universidad del Valle, Cali, Colombia
J.P. Rajapakse	Department of Civil and Municipal Engineering, University College London, UK
J.E.M. Smet	IRC International Water and Sanitation Centre, The Hague, The Netherlands
M. Wegelin	International Reference Centre for Waste Disposal (IRCWD), Dübendorf, Switzerland
D. Wheeler	Environmental Health Unit, The Robens Institute, University of Surrey, UK
H. Wolters	IRC International Water and Sanitation Centre, The Hague, The Netherlands

PREFACE

Slow sand filtration is a well proven water treatment technique in developing countries as well as in the western world. It has great advantages over other filtration techniques specifically in reliability, cost, and performance. However filters become quickly clogged as a result of high suspended solid contents of the raw water. This presents a major problem in developing countries, particularly during the rainy season. While source protection to prevent erosion may improve the situation, often raw water requires pre-treatment before slow sand filtration. This is one of the main findings of IRC's development and demonstration project which, in collaboration with national institutions, has stimulated the application of slow sand filtration.

In the last few decades, several new pre-treatment techniques have been tested in the laboratory and in the field. Information on these techniques is presented in this document. It is being prepared in the context of a new development and demonstration project being undertaken by the Universidad del Valle, Cali, Colombia and IRC. Financed by the Netherlands Directorate-General for International Co-operation (DGIS), this project is testing several simple pre-treatment systems for rural and urban fringe water supply. Project activities include comparative studies of five pre-treatment systems and dissemination of available knowledge on pre-treatment.

The present document is a first step in establishing a knowledge base and in sharing information on pre-treatment with professionals in this field. Some of the pre-treatment techniques are dealt with in detail while others are covered in lesser depth because information is not readily available or experiences are scarce. This document and comments on it received from the field will provide the basis for a more comprehensive state-of-the-art publication.

Many thanks are expressed to those who contributed to this project work-document: A. Castilla, C. Engels, G. Galvis, L. Huisman, K.J. Ives, T.S.A. Mbwette, M. Pardón, W. Poggenburg, E. Quiroga, J.P. Rajapakse, J.E.M. Smet, M. Wegelin, D. Wheeler and H. Wolters.

1. INTRODUCTION

Surface water in tropical regions often contains high loads of suspended solids of discrete and colloidal nature. Loads are particularly high during the rainy season, when concentrations of several thousands milligrams per litre are not exceptional. Very often these surface waters are the only source of drinking water for many people especially in developing countries. Extensive treatment is required to ensure water of reasonable quality. Conventional treatment techniques requiring large amounts of chemicals are costly and adequate operation and maintenance is often well beyond local capacities. Slow sand filtration is thus a suitable alternative in situations where skilled staff is limited and chemicals' supply difficult. However, performance is poor in very turbid surface waters. Filter runs are very short and the effluent quality is poor because the biological action cannot develop in raw water with turbidity levels exceeding 50 NTU for longer than a few weeks. For good performance, average turbidity levels of the raw water should not exceed 10-15 NTU.

Solutions are coming to the fore. Simple treatment of raw water before slow sand filtration are giving promising results, particularly for small water supply systems. The primary objective of pre-treatment is to reduce the excessive of suspended material in surface water. Bacteriological, biological and chemical water quality improvements may be also achieved but these do not usually justify construction of a pre-treatment system.

A number of techniques have been developed. Some have been applied for centuries while others are quite new or even in a development stage. A brief introduction is given on the following techniques:

- riverbank filtration
- river bed filtration
- modular sub-sand abstraction
- plain sedimentation
- tilted plate settling
- downflow roughing filtration
- upflow roughing filtration
- horizontal-flow roughing filtration
- pebble matrix filtration.

Riverbank filtration

One of the oldest pre-treatment techniques is riverbank filtration. This system makes use of permeable soil formations, such as alluvial sand and gravel sediments. The chemical, biological and physical processes improve the quality of surface water usually river or stream water as it passes through these sediments to the abstraction site. The capacity of abstraction wells is determined by the soil characteristics and size of soil formations, distance from the river and water level, and the contribution of the landward groundwater. These systems produce water of excellent quality provided the surface water quality is within acceptable limits. In many industrialized countries this technique is still used to treat water for human consumption. However, the industrial and agricultural surface water pollution with non-biodegradable organic compounds which may be carcinogenic or mutagenic for humans, is a serious constraint to use biological treatment systems. In this document, the background of riverbank filtration along the Rhine River in Germany, is described together with the natural purification processes and the problems encountered.

River bed filtration

Widely used in developing countries, river bed filtration ranges from a hole in the dry river bed to complete abstraction systems behind dams. The technique is based on filtration of surface water through a river bed of natural permeable layers or a river bed constructed of coarse filter medium. The main difference between this technique and bank filtration is that the water collection system is in the river bed. Biological, chemical and physical processes improve the water quality as water passes through the river bed filter to the abstraction system. Several systems applied in developing countries are described in this document.

Modular sub-sand abstraction

Specially designed and commercially manufactured modular sub-sand abstraction systems have been used in many African countries. The first such systems consisted of an inverted filter box, filled with coarse medium and buried in the river bed. The system was improved to consist of a box filled with coarse filter material and covered with a filter mat. Placed on top of a river bed or in a reservoir the filter system should produce water of sufficient quality to supply small communities. Several field and laboratory studies, and evaluations of this technique are reviewed in this document.

Plain sedimentation

In plain sedimentation water at low velocity passes through a large basin in which suspended solids settle. This is one of the simplest water treatment techniques by which considerable improvement in water quality is achieved at relatively small cost. In this document general design criteria are presented for optimum operating conditions and results.

Tilted plate settling

Problems with uniform flow conditions in plain sedimentation basins have led to research and experiment to improve sedimentation techniques. Multi-layer and tube settling are possible solutions. In the tilted plate settlers raw water passes through narrow spaces between inclined plates, which increases the total settling area. In view of construction, cost and operation and maintenance this seems to be the most appropriate technique for developing countries. The technique has also potential to expand capacities for existing plain sedimentation tanks. Experience in Latin America is reported.

Downflow roughing filtration

Downflow roughing filtration is similar to slow sand filtration except for the design of filter medium size, drain construction and filter velocity. In South-East Asia experiments have been undertaken with coconut fibers as filter material, while in South America gravel was commonly used. The results have been used to design full-scale plants. These and other experimental results and plant experiences are covered.

Upflow roughing filtration

Upflow filtration using coarse filter material has the advantage that the first filtration occurs at the bottom of the filter. The cumulation capacity for filtered solids is usually higher than for downflow filtration, as the process makes better use of the depth of the filter bed. Because of counter current sedimentation, the effluent may be of lower quality than from comparable downflow systems. The under drainage is an essential design component of the filters.

Horizontal-flow roughing filtration

In horizontal-flow roughing filtration, the raw water is directed by horizontal flow through filter material of decreasing size. The filter bed is divided in three or four sections of decreasing length and filled with medium of decreasing size. Field experience has shown that this method is suitable for raw water with high levels of suspended solids. The process, design, cost, and operation and maintenance as well as field experiences are discussed in this paper.

Pebble matrix filtration

The filter medium consists of a matrix of large pebbles of about 50 mm diameter, partly packed with sand. The water flow is downward through this double layer filter. Preliminary results of laboratory tests with different turbidities and filtration rates are discussed in this document.

CHAPTER 2

RIVERBANK FILTRATION

C. Engels and W. Poggenburg

SUMMARY

The processes taking place in riverbank filtration are described at a water abstraction site along the Rhine River. Quality of the river water and riverbank filtrate, and the bio-chemical processes occurring during riverbank filtration are discussed. Improvements in water quality are expressed in characteristic parameters, such as bacterial densities, turbidity, anorganic compounds (for example, trace metals) and organic compounds (dissolved organic carbon (DOC), chemical oxygen demand (COD) and adsorbable organohalides).
The retention time, that is from river to abstraction point, was measured using chloride as parameter.

Riverbank filtration functions as a "natural" treatment of the Rhine river water. Based on bacteriological indicators, turbidity, and levels of organic and anorganic constituents, this treatment brings about a significant improvement in water quality. Up until 1951, Rhine river water which was riverbank filtered only needed chlorination as treatment prior to distribution.

1. INTRODUCTION

The Rhine river has a very high discharge. From its source in Switzerland, the river passes through Lake Constance, along the border between West Germany and France, through West Germany and the Netherlands and finally discharges into the North Sea. From Lake Constance to Rotterdam the river is 1007 km long. It has a relatively steady discharge due to the tributaries in the higher parts of the Alps, the low lands of the Alps and the swiss Jura. The mean discharge at Basel, Switzerland of $1{,}037 m^3/s$, is half of that at Lobith on the border between West Germany and the Netherlands.

There are several important groundwater reservoirs in river and river-glacial sediments of the Quarternary. In the Lower-Rhine very high yielding aquifers occur. They are 20-30 m thick and of high permeability.

Groundwater hydraulics directly correspond to the river regime, which creates the basic condition for a river to act as reliable riverbank filtration source. Another important condition for riverbank filtration is the water velocity. The high water velocity scour and sediment transport result in a natural cleaning (de-silting) of the river bottom and banks. Consequently the infiltration surfaces remain open and clogging of the aquifer between river and recovery site is limited.

On the one hand the Rhine river the main water source for industrial and municipal water utilities on the other hand considerably quantities of cooling water are discharged into it. Water is rarely taken from the river in

West Germany mainly because of uncertainty about its quality and quantity. In general, groundwater abstracted is a mixture of riverbank filtrate and groundwater from the landward side. The retention time of the river water in the aquifer before abstraction may be several weeks. This has a number of advantages:

- reduction of suspended solids, germs, bacteria and viruses;
- balancing fluctuations of chemical constituents;
- considerable reduction of bio-degradable substances present in the river water.

Up until the early 1950s, the riverbank filtrate (actually groundwater of mixed composition) could be distributed without treatment. Deterioration of the river water quality due to discharge of insufficiently treated sewage or industrial waste water into the river made treatment necessary near Dusseldorf. Accelerated water pollution led to the need for increasingly complex treatment. However to keep treatment costs at a reasonable level, pollution levels should be contained as much as possible. The use of riverbank filtrate received priority over other utilization purposes of the Rhine river water. Measures were required to ensure the quality of river water for the water works along the river. To stress the need for action the "Co-operation of Rhine Water Works" was established, and by about 1976/1977 water quality of the river had considerably improved.

2. RIVERBANK FILTRATION IN DÜSSELDORF

In 1985, the Dusseldorf Municipal Water Works was supplying approximately 592,000 people with a total of 60 million m^3/year. The highest daily consumption recorded was 404,000 m^3 in 1976.

The source is groundwater mainly from the river and to a lesser extent from the landward side. Water is pumped from boreholes and galleries at a depth of 18-25 m to three intakes works over a distance of 32 km. The wells are situated 50-250 m from the river.
The proportion of riverbank filtrate and water from the landward side varies with the river level and the amount of water abstracted.

Retention time in the aquifer (from river to abstraction site) is remarkably high. Tracer investigations were carried out to determine the retention periods. The tracer must fulfil criteria such as stability against bio-chemical processes, no adsorption or molecular exchange with soil particles. The chloride anion is a good tracer because ion exchange or absorption and adsorption to soil particles and bio-chemical decomposition of Cl^- do not occur. Therefore, the chloride ion concentration can easily be used to determine the retention time of the riverbank filtrate in the aquifer. Chloride concentration can also be used to determine the ratio of riverbank filtrate to water from the landward side in the abstracted water. The chloride content varies with the river discharge.

During the low-discharge period of 1983/1984 the proportion of the riverbank filtrate (Schubert, 1984) was:

site no.1 (Auf dem Grind): 80-85%, retention time ± 7 weeks;

site no.2 (In der Flehe) : 55-60%, retention time ± 3 weeks

site no.3 (Am Staad) : 70-75%, retention time ± 4 weeks

Since 1961 water for drinking has been treated as follows (Poggenburg,1976):

- ozonation to oxidize anorganic and organic substances;
- iron and manganese removal by a two-step activated carbon pressure filter;
- adsorption of organic substances in the next activated carbon filter (this effluent is of drinking water quality);
- pH correction with sodium hydroxide;
- chlorination before water is pumped into the distribution network.

The quantities of ozone, activated carbon and sodium hydroxide are directly related to the quality of the riverbank filtrate.

3. RIVERBANK FILTRATION PROCESS AND RELATED CHARACTERISTIC PARAMETERS

Discharge of treated sewage greatly affects the bacteriological quality of river water. High bacterial and viral densities are prevalent. The suspended solids reduce the clearness of the water. Anorganic and organic substances adsorbed by the suspended solids cause a odorous smell and taste. The level of these substances, both qualitative and quantitative is determined by the amount of water and industrial waste water disposed of in the river.

Urban and industrial waste water disposal and river discharges determine the presence of substances in the river water both in qualitative and quantitative respect.
When studying the water quality improvement of the river water due to riverbank filtration, the retention time appears to be an important parameter.

3.1 Reduction in germs

The bacteriological quality is monitored by the standard plate count (number of colonies per ml), the total coliform count and the thermotolerant coliform count per 100 ml. These are generally applied tests for drinking water quality.
The sampling was done during winter conditions (low discharge and low water temperature) and in summer (average water level and higher water temperature). The analysis results are shown in table 1.

The reported counts are the geometric means of three tests using different dilutions.

Bacterial densities tend to be lower in summer and higher in winter. However in terms of bacteriological quality the Rhine River is permanently contaminated.

Table 1: Results of bacteriological analysis of Rhine water at Düsseldorf

DATE	COUNT OF COLONIES/ML WATER	E.COLI/ 1oo ML WATER	PSEUDO COLI/ 1oo ML WATER	RHINE WATER-LEVEL M	RHINE °C
11.12.85	1.5oo	dotted	dotted	2,17	8,o
12.12.85	1.2oo	dotted	dotted	2,15	7,6
16.12.85	7.13o	7.o7o	8.67o	1,77	8,8
o7.o1.86	29.97o	7.7oo	5.67o	2,4o	4,4
o8.o1.86	19.38o	39.33o	dotted	2,21	4,o
o9.o1.86	1o.83o	41.5oo	67.5oo	2,12	4,4
21.o7.86	73o	1.33o	4.2oo	2,6o	2o,4
22.o7.86	17o	1.57o	15.o7o	2,59	21,2
24.o7.86	o	17o	4.8oo	2,63	2o,o

E.COLI = ESCHERICHIA COLI

However, results of daily analysis of the bank filtrate indicate that the bacteriological requirements set for drinking water are met most of the year round. Incidentally, for instance in January/February 1986, the raw water (bank filtrate water) contained some coliform but the levels remained low (less than 10/100 ml) both for E.coli and total coliform. Standard plate counts of the raw water are rarely positive, being mostly less than 10/ml. This can be attributed to the fact that during normal river discharges bacteria stick to the suspended solids retained during filtration through the aquifer. Bacteria can break through the aquifer, which acts as a filter, when the water level quickly rises. This results in shorter retention times, or during periods of low discharge, in higher turbidity levels in the bank-filtrate.

3.2 Reduction in turbidity

Suspended solids reduce light penetration and thus the clearness of river water. Turbidity levels are expressed in FTU, formazine turbidity units. The amount of suspended matter is related to the level of the river water, as shown in Figure 1.

Samples were taken on all working days during the low discharge period in 1985/1986. During this time there was also a period when discharges were about average. Samples were taken from a few centimeters below the water surface.

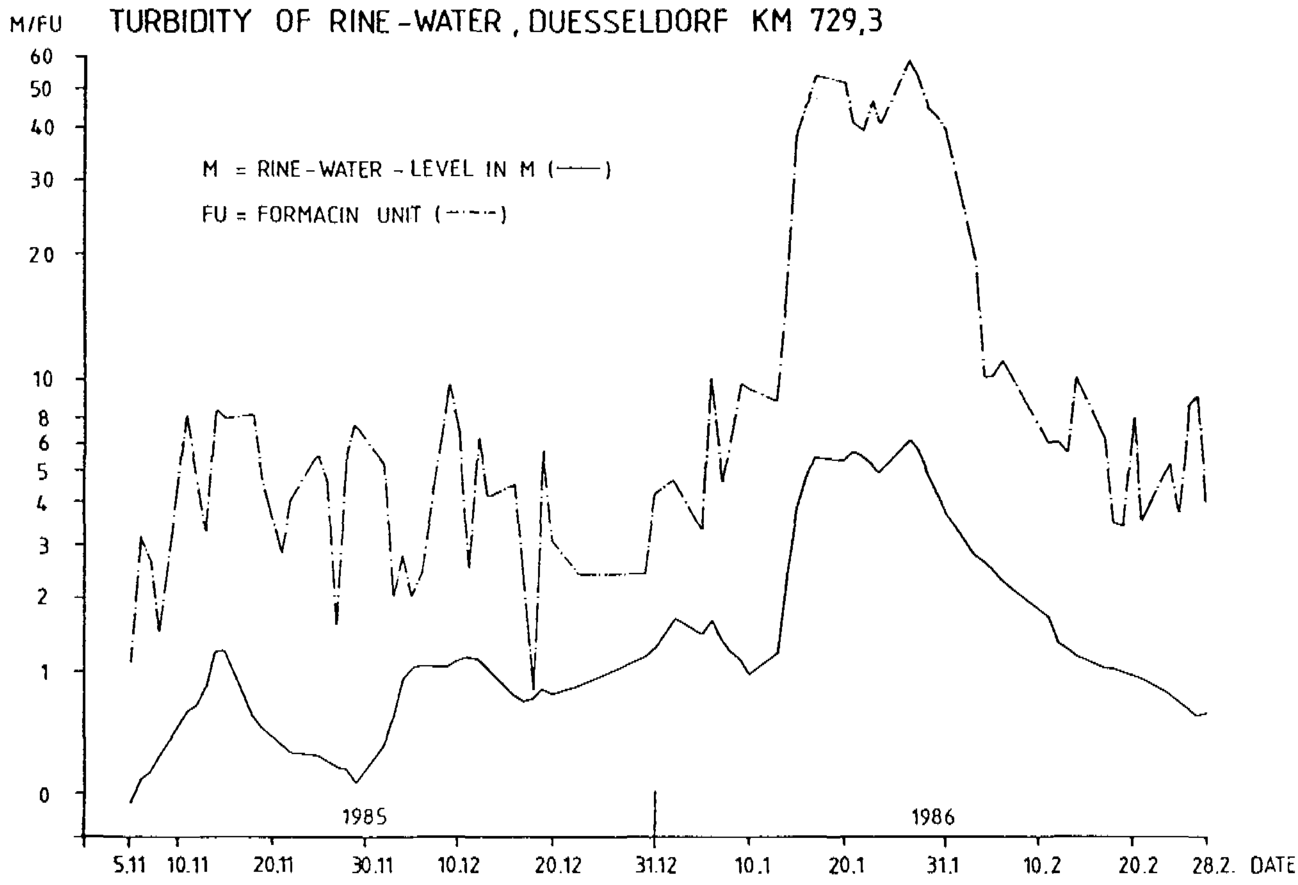


Figure 1. Relationship between river water level and turbidity

In the Rhine River pollution levels increase during lower discharges, but the amount of suspended solids decreases at lower discharges because the reduced water velocity enhances sedimentation of suspended matter. Furthermore, organic and inorganic matter from the riverbanks is not picked up because floods are rare.

At a river depth of less than 2.0 m, the mean turbidity is about 5 FTU. But when the water level rises quickly, 100% increase in turbidity may occur, as measured on 14.01.86. Turbidity may reach values of up to 60 FTU. Subsequent lower water levels do not automatically result in a rapid decrease in turbidity. Suspended matter takes some time to settle.

A positive effect of the filtration of suspended materials is that bacteria, anorganic and organic substances adsorbed by the suspended materials, are also filtered. However, riverbank filtration does not permit backwash to remove the suspended material retained. This feature was studied using a diving-bell in 1954. A "window" was dredged in a hard layer close to one of the water intakes to the Dusseldorf water works. As a result the yield from the source increased, but after some time the old situation was re-established. With time the infiltration area had shifted from the banks to the centre of the stream. The mid-stream river bottom is kept free of sediments because of the scouring action of the water and the movement of sand and gravel. Therefore, riverbank filtration will continue to function, which is proven by 100 years and longer functioning of one of Dusseldorf's intake works.

Compared to river water, turbidity in river bank filtrate is low. Turbidity peaks occurring during high water discharges level out. During the research period, the bank filtered water had a high clarity. The geometric mean for turbidity was 0.38 FTU, in a range of 0.22 - 0.82 FTU. Treatment reduced turbidity further to 0.2 FTU.

3.3 Changes in concentrations of anorganic and organic compounds in the water

The type of processes taking place during the riverbank filtration are evident from changes in:

- trace metals, oxygen, manganese, ammonium and nitrite;
- parameters expressing the total amount of organic and biological degradable compounds (sum parameters such as: dissolved organic carbon (DOC), adsorbable organohalides and chemical oxygen demand (COD)).

3.3.1 Reduction of trace metals in riverbank filtration

Trace metals in groundwater and surface water come from both natural and man made sources. The following metals were monitored in river and bank filtered water: arsenic, lead, cadmium, chromium, copper, nickel, mercury and zinc. Iron and manganese may be oxidized in the soil but oxides present in the soil may also be dissolved. The metals are present in the mg/l range, while the trace metals are in the μ/l range.

The highest concentrations of trace metals in river water were detected in 1975-1976. Reductions in concentrations since then can be attributed to the waste water treatment and recycling plants. At present the concentrations of trace metals are low. However, monitoring should continue and any increases in levels controlled. Concentrations of trace metals in Rhine river water and riverbank filtrate in 1976 are given in Table 2. Levels of riverbank filtrate have never reached alarming values, not even when high concentrations have been recorded in the river itself.
Trace metals are retained in the soil with the suspended materials which act as natural flocculants. Exchange, absorption and adsorption processes contribute to this as well as formation of insoluble metal oxides.

Tabel 2: Concentrations of trace metals in the Rhine river and the riverbank filtrate.

METALS, 1976

RHINE-WATER, BAD HONNEF, KM 645 F I 1 I

RAW WATER, DÜSSELDORF TREATMENTPLANT FLEHE, KM 729,3

RHINE-WATER: EIGHT WEEKS COLLECTING SAMPLES

RAW WATER: ONE SAMPLE PER MONTH nd = NON DETECTABLE

TOTAL METAL	RHINE-WATER VALUES			RAW WATER VALUES		
	MEAN-	MIN-	MAX-	MEAN-	MIN-	MAX-
ARSENIC						
µg/l	7,7	6,o	9,o	nd	nd	nd
mmol/m³	1o·1o^{-2}	8·1o^{-2}	12·1o^{-2}	nd	nd	nd
CADMIUM						
µg/l	5,3	1,o	26,o	o,5	o,2	o,8
mmol/m³	4,7·1o^{-2}	o,9·1o^{-2}	23,1·1o^{-2}	o,4·1o^{-2}	o,2·1o^{-2}	o,7·1o^{-2}
CHROMIUM						
µg/l	7,3	5,o	1o,o	o,8	nd	2,1
mmol/m³	14,o·1o^{-2}	9,6·1o^{-2}	19,2·1o^{-2}	1,5·1o^{-2}	nd	4,o·1o^{-2}
COPPER						
µg/l	21,3	1o,o	35,o	8,3	5,4	12,3
mmol/m³	4,9·1o^{-1}	1,6·1o^{-1}	5,5·1o^{-1}	1,3·1o^{-1}	o,8·1o^{-1}	1,9·1o^{-1}
LEAD						
µg/l	22,3	1o,o	31,o	2,3	1,3	5,8
mmol/m³	1o,8·1o^{-2}	4,8·1o^{-2}	15,o·1o^{-2}	1,1·1o^{-2}	o,6·1o^{-2}	2,8·1o^{-2}
MERCURY						
µg/l	o,15	o,o2	o,35	o,2	nd	o,8
mmol/m³	7,4·1o^{-4}	1,o·1o^{-4}	17,4·1o^{-4}	1o,o·1o^{-4}	nd	39,9·1o^{-4}
NICKEL						
µg/l	8,3	5,o	1o,o	2,9	o,8	5,4
mmol/m³	14,1·1o^{-2}	8,5·1o^{-2}	17,o·1o^{-2}	4,9·1o^{-2}	1,4·1o^{-2}	9,2·1o^{-2}
ZINC						
µg/l	62,3	4o,o	11o,o	6,8	1,8	22,6
mmol/m³	9,5·1o^{-1}	6,1·1o^{-1}	16,8·1o^{-1}	1,o·1o^{-1}	o,3·1o^{-1}	3,5·1o^{-1}

Each trace metal has its specific reduction factor. Whether trace metals are dissolved or bonded, depends on a number of factors, such as the type of bond in the metal compound, conditions and criteria for the dissolved state (pH, redox potential, conductivity, presence of organic complex binding agents, biological activities, temperature) and the geohydrology (Förstner et al, 1985).

Settled oxides may dissolve again if the geohydrology or groundwater composition changes. This applies, for example, to the manganese concentration in riverbank filtrate.

Since 1950 the oxygen content of Rhine water has been decreasing because of increasing surface load of organic compounds. This has also had an adverse effect on bio-chemical processes in the groundwater and subsequently on the quality of river filtrate. Bio-degradable material is not completely decomposed in the river and the remaining oxygen is not sufficient for complete oxidation in the aquifer between riverbank and abstraction point. Oxygen levels in riverbank filtrate of up to 5 mgO_2/l (125 to 157 $mmol/m^3$) were recorded until mid-1953, and since then values have not been higher than 1 mgO_2/l (less than 31 $mmol/m^3$).
Oxygen demand of the micro-organisms present in the subsoil have been met by reduction of nitrate to nitrite and ammonium. This reduction process only occurs if biological degradable organic substances are present and the oxygen content is less than 1 mgO_2/l. Further biological reduction such as of manganese and ferrous-oxides (which are insoluble compounds) will occur. Then concentrations of nitrite, ammonium and manganese in river bank filtrate increase.
Since 1976, oxygen level content in the Rhine river have increased because pollution upstream is less, consequently levels are higher again in bank filtrate. The reduction processes no longer takes place and the natural levels have been re-established (Engles, 1985) as follows:

- nitrite and ammonium concentration (1.1 $mmolNO_2/m^3$, 2.8 $mmolNH_4/m^3$)	± 0.05 mg/l
- manganese (3.6 $mmolMn/m^3$)	± 0.2 mg/l
- nitrate (161-242 $mmolNO_3/m^3$)	± 10-15 mg/l

3.3.2 Reduction of organic compounds by riverbank filtration

Several thousands of organic constituents are present in the Rhine River. Sum parameters used are dissolved organic carbon (DOC), adsorbable organohalides, adsorbable organic sulphur and the spectrum absorption coefficient at 254 nm.

The degradability of the organic substances is measured by the COD parameter, using potassium dichromate or potassium permanganate.

DOC and COD (using $K_2Cr_2O_7$) values for the Rhine river and the riverbank filtrate between 1970 and 1985 are given in Figure 2. Water quality of both river water and the bank filtrate improved as a result of reduction in organic substituents. Purification is augmented by subsoil bio-chemical processes and the reduction in turbidity and materials adsorbed to the suspended material.

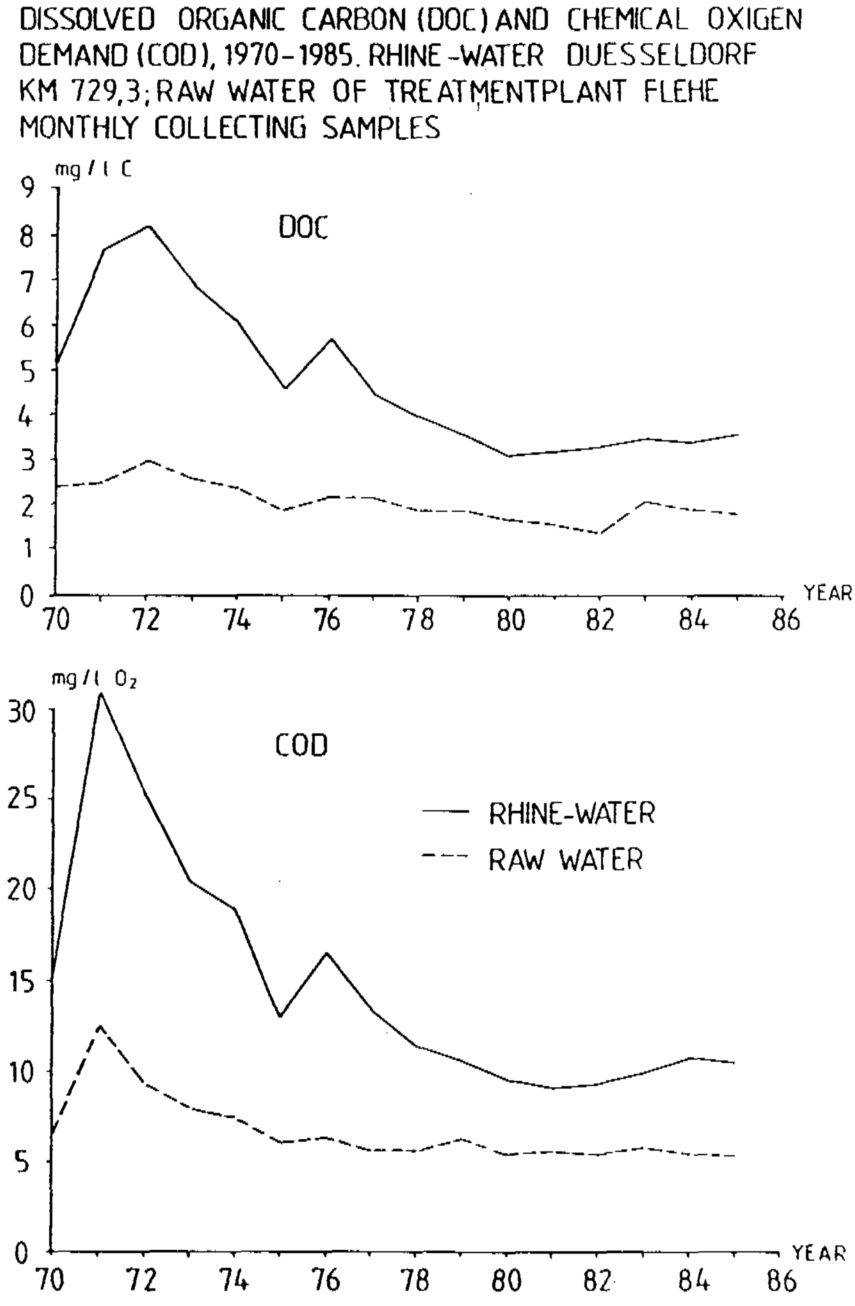


Figure 2. Dissolved organic carbon (DOC) and chemical oxygen demand (COD) in the Rhine River and at one of the intake sites of the Düsseldorf Water Works, between 1970 and 1985.

Levels of organic halogenated compounds were also reduced during bank filtration. In 1985 the mean monthly adsorbable chloro-organics concentrations (mg/l) were:

	mean value	minimum	maximum
Rhine	0.067	0.043	0.104
riverbank filtrate ("In der Flehe")	0.028	0.020	0.043

In addition, single constituent analysis were done for chloroform, tri- and tetra-chloroethylene, chloro- and nitro-benzene, toluene and nitrilotriacetate (NTA) and other substances having toxic effects, disturbing the treatment process or being difficult to remove.

REFERENCES

Engels, C. (1985). Eliminationsprozesse bei der Trinkwasseraufbereitung hinsichtlich der organischen und anorganischen Schadstoffe. Gewässerschutz, Wasser, Abwasser. Vol. 69, p. 447-460.

Förstner, U., Calmano, W. and Schoer, J. (1985). Distribution of Trace Metals between Solution and Particulates - Current Questions of Sediment Research from Water Quality Practice. Vom Wasser. Vol. 64.

Poggenburg, W. (1976). Die aufbereitung von Rohwasser zu Trinkwasser - Verfahren, Betrieb Probleme. Deliwa, No.3. p. 76-80.

Schubert, J. (1984). Auswirkungen der Niedrigwasserperiode 1983 auf die Beschaffenheit des Mischgrundwassers. Jahresberichte der Arbeitsgemeinschaft der Rheinwasserwerke. p. 192-220.

CHAPTER 3

RIVER BED FILTRATION SYSTEMS

J.E.M. Smet, E. Quiroga and H. Wolters

1. INTRODUCTION

In river bed filtration, water is filtered through the natural river bed material, or if the permeability is too low, through an artificial bed of suitable material, such as coarse sand and gravel. The filtered water is then collected by perforated or porous pipes dug in the river bed (Figure 1). In rivers discharging coarse sand, a dam could be constructed across the river behind which a sand body will be collected over a period of time. This could be used as water storage and infiltration bed.

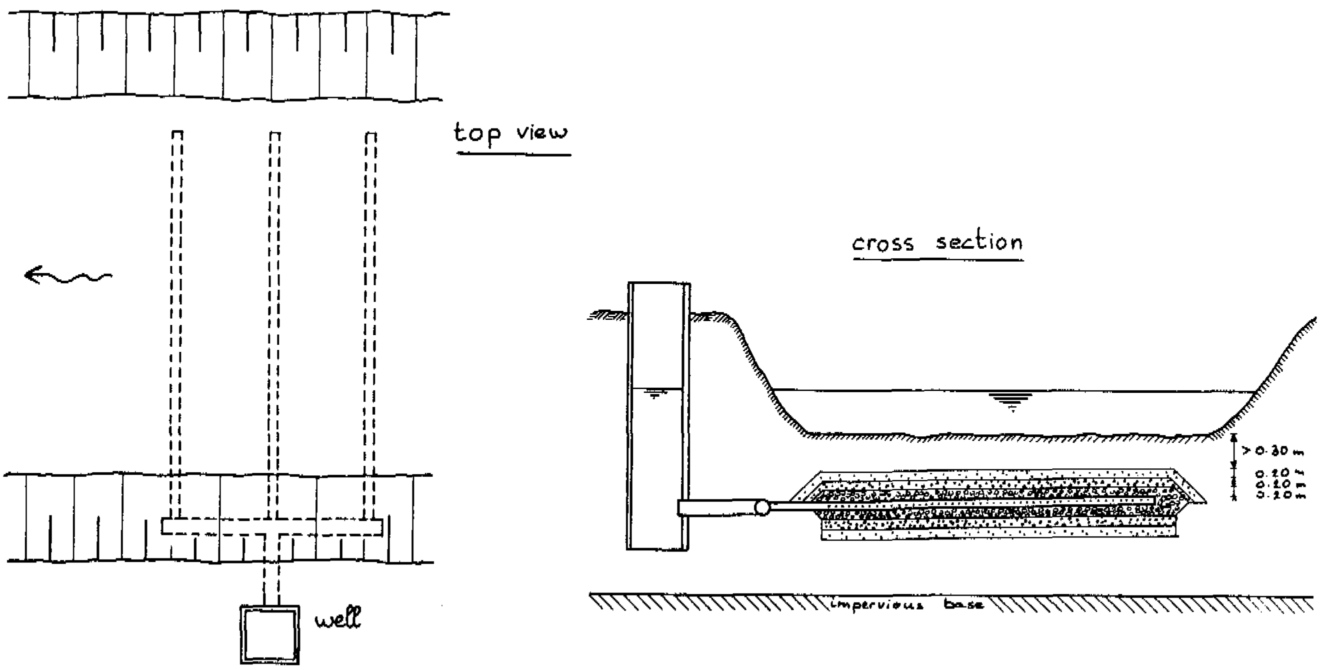


Figure 1. River bed filtration

As water of high turbidity passing through such a river bed filter is particularly treated for substantial removal of suspended solids and bacteria, river bed filtration is a potential pre-treatment option prior to slow sand filtration.

River beds have been used in many countries for centuries as safe and reliable water sources. The simplest type of river bed filtration, a small temporary hole dug in the dry river bed, can still be seen in many developing countries. In more advanced systems using this principle dams are constructed to surface water and an underdrain and pipe system is connected to an intake chamber or sump to collect the water. The water is abstracted from the collection system using engine driven pumps or hand pumps.

The amount of water to be drawn or pumped from a filtration site will depend largely on the permeability, the filter bed storage capacity and the abstraction means.

Several river bed filtration systems used with different water retaining methods such as sub-surface dams, sand dams and concrete intake works are described:

1. Longitudinal drain system. This filtration system which may be used for small capacity water supplies often has a single collection drain usually placed in a narrow river bed in the direction of the current. A dam may need to be constructed in the river to increase the capacity of the filtration system.

2. Lateral drain system connected to a well. Often these drain systems consist of a collection pipe dug in the natural river bed. This method is suitable for small communities, but for larger communities, several radial collector drains need to be installed. Construction of a dam could increase the yield of the system.

3. Vertical river bed filtration. In principle, the system consists of a vertical filter artificially installed in the river bed with collectors in the bottom of the filter. The filter area depends on the required capacity. The filtered water is collected in an intake chamber and pumped or supplied by gravity to the treatment plant.

4. River dam filtration system. In this system, a gravel pack with collectors is installed right behind a dam in the river. The gravel may be placed either in or on the river bed, depending on river characteristics such as turbidity levels, sediment transport, and gradient.

The modular sub-sand abstraction systems which are often used in Africa are described in chapter 4.

In the tropics river discharge vary considerably throughout the year, and are sometimes negligible at the end of the dry season. Thus dams could be constructed to increase the storage capacity to meet water demand.

River bed filtration systems use devices or materials installed in the river bed to permit abstraction of river water, while in riverbank filtration, production wells are located some distance from the river.

2. DESCRIPTION OF THE SYSTEMS

Process

In river bed filtration surface water passes through a filter, which is either naturally present or is constructed in the river bed. The composition of the filter bed and the filtration rate determine the degree of water treatment. A filter bed usually comprises coarse material, such as gravel of 2-25 mm diameter built up in layers. Gravel size increases towards the slotted drains and should be of sufficient diameter not to block the pipe orifices. The top layer of finer material will retain most of the solids suspended in surface water. If coarse filter material is used in the top layer, deep penetration of the suspended solids will clog the entire filter. This will then require cleaning of the filter which involves complete removal and washing of all the filter medium.

Sedimentation and straining in the filter bed contribute most to the physical compound reduction process. Microbiological and physico-chemical processes may also occur, but because of the relatively high filtration rates, are of minor importance. Adsorption of constituents to biological slime layers formed around gravel may be substantial.

Coarse filter material may not permit removal of very small suspended material, particularly colloidal material. The capacity of river bed filtration to remove oxidizable organic material, colour, iron and/or manganese depends on the oxygen content of the water. However, the capacity is usually not high because of the short detention time (CEPIS/PAHO, 1982).

Design

River water characteristics, such as river discharge, suspended solids content and particle distribution, and bacteriological quality, should be investigated prior to the design. This should be done preferably over a one-year period. This information could be used to determine the required filter bed composition and thickness. Ideally, pilot tests should be carried out to determine the optimal gravel pack composition to meet influent turbidity requirements for slow sand filtration, being preferably less than 10 NTU.

Dams or weirs could be constructed to ensure continuous water supply also during the dry season when the river flow may decrease substantially. Apart from increased storage capacity, the storage basin also acts as a plain sedimentation reservoir in which much of the heavier particles suspended in the water will settle. Water quality may also improve due to the extended retention time and the consequent microbiological and physico-chemical processes, provided the water source is protected from pollutants and contaminants. Evaporation from open reservoirs can be considerable and should be taken into account, as well as the resultant increase in dissolved solids in the impounded water.

In rivers of high discharges, the filter could be constructed in a branched-off channel. A weir in the main river and a sluice gate in the channel will regulate the flow over the filter. Diversion channels or spill ways should be included in the design to prevent damage to dams and weirs during periods of high river discharges.

The design filtration velocity of surface water through the filter bed ranges from 0.25-1.5 m/h, depending on turbidity levels and effluent requirements. When the daily design consumption is known, the required filter bed area can be calculated.

The filter bed could best be placed in the middle of the river bed because the highest water velocities there give good scouring action. This would reduce problems, such as blocking of the filter bed because of caved river banks.

The gravel pack composition of the filter bed depends on the raw water turbidity and the effluent requirements. Pilot testing is required. Gravel for a vertical river bed filter to treat water of turbidity level of 500 NTU could be as follows:

- upper layer (0.20 m), gravel size 2- 5 mm
- second layer (0.20 m), gravel size 5-10 mm
- third layer (0.20 m), gravel size 10-15 mm
- bottom layer (0.20 m), gravel size 15-25 mm

The gravel in the bottom layer should be two to three times larger in diameter than the orifices.

The diameter of the main and lateral collectors should be calculated for a velocity between 0.1 and 0.5 m/s. The drains are laid at a slope of 1% towards the main and at intervals of 1.0 to 1.5 m. Orifices of 5-10 mm diameter are drilled in two lines at a 30° angle with the horizontal line at 150 mm intervals (Figure 2).

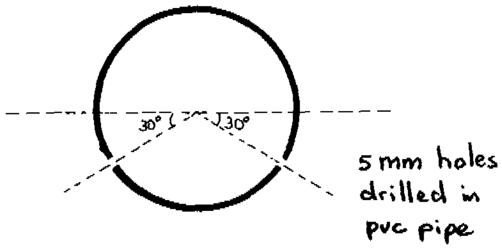


Figure 2. Orifices in collection drain

Operation

If the surface water quality for a period of the year meets the criteria set for the influent of the subsequent treatment, then river water may be abstracted directly, thus bypassing the filter. On the other hand, when the pollution level is temporarily very high, abstraction may be stopped until water quality has improved. Therefore, sufficient storage capacity should be included in the design.

Suspended material which settles on and in the river bed will gradually block the filter. If the flow velocity is high, for instance during the rainy season, these particles may be resuspended by scour forces and transported. Raking the filter bed with a long dented rake also resuspends material retained in the upper part of the filter. This will only be effective if there is sufficient flow to discharge the very turbid water. Flushing the impounded water through dam gates is also a feasible option. This is best done during the rainy season when re-fill of the reservoir is ensured. The aim is a dynamic equilibrium between settlement of suspended material and resuspension, and the subsequent transport of settled material. Where there is sufficient water pressure, such as in mountainous areas, and sufficient water quantity, backwashing with raw water from upstream via the collection drains may be considered. The purpose of backwashing is not to obtain a filter bed expansion as in rapid sand filtration practice.

The rate of clogging depends on the filter composition and raw water characteristics. After some time (up to several years) when clogging has increased filter resistance to the maximum, the filter material has to be dug out manually, washed and replaced.

3. DESCRIPTION OF DIFFERENT TYPES OF RIVER BED SYSTEMS

3.1. Longitudinal Drain System

3.1.1 Process

In the longitudinal drain system the collectors are placed longitudinal to the river bed (Figure 3). The required capacity of the system and river characteristics determine whether a dam needs to be constructed. The geo-hydrological situation determines the flow characteristics towards the drain collection system. Mostly the system is fed from the direct river discharge. In the Waiya Dam case in Kenya (Volkhard, 1983), the river water was forced to infiltrate the neighbouring groundwater aquifers before flowing into the filter. This can only be done where soil characteristics are suitable.

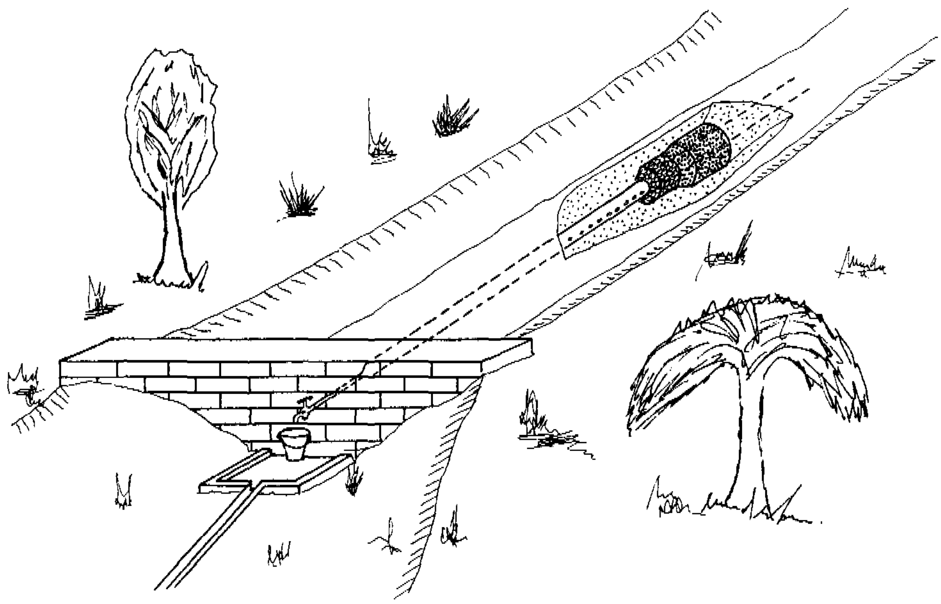
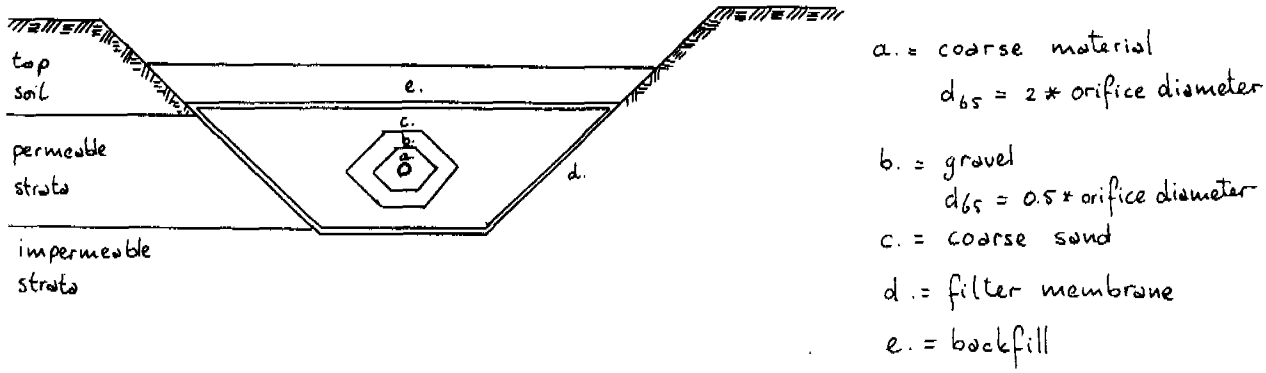


Figure 3. Longitudinal drain system

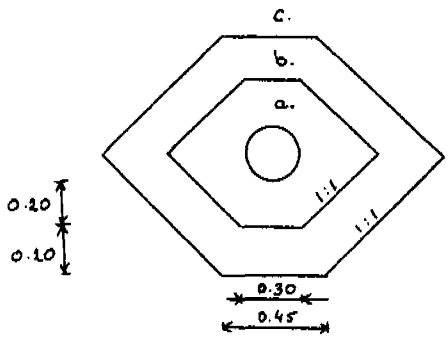
3.1.2 Field Experience

Waiya Dam, Kenya

A longitudinal drain system was placed in the bed of the reservoir behind the Waiya Dam, Machakos District, Kenya. Volkhard (1983) reported on the performance of the filter, which consisted of a perforated pvc pipe ϕ 100 mm of 21 m length embedded in gravel (Figure 4). The pipe had orifices of 5 mm diameter at intervals of 50 mm. The gravel pack around the collection drain was carefully graded and installed. This was covered with an impermeable and slightly compacted backfill (0.5 m) to prevent direct seepage into the filter. The filter is fed from the permeable strata of the reservoir. Subsoil composition near the filter largely determines its performance. Tests results indicating filter performance are given in Table 1. The drain pipe is connected to a draw-off pipe.



Cross-section



Detail of filter

Figure 4. Cross-section and detail of longitudinal drain system (Source: Volkhard, 1983).

Table 1: Test results of raw and filtered water at Waiya Dam

	Unit	Raw water	Filtered water	Percentage increase/ decrease
pH		6.5	7.9	+ 21
turbidity	NTU	325	< 5	- 98
conductivity	$\mu S/cm$	62	680	+ 997
total hardness	mg/l $CaCO_3$	126	376	+ 198
calcium	mg/l	1.3	114	+ 8669
magnesium	mg/l	3.3	22	+ 567
sodium	mg/l	3.8	58	+ 143
total alkalinity	mg/l $CaCO_3$	12	174	+ 1350
chloride	mg/l	2.3	41	+ 1683
total coliforms	per 100 ml	>1800	NIL	- 100
faecal coliforms	per 100 ml	25	NIL	- 100

(single sampling)
Source: Volkhard, 1983.

Considerable changes in concentrations of chemical compounds occurred, in particular the mineral content of the water increased. This was probably due to the high mineral content of the adjacent soil strata and the leaching effect of the water passing through. Turbidity levels and bacterial densities were greatly reduced. Most of the effluent parameters values remain within the Guidelines for Drinking Water Quality as published by WHO (1984).

Tested over a three hour period Waiya Dam yielded 22 l/min (1320 l/h). Yield was expected to decrease because of sedimentation of fine material in the reservoir.

The limited operation and maintenance required means that these systems can be maintained at village level and recurrent costs are very low. Only when the strata adjacent to filter or the coarse medium layers covering the filter and drains become blocked, are major inputs of funds and labour needed.

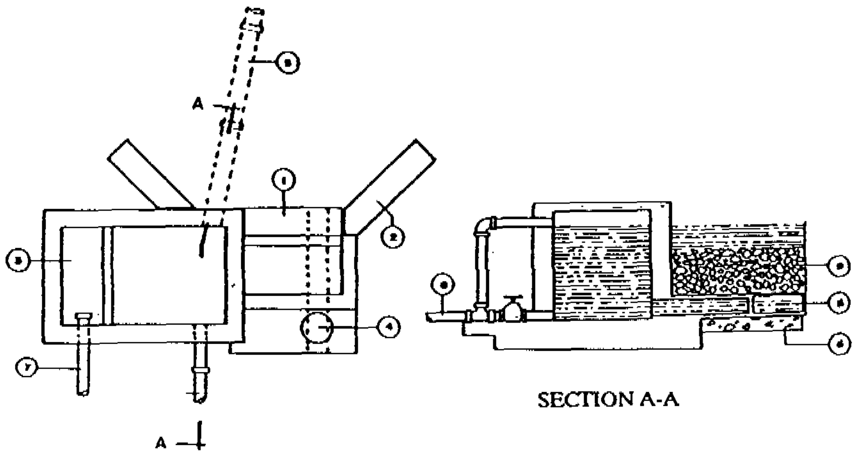
The cost of filter construction at the Waiya Dam, including 21 m of filter, was approximately US$ 7250 or about US$ 350 per metre (1981 price level). This system was estimated to cost 3-8 times more than that for the conventional intake method from a reservoir.

<u>Small Dynamic Roughing Filter</u>

A small dynamic roughing filter of 3.6 to 36.0 m 3/h (1-10 l/s) capacity was designed by Servicio Seccional de Salud del Cauca, Cauca Department, Colombia (Figure 5). The term dynamic applies to the continuous washing/cleaning of the upper filter layer by the water flow. A balance is sought between the settling of small particles in the upper layer and re-suspension of these particles by the river flow.

The main components are the small dam, collection drain around the filter and filtered water sump. The filter comprises four layers of gravel 6 to 36 mm in size, with a collector of 200 mm diameter and orifices of 9 mm diameter.

After chlorination the filtered water is distributed. No data are available on the performance of the system as yet.



1. dam
2. side walls
3. filtered water sump
4. drainage valve
5. collecting drain
6. filter's base
7. filtered water outlet
8. drainage valve for filtered water sump
9. filter bed

Figure 5. Small dynamic roughing filter
(Source: Servicio Seccional de Salud del Cauca).

3.2 Lateral Drain System

3.2.1 Process

In a lateral drain system the collection pipes are placed perpendicular to the course of the stream. The simplest lateral drain system is a horizontal drain buried in the river bed and connected to a dug well in the river bed or on the river banks (Figure 6). Various materials can be used for the drains. The productivity of the system largely depends on the permeability of the river bed material. This can easily be improved by digging a channel and installing a longitudinal drain system as described in section 3.1.

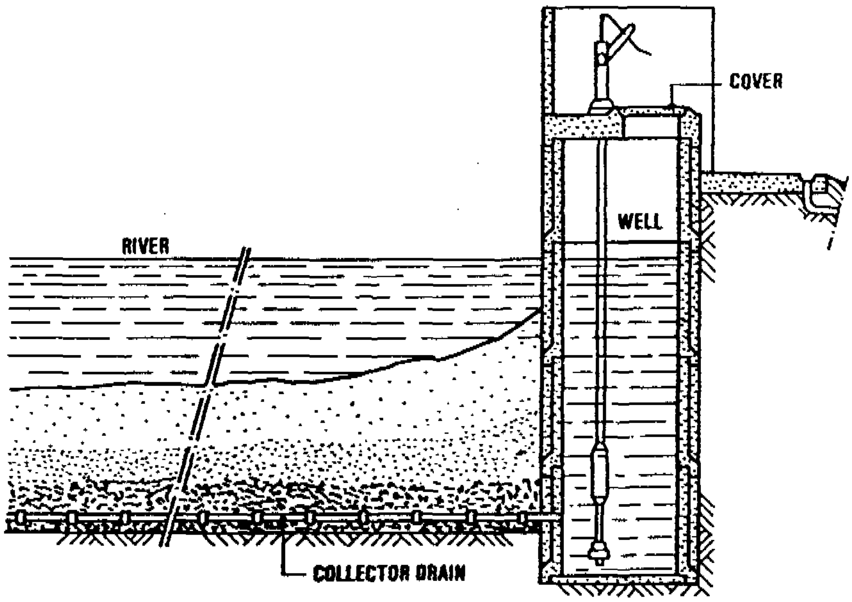


Figure 6. Lateral river bed system connected to dug well.

When discharge and bed characteristics of the river are favourable, large amounts of water can be filtered by river bed filtration. The large collection area required then is obtained by installing in the river bed radial collectors which are connected to a central infiltration well (Figure 7). Alternatively several infiltration galleries with radial collectors could be constructed. Both systems have been used in India for capacities between 10,000 and 20,000 m^3/d.

A dam or weir could be constructed to ensure continuous production. If accumulation of river sediments behind the dam is to be prevented then provisions for flushing have to be included in the design.

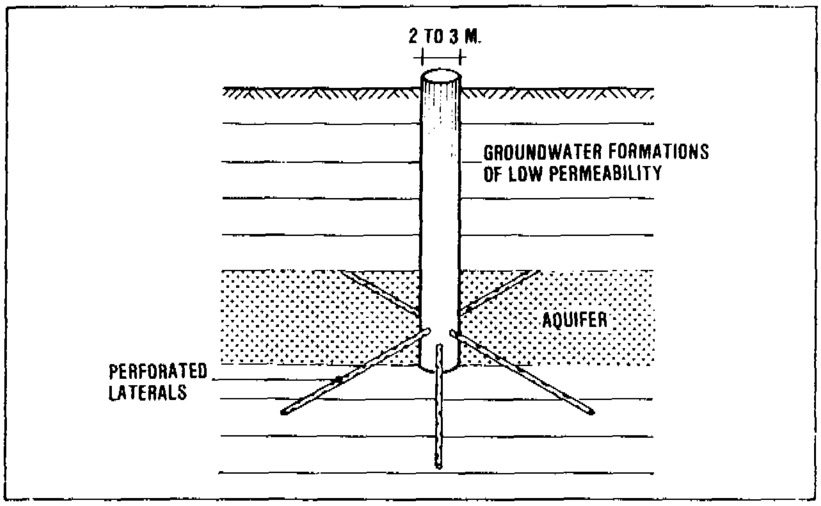


Figure 7. Infiltration well with radial collectors

3.2.2 Field experience

In the Indian State, Tamil Nadu, several river bed infiltration works have been constructed of capacities up to 20,000 m^3/d. The water is chlorinated before distribution. NEERI did a rapid appraisal of the function and effects of several of these works (NEERI, 1988). The Kochadai installation near Madurai city is an infiltration gallery of 20,000 m^3/d designed capacity. The gallery comprises open jointed pipes (diameter 450 mm) surrounded by graded gravel and sand. The total length of the collectors is 680 m. The surface flow of the Vaigai River is ensured by flow regulation at the upstream dam. Performance data are given in Table 3.

The BHEL installation near Puthapuram in Trichy District, Tamil Nadu State, is an infiltration well of 13,600 m^3/d designed capacity. The three radial collectors of 375 mm diameter are 240 m in total length. The collector well is 6 m in diameter (Figure 8). Performance data are also presented in Table 2.

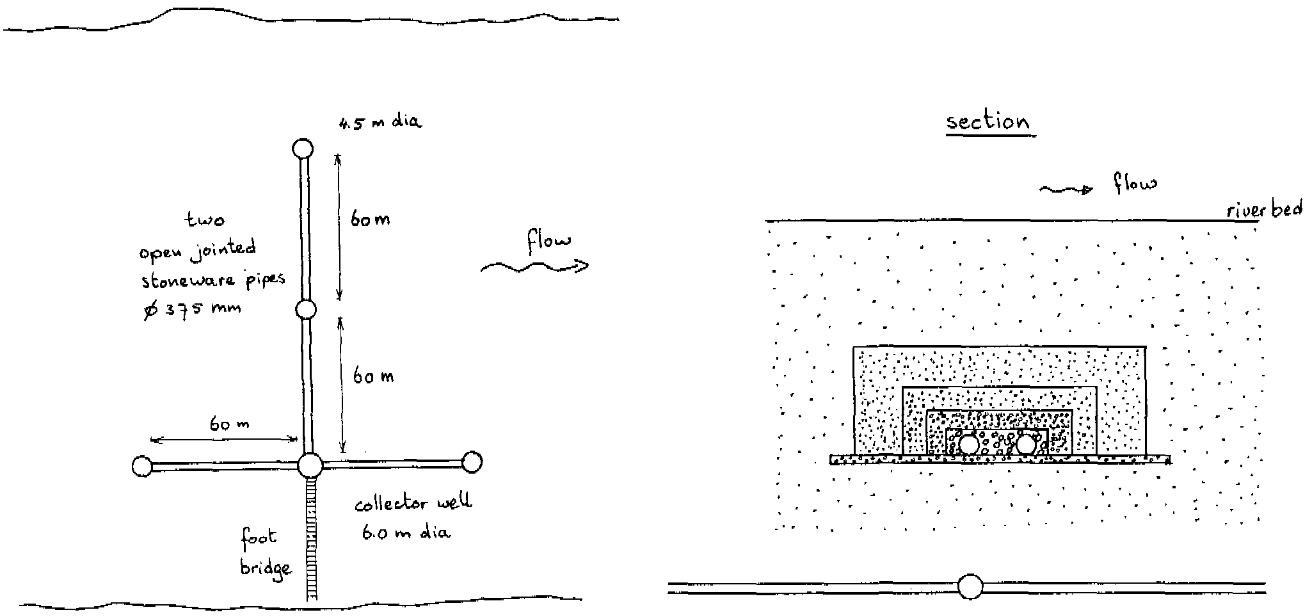


Figure 8. Infiltration well with radial collectors in Tamil Nadu, India (NEERI, 1988)

Experience with river bed filtration works in Tamil Nadu indicates a gradual decrease in yield over a number of years as a result of clogging of the filter bed.

Table 2. Performance data of two river bed filtration works in Tamil Nadu State, India

	Kochadai near Madurai		BHEL near Puthapuram	
	raw water	filtered water	raw water	filtered water
Turbidity (NTU)	4.75	0.4	3.4	0.4
Iron (mg/l)	0.22	traces	0.2	nil
COD (mg/l)	6.2	3.6	16.0	4.9
Total coliforms (MPN/100ml)	240	5	35000	540
Faecal coliforms (MPN/100ml)	33	2	1700	27

(single sampling)
Source, NEERI, 1988.

3.3 Vertical River Bed Filtration

Process

In principle, the filter consists of a vertical filter installed in the river bed. This is an area dug out to a depth of about 1.00 m (Figure 9). The size of the area depends on the volume of water required and the filtration velocity selected (0.25-1.50 m/h).

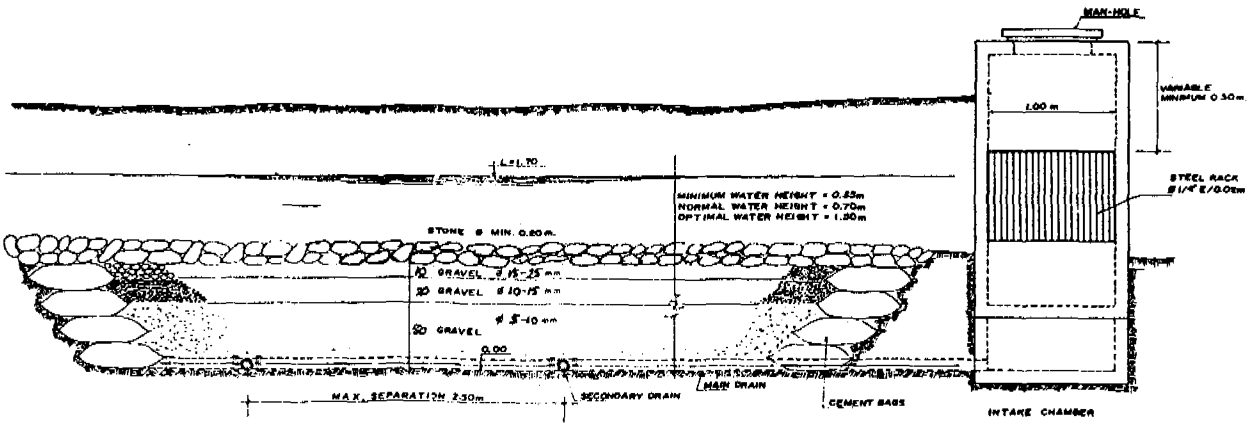


Figure 9: Vertical river bed filter (Source: CEPIS/PAHO, 1981).

At the bottom of the excavated area, slotted pvc pipes are laid (intervals between 1.00 and 2.00 m) to collect and convey the filtered water to the main drain. The filter bed is lined with impermeable and durable material, such as pvc or pe foil covered with concrete tiles, large stones, or plastic woven bags filled with a cement and sand mixture. The excavated area is filled with a filter pack of several gravel layers ranging from coarse (25 mm) to fine (2 mm), and may be covered with large stones to prevent scour of the filter material.

The main drain, dug in the river bank, is connected to the intake chamber constructed of masonry or concrete. The intake chamber could be fitted with a screen gate to direct water intake if the river bed filter is to be bypassed (Figure 10). The water passes to the treatment plant or directly to the distribution system.

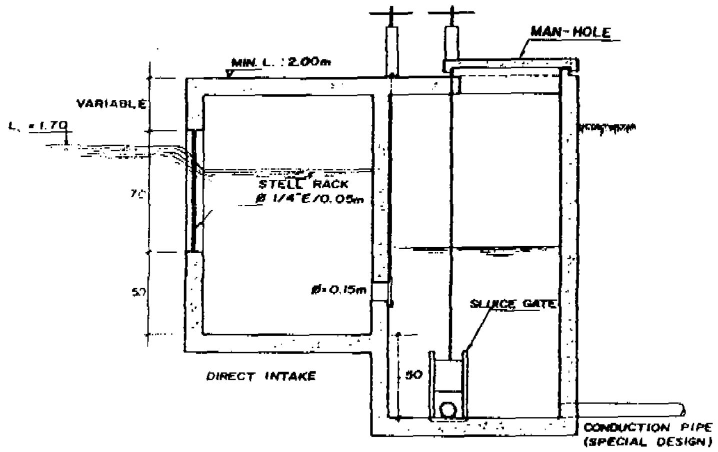


Figure 10. Intake chamber of a vertical river bed filtration system (Source: CEPIS/PAHO, 1982).

A minimum water level over the filter of 0.35 m should be maintained. This may require construction of a dam downstream of the filter bed.

For the given vertical river bed filter CEPIS/PAHO developed tables for dimensioning filters for the recommended filtration rate of 0.25 m/h. A sluice gate or other type of valve is to be used to control the filtration rate. As already discussed, it is not advisable to use the full river width. Diameters of main and secondary drains can be calculated using the CEPIS/PAHO design table (see Table 3).

Table 3. Filter area and drain pipes dimensions for vertical river bed filters (filtration rate 0.25 m/h)

Capacity (m^3/h)	Area (m^2)	Criteria for drain selection: Max. drainable area (m^2)	Criteria for drain selection: Diameter (inches)
3.6	14.4	7.4	2
5.4	21.6	16.8	3
7.2	28.8	30.0	4
9.0	36.6	48.2	5
10.8	43.2	53.0	5
14.4	57.6	53.0	5
18.0	72.0	94.0	6
21.6	86.4	94.0	6
25.2	100.8	230.0	8
28.8	115.2	230.0	8
32.4	130.0	230.0	8
36.0	144.0	230.0	8

Source: CEPIS/PAHO, 1982.

This river bed filtration system can supply water up to a capacity of 36 m^3/h.

3.4 River Dam Filtration Systems

3.4.1 Process

A gravel pack with a piping system is installed behind a dam in the river to collect the filtered water. Instead of a direct intake, the water could firstly pass through a gravel pack which retains most of the suspended solids. Prior to this plain sedimentation occurs in the reservoir created.

The intake chamber may also act as a sedimentation tank prior to transport of the water to the treatment plant. Flush arrangements in this tank such as a drain pipe for the accumulated sediments should be included in the design.

WHO/SEARO developed the following river dam filter (Figure 11) comprising of layers of 0.20 m thickness each.

- coarse sand 0.3 - 2.5 mm
- gravel 2.5 - 5.0 mm
- coarse gravel 5.0 - 10.0 mm
- coarse gravel 10.0 - 20.0 mm

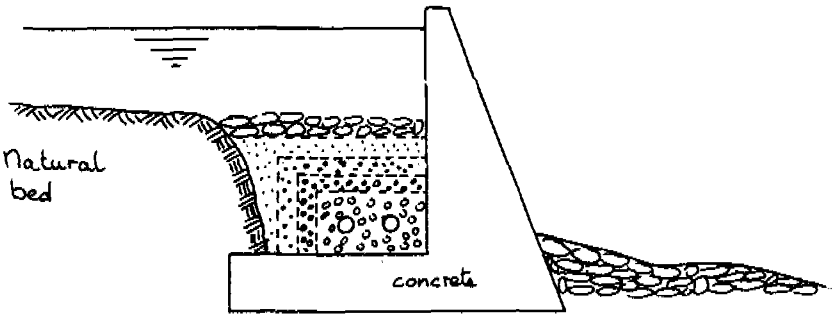


Figure 11. River bed filtration behind dam (Source: adapted from WHO/SEARO, 1976).

3.4.2 Field experience

Dynamic Roughing Filtration, in Colombia

In Colombia two designs have been used depending on the capacity (Salazar, 1980):

- Design 1: capacity 12 to 36 m^3/h (Figure 12)
- Design 2: capacity 36 to 144 m^3/h (Figure 13).

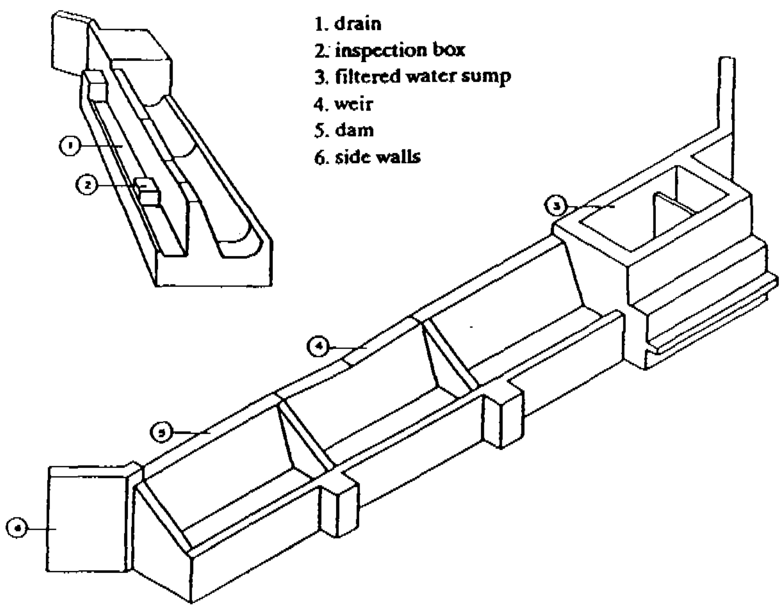


Figure 12. A small river dam filtration system, design capacity 12-36 m^3/h (Source: Salazar, 1980).

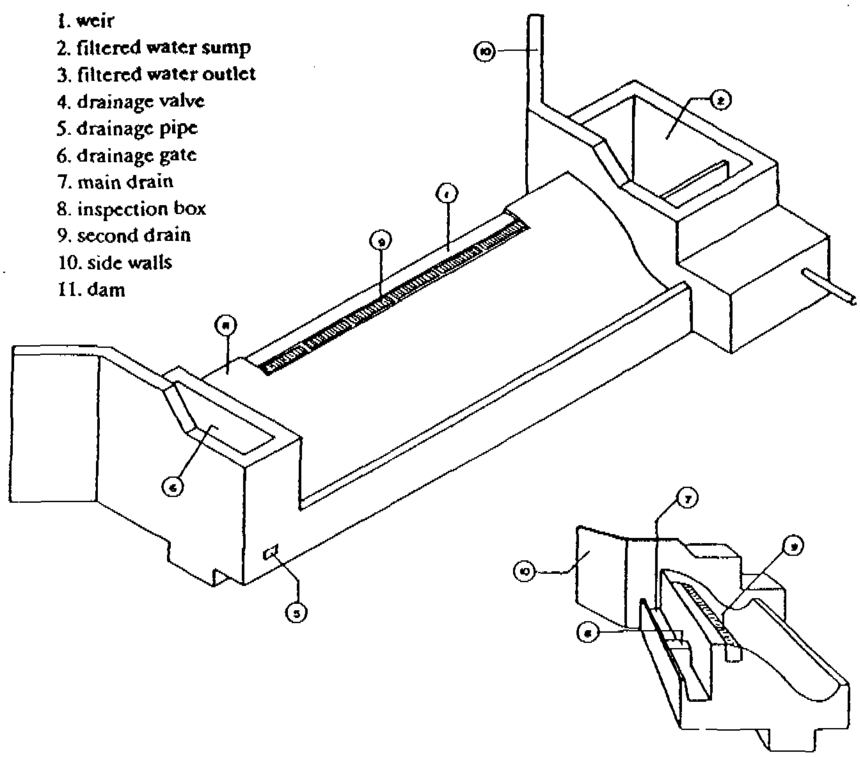


Figure 13. Larger river dam filtration system, design capacity 36-144 m^3/h (Source: Salazar, 1980).

More than 30 of these dynamic roughing filters have been constructed in Cauca Department, Colombia.

The basic components of the pre-treatment works include:
- a small concrete or masonry dam with a U-shaped weir to attain a minimal overflow to remove floating material;
- dam wings to prevent damage to the banks and seepage;
- gravel bed;
- perforated pipes to collect filtered water;
- intake chamber annex settling tank.

A gravel bed, in a triangular shape is placed on top of the collecting drain at a slope of 2:1 or 3:1. The gravel bed acts as the main filter medium but also as a basis for a naturally developed sand bed, on top of the gravel bed by accumulation of settled coarse material. This layer will support filtration of suspended solids. The dynamic characteristics refer to the dynamic equilibrium of the filter's naturally developed sand bed, that is as particles settle continuously on top of this filter bed, others are washed out and flushed away by the river stream (Figure 14).

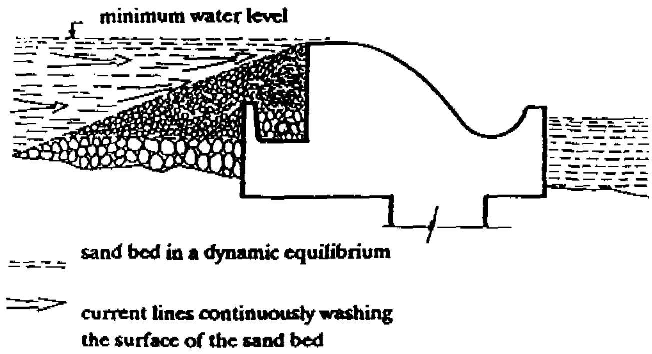


Figure 14. Dynamic equilibrium of the filter's sand bed.

In the dynamic roughing filters in Cauca Department, Colombia, gravel bed layers were installed (thickness measured perpendicular to the slope of the gravel pack) as follows:

	Gravel size	Height
1st layer (top)	6 mm	0.25 m
2nd layer	9 mm	0.25 m
3rd layer	24 mm	0.20 m
4th layer (bottom)	36 mm	0.30 m

In Colombia bacterial density reductions of 85-95% have been obtained with this river bed filtration system (Salazar, 1980). Turbidity removal levels by dam filtration are given in Table 4.

Table 4: Turbidity removal levels by river dam filtration at several sites in Colombia

Site	Turbidity raw water	Turbidity filtered water	Percentage reduction
El Crucerero - Caloto	200	0.8	99.6
Honduras - B. Aires	65	1.5	97.7
Las Cruces - Timbio	48	0.8	98.3

(Source: Salazar, 1980).

Performance during periods of high river discharge was poor because frequent cleaning was needed (sometimes every three days) (Wolters, 1988). During such periods up to 5 cm of the upper filter material was washed away. Filter runs lengthened during the dry season.

Standard designs were not considered to be appropriate because of variation in local conditions and requirements (Wolters, 1988). Construction is likely to be costly in rural areas, although costs may be reduced by leaving out the sedimentation tank and constructing a simpler dam.

As the prefiltration capacity is limited, this design is only suitable for surface waters of moderate turbidity levels. Consideration should be given to simplifying the design to make it feasible for rural communities. The option described may also be suitable for construction on existing dams.

Branched-off River Dam Filtration, Colombia

A branched-off River dam filter has been developed by the Universidad del Valle, Cali, Colombia, and used with slow sand filtration in Choro del Plata, Valle de Cauca Department (Figure 15).

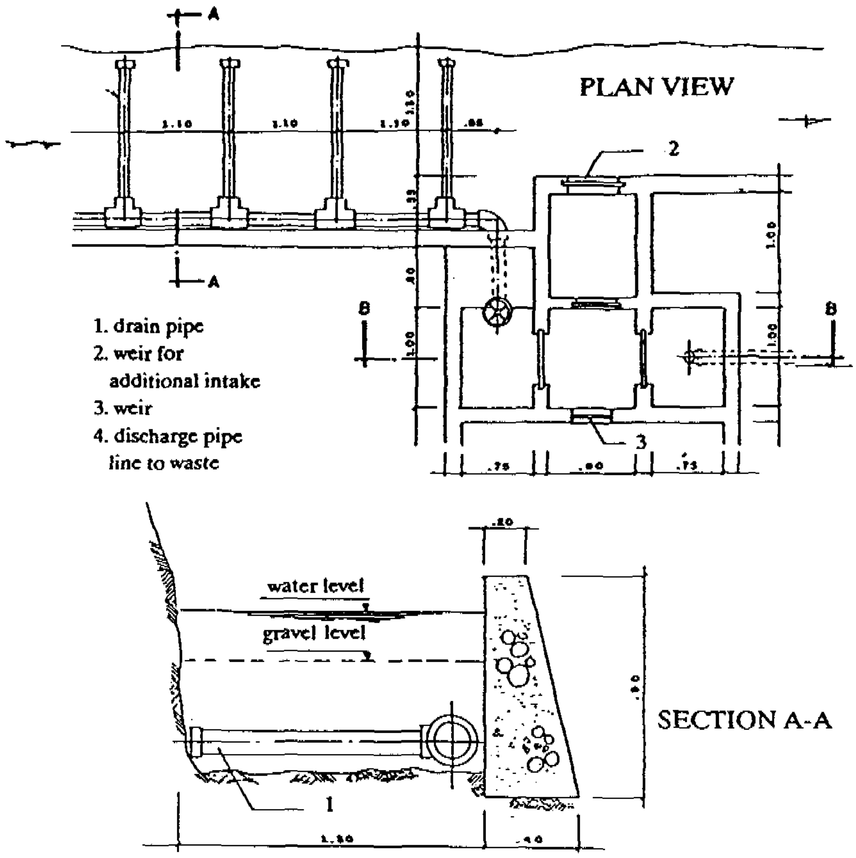


Figure 15. Branched-off River Dam Filtration
(Source: Area de Abastecimiento y Remoción de Agua. Universidad del Valle, Cali)

This type of filter may be placed in small streams having low to moderate suspended material load or in a branch of a larger river.

The basic filter components are:

- a concrete or masonry filter box containing the gravel bed and drain system. A little foot upstream of the filter box protects the gravel bed, while a small dam is constructed downstream to maintain a minimum water level.

- collection drain system, consisting of a main and laterals. The laterals have orifices of 10-12 mm, drilled under a 30° angle with the horizontal.

- the gravel bed comprises:

		Height
1st layer (top)	coarse sand	0.05 m
2nd layer	gravel 5-10 mm diameter	0.10 m
3rd layer	gravel 10-15 mm diameter	0.10 m
4th layer (bottom)	gravel 15-20 mm diameter	0.15 m

One of the major advantages of this technology is ease of operation (Wolters, 1988). Only periodic cleaning of the upper part of the filter bed is required. This is done by stirring small quantities of the upper layer of the gravel bed. Accumulated deposits are washed out and removed by the stream of the branched-off river. Eventually the whole gravel bed may become clogged and then all filter material has to be removed, washed and replaced.

Consideration should be given to regulating the flow of the branched-off river at the diversion point of the main river as an operational instrument in filtration. Removal efficiencies for suspended solids may not be very high because of the relatively high filtration rates (1.0-2.5 m/h) and the limited height of the filter bed. The main advantage is expected to be the reduction in high peaks of suspended solids in surface water for short periods.

3.5 Sand and sub-surface dams

Sand and sub-surface dams store water below ground level in the soil compartment upstream of the dam. The dam blocks the groundwater flow, thus resulting in higher groundwater levels behind the dam and in the adjacent aquifers. Provided the storage capacity of the dam meets the water demand. Aquifers may then supply water throughout the dry season when they normally dry up.

Sub-surface dams are constructed in fairly permeable aquifers of alluvial sediments. The foot of the dam should preferably reach the bedrock (Figure 16). A dam can be built of any durable and impervious material, such as concrete, burnt bricks, puddled clay, and steel. A water collection system is built, upstream of the dam. This could range from a single dug infiltration well to a well with radial collectors.

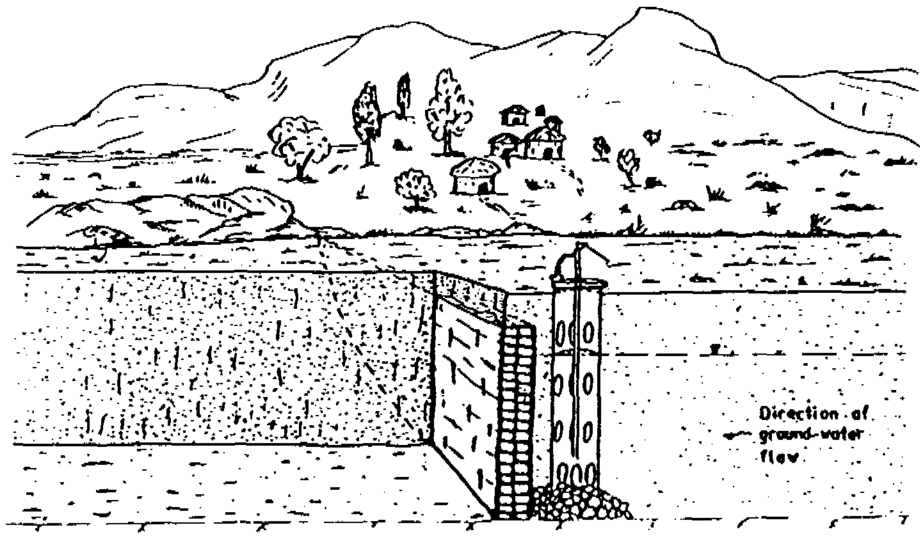


Figure 16. General principle of a sub-surface dam (Source: Nilsson, 1984).

With time, a sand body will gradually be created by constructing a low dam, which is built higher each year. During heavy flows, coarser sediments will settle behind the dam, filling the reservoir (Figure 17). The smaller solids will remain suspended and be discharged over the dam. The aquifer behind the dam is built up in stages. Water can be abstracted by gravity through a drain and collection system in the aquifer connected to a distribution pipe or system.

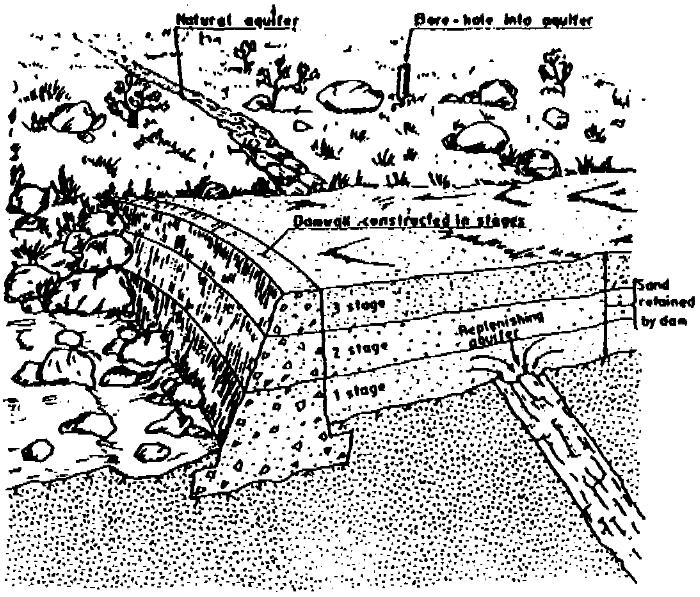


Figure 17. General principle of a sand dam, adapted from Sauermann (date unknown).

4. CONCLUSIONS

The simple principles of river bed filtration make this a reliable system with potential for application in rural water supplies. Some experience has been gained with different systems in several countries. The main problem encountered is the composition of the filter bed for particular surface waters. Often the filter becomes clogged very quickly and operation and maintenance is not adequate to keep the filter operating. The performance results show that river bed filtration is not suitable as a single water treatment method because effluent quality does not meet levels set in the international guidelines. However, this method may be very satisfactory as a pre-treatment prior to slow sand filtration.

It is recommended that present systems be monitored and evaluated further. This should be combined with research to make designs economically and technically optimal for local conditions and requirements. Specific attention should be paid to operation and maintenance requirements.

REFERENCES

CEPIS/PAHO (1982). Modular Plants for Water Treatment. Technical Documents no.8, Vols. I and II. Lima, Peru, CEPIS.

NEERI, (1988). Report on rapid appraisal of a few river bed infiltration works in Tamil Nadu. Unpublished report. Nagpur, India, National Environmental Engineering Research Institute.

Nilsson, A. (1984). Groundwater dams for rural water supply in developing countries. Stockholm, Sweden, Department of Land Improvement and Drainage, Royal Institute of Technology.

Salazar S., L.G., (1980). Agua Potable para la Zona Rural Colombiana, in ACODAL, no. 96. Bógota, Colombia, Acueductos y Alcantarillados.

Sauermann, H.B., (date unknown). Sand versus Sun. South-West Africa.

Volkhard, W.W., (1983). Integral Filter Systems for Reservoirs of Small Earth Dams. Paper presented at UNESCO Seminar on Soil and Water Conservation, University of Nairobi, March 1983.

WHO (1984). Guidelines for Drinking Water Quality, Vol.1 Recommendations. Geneva, Switzerland, WHO.

WHO/SEARO, (1976). Typical designs for engineering components in rural water supply. (WHO Regional Publications - South-East Asia). New Delhi, India, WHO, South-East Asia Regional Office.

Wolters, H. (1988). Roughing filtration: a literature study. M.Sc Thesis. The Hague, The Netherlands, IRC International Water and Sanitation Centre.

CHAPTER 4

MODULAR SUB-SAND ABSTRACTION SYSTEMS

by J.E.M. Smet and D. Wheeler

1. INTRODUCTION

For centuries sub-sand abstraction has been used in many countries as a safe and reliable means of water filtration. The simpliest and most basic form of sub-sand abstraction is a small temporary hole dug in a dry river bed in search of water. Based on the same principle, more sophisticated abstraction systems have been developed incorporating some type of pumping device or using gravity flow when appropriate.

In sub-sand technology water under a river bed or another surface water source is collected by means of a perforated pipe or collection box. An artificial bed of coarse material may have to be installed down if the natural river bed material is too fine to permit a reasonable filtration rate. The water flows from the perforated drain pipe to the collection chamber and from there by gravity or is pumped to the distribution area (Figure 1).

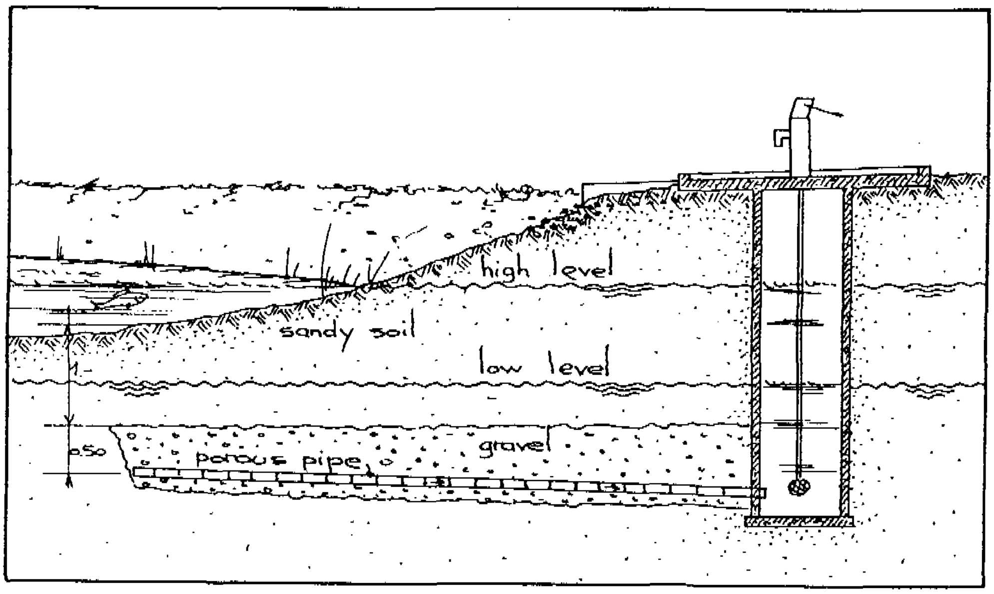


Figure 1. Sub-sand abstraction

Construction of a dam may improve the production capacity of a system as this would overcome the problems of flow off, whether it be surface water or groundwater.

Sub-sand abstraction is both financially and technically suitable for water supply systems for small to medium-sized communities. In many cases further water treatment before domestic consumption could be limited to slow sand filtration.

In this paper a modular sub-sand abstraction method suitable for small communities is described and discussed in relation to research carried out on this technique as well as field studies undertaken in four countries in Africa. Finally recommendations are made about design criteria, construction materials, operation and maintenance, and costs.

2. SUB-SAND FILTERS

This method was developed by SWS Filtration Ltd., United Kingdom, initially to supply clean water to marine aquaria. Performance monitoring of the filter, which was buried in the sand on the beach, revealed that even the smallest plankton and organic particles were removed from the sea water and the bacterial density reduced (Cansdale, 1982)

The filter was subsequently developed for small community water supplies. It consists of an inverted plastic filter box from which water is either hand-pumped to a slow sand filter or directly to consumers (Figure 2).

Based on the design several types of filters have been developed:

- Village unit. This consists of an inverted reinforced plastic box of 600 x 300 x 300 mm and weighing approximately 8 kg. It has a false ceiling (slotted plated) which is perforated with numerous 3 mm holes (Figure 2).

- Camp unit. This is similar to the village unit but is smaller being 300 x 300 x 300 mm in dimensions.

- Mini unit. This is a stainless steel cylindrical screen 60 mm in diameter and 80 mm in length which is connected by a 25 mm diameter plastic suction pipe to a hand pump (Figure 3).

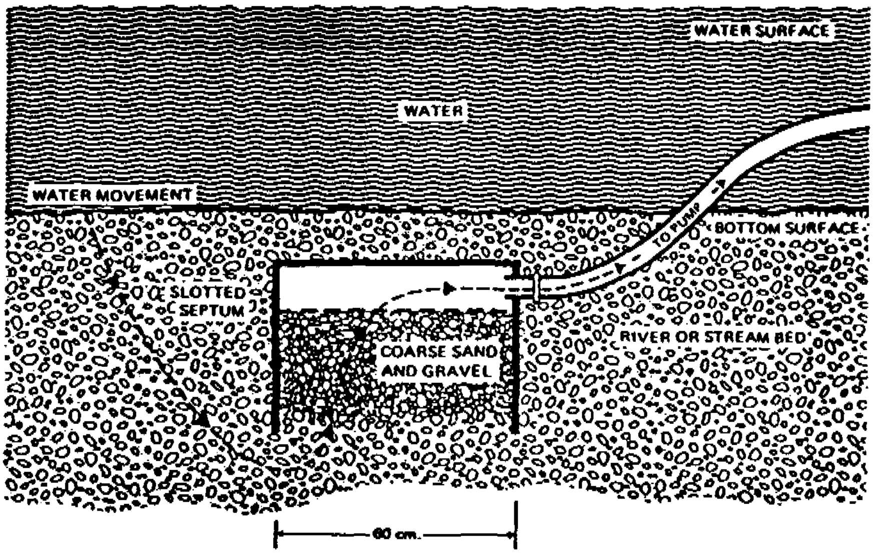


Figure 2: The Village Filtration Unit (reproduced from Waterlines, Vol.1, no.1).

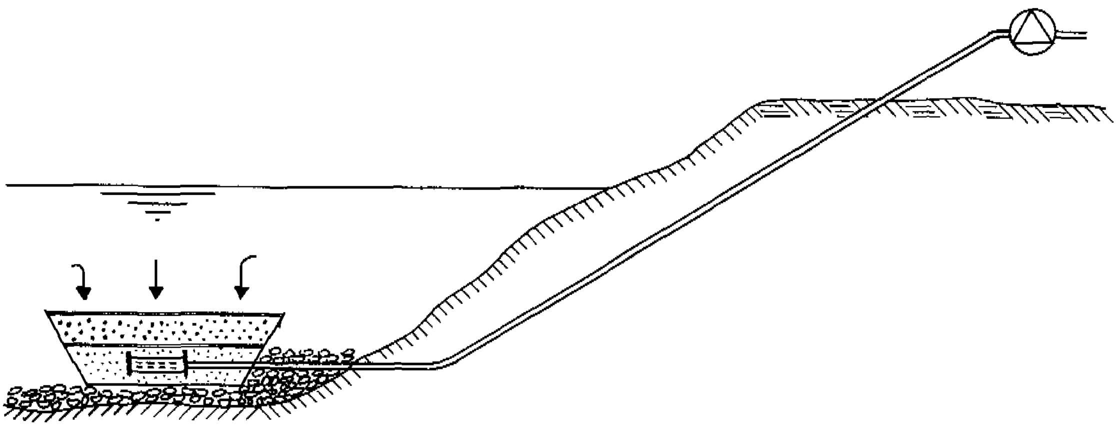


Figure 3: Mini filter in gravel filled container covered with filter matting.

Several types of pumps can be connected to these units depending on the required and potential discharge. These include low head suction, diaphragm and irrigation pumps, in particular the Rower pump.

Village and camp units

The box is buried in the surface water bed, open end down. The top of the unit should be at least 150 mm below the sandbed and the minimum water depth 300 mm. The space below the slotted plate is filled with coarse sand and gravel, and the area around the filter unit refilled with well packed coarse sand. If the particle size of the river bed materials is unsuitable, then the filter area may have to be excavated and filled with material, ideally of 1 to 5 mm in diameter. The unit is connected to a pump which draws the water through the slotted plate. During installation, the sand particles within and around the unit need to be graded by a series of alternating and reverse pumping operations.

The sandbed acts as an induced gravity filter with a maximum pressure of 1 atmosphere. The size of the sandbed abstraction area depends on the pumping rate of the unit, the depth of the installation, and the filter medium. It is assumed that per m^3 water pumped per hour approximately 4 m^2 of sandbed surface acts as filter surface. This corresponds to a filtration rate of 0.25 m/h, which is equivalent to the usual slow sand filtration rate. However, uniform velocities of the water over the assumed abstraction area are unlikely. Higher filtration rates may be expected just above the filtration unit than a few metres from the unit. A constant pumping rate is essential for stable biological filtration and because overpumping may cause fine particles including bacteria to break through the filter.

The filtration capacity of the village unit is up to 22 m^3/h with an abstraction area of 10 m in diameter. This corresponds to an average filtration rate of 0.275 m/h, provided the filter bed is composed of optimal material.

Mini Unit

The screen of the mini unit can be buried in the river bed or in a container filled with filter material. It may also be covered with synthetic filter mat material as illustrated in Figure 4. The filter mat is 1 cm thick non-woven fabric. The bottom quarter of the container is filled with fine gravel (2-5 mm diameter) to ensure adequate coverage of the filter and covered with a layer of filter matting. The container is then filled with coarse sand (0.5-2 mm diameter) and covered with two layers of well secured filter matting. It is placed in the surface water above the bottom sludge with the top at least 300 mm below the water surface. The unit should be placed some distance from the bank to prevent high concentrations of schistosome cercariae occurring.

The mini containerized filter has a maximum capacity of 3.6 $m^3/m^2/h$. It can also be used, for example as an "on shore" filter, a gravity filter from a small dam and a spring water filter (Gifford et al., 1986). When used for springs, a large hole or trench should be dug into the eye of the spring and lined with stones and coarse gravel. The mini filter is connected to a durable hose, placed on the bed and covered with fine gravel and coarse sand. A retaining wall is constructed to make water collection site. A tap is attached to the outlet of the pipe. Sufficient storage capacity and an overflow should be provided for the system.

This type of small modular sub-sand abstraction systems is relatively simple to install. Special skills or tools are not usually required, and components are not difficult to transport even by foot.

Controlled conditions are necessary to attain high performance of modular sub-sand abstraction. This means continuous operation, low flow rate, smooth pumped abstraction and diligent maintenance (Wheeler et al., 1985).

A. Filter buried in natural sand or gravel bed of stream

B. Filter buried in graded bed of stream.

C. On shore filter: water is pumped in gravel-filled container and flows out by gravity

D. Filter is buried in gravel bed of small dam; water flows by gravity to reservoir

E. Filter is buried in gravel bed of protected spring

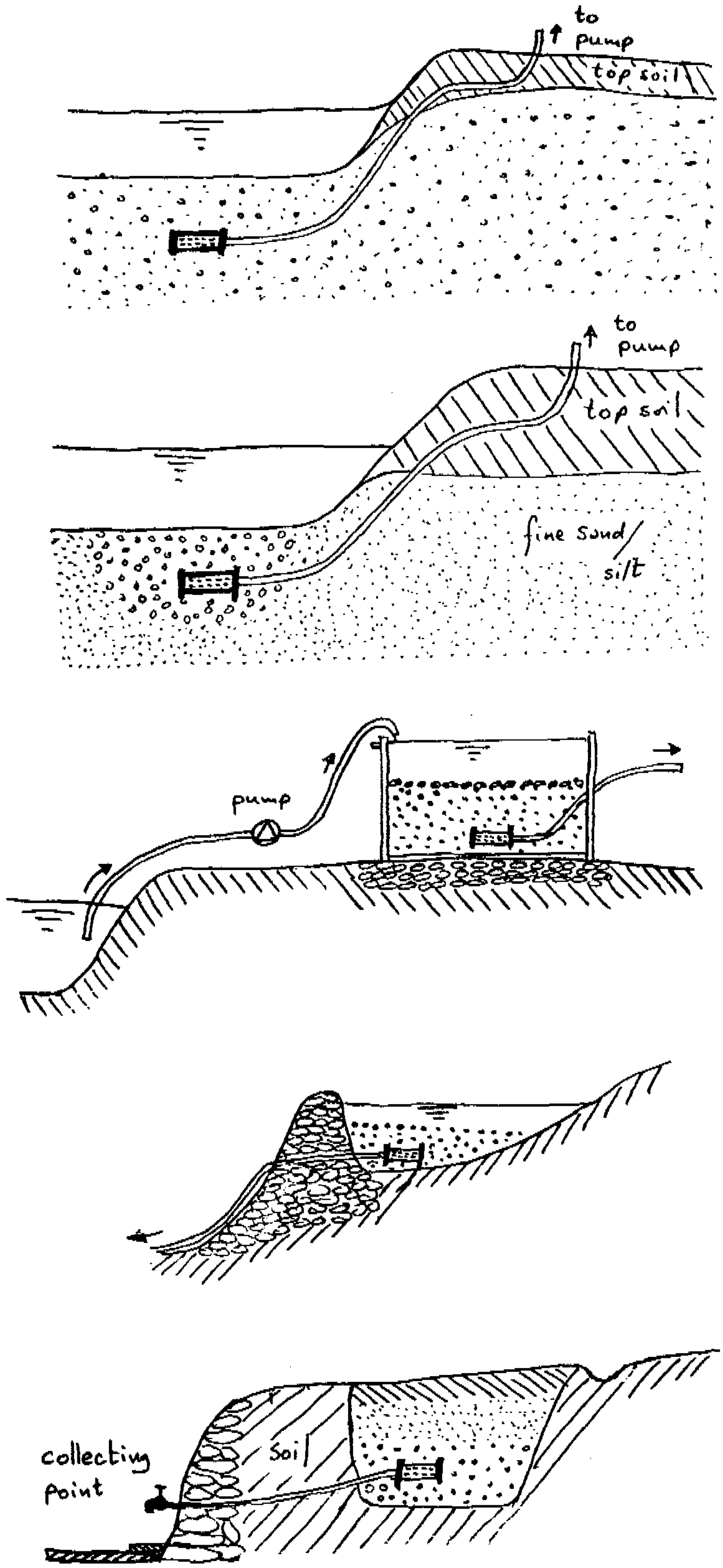


Figure 4: Mini Filter Installations

3. STUDIES ON SMALL MODULAR SUB-SAND WATER ABSTRACTION SYSTEMS

Wheeler et al., (1985) studied modular sub-sand abstraction under summer and winter conditions in the United Kingdom. They concluded that with careful management, these systems can improve the hygienic and physical quality of surface waters. For reducing levels of faecal coliform and turbidity, the sub-sand filtration performed better under warm water conditions, that is above a temperature of 10°C. This is probably because biological activity in the filtration process is enhanced in warm water. Significant reductions in faecal bacterial densities were achieved at equivalent river bed filter flow rates of 0.25-0.30 m/h, until abstraction resistance became too high (Wheeler et al., 1983).

Hurst (1977) and Caddy and Hurst (1978) reported a mean reduction for Escherichia coli of 98.8% in river water after modular sub-sand abstraction in the United Kingdom. The tests were done after a ten days ripening period of the filter during which water was pumped through the filter at a constant rate of 29 m^3/d. Tracer studies were undertaken to determine the river bed area used in the abstraction, and consequently the filtration rate was estimated at 0.17 m/h. The mean removal rate of chemical constituents is given in Table 1.

Table 1: Mean removal rate (%) of chemical constituents by sub-sand filtration as recorded in three studies in the United Kingdom.

Test	Hurst 1977	Hurst 1981	Wheeler 1985
Turbidity	--	98	90
Suspended solids	71	--	--
B.O.D.	49	--	--
Ammoniacal nitrogen	71	--	--
Nitrate nitrogen	12	--	--
Anionic detergents	18	--	--
Phosphate	21	--	--
Iron	59	--	--
E.coli	99	96	90-98
Abstraction rate [m^3/h]	1.2	1.2	1.1-1.5

Wheeler (1985) found that filter runs varied from 7 to 10 days in summer, to 5 to 10 days in winter when the suspended solids load was higher. The abstraction units needed some time to build up faecal coliform reduction levels in excess of 1 log unit (90%). Under winter conditions (<10°C) this took 5-6 days and under summer conditions (>10°C) about 2 days. In summer at the end of the filter run, faecal coliform reduction levels approached 2 log units (99%).

The reliability of faecal coliform reductions decreased when the abstraction resistance exceeded 0.6 atmosphere as measured at the suction side of the pump (Wheeler, 1985). This phenomenon of reduced purification with increased resistance was not found for turbidity removal.

Shridar et al., (1985) reported on the effectiveness of simple sand filters in the reduction of the prevalence of cyclops (water fleas which may carry guinea worm disease). The sand filter comprised a bottom layer of 12 cm of gravel (2-4 mm diameter) on top of which was 15 cm layer of river sand. The supernatant water layer was approximately 20 cm. The sand filter was able to filter the cyclops completely.

Oxygen depletion of up to 50% may occur during sub-sand abstraction. If secondary treatment is used, for example slow sand filtration, oxygen levels may recover in the supernatant. But Wheeler (1983) recorded further oxygen depletion due to biological treatment to less than 1 mg/l.

After backwashing of the sub-sand filter, Wheeler et al. (1983) found an increase of bacterial densities in the filtrate of about 1 log unit. Bacterial density increased by 1.5-1.8 log units when cleaning of downstream slow sand filters coincided with backwashing of sub-sand filters. The same was noted for turbidity, although in the filtrate the level remained below 1 NTU.

If the raw surface water is subject to chronic organic pollution or occasional high levels of organic pollution, sub-sand abstraction is not recommended as the pre-treatment system prior to slow sand filtration. In such cases the filter medium may need to be replaced totally but this may not always be feasible (Wheeler et al., 1983).

4. FIELD EXPERIENCE

In this section modular sub-sand abstraction systems constructed in Zaire, Uganda, Sudan and Nigeria, and evaluated by Gifford and Partners, and the London School of Hygiene and Tropical Medicine are discussed (Gifford et al., 1986).

Village and camp units

Operation. Except for the hand pumps employed, no major operational problems were encountered with the units. When the filter containers were buried deep enough in the river bed, they continued to supply water even during the dry season when there was no river flow. Several systems did not function because parts had been stolen thus demonstrating the vulnerability of portable systems to theft (Gifford et al., 1986).

Maintenance. The units required minor maintenance and the filter containers showed a good durability (Gifford et al., 1986).

Filter performance. Insufficient data were available for the evaluation of filtration performance.

Mini Filter

Operation. Where used in springs, test results showed little difference in water quality between this system and traditional spring protection systems. The main advantage of the mini filter was the reduction of clogging (Gifford et al., 1986). The amounts of silt in the filled mat was found to be high in mini filters installed in irrigation channels and rivers with a high suspended solids load. Nevertheless, filters continued to operate throughout the ten-day monitoring period. (Gifford et al., 1986).

Maintenance. Gravity systems were found to require relatively little maintenance, depending on the quality of the raw water. Spring water caused very little blockage and therefore cleaning of the filter was not necessary.

A specially trained team were charged with filter maintenance in Sudan. Some of the difficulties encountered include:

- lifting the gravel/sand filled units out of the canals since the container does not have handles;
- use of several parts of the box with low durability leading to rust and breakage;
- the filter matting being too small to fix in place with the clamping device; as a result snails were found in the gravel making schistosoma infection likely.

The system was reported not to be very durable (Gifford et al., 1986). About 30% of the systems did not function, although many breakdowns were due to pump failure. Problems with the filter units were not in general with the box but were mostly breakage of the hoses, rusting and subsequent breakage of the clamping device and weakness of the plastic rims of the container. In a number of cases the filter cloth had to be replaced because of the action of the sun, water quality, silt burden, algal growth or rough handling. The cloth could not stand up to very silty conditions in particular.

Filtration rates. Filtration rates of 19 $m^3/m^2/h$ were observed with a heavier diaphragm pump but at this rate turbidity reductions were negligible in irrigation water in Sudan.

Pumps. Breakdowns were primarily caused by pump failure, particularly with the mini containerized filter, many breakdowns occurred because of severe operation conditions. The "Rower" pump (irrigation type pump) delivered with the filter units proved to be very reliable. Some pumps have operated maintenance free for over a year. The "Rower" pump is considered to be a suitable low lift pumps for rural water supply (Gifford et al., 1986).

Microbiological performance. The effectiveness of a fabric mat in the filtering of schistosoma cercariae has not been investigated sufficiently.

For all the modular sub-sand abstraction systems tested in the field the faecal coliform level of water was reduced after passing through the filters although not significantly. The level was reduced to one log unit at only a few sites (Gifford et al., 1986).

5. DESIGN CRITERIA

According to the supplier, the village unit can draw up to 22 m^3/h provided the filter bed is composed of optimal material (1 to 5 mm in diameter). This discharge would be sufficient for a population of 3500 assuming a daily water consumption of 50 litres per capita. With the use of a hand pump the output could be increased to 4 m^3/h.

Maximum number of users for the mini unit is considered to be 200. The unit is capable of supplying more water of adequate quality depending on the raw water characteristics.
Filter composition and required materials are described under section 2 "Installation and Functioning".

Care must be taken to select an appropriate site for the filter. Aspects which require consideration include river bed composition, flow characteristics, and suspended solids content. These should be monitored for a period of at least a year. Filters need to be installed at the end of the dry season and as a carefully planned community participation action (Joy, 1983).

6. CONSTRUCTION MATERIALS

The modular sub-sand abstraction systems described are factory-made in the United Kingdom. However, similar systems could be constructed in developing countries from locally available materials, and so reduce the transport cost.

7. OPERATION AND MAINTENANCE

Small modular sub-sand abstraction systems in general are relatively simple to operate and maintain.

Various cleaning techniques were applied by Wheeler et al. (1985), including pumped backwash, gravity backwash, and in situ skimming. Both backwashing techniques proved to be successful. Only a negligible reduction of silt level in the filter medium was attained with underwater skimming.

Where raw water regularly exceeds turbidity levels of 10 NTU, a regular weekly or biweekly maintenance programme should be observed to ensure continuity of water supply. In most circumstances skilled labour is needed for this task (Wheeler et al., 1985).

The containerized mini filter only needs replacement of the upper matting when high filter resistance has built up. Only the upper sand layer needs to be washed and replaced. A fresh, clean mat is put over the sand and the container is closed with the old underlying, ripened filter mat. After several "top layer cleanings" the entire sand body has to be replaced with fresh sand. Some old, dirty sand is retained to mix and "seed" the fresh sand for quick ripening of the biological filter. The old sand can be washed and reused.

Simultaneous cleaning of the sub-sand abstraction and the secondary treatment unit such as slow sand filtration results in serious deterioration of the bacteriological quality and turbidity level of the final filtrate. Therefore this practice should be avoided unless disinfection is applied.

Villagers could be trained in relatively simple maintenance procedures. This would reduce the costs considerably and ensure more reliable and continuous water supply.

8. COST

Costs could be reduced by developing screens which could be manufactured in developing countries.

The installation costs, including local components and transport to the site, range from US$ 100 (1988 price) for mini unit in a protected spring with gravity supply to US$ 750 for a two staged mini containerized unit. The installation costs and costs per capita costs for systems to serve a population of 200 people are compared in Table 2.

Operation and maintenance costs depend on whether the water supply system is by gravity or hand-pumped. For the sub-sand river bed unit the maintenance costs were calculated at 5% of the investment costs.

Table 2: Total and per capita investment costs in US $.

Type of Installation	Total Investment Cost (US$)	Capacity (Number of People)	Per Capita Cost (US$)
Traditionally protected spring	17	550	0.03
Protected spring unit			
hillside, gravity	120	200	0.60
valley bottom, with pump	230	200	1.15
Sub-sand river bed unit	568	200	2.84
Mini Containerized unit			
one stage	398	200	1.99
two stage	759	200	3.80
Slow sand filter, two hand pumps	2398	200	11.99

Source: Gifford et al., 1986

9. CONCLUSIONS

With careful management, modular sub-sand abstraction systems can improve the hygienic and physical quality of surface waters (Wheeler et al, 1985). The enhanced biological activity in the filtration process in warm water conditions are encouraging for application in tropical environments.

Modular sub-sand abstraction is a useful means of clarifying turbid waters preparatory to secondary treatment. The process can provide substantial improvements in microbiological quality and thus overall hygienic safety. The reductions in faecal coliform content found in several studies are promising. However trails in Africa have shown that the reductions are not impressive under field conditions. Conditions for optimal filtration require careful investigation such as, river bed material, installation procedures, filtration rates, and filter box components.

The process can not be compared with slow sand filtration. Modular sub-sand abstraction is a type of pressure filtration and may be subject to breakthrough of suspended solids and faecal bacteria, and erratic performances. The system is susceptible to pollution resulting in operation and maintenance problems if the raw water deteriorates seriously. The filter bed may then need to be removed completely which may be difficult.

The oxygen depletion arising from the process may have deleterious effects on subsequent biological filtration.

No problems were found in the function or performance of mini unit installed as a spring filter. Results were comparable with conventional spring water protection but there are indications that maintenance is less expensive.

Containerized mini filter systems need careful operation and maintenance. Maintenance by semi-skilled people is a necessity. Recurrent costs may become high when the filter mat needs to be replaced and filter box parts repaired. However, the use of more durable materials could overcome this.

Small sub-sand abstraction systems often fail because of pump breakdown, especially the "diaphragm type". The "Rower" pump appears to perform better. Other locally manufactured suction pumps of proven performance could be introduced. Several parts of the filter unit must be improved to make the unit more durable. Operation and maintenance training should be given to the communities in order to make them more independent of maintenance from outside the community.

Presently, introduction of these systems in rural communities could be considered provided the community is well organized, skilled and financially capable of operating and maintaining the system.

REFERENCES

Caddy, D.E. and Hurst,M.J. (1978). Applying the SWS Unit in the UK, Anglian Water Authority. World Water, Vol. 1, no. 1, p.30-31.

Cansdale, G. (1982). The SWS Sand Filter and Pump. Waterlines, Vol.1, no.1, pp11-13.

Gifford and Partners[1], and London School of Hygiene and Tropical Medicine[2]; (Buhl-Nielsen, E.[1], Cox, N.P.[2], Stern, P.H.[1], Cairncross, A.M.[2]) (1986). Evaluation of SWS Filtration System. Report for the UK Overseas Development Administration (Project no. R 3957), London, UK.

Hurst, M.J. (1977). Treatment of Water by the SWS Filter Unit. Cambridge, UK. Agricultural Development and Advisory Service, Cambridge, UK, Ministry of Agriculture, Fisheries and Food.

Hurst, M.J. (1981). Studies on SWS Water Filtration Units. Cambridge, UK. Ministry of Agriculture, Fisheries and Food.

Joy, D. (1983). Cleaner water from fast-flowing rivers. Waterlines, Vol.1, no.4, pp.26-28.

Madely, J. (1982). Freeing the Blue Nile from Bilharzia. Waterlines, Vol.1, no.1, pp.10-11.

Sridhar, M.K.C., Kale, O.O. and Adeniyi, J.O. (1985). A Simple Sand Filter to Reduce Guinea Worm Diseases in Nigeria, Waterlines, Vol.4, no.2, pp.16-19.

Wheeler, D, Symonds, C.H., Lloyd, B.J.and Pardon, M. (1983). Progress Report (Phase II) on Project 'Rural Water Treatment Package Plant', Guildford, UK. University of Surrey.

Wheeler, D., Pardon, M. and Lloyd, B.J. (1985). Sub-sand Abstraction as a Prefiltration Step in the Treatment of Surface Water for Small Community Water Supplies in Developing Countries, (unpublished paper), Guildford, UK. University of Surrey.

CHAPTER 5

PLAIN SEDIMENTATION

by L. Huisman

1. Introduction

Sedimentation is the clarification process, which occurs when water flows at low velocities through a basin. Due to the slow movement, turbulence is no longer able to keep the particles in suspension, after which those having a mass density higher than that of the surrounding water will move downward by gravity. Ultimately these particles will be deposited on the bottom of the basin, forming a sludge layer, while the water leaves the basin in clarified condition.

Sedimentation occurs in any basin. In the field of water management it takes place in natural lakes and artificial reservoirs, often with disastrous results as the silting up decreases the usable volume. In water works engineering the same applies for storage reservoirs, which therefore must be cleaned in one way or another at regular intervals. When water is abstracted directly from a turbid river, sedimentation tanks are necessary or at least desirable to lessen the load on subsequent treatment processes, for instance allowing slow sand filters to operate at higher rates for prolonged periods between cleaning. In such tanks, removal ratios are rather limited, not more then 80-90%. Sedimentation is therefore not able to produce drinking water quality, it is always a preliminary and never a final treatment.

Water may contain various kinds of particles, of mineral, vegetable or animal origin. The latter two in particular tend to coalesce, forming layer flocs with a higher settling velocity. Due to turbulence in a flowing river, however, this process of natural aggregation will not have been completed when the water is taken in. In the subsequent settling process, the particles will maintain their identity and consequently move down at a constant rate (discrete settling). It goes without saying that the intakes for river water should be constructed in such a way that no floating matter nor coarse particles rolling and tumbling over the river bottom are abstracted.

Consequently the particle size is well below 1 mm, for which the upward flow of water along the downward moving particle occurs under laminar conditions. The settling velocity s is now given by Stoke's law.

$$s = \frac{1}{18} \frac{g}{\nu} \frac{\rho_s - \rho_w}{\rho_w} d^2$$

with g as gravity constant, ν as kinematic viscosity, ρ_s and ρ_w as mass densities of particle and water respectively and d as the diameter of the spherical particle.

Table 1: Settling velocity in relation to temperature and particle characteristics

		0	10	20	30
t	(°C)	0	10	20	30
ν *10^6	(m^2/s)	1.79	1.31	1.01	0.80
sand grains (ρs = 2650 kg/m^3) of 0.05 mm size					
s	(mm/s)	1.26	1.72	2.23	2.81
flocculated mud particles (ρs = 1050 kg/m^3) of 0.5 mm size					
s	(mm/s)	3.81	5.20	6.75	8.52

With plain sedimentation as described above, particles can be removed down to a size of about 1 um. Still smaller particles are kept suspended by Brownian movement and for their removal they must first be aggregated into larger flocs, by chemical coagulation and flocculation. Even dissolved impurities can be eliminated by settling, when chemicals are added to throw them out of the solution. Chemical water treatment, however, has many disadvantages. It is expensive, difficult to manage and it increases the mineral content of the water. In this chapter it will be left out of consideration.

2. Settling in an ideal horizontal flow basin

As mentioned before, particles from river water may be expected to settle discretely. For their removal rectangular horizontal flow tanks offer the best solution. In idealized form such a tank is shown in Figure 1 where four zones can be distinguished:

a. the inlet zone, constructed in such a way that the influent is distributed evenly over the full width and depth of the tank;
b. the settling zone with a constant displacement velocity v of the water;
c. the outlet zone where the clarified water is abstracted evenly over the full cross-sectional area;
d. the sludge zone at the bottom where the settled out particles are stored without the danger of re-suspension.

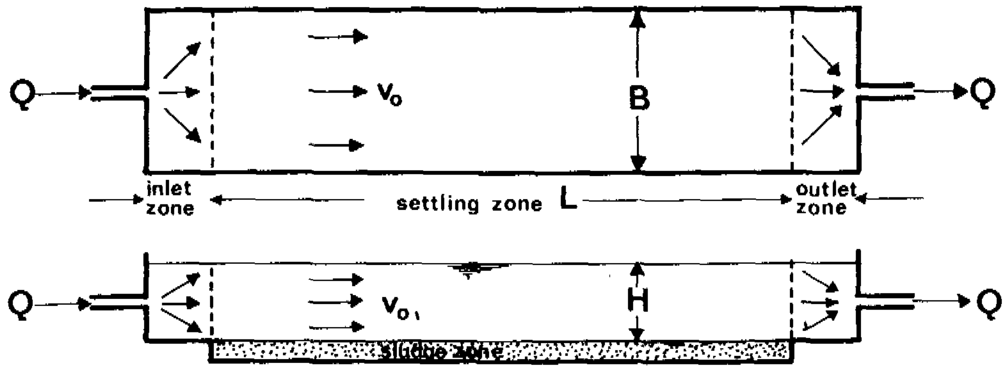


Figure 1. Retangular horizontal flow settling tank

The actual work of the sedimentation basin is restricted to the settling zone, where the discrete particles trace straight paths, following the sector sum of the displacement velocity v of the water and the settling velocity s of the particle. The horizontal velocity v has been assumed constant over the whole settling zone, but the settling rate s varies from one particle to another, depending on size and mass density. As an example to be used in subsequent calculation, Figure 2 shows the cumulative frequency distribution of suspended particles in water from a particular river.

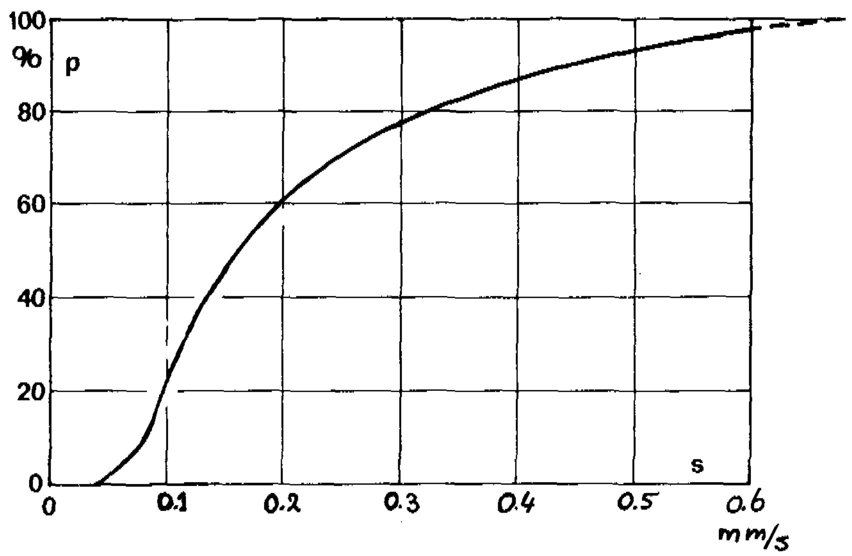


Figure 2. Cummulative frequency distribution of settling velocities

For different settling velocities s the paths followed by discrete particles are shown in Figure 3. At the top a very special settling rate, subsequently called s_o, has been assumed, of such a value that a particle starting at the top left just reaches the bottom at the far right and is still returned. Particles with a higher settling velocity reach the bottom (far) before the outlet, while some particles with a smaller settling rate escape with the effluent.

With the notations of figure 3 this gives as removal ratio r for individual particles of

for $s \geqslant s_o$ $r = 1$

for $s < s_o$ $r = \frac{h}{H} = \frac{s\,T}{s_o T} = \frac{s}{s_o}$ with T as detention time

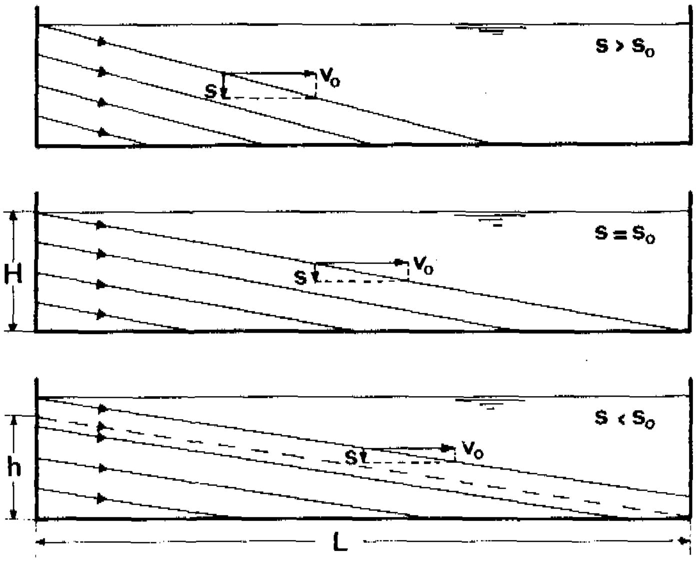


Figure 3. Paths traced by discrete particles in a retangular horizontal flow settling tank.

According to Figure 4, the removal ratio for the suspension under consideration becomes

if $s \geqslant s_o$ $r = 1-p_o$

if $s < s_o$ $r = \int_0^{p_o} \frac{s}{s_o}\,dp$

together $r = (1-p_o) + \frac{1}{s_o} \int_0^{p_o} s\,dp$

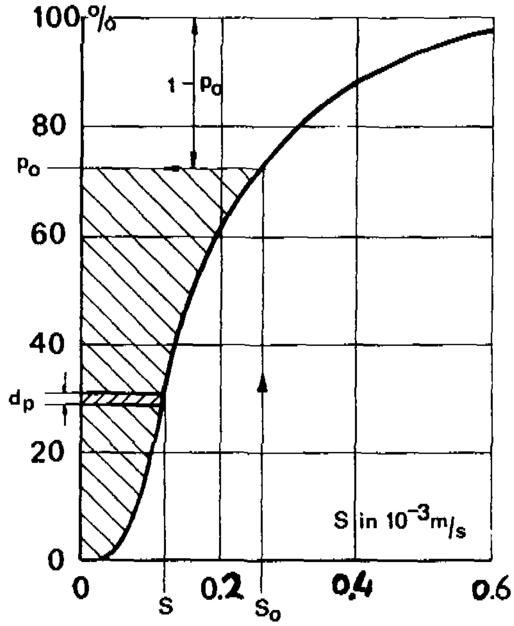


Figure 4. Calculation of setting efficiency (1)

In Figure 4 s.dp is the hatched area and $\int$s.dp the dottedd one. Measuring this area with a planimeter, dividing it by s_o and adding ($1-p_o$) gives the total removal ratio r (Figure 6). This calculation can be carried out for different values of s_o, giving for the suspension of Figure 2 the results shown in Figure 5.

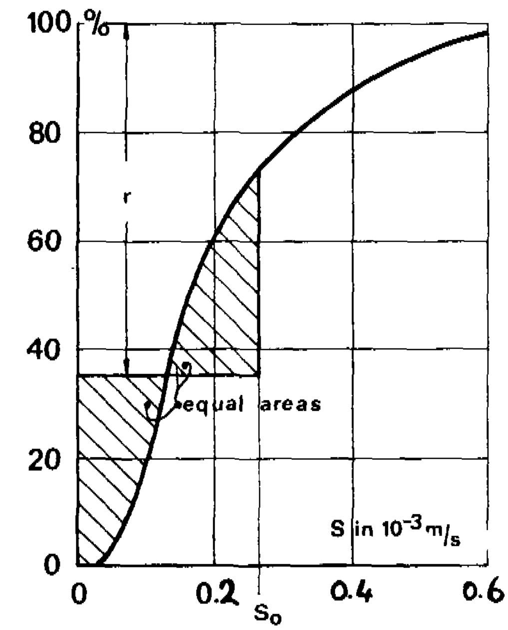


Figure 5. Calculation of settling efficiency (2)

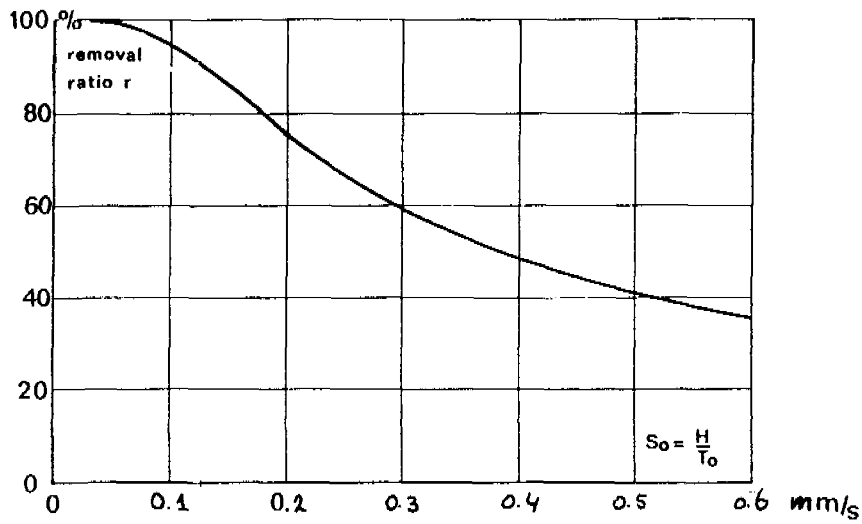


Figure 6. Removal ratio as function of overflow rate

From the foregoing it will be clear that the clarification efficiency of a sedimentation tank depends on two factors, on the frequency distribution for the settling velocities of the suspended particles and on the value of s_o. Only the latter factor can be influenced by the design of the tank. According to Figure 3 at the top

$$\frac{s_o}{v} = \frac{H}{L} \quad \text{in which} \quad v = \frac{Q}{WH} \quad \text{(see Figure 1)}$$

$$s_o = \frac{Q}{WL} = \frac{Q}{A}$$

with A as surface area of the tank. Derived by Hazen as far back as 1904, this formula states that for a particular suspension of discrete particles, the clarification efficiency of a continuous horizontal flow sedimentation basin solely depends on the rate of flow and the surface area of the tank, which together contributes the surface loading or overflow rate s_o. The efficiency is independent of the depth of the tank and of the detention time. When river water with the suspended particles of Figure 2 must be treated for 80% removal,
Figure 5 asks for s_o = 0.175 mm/s = $(0.175)10^{-3}$ m/s and for an amount of 35 m^3/h = $(9.72)10^{-3}$ m^3/s

$$A = \frac{Q}{s_o} = \frac{(9.72)\ 10^{-3}}{(0.175)\ 10^{-3}} = 56\ m^2$$

With a depth of say 2 m, the detention term equals

$$T = \frac{A.H}{Q} = \frac{(56)(2)}{(9.72)\ 10^{-3}} = (11.5)\ 10^3 \text{ seconds} = 3.2 \text{ hours}$$

but greater depths and longer detention times have no influence on basin efficiency.

The conclusions obtained above have been derived for rectangular tanks, but also hold true for other shapes provided that the depth H is constant. In the detention time T all particles reach the bottom of the tank and are removed from the flowing liquid when thin settling velocity is larger than

$$s = \frac{H}{T}, \text{ or with } T = \frac{AH}{Q}$$

$$s = \frac{Q}{A} = s_o$$

In practice, circular tanks are used next to rectangular ones (Figure 7). In sewage treatment they have the great advantage of a simple and rapid sludge removal without the danger of putrefaction, but for the purification of river water this is of less importance.

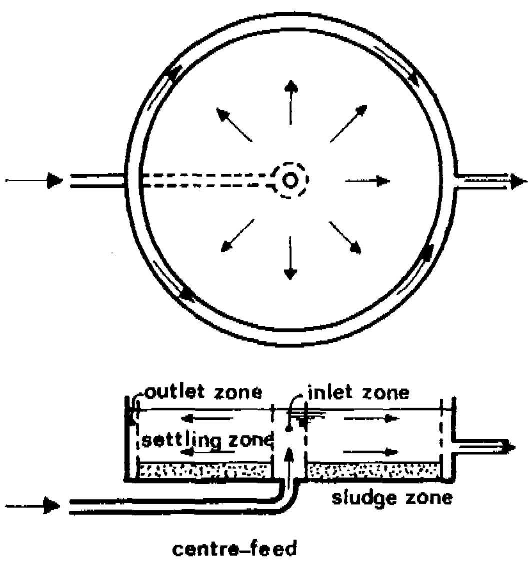


Figure 7. Circular horizontal flow settling tank

3. Field tests

In a judicious design of sedimentation tanks, the cumulative frequency distribution for the settling velocities of the subsiding particles in the water to be treated, is indispensable. It cannot be taken from textbooks, but must be determined by field tests for the chosen inlet location. Preferably these tests should be carried out a number of times to get acquainted with seasonal variations in settling characteristics. The test uses a cylinder closed at the bottom, preferably made of clear plastic, with an inside diameter of about 0.2 m, a depth of 2 m or more and provided with at least two sampling cocks, at distances of about 0,7 and 1.6 m below the top (Figure 8).

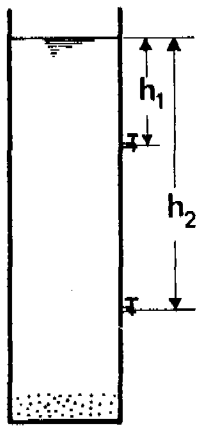


Figure 8. Cylinder for settling velocities analysis

The test starts by filling the cylinder with the river water to be investigated, stirring the contents gently to distribute the suspended matter equally over the full volume. After the water has come to rest, testing begins by taking water samples from the cocks and at various time intervals thereafter. The samples are investigated for suspended matter content or any other substance that has to be reduced by settling. Care should be taken that during the test no change in water temperature occurs as this may give rise to density currents, disturbing the settling process.

The cumulative frequency distribution of settling velocities given in Figure 2, may have been based on the observations given in Table 2.

Table 2: Suspended matter content at 0.6 m and 1.5 m measured at different times

t	(s)	0	900	1800	3600	7200	10800
for h_1 = 0.6 m:							
c	(g/m^3)	84	83	71	44	12	4
c	(%)	100	99	85	52	14	5
h_1/t	(mm/s)		0.667	0.333	0.167	0.083	0.056
for h_2 = 1.5 m:							
c	(g/m^3)	84	84	83	75	50	31
c	(%)	100	100	99	89	60	37
h_2/t	(mm/s)		1.667	0.833	0.417	0.208	0.139

In Figure 9 c in % is plotted against s = h/t. The smooth curve obtained through these observations presents the required frequency distribution.

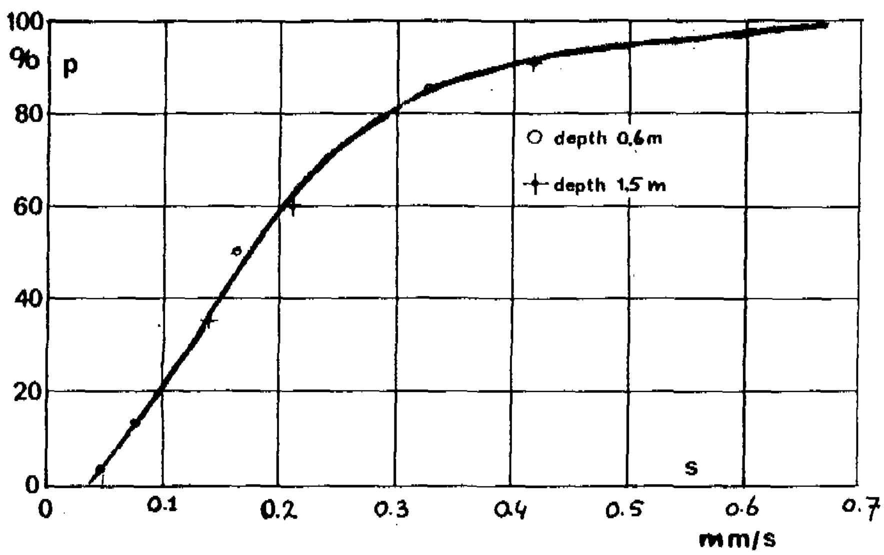


Figure 9. Cummulative frequency distribution of settling velocities

When the apparatus of Figure 8 is not available, settling tests can also be carried out by using a number of beakers with a depth of at least 0.3 m. All the beakers are filled with the water to be investigated, stirred and then left standing for various periods of time t. At the end of the respective periods, the upper 0.2 m of clarified water is siphoned off, homogenized by mixing and the average suspended matter content c_a determined. For the same suspension as applied before, results might have been as shown in Table 3.

Table 3: Suspended matter content and settling velocities determined with beaker tests

t	(s)	0	600	1200	1800
c_a	(g/m^3)	84	37	16	7
c_a	(%)	100	44	19	8
200 mm/t	(mm/s)	∞	0.333	0.167	0.111

In this table ($100-c_a$) in % represents the removed ratio r and 200/t the surface loading s_o of Figure 6. The observations are plotted in figure 10 and can be used directly for the design of a settling tank. Due to the smaller sample size and to disturbances while siphoning off the top 200 mm of water, however, they are less reliable.

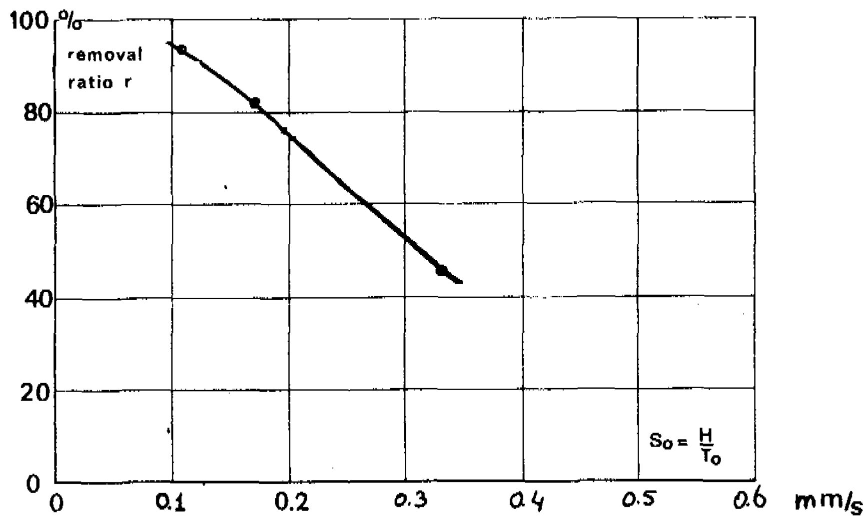


Figure 10. Removal ratio as function of overflow rate

4. Disturbing factors

Without saying so explicitly, it has been assumed in section 2 that settling occurs without any influence of the movement of water in a continuous horizontal flow in the sedimentation basin. In reality, however, various disturbing factors are present. When the depth and width of the basin are small and the horizontal velocity of flow correspondingly large, basin efficiency may suffer from turbulence and bottom scour. In the opposite case of a large depth and width and a small horizontal flow velocity, basin instability and short-circuiting may lower settling efficiency. In all cases finally, the downward movement of the suspended particles results in an upward flow of water, reducing the effective settling velocity.

The horizontal flow of water through the settling tank occurs under laminar conditions, when the Reynolds number Re is smaller than 2000.

$$Re = \frac{vR}{\nu} \quad \text{with R as hydraulic radius}$$

With $t = 20°C$ and $\nu = (1.01)\ 10^{-6}\ m^2/s$ and provissionally assuming $R \simeq 1$ m, this gives as requirement

$$v < 2 \text{ mm/s}$$

Higher velocities, up to 5 times larger, do lower settling efficiency, but the reduction is slight and can easily be compensated by a small increase in surface area.

According to Shields, the horizontal velocity at which scour and resuspension of settled out material starts to occur, is given by

$$v_s = \sqrt{\frac{40}{3}\ \frac{\rho_s - \rho_w}{\rho_w}\ g\ d}$$

With the formula for the settling velocity

$$s = \frac{1}{18}\ \frac{g}{\nu}\ \frac{\rho_s - \rho_w}{\rho_w}\ d^2$$

this can be rewritten as

$$v_s^{\ 4} = (3200)\ \nu\ g \frac{\rho_s - \rho_w}{\rho_w}\ s$$

Assuming as before $\nu = (1.01)\ 10^{-6}\ m^2/s$, $g = 9.81\ m/s^2$ and as lowest mass density of river silt $\rho_s = 1050\ kg/m^3$ (98% water), this formula simplifies to

$$v_s^4 = (1.6)\ 10^{-3}\ s$$

In Figure 2, 10% of the particles have a settling velocity s smaller than 0.08 mm/s. Substituted:

$$v_s^4 = (1.6)\ 10^{-3}*(0.08)\ 10^{-3},$$

$$v_s = (19)\ 10^{-3}\ m/s.$$

giving as requirement:

$$v < 19 \text{ mm/s}$$

This velocity is so high, that it is seldom a factor in the design of the settling zone.

For the rectangular basin of Figure 1, the theoretical detention time equals to:

$$T = \frac{W\ L\ H}{Q}$$

Due to differences in mass density as a result of variation in temperature and suspended matter content and in particular due to wind, actual detention times of the various water particles vary greatly, with minimum values only a fraction of the theoretical one. A smaller detention time, however, means a larger surface loading

$$s_o = \frac{H}{T}$$

and a corresponding decrease in efficiency. To keep this decrease limited the Froude number Tr should be high, at least equal to 10^{-5}

$$Tr = \frac{v^2}{g\ R}$$

Assuming again R   1 m, this gives as requirement

$$v > 10 \text{ mm/s}$$

Already a small reduction, for instance by a factor 3, gives a noticable drop in basin efficiency, to be compensated by an increase in surface area, say by 5-10%.

Summing up, it will be clear that with the assumption R   1 m, it is impossible to satisfy simultaneously the requirements of no turbulence (v 2 mm/s) and no shortcircuiting ($v > 10$ mm/s). In practice a value in between is applied for which v = 6 mm/s seems a good compromise.

When the subsiding particles move downward, water is displaced upward, reducing the effective settling velocity from s to s'. In the field tests of section 3, this phenomenon has already been taken into account, but in other cases calculation is necessary. With a volumetric concentration c_v, water flows upward in the remaining space $(1-c_v)$ at a rate v_d. This gives

$s' = s - v_d$ and $s'\ c_v = v_d\ (1-c_v)$, combined
$s' = s\ (1-c_v)$

For a concentration of 1000 grams dry matter per m^3 and a water content of 98%, the concentration of silt and adhering water equals

$$c = \frac{100}{2}\ 1000 = 50\ 000\ g/m^3 \quad \text{and}$$

$$c = 50\ 000\ cm^3/m^3 = 0.05\ m^3/m^3$$

$$s' = (0.95)\ s.$$

giving on basis of Figure 6 a reduction in basin efficiency less than 2%

For circular basins (Figure 7), the velocity of flow is high near the inlet, resulting in turbulence and bottom scour, and low near the outlet where the wind will result in short-circuiting. The latter phenomenon cannot be avoided, but by giving the inlet well a larger diameter than hydraulically required, the flow velocity here can be kept within bounds.

5. Design of settling zone

According to section 2, the depth H of the tank has no influence on settling efficiency, making a shallow basin with low construction cost rather attractive. On the other hand, however, the depth should be large enough to reduce turbulence and bottom scour and to provide space for sludge storage when the basins are cleaned manually at intervals or space for sludge removal equipment when the basins are cleaned continuously by mechanical means. A value of around 2 m is a good compromise for the small basins under consideration here. Again according to section 2, the length L of the basin can be calculated with

$$L = \frac{v}{s_o} H$$

The value of s_o depends on the settling characteristics of the suspended matter to be removed from river water and various in practice between about 0.03 and 0.3 mm/s. With H = 2 m and v = 6 mm/s as mentioned in the preceeding section, this gives

L = 40 to 400 m

For the small basins under consideration, this is much too large.

Again a compromise is necessary, choosing the ratio L/B as high as financially acceptable. The example at the end of section 2 used $Q = (9.72)\ 10^{-3}\ m^3/s$, $s_o = (0.175)\ 10^{-3}$ m/s and $A = 56\ m^2$. With regard to disturbances, A is increased to 60 m^2, giving at t = 20°C, $\nu = (1.01)\ 10^{-6}\ m^2/s$ for the basin of 2 m depth.

Table 4: Values of important parameters for varying width and length of a basin

A = 60 m^2	D = 2 m					
Parameter	Units	Parameter values				
Width W	m	7.75	5	4	3	2
Length L	m	7.75	12	15	20	30
Circumference 2L + 2 W	m	31	34	38	46	64
Circumference 2L + 2 W	%	100	110	123	148	207
Horizontal velocity v	mm/s	0.63	0.97	1.22	1.62	2.43
Hydraulic radius R		1.32	1.11	1.00	0.86	0.67
Reynolds number Re		820	1070	1200	1380	1600
Froude number Fr	$*10^{-6}$	0.03	0.09	0.15	0.31	0.90

For all combinations of length and width the flow is laminar and unstable, but the latter factor is less pronounced when the ratio between length and width goes up. At the same time this increases the combined length of the vertical walls, augmenting the cost of construction. The best compromise will be between L = 15 m, W = 4 m and L = 20 m, W = 3 m. In case for cleaning and maintenance two basins are required, each for half the capacity, the basis of 15 x 4 m^2 could be split lengthwise. The horizontal velocity remains unchanged, but the hydraulic radius goes down, meaning a lower Reynolds number (800) and a higher Froude number (0.23×10^{-6}). For larger capicities, the situation is more favourable, but to obtain an acceptable value of the Froude number, large ratio's between length and width, 4 to 8, are still necessary.

6. Use of baffles, trays and tilted plates

Flow conditions in the settling zone can be improved by the use of continuous vertical baffles, as indicated in Figure 11. These leave the horizontal velocity unchanged. but reduce the hydraulic radius by which the Reynolds number goes down and the Froude number goes up. For the tank of 15 x 4 m^2 of the preceeding section and 3 baffles at 1 m interval, R = 0.40 m, giving Re = 480 and Fr = $(0.38)10^{-6}$. The water pressure at both sides of the baffle is the same and consequently they can be made of light-weight material such as corrugated aluminium.

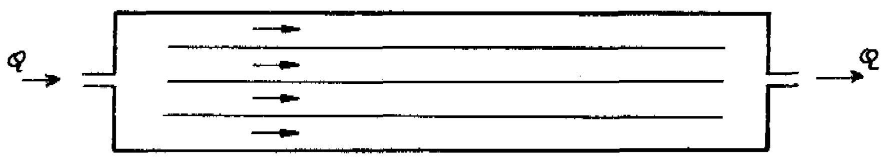


Figure 11. Plan of settling tank with vertical baffles

Horizontal baffles are called trays and serve on one hand the same purpose as mentioned above. In the other hand, however, they reduce the surface load as in Figure 12.

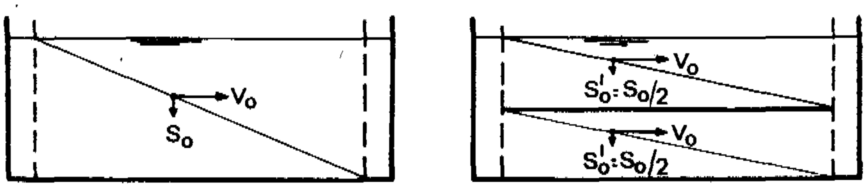


Figure 12. Longitudial section of a tray settling tank.

The two compartments may be considered as two tanks operating in parallel. For the same efficiency the tank of 15 x 4 m^2 of the preceeding section may be reduced to 15 x 2 m^2, giving with v = 2.43 mm/s and R = 0.33-0.50 m Re = 800-1200 and Fr = (1.8) - (1.2) 10^{-6}. Again the Reynolds number is excellent, but the Froude number a little too low. Still better results can be obtained with more trays, but with regard to sludge removal, the vertical distance between them may not drop below 0.7 m, asking for an increase in the total depth of the tank, which will augment construction cost. The trays are loaded by the weight of the accumulated sludge and must be of sufficient structural strength. Commonly they are made of reinforced concrete or wood.

Trays set at a short distance, 5-10 cm, must be self-cleaning, which can be obtained by setting them at a steep angle, 50-60° with the horizontal. In these so-called tilted plate separators (Figure 13), the sludge does not adhere to the plates, but slides down to the bottom of the tank.

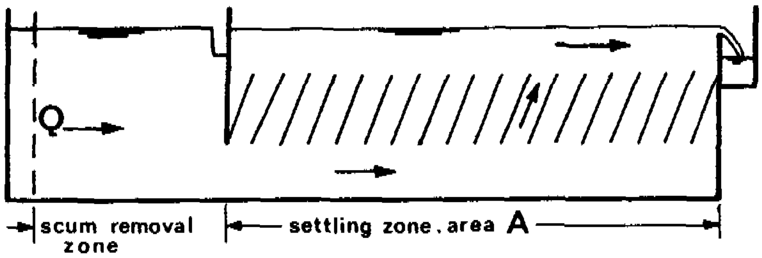


Figure 13. Tilted plate separator

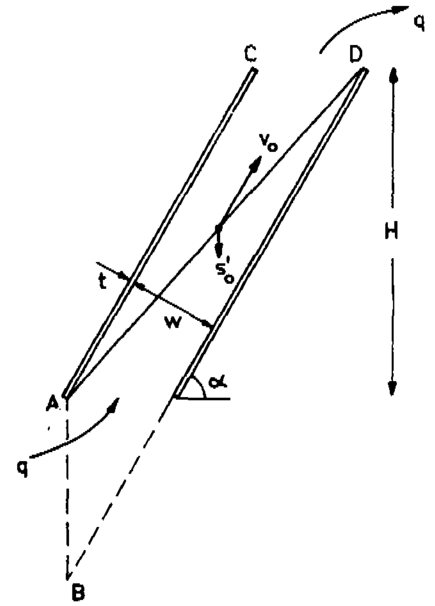


Figure 14. Tilted plate separator with upflow

The hydrodynamics of a tilted plate separators are shown in Figure 14, from which follows

$$\frac{s'_o}{v} = \frac{AB}{BD} = \frac{\dfrac{W}{\cos}}{\dfrac{W}{\sin \ \cos} + \dfrac{H}{\sin}} = \frac{W \sin}{W + H \cos}$$

With $v = \dfrac{q}{W}$ and the apparent surface loading equal to

$$s_o = \frac{q}{CD} = \frac{q}{\dfrac{W + t}{\sin}} = \frac{q \sin}{W + t}$$

$$s_o' = \frac{W + t}{W + H \cos} s_o$$

With for instance H = 1.5 m, W = 0.07 m, t = 0.005 m and
= 55°

$$s_o' = \frac{s_o}{12.4}$$

reducing the surface area of 60 m^2 of the preceding section to 4.8 m^2 only, say a tank of 5 m length and 1 m width. The flow between the plates is governed by

$$v = \frac{W + H \cos}{W \sin} s_o' \quad , \; R = \frac{W}{2}$$, giving for the example quoted

Re = 98   Fr = (2.3) 10^{-6}

that is to say a flow which is laminar and very stable.

Plates only 1 m wide can be made of light-weight material such as asbestic cement or corrugated aluminium.

The construction of Figure 13 has the disadvantage that in the space below the tilted plates, the downward falling sludge is disturbed by the horizontal movement of the raw water (Figure 15).

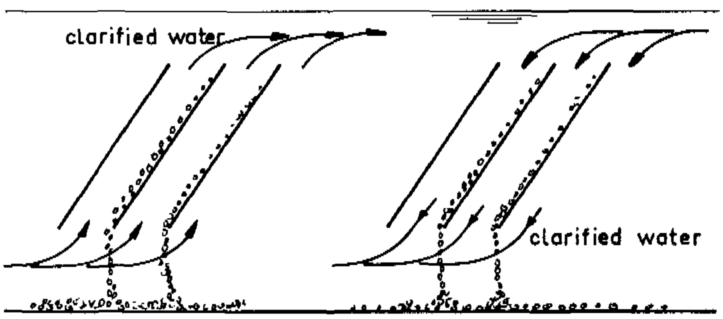


Figure 15. Re-suspension of settled out material

In industrialized countries this is prevented by a special construction of the tilted plates, using also downward and horizontal water movement between them. For the small plants under consideration in this paper, this is too complicated and here simple flat or corrugated plates should be used. Also these add to the cost of construction and in many cases a tilted plate separation is not cheaper than the plain basin of Figure 15 and more difficult to operate.

7. Inlet and outlet constructions

The inlet construction of a sedimentation tank serves to distribute the incoming water equally over the full cross-sectional area of the settling zone, with equal flow velocities over the width of the basin being most important. This can best be achieved by constructing an inlet channel over the full width of the tank, provided with holes in the bottom at regular intervals (Figure 16).

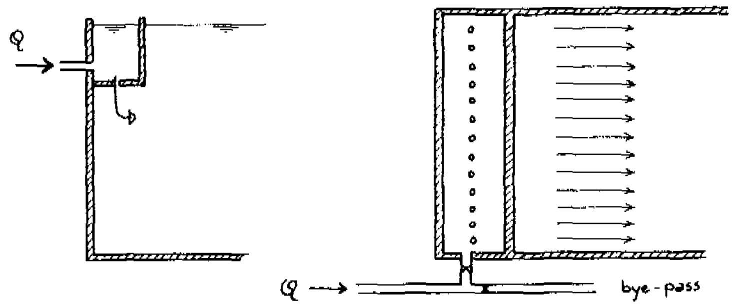


Figure 16. Inlet construction

Whenever possible, the cross-sectional area of this channel should provide an initial velocity of about 0.3 m/s. On the other hand it should be 1.5 to 2 times as large as the combined cross-sectional areas of the holes, which in their turn should be big enough to prevent clogging by larger suspended particles.

For the example at the end of section 2, $Q = (9.72)10^{-3}$ m^3/s, an area of 0.2×0.3 m^2 gives an initial velocity of only 0.16 m/s. In the bottom 10 holes of 6 to 7 cm diameter should now be constructed.
Instead of through holes, the water can also be brought into the basin over a weir. For $Q = (9.72)$ 10^{-3} m^3/s and $W = 4$ m, the overflow height is only 12 mm, giving a rather unequal distribution when the top of the weir is slightly out of the horizontal. A better solution is to provide the weir with 90° V-notches, say at intervals of 0.25 m, giving an overflow height of 45 mm and constructing the weir crest of a separate metal strip fastened to the concrete wall by bolts in slotter holes. When using inlet holes, the head loss of the inlet construction is only a few cm's, with a V-notch weir somewhat larger, 5 to 10 cm.

The outlet should again be constructed in such a way that the clarified water is abstracted evenly over the full cross-sectional area of the settling zone. In this zone, the water is accellerated and thereby, stabilized and with a simple outlet weir (Figure17) satisfactory results can be obtained. To avoid disturbance of the settled out material at the bottom of the tank, the weir loading should be small, which can easily be obtained by installing a number of weirs (Figure 17).

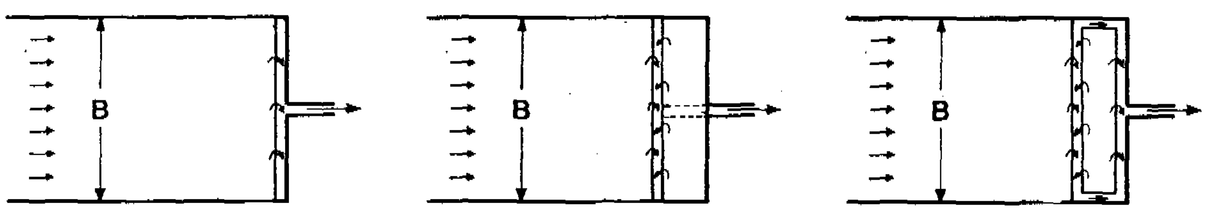


Figure 17. Outlet construction with 1, 2 and 3 weirs

For good results this number should equal

$$n > \frac{L}{5H}$$

In the example of section 5, L = 15 m and H = 2 m

$$n > \frac{15}{(5)\ (2)} \text{ or } n = 2$$

The overflow height is now very small and adjustable V-notch weirs should be applied.

8. Sludge deposition and sludge removal

The average sludge deposition at the bottom of a settling tank can easily be calculated. When the raw water has a suspended matter content of for instance 70 g/m^3 of which 80% is retained, then with $Q = (9.72)\ 10^{-3}\ m^3/s$ and $A = 60\ m^2$ the deposition equals

$$a = \frac{(70)\ (0.8)\ (9.72)\ 10^{-3}}{60} = (9.07)\ 10^{-3} \text{ gram dry matter}/m^2,s$$

In reality the sludge at the bottom of the tank has a high water content, say 97% increasing the deposition to

$$a = (9.07)\ 10^{-3}\ \frac{100}{100 - 97} = 0.302 \text{ gram wet sludge}/m^2,s$$

and with the mass density of wet sludge at 1.05 g/cm^3

$$a = \frac{0.302}{1.05} = 0.288\ cm^3/m^2.s = (0.288)10^{-6}\ m/s$$

Allowing an everage deposition of 0.25 m, gives an interval between cleanings equal to

$$t = \frac{0.25}{(0.288)10^{-6}} = (0.878)10^6\ s \cong 10 \text{ days}$$

In reality sludge deposition is larger near the inlet and smaller near the outlet of the tank, as shown in Figure 18. The maximum rate varies between 1.5 and 2.5 times the average value calculated above.

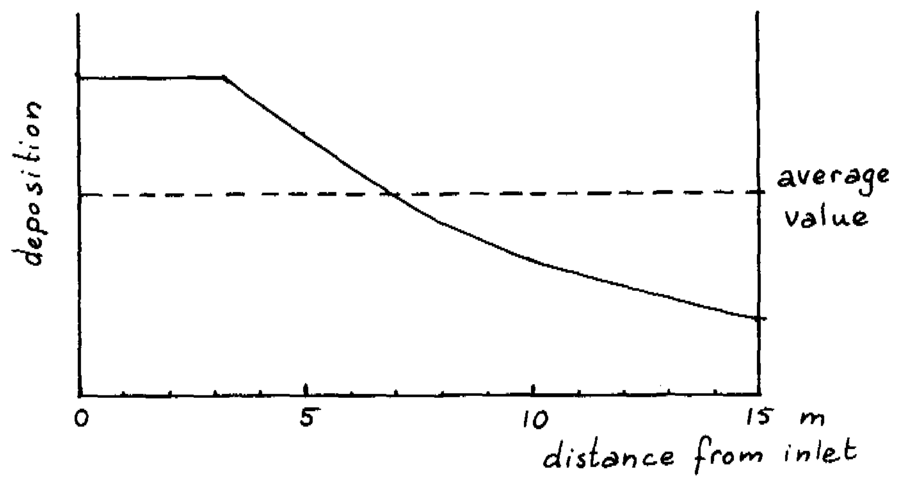


Figure 18. Sludge curve calculated for the settling velocity distribution of Figure 2.

When sludge accumulation is slow, the tank can be cleaned manually at intervals of one week to several months. The tank is first drained after which the sludge is flushed to the inlet end with the help of a jet of water (firehose), here to be removed with a sludge pump. To facilitate this process, the tank bottom is sloping, from the longitudinal walls to the centre (5%) and from the outlet to the inlet end (1-2%) as shown in Figure 19.

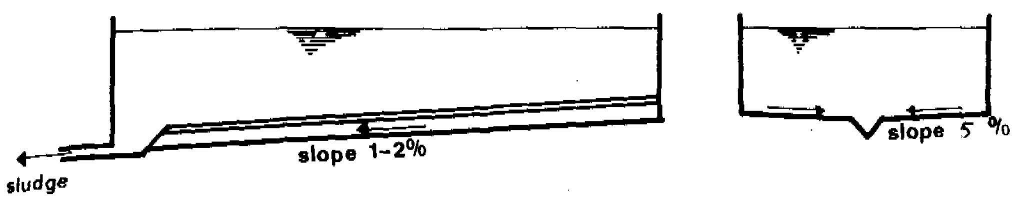


Figure 19. Sloping bottom in settling tank to facilitate manual sludge removal

When the interval between cleaning is small, a few weeks only, cleaning cannot be postponed to the time that the river water has a low suspended matter content. At least two tanks must now be built, with a combined surface area equal to 1-1.5 times the calculated value.

When sludge accumulation is rapid and the interval between cleanings drops to a few days, mechanical sludge removal is required. The tank shows the same longitudinal slope as indicated in Figure 19, but transversely the bottom is flat. In Figure 20 the equipment consists of two chains, carrying scraper blades at intervals of 2-3 m and moving at a velocity of about 1 cm/s. The sludge is carried to a sludge pit at the inlet and of the tank from where it is removed by gravity or pumping.

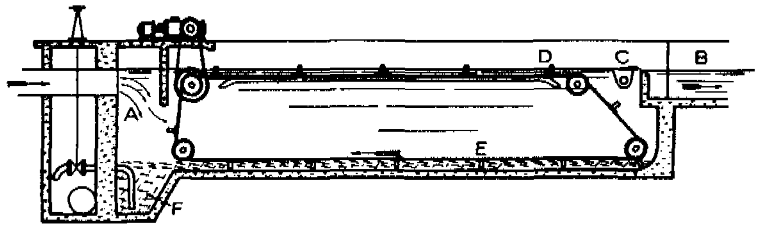


Figure 20. Mechanical sludge removal by chain carried scraper blades

Moving equipment below water has disadvantages and for this reason the scraper blade of Figure 21 is suspended from a travelling bridge. For maintenance and repairs, the rake can now be lifted out of the water without interrupting the settling proces.

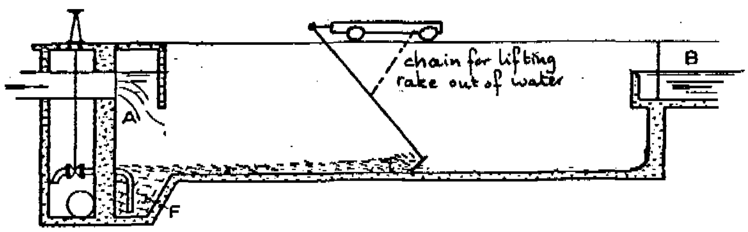


Figure 21. Mechanical sludge removal by rake suspended from travelling bridge

9. Construction

In industrialized countries, sedimentation tanks are made of concrete or steel, providing a rectangular cross-section, a water-tight construction and easy cleaning. In developing countries where cement and steel are at a premium, the same advantages can be obtained with reinforced or massive masonry work or with massive concrete (Figure 22).

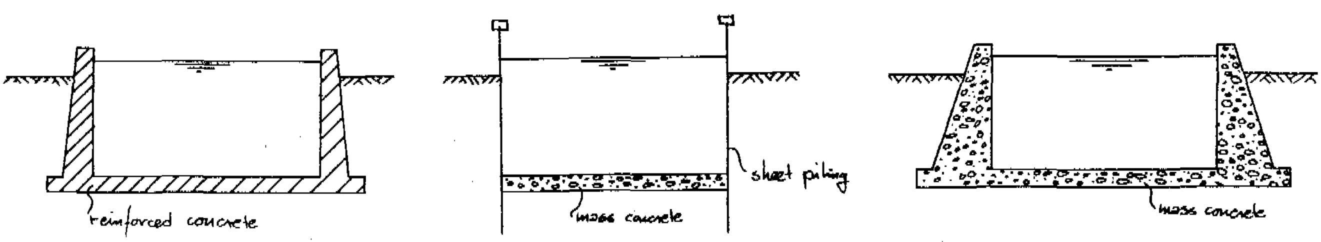


Figure 22. Cross-section over settling tank constructed of concrete and steel

To save on building materials, clay basins may also be considered. When bottom and sides are covered by a layer of chicken-wire reinforced concrete, by pavement tiles or similar materials, the loss of water will be small. To prevent the lining from bursting upward when the basin is empty for manual cleaning, however, the bottom should be above the highest groundwater table. Without a lining, the loss of water may be acceptible, but it will decrease with time due to clogging of the pores by settled out suspended matter. Cleaning may now be a problem. The basin should be drained and left in this position for several weeks, allowing the sludge layer to consolidate after which it can be removed with shovels and wheel barrows. This is a lot of work, which can be reduced by applying a very low surface loading. In all cases of sloping sides, the design width of the basin is measured at half the depth (Figure 23).

Figure 23. Basins with and without lining

10. Selected literature

1. American Water Works Association, (1971), Water quality and treatment. McGraw-Hill. New York. (pp 139-155).

2. CEPIS, (1982). Modular plant for water treatment. Lima. (Vol. I, pp 19, 20, 24-26; plan no. 4, 12, 13).

3. Davies, V.D.(1942). Handbook of applied hydraulics. Mc Graw-Hill Book Company, New York (pp. 791-800).

4. Huisman, L. and others (1981). Small community water supplies (T.P. 18) ed. by E. Hofkes, IRC, The Hague (pp 229-241)

5. Schultz, C.R. and Okun, D.A. (1984). Surface water treatment for communities in developing countries. John Wiley & Sons. New York (pp 30-39).

6. Fair, G.M., Geyer, J.C. and Okun, D.A. (1968). Water and Waste Water Engineering, John Wiley & Sons. New York (pp 25/1-29).

CHAPTER 6

TILTED PLATE SETTLING

by A. Castilla and J.E.M. Smet

1. INTRODUCTION

The theory of multilayer sedimentation was developed by Hazen early this century. But it was not until the 1960s that this theory was put into practice in tube settlers and soon after in the tilted plate settlers. The plate settler with well designed inlet and outlet structures permits uniformity of flow and minimizes short circuiting problems. Lack of uniform flow resulting in short circuiting and decrease detention time are serious problems in horizontal sedimentation tanks.

The basic concept of plate settlers is a series of parallel trays inclined at an angle sufficient to stimulate self-cleaning (Arboleda, 1986) (Figure 1). Problems were encountered with sludge removal in early tests of horizontal plates. The water flow is usually upward while the settled sludge moving slowly downwards against the current collects on the bottom of the settling tank and has to be removed either manually or mechanically.
The effective gravity settling area of tilted plate settlers is equal to the sum of the projected area of each plate on the horizontal. That results in general a ten times more efficient use of the area.
The detention time can often be reduced to less than 15 minutes. This means significant reductions in construction costs.

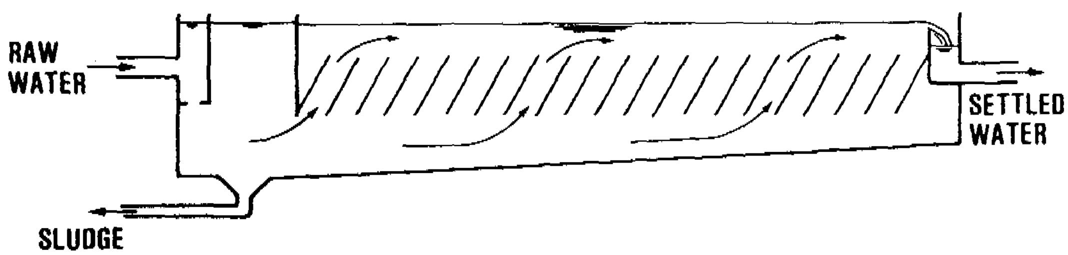


Figure 1: Typical plate settler

2. THEORY OF THE PROCESS

2.1 Counter current plate settlers

The hydrodynamic principle of inclined plate settlers is shown in Figure 2 (angle of inclination of θ). The water flows between the plates at a mean velocity V_0 (laminar flow is assumed). A suspended particle that enters at B will settle in C if its fall velocity is Vsc.

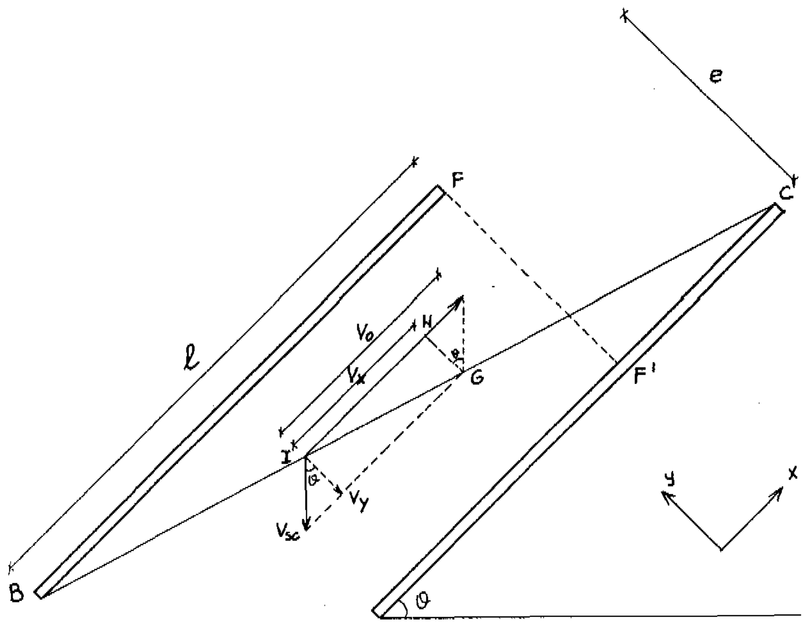


Figure 2. Hydrodynamic forces in plate settlers

For this type of plate settlers the theoretical design equations are derived for required total sedimentation area, equivalent surface load and overflow rate.

Area of a plate settler

The required total sedimentation area can be easily calculated if the velocity of the flowrate V_0 between the plates is chosen (Figure 2). This velocity is equal to:

$$V_0 = Q / a.e.(n-1)$$

where a = the width of the plates
n = the number of plates (the number of conduits between the plates is (n -1))
Q = the capacity of the settler
e = distance between the plates

The horizontally projected cross-sectional area (of the (n-1) plates) on which sedimentation can take place is:

$$A = (n-1).a.(e + e_p)/\sin\theta$$

or $$A = (Q/V.\sin\theta).(1 + e_p/e) \quad \text{[Eq. 1]}$$

where e_p = thickness of the plates (measured perpendicular to the plates)

Equivalent surface load

The particle that determines the critical velocity enters the settler at point B, follows the traject BC and should settle at C (Figure 2). The particle moves upward through the plates at a mean velocity V_o parallel to the plates. The critical velocity of this particle to settle is V_{sc}. The resultant of V_o and V_{sc} can be divided in V_x and V_y. In Figure 2, the two triangles F'FB and GIH are similar, which means:

$$e/V_y = 1/V_x$$

Assuming that the relative length L is equal to: $L = 1/e$ [Eq. 2]

From the geometrics in Figure 2 can be derived:

$$V_y = V_{sc}.\cos\theta$$

$$V_x = V_o - V_{sc}.\sin\theta$$

and the critical fall velocity $V_{sc} = Vo/(\sin\theta + L.\cos\theta)$

[Eq. 3]

(the actual velocity along y-axis slightly varies because of non-ideal uniform flow conditions)

These determinations are only valid if the flow between the plates is laminar.
This laminar flow develops gradually over a length of L_T after the water has entered between the plates. This length L_T is called the transitional zone, and must be added to the calculated relative plate length because possible turbulences may disturb the sedimentation of the suspended solids (Yao,1970).

$$L_T = 0.013\ R_N = 0.026\ {}^{*}\ V_0\ .\ e\ /\ v \quad \text{[Eq. 4]}$$

with V_0 in m/s,
v (kinematic viscosity of water) in m^2/s
e in m

Equivalent overflow rate and the relative length L

The overflow rate s_0 which is the flow rate per unit tank area, is commonly used in the design of horizontal flow rectangular settling tanks. For these tanks the overflow rate equals the critical particle velocity V_{sc}. This implies that any particle present in the water with a settling velocity of at least the critical particle velocity would be removed.

This concept also applies to tilted plate settling:

$$V_{sc} = C * V_0 / (\sin \theta + L.\cos \theta)$$

where C is a constant with a value of 94.13 if V_0 is expressed in m/s and V_{sc} in $m^3/m^2.h$.

$$V_{sc} = 94.13 * V_0 / (\sin \theta + L.\cos \theta) \qquad \text{[Eq. 5]}$$

V_{sc} for plate settlers is also the "equivalent overflow rate"

This equation is commonly used in plate settlers design when an appropriate overflow rate and an angle of inclination are first selected.

The total plate length l_{tot} is derived from

$$l_{tot} = L_{tot} * e \qquad \text{[Eq. 6]}$$

with $L_{tot} = L + L_T$

The relative length has considerable effect on the critical velocity decrease. The optimal value of L for a significant decrease in V_{sc} is about 20. Values of L above 40 have a minimum effect on the critical velocity decrease.

For solid removal, the most effective angle of inclination is 0°. The settler area required is then at the minimum. To ensure removal of settled sludge, the angle should be sufficient to facilitate self-cleaning of the plates. Theoretically, an angle of inclination of 90° gives an efficiency of nil, but in practice particles of $V_{sc} > V_o$ still settle. Performance of the settler deteriorates rapidly if the angle is greater than 40° (Figure 3). This is indicated by the sudden increase in V_{sc}. However, from experience desludging is best achieved at a 60° angle of inclination.

Sludge accumulates over the entire surface of the plates. As the sludge blanket grows, most of the sludge will move slowly downwards. However, if the sludge accumulation at point C becomes critical, that is settled particles are resuspended by the effluent which may also occur for θ=60°, the plates should be cleaned using water jets to prevent increase of effluent turbidity.

The settling particles are obstacles for small particles present in the raw water. Through various actions (collision, adhesion), their upward velocity is reduced and they settle on the plates. The sludge blanket formed on the plates reduces the settling depth and accelerates the retaining processes of the system. This phenomenon is not incorporated in the given equations.

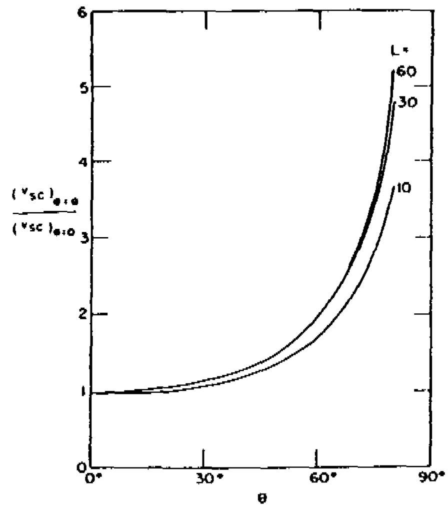


Figure 3. Angle of inclination and rate sedimentation (Yao, 1970)

2.2 Cross current plate settlers

The raw water in a cross current plate settler flows horizontally between the plates. The mechanisms are similar to the conventional plain sedimentation but the settling depth is less (Figure 4). A cross-current plate settler for discrete particulates with a capacity of 1.0-10.0 l/s has been included in the modular plant design developed by CEPIS (1982).

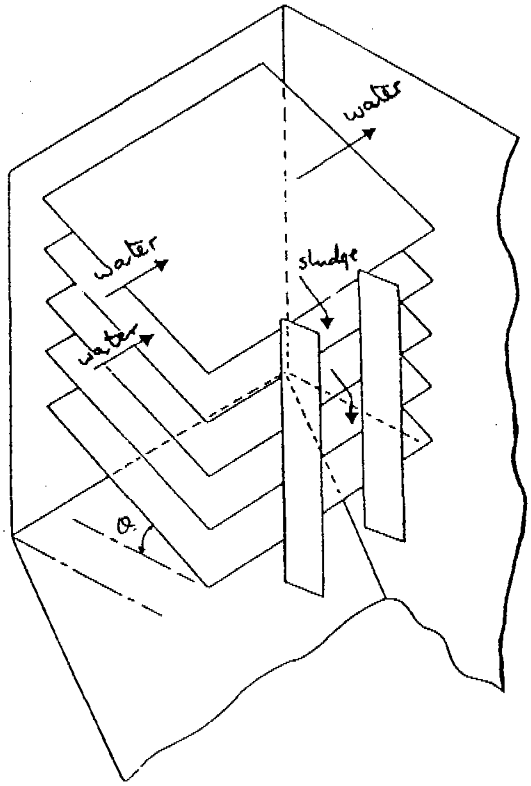
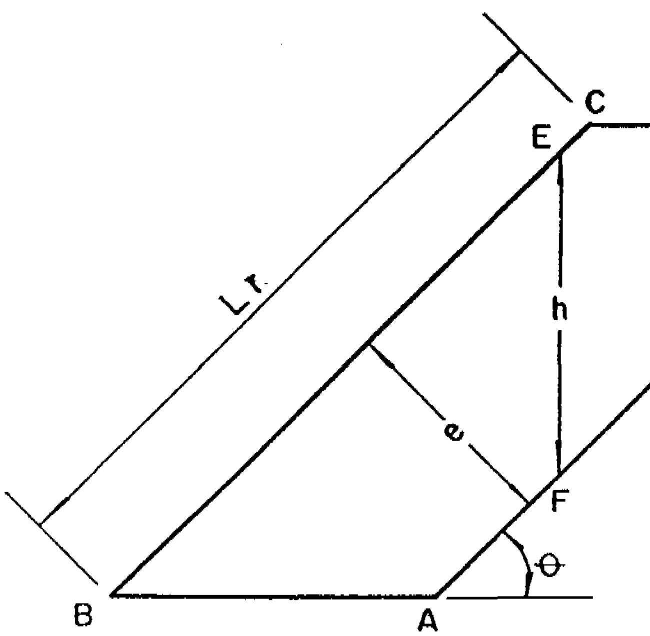


Figure 4. Typical cross current plate settler



e = distance between the plates
θ = angle of inclination
k = sedimentation depth

Figure 5. Cross current plate settler geometric principle. The current is perpendicular to the drawing.

The geometric principle of a cross-current plate settler is demonstrated in Figure 5. The hydraulic radius is (assuming L_r>>e)

$$R = \frac{L_r \cdot e}{2\ L_r} = \frac{e}{2} \qquad \text{[Eq. 7]}$$

$$V_a = \frac{R^{0.167}}{n} \sqrt{K \frac{(\rho_s - \rho) \cdot d}{\rho}} \qquad \text{[Eq. 8]}$$

where V_a = drag velocity
n = Manning's roughness coefficient
K = Camp's drag coefficient
s = specific density of the particle
= specific density of the water
d = particle diameter

All variables are in SI units.

or:

$$V_a = \sqrt{\frac{8\ K.\ g}{f} \frac{(\rho_s - \rho).\ d}{\rho}} \qquad \text{[Eq. 8]}$$

where f = Darcy-Weisbach's friction coefficient
g = gravity constant

$$T = \frac{h}{V_h}$$

where T = settling time

$$L_c = V_h \cdot T \qquad \text{[Eq. 9]}$$

where V_h = horizontal velocity
L_c = theoretical length of the settling basin

Uniform flow conditions help to prevent re-suspension of settled particles. Proper operation procedures of cross current plate settlers are essential. Well designed and adequate inlet devices to permit uniform distribution of the influent over the total height contribute to good flow conditions.

3. RESEARCH EXPERIENCE WITH THE PROCESS

A lamella separator to clarify the water for the New York City Metropolitan was tested. The coagulant dosage was constant throughout the test. The total suspended matter at the inlet of the settler ranged between 10 and 16 mg/l because of variation in the load of suspended solids in the raw water. As shown in Figure 6 there is a linear relationship between solids removal efficiency and overflow rate (Fulton, 1981).

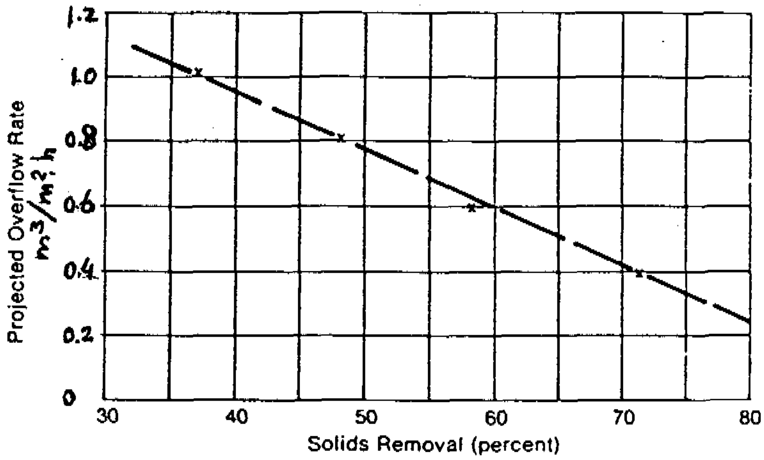


Figure 6. Relationship between solids removal and projected overflow rate in a tilted plate settler (Source: Fulton, 1981).

Tariq and Yao (1976) studied the performance of a plate settler without inclination. The tank dimensions were 89.5 cm x 18.5 cm, and the inter-plate distance, 63 mm. Tests were done with tap water mixed with Fuller's earth and with raw canal water without coagulation. Turbidity removal efficiencies ranged between 64% (influent turbidity of 280 JTU) and 89% (influent turbidity of 70 JTU), for various very low flow rates (V_0 range 0.042-0.385 m/h (0.012 - 0.107 mm/s)).

Several experiments have been undertaken on 'co-current' and 'counter-current' plate settlers. Rao and Paramasivam (1980) concluded that downflow units are preferable because sludge removal is a self-action but the collection of the clarified water demands a complicated construction (Figure 7). Upflow units are also self-desludging and recovery of the treated water is less complicated.

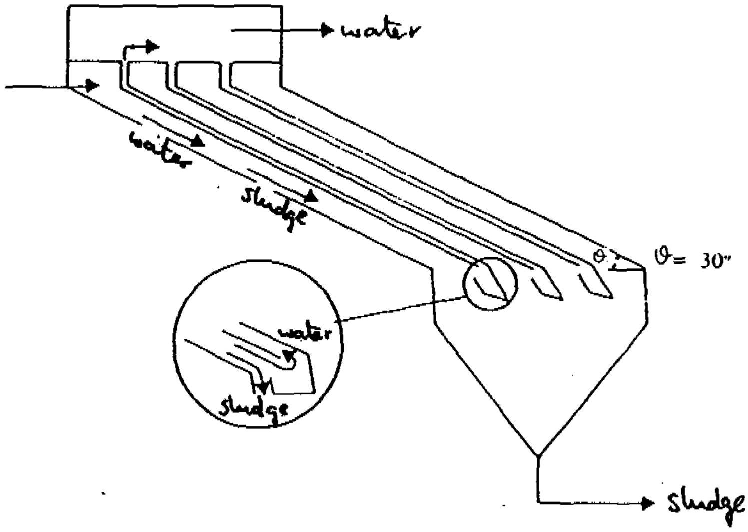


Figure 7. Co-current lamella separator

4. FIELD EXPERIENCE

Plate settlers are widely used in Latin America for both new plants and upgrading existing facilities. In most plants coagulation is applied prior to sedimentation. Surface loads are 5 - 7.7 m/h (1.39 - 2.13 mm/s) with removal efficiencies of more than 90% for coagulated water. There is little experience with discrete particle sedimentation in tilted plate settlers.

La Toma, Guayaquil, Ecuador

Turbidity removal efficiencies of two systems, conventional sedimentation and plate settler, both receiving flocculated water, were compared over a one-year period at the La Toma treatment plant in Guayaquil, Ecuador (Arboleda, 1986). The plate settler consisted a wooden box (0.6x1.0x2.2 m) containing 17 plates placed at an angle of 60° and spaced 55 mm apart. The surface load for the conventional sedimentation was 1.25-1.54 m/h (0.347-0.423 mm/s), and varied for the plate settler. The turbidity removal efficiency was similar for both systems at an equivalent surface load of 1.0 m/h (0.278 mm/s). That is 0.63-0.83 times the surface load of a conventional system (Figure 8). The raw water turbidity ranged between 200 and 1,000 NTU.

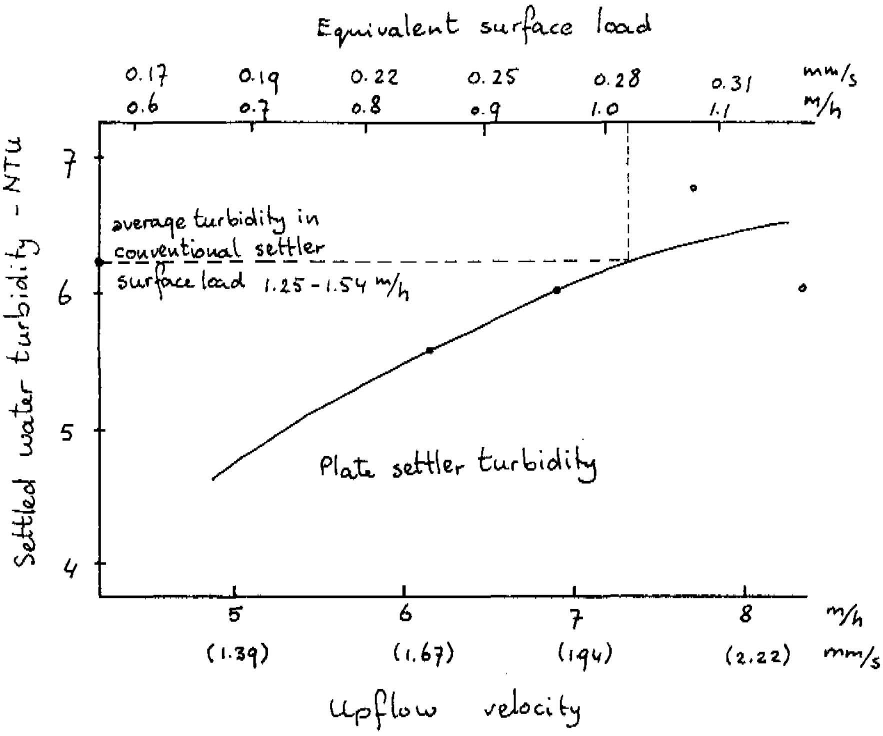


Figure 8. Relation between effluent turbidity and surface load of a conventional sedimentation tank and a plate settler module at Guayaquil, Ecuador (Source: Adapted from Arboleda, 1986).

Prudentopolis, Brazil

The experimental plate settler at the Prudentopolis, Brazil, had an overflow rate of 1.1 m/h which is comparable to 7.7 m/h in a conventional sedimentation. For more than 80% of the time, turbidity of the treated water was less than 3 NTU (Arboleda, 1986).

Cali, Colombia

The horizontal sedimentation tanks at the Rio Cali treatment plant in Cali, Colombia, were converted to plate settlers because of required extension of the plant capacity and because of poor functioning due to the existence of dead space and short circuiting (Medina and Hudson,1980). The asbestos-cement plates are 2.4 x 1.2 m, and the angle of inclination is 60° with an interspace of 60 mm.
Plate settler effluent is collected through perforated pipes bringing the water to a central channel in each tank. If well designed this system of collection should ensure reasonably uniform flow.

Cleaning is done manually. Firstly the tanks are drained downwards to discharge most of the settled particles. Then high pressure hoses are used to clean the plates. Because of the increased sludge load each tank needs cleaning monthly. Regular removal of the sludge is imperative to maintain good quality effluent and so prevent the settled sludge becoming re-suspended.

In general plate settlers are more efficient than tube settlers because of the better relationship of length to width and the hydraulic characteristics.

5. DESIGN CRITERIA

Pilot studies are required to develop final designs for non-coagulated water.

Using the equations the three main parameters in plate settling can be calculated:

- Reynolds Number
- required area of the settler
- equivalent surface load.

Counter current plate settlers

The Camp column settling test (Yao, 1979) is a reasonable alternative for pilot studies in determining the surface load. Values obtained from settling tests should be divided by the safety factor 2. In Latin America surface loads of plate settlers treating coagulated waters are used between 5 - 7.7 m/h (1.39 - 2.13 mm/s) with removal efficiencies of more than 90% (Arboleda, 1986). Surface loads for non-coagulated waters should be much lower.

A common plate size is 1.2 x 2.4 m, placed 50-60 mm apart at a 60° angle inclination.

The horizontal velocity of the water greatly influences flow distribution between the plates. In studies on the inflow velocity of water and the effects on flow uniformity between the plates, Garcia (1978) found that the horizontal inlet flows right under the plates gave a very disturbed flow distribution between the plates. More recently in Latin America the plate settlers for coagulated waters are designed with a perforated duct under the plates. This duct acts as an inlet manifold to direct the floc to the plates, without breaking the floc (Figure 9 and 11).

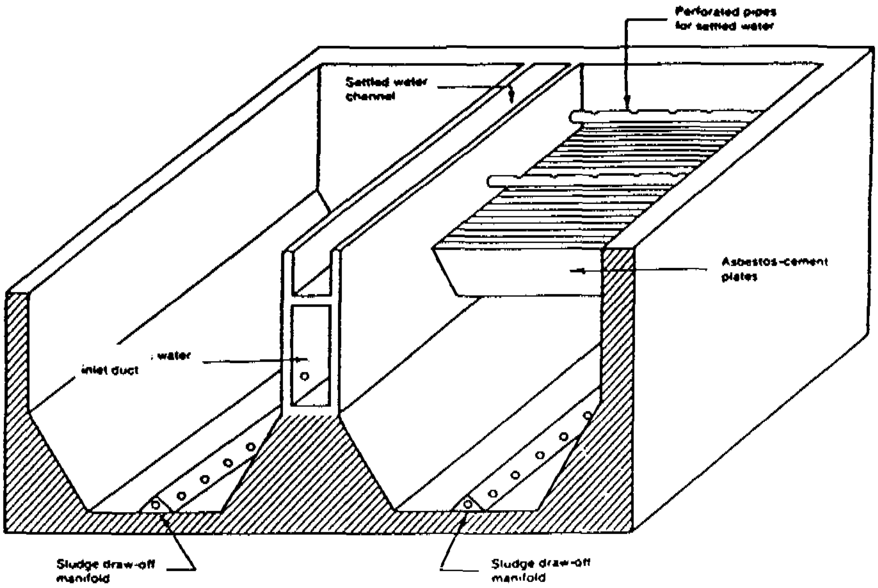


Figure 9. Plate settlers with longitudinal hoppers at the bottom and an inlet water manifold (Source: Arboleda, 1986).

A better solution are laterals connected to a main pipe. The diameter and the interspacing of the laterals and the orifices should ensure uniform flow to the plates. The orifices are drilled at an angle of 30° with the horizontal to prevent blocking of the holes with settling solids (Figure 10).

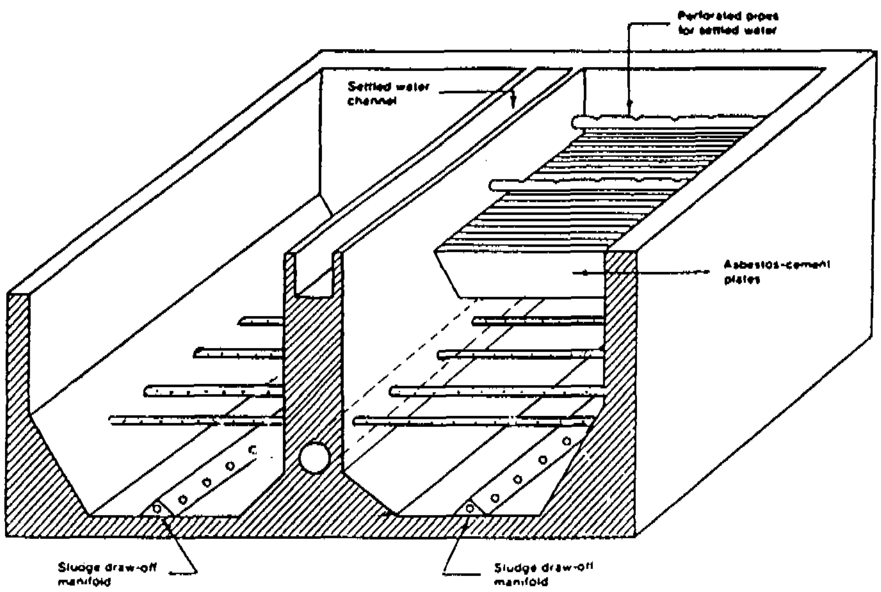


Figure 10. Inlet construction using main and laterals (Source: Adapted from Arboleda, 1986).

Cross-current plate settlers

CEPIS (1982) recommends a surface load for coagulated waters of 6.0 m/h (1.67 mm/s) with a retention time of approximately 25 minutes.

Water of turbidities up to 500 NTU can be treated sufficiently if turbidity is caused by particles larger than 1 μm (CEPIS, 1982).

General

There is considerable accumulation of sludge in tilted plate settlers mainly because the accumulation area is much smaller than for a conventional sedimentation tank. Sludge removal can either be done continuously or periodically. For this purpose sludge hoppers and drain valves are commonly used.

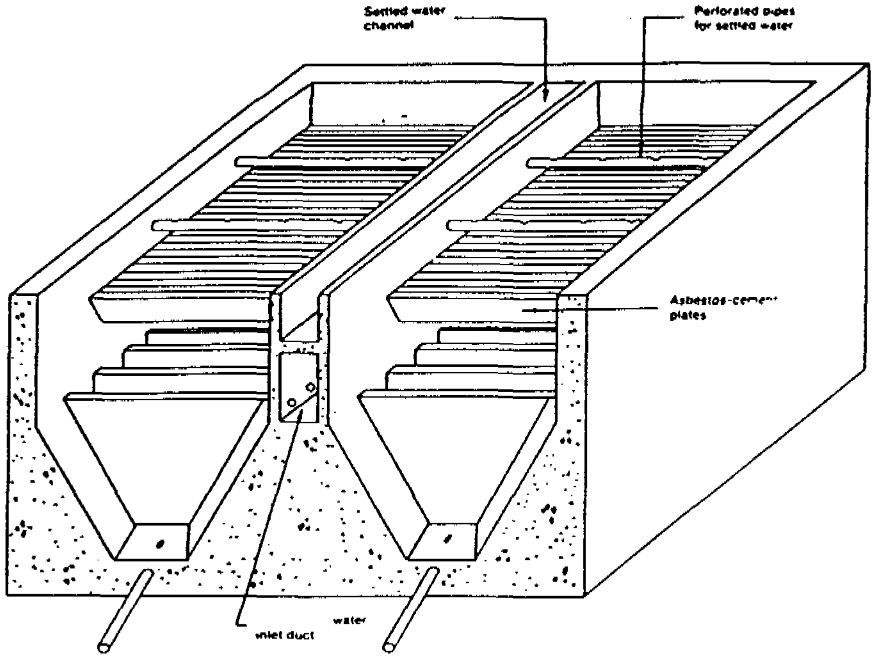


Figure 11. Plate settler without multiple hoppers at the bottom and with an inlet hopper manifold (Source: Arboleda, 1986).

Sludge removal should be uniform over the bottom to prevent flow disturbance. Accumulation of sludge in heaps could result in re-suspension of the settled solids. Careful consideration should be given to the dimensions of the sludge removal manifold to empty the bottom hoppers. The summation of the total area of inlet ports should be half the area of the main pipe or duct (Arboleda, 1986.

In water of turbidities of 100 - 800 NTU sludge production can be estimated using the equation of Ruiz et al. (1983).

$$V = Q.(K_1.D + K_2.T)/100$$

where
V = volume of sludge per day (m^3/d)
Q = water flow (m^3/d)
D = optimal dosage of coagulants (g/m^3)
T = water turbidity (NTU)
K_1 = coefficient between 0.015 and 0.025
K_2 = coefficient between 0.0014 and 0.0009

Effluent collection

The outlet should also be careful designed to obtain a uniform flow distribution. Lateral perforated pipes or lateral troughs placed at intervals of up to 1.5 m, and connected to a main duct guarantee undisturbed flow conditions. For lateral perforated pipes the summation of the ports should be 0.144 times the area of the main pipe ($\Sigma a = 0.144 * A$).

Pipe and trough diameters are determined using the equations (Fair and Geyer, 1956):

$$Q = 1.375\ w\ H_o^{3/2} \qquad \text{for troughs}$$

where w = width,
H_o = initial water depth

$$d = Q^{0.4} \qquad \text{for pipes}$$

where d = diameter of the pipe

The distance between the top of the plates and the water level in the tank is at least 0.6 m, but preferably 0.9 m.
Outlet troughs are installed with V-notches to compensate for any error in levelling of the laterals.
Lateral pipes are perforated at the top with 25 mm orifices, at intervals of 100 to 200 mm in order to maintain a hydraulic head of 50-100 mm above the perforations.

To obtain the optimal design pilot testing should be done under the local conditions and water characteristics. Consideration needs to be given especially to the angle of inclination of the plates, the distance between the plates, and the velocity V_0.

6. CONSTRUCTION MATERIALS

The selection criteria for construction materials are low cost, not soluble in water, durability, and non toxic and not hazardous to health (Arboleda, 1986). Asbestos-cement and marine wood are acceptable and are frequently used. Asbestos-cement plates of 10 mm are light in weight. A common size is 1.2 x 2.4 m. A coating with in case of corrosive water, painting of the plates with bituminous paint may be advisable for use in corrosive water to avoid leaching of the asbestos fibres.
Careful consideration must be given to the presence of phenol leachates in plywood (Arboleda, 1986).

7. OPERATION AND MAINTENANCE

The major operating procedures of tilted plate settlers are regulation of the water inlet and regular desludging by opening the sludge valves at the bottom of the settlers.

Depending on the raw water characteristics, sludge accumulated on the tilted plates may need to be removed from time to time. This is done with water jets to move the sludge downward into the hoppers and drained through the sludge valves.
The settlers tank should be refilled carefully to re-establish uniform flow conditions. It may be necessary to flush the first effluent until the normal effluent quality is reached.

8. COSTS

Plate settlers represent 50-75% of the cost of tube settlers (Arboleda, 1986).

9. CONCLUSIONS AND RECOMMENDATIONS

Plate settlers are more economical to build and they produce better effluent quality than horizontal conventional sedimentation tanks. Fewer problems in operation are encountered because the process is hydraulically stable and reliable.

In comparison with plain sedimentation plate settlers are more compact, about one tenth of the detention time is required, and high overflow loadings are possible, flow conditions are uniform and short circuiting is minimized.

With or without coagulation, tilted plate settlers have a high potential both technically and economically as pre-treatment step of surface water prior to slow sand filtration. As a rule, extensive pilot testing should be carried out to optimize the design according to the raw water characteristics.

REFERENCES

Arboleda-Valencia, J. (1986). A New Approach to Treatment Plant Design and Construction in Latin America, Journal American Water Works Association, 78, 7, 92-105.

CEPIS (1982). Modular Plants for Water Treatment, Vol I and II, Technical Document Series 8, Lima, Peru.

Davies, H.L. and White, C.M. (1928). Proceedings of the Royal Society of London. Vol. 119, pp.921.

Fair, G.M. and Geyer, J.Ch. (1956). Water Supply and Wastewater Disposal. New York, USA, John Wiley & Sons.

Fulton, G.P. (1981). Control Settling to Optimize Water Filtration, Water, Engineering and Management. March 1981.

Garcia, E. (1978). Compartamiento Hidráulico de los Sedimentadores de Alta Rata. Proyecto de grado. Universidad de los Andes. Facultad de Ingeniería (December 1978), Bogotá, Colombia.

Hazen, A. (1904). On Sedimentation. Transactions of the American Society of Civil Engineers, Vol. 53, pp.45.

Medina, L.E. and Hudson, H.E. Jr. (1980). Upgrading the Old, While Building the New. Journal of American Water Works Association, 72, 12, 666-671.

Rao, C.V.C and Paramasivam, R. (1980). High Rate Settlers - A Review. IAWPC Technical Annual VI & VII, pp.91-106.

Ruiz, G. et al. (1983). Determinación de Volumenes de Barro y su Hidraulica de Extracción en una Planta de Tratamiento de Agua Potable. Tesis de grado, Universidad de Chile, Santiago de Chili.

Tariq, M.N. and Yao, K.M. (1976). A Laboratory Study on Tube Settling. Report no. 028-02-76. Lahore, Pakistan, Institute of Public Health Engineering and Research, University of Engineering and Technology.

Yao, K.M. (1970). Theoretical Study of High Rate Sedimentation. Journal of Water Pollution Control Federation. 42, 2, 218.

Yao, K.M. (1979). Column Settling Test and Tube Settling. Journal of American Water Works Association 71, 2, 109-112.

CHAPTER 7

DOWNFLOW ROUGHING FILTRATION

H. Wolters, J.E.M. Smet and M. Pardón

1. INTRODUCTION

Downflow roughing filtration was introduced to overcome problems of rapid clogging of slow sand filters caused by large quantities of suspended solids. In this method filter water passes at low velocity through a coarse medium and a large proportion of the suspended solids is retained. Compared to horizontal-flow roughing filters, downflow roughing filtration provides moderate silt storage capacity and hence requires periodic hydraulic cleaning, and at longer intervals removal and cleaning of the filter medium.

In South-East Asia experiments and pilot plant studies have been undertaken using coconut husk fibre as filter material. When the filters got clogged, the husk fibres were discarded and replaced with new filter material. Much research has been done by the Asian Institute of Technology, Bangkok, Thailand (Frankel, 1974 and Thanh et al., 1976) in using alternative filter materials for both pre- and final filtration.

Experiments carried out in Peru and Colombia using gravel as a filter medium together with periodic hydraulic cleaning to overcome clogging are promosing and therefore this chapter is mainly confined to downflow roughing filtration using gravel as filter medium.

Only in Peru as a result of the pilot experiments a full-scale plant was constructed in a rural water supply system in San Vicente de Azpitia. At this stage the preliminary results obtained are not sufficient to form the basis of the design of a full-scale plant.

Downflow roughing filtration is not to be confused with conventional rapid sand filtration because of lower filtration rates, coarser filter material and different cleaning methods.

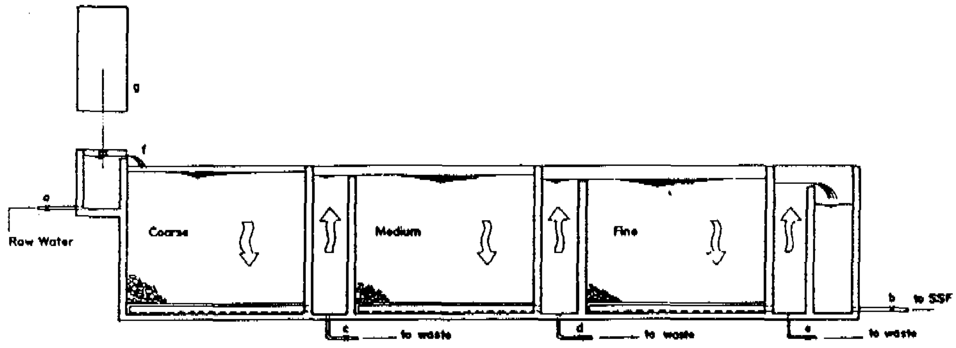
2. PRINCIPLES

2.1 Basic components

The basic components of a downflow roughing filter are:

- filter box divided in two or three compartments
- gravel bed
- underdrain system
- inlet and outlet structure
- set of filter regulation and control devices.

An inlet-controlled downflow roughing filter with three compartments is shown in Figure 1.



a. valve for raw water inlet and regulation of filtration rate
b. effluent outlet valve
c. valve for drainage of first gravel bed compartment
d. valve for drainage of second gravel bed compartment
e. valve for drainage of third gravel bed compartment
f. inlet weir
g. calibrated flow indicator

Figure 1. Basic components of an inlet controlled downflow roughing filter in series

Filter box

The total height of the filter box may range from 1.3 to 2.0 m, depending on whether extra water is needed for hydraulic. The filter box may be either circular or rectangular in shape and is usually constructed of a combination of masonry and reinforced concrete.

Gravel bed

Downflow roughing filters consist of three compartments, each containing gravel of a specific size, ranging from coarse in the first to fine for the last compartment.

Inlet and outlet structure

The inlet and outlet structure should ensure even distribution of raw water and abstraction of the filtered water. Flow regulation devices and water level control can be included in these structures.

Underdrain system

The underdrain system, which supports the gravel bed, provides an outlet for water passing through the filter and also permits filter cleaning by rapid drainage. The system may consist of drain pipes or a small trough covered with layers of coarse gravel. A valve or gate is connected to the underdrain system to facilitate drainage (Figure 2).

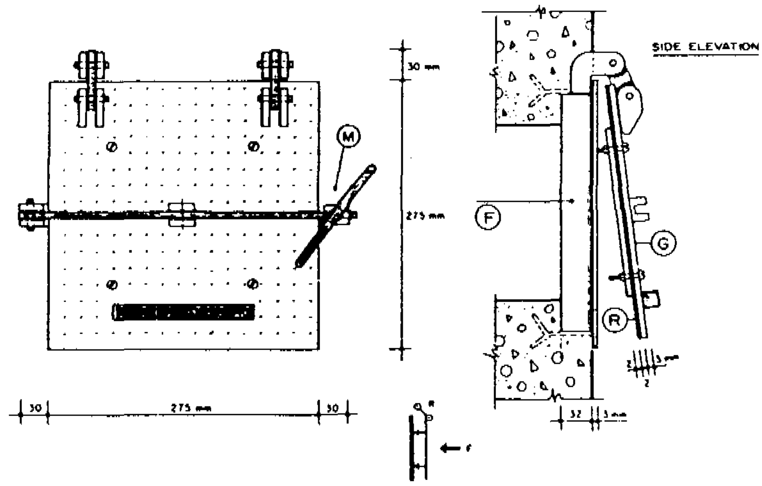


Figure 2. Drainage gate (Source: Lloyd et al., 1986)

2.2 Treatment process

Physical, biological and chemical processes all play a role in a downflow roughing filtration. The gravel bed offers a large surface area for sedimentation to take place and with filter runs of several weeks or months, biological processes have sufficient time to develop. Experience to date is limited and little is known about the mechanisms responsible for removal of suspended and colloidal material. The removal efficiency of inert suspended material is attributed to sedimentation, but clearly this is not the only mechanism. Experimental observations on the stable part of the suspended material show that removal efficiency depends little on the height of the filter bed (Babenkov, 1982). In field experiments not only turbidity but also coliform, true colour and iron content have been reduced considerably (Quiroga, 1988).

The efficiency of retention of destabilized suspended matter is determined by the ratio of effective diameter of the media and the height of filter bed d/D and by the hydrodynamic characteristics of the flow (Babenkov, 1982). Three features of hydrodynamic flow are distinguished. Firstly, formation and accumulation of destabilized sediments occurs predominantly on the underside of the granules, in zones of hydrodynamic shadow. Secondly, the sediment is arranged in chain units oriented along the current lines. Thirdly, the transport of filtered sediment to lower filter bed layers results from periodic breakthroughs of individual pore channels, accompanied by a rather prolonged (magnitude of seconds) movement of the filtered sediment along a certain route (Babenkov,1982).

3. RESEARCH

3.1 Laboratory experiments with coconut husk fibres as filter medium

Laboratory experiments have been done by the Asian Institute of Technology, Bangkok, with various filter media such as pea gravel, charcoal, coconut husks and rice husks. Shredded coconut husks were found to be most promising (Frankel, 1974a). A removal efficiency of 98% (turbidity influent 24-130 JTU, effluent turbidity 0.75-0.1 JTU) was obtained during a 440 h laboratory test with shredded coconut husk as filter material at a filtration rate of 0.25m/h (Sevilla, 1971; see Figure 3).

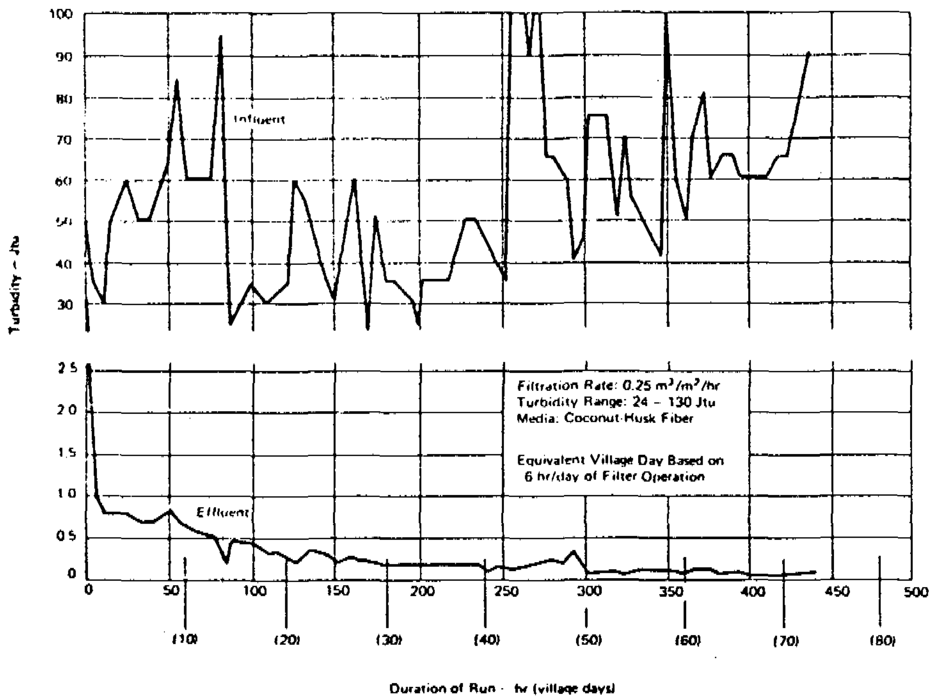


Figure 3. Filter performance of coconut-husk fibre at low rate
(Source: Frankel, 1974a)

Suspended matter penetrated further in the fibre than has been found with other filter media. The filter started to clog in the top layer, but as the filter run progressed the filtered suspended matter penetrated deeper into the bed. At higher filtration rates up to 2.5 m/h, turbidity removal efficiencies were 80-90% (Frankel, 1974a). This type of pre-filtration followed by final filtration using burnt rice husks as filter medium results in bacteria reduction of 60-90% (Frankel, 1981).

The filter medium was very difficult to clean. Disposal rather than cleaning would be more appropriate in areas where coconut fibre is very cheap (especially in coastal low land areas). Filter material would need to be replaced about once a month.

3.2 Pilot plant experiments with gravel as filter medium

A pilot plant was set up at the treatment works of Lima, Peru, using raw water from the Rímac river with influent turbidities under 50 NTU. For influent turbidities over 50 NTU, a mixture of raw river water and material from sedimentation tanks of the treatment plant was used. Both single and series downflow roughing filters were investigated using painted plexiglass pipes of 150 mm diameter. Filter bed height varied from 0.50 to 2.00 m and gravel diameters ranged from 12 to 50 mm. The filters were operated at filtration rates ranging from 0.10 to 0.80 m/h. Turbidity removal efficiency was recorded for various initial turbidity loads: < 50 NTU, 100 - 200 NTU, 250 - 350 NTU and 600 - 800 NTU.
Downflow roughing filter performance was evaluated on turbidity removal and resistance development using varying raw water turbidity, filtration velocity, height of filter bed, and gravel size (Perez et al., 1986). It was found that most particles larger than 5μm were retained. Turbidity removal efficiency increased to over 90% for influent turbidity of up to 200 NTU but decreased again for levels over 300 NTU (Figure 5). This is probably due to the fact that heavy loads of suspended material easily destabilize the unstable deposits within the prefilter.

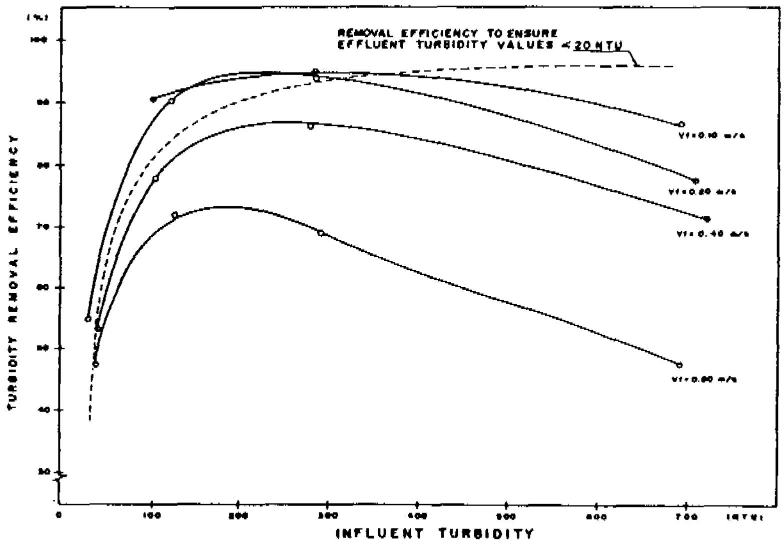


Figure 4. Relationship between influent turbidity and removal efficiency at various filtration rates for downflow filtration in series (Source: Perez et al., 1985)

The first 50 to 100 cm of the filter bed have a higher removal efficiency than the remainder of the bed. Deeper layers permit extended roughing filtration after the removal and storage capacity of the first centimetres have been exhausted.

Turbidity removal was higher in downflow roughing filter in series than in single filters. A sequence of three filters was found to be the most efficient and economical (Peres et al., 1985).

The working group 'Area de Abastecimiento y Remoción de Agua' of the Universidad del Valle, Cali, Colombia, carried out field tests at three sites to compare downflow and upflow roughing filtration (Quiroga, 1988). At Puerto Mallarino raw water from the Cauca river was used. PVC pipes of 150 mm diameter with gravel of 18, 12 and 6 mm respectively were used. Filtration rate was 0.70 m/h. Turbidity removal for various influent turbidities is given in Figure 5. Most turbidity levels were reduced to values under 20 NTU.

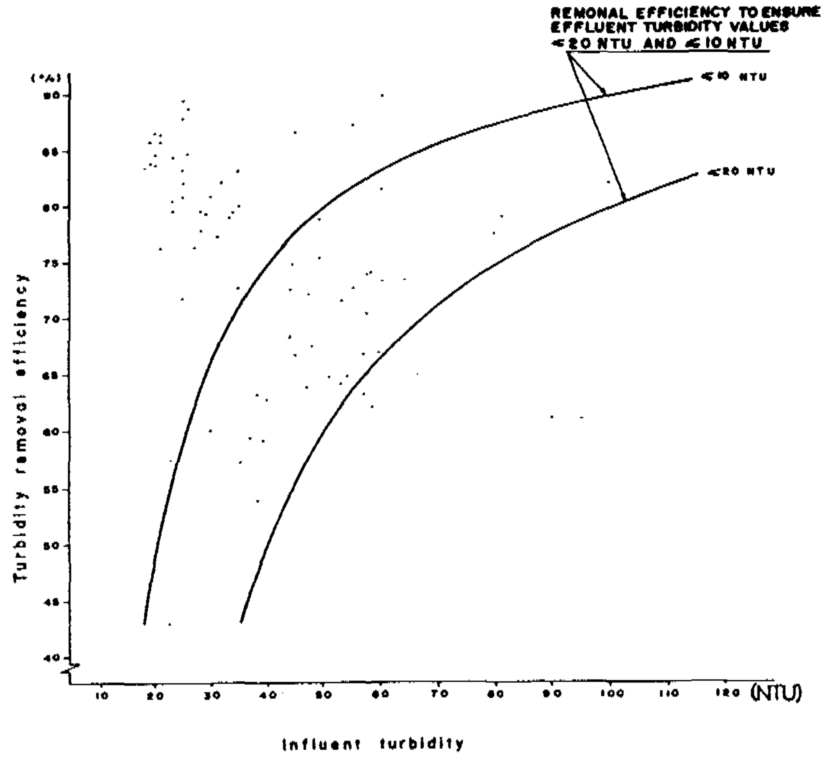


Figure 5. Relationship between influent turbidity levels and turbidity removal for downflow roughing filters in series, Puerto Mallarino, Cali, Colombia (Source: Quiroga, 1988)

The pilot filters also reduced apparent colour of the water considerably, by

45-80% (Figure 6). At least 85% of the effluents had an apparent colour of less than 50 mg Pt/l. Reduction of total and faecal coliforms ranged from 70 to 99.9%.

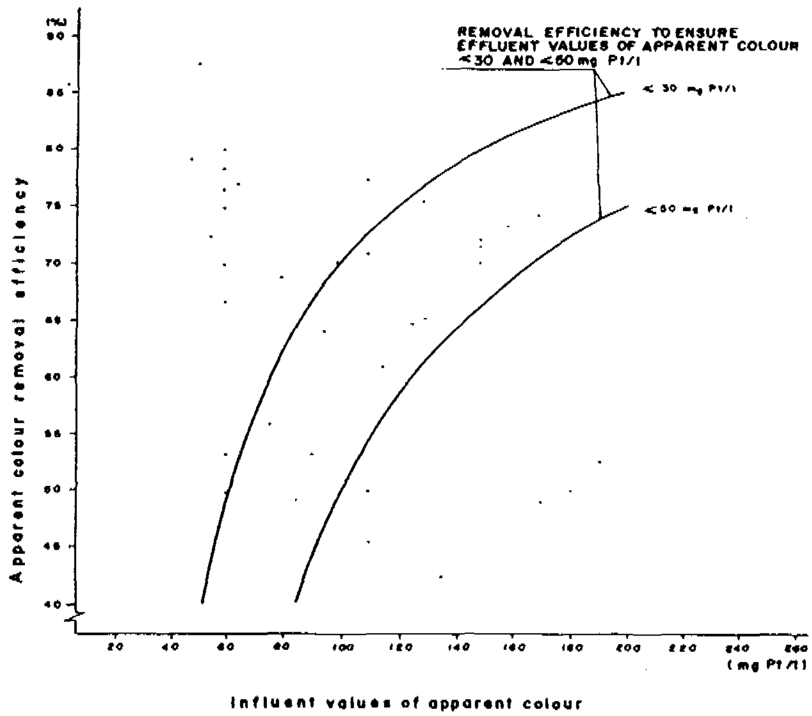


Figure 6. Relationship between influent apparent colour and apparant colour removal for downflow roughing filters in series, Puerto Mallarino, Cali, Colombia (Source: Quiroga, 1988)

4. FIELD EXPERIENCE

4.1 Coconut husk fibre filter

More than 100 two stage filtration units using shredded coconut fibers and burnt rice husks (Figure 7) have been installed in villages in South-East Asia (Frankel, 1981).

For one filter (Ban Som, Changwat Korat, Thailand) average filter run was three months at a filtration rate of 1.5 m/h. Only total efficiencies of the two-stage filtration were available. Effluent turbidity was generally less than 5 JTU and average removal was 90% (raw water turbidity range 5-140 JTU). The average removal of coliforms and faecal coliforms was 60 and 63% respectively. Absorption capacity of this type of filter for removing colour and tastes was significant (Frankel, 1981). The filters operate intermittently thus increasing the likelihood of break through of bacteria.

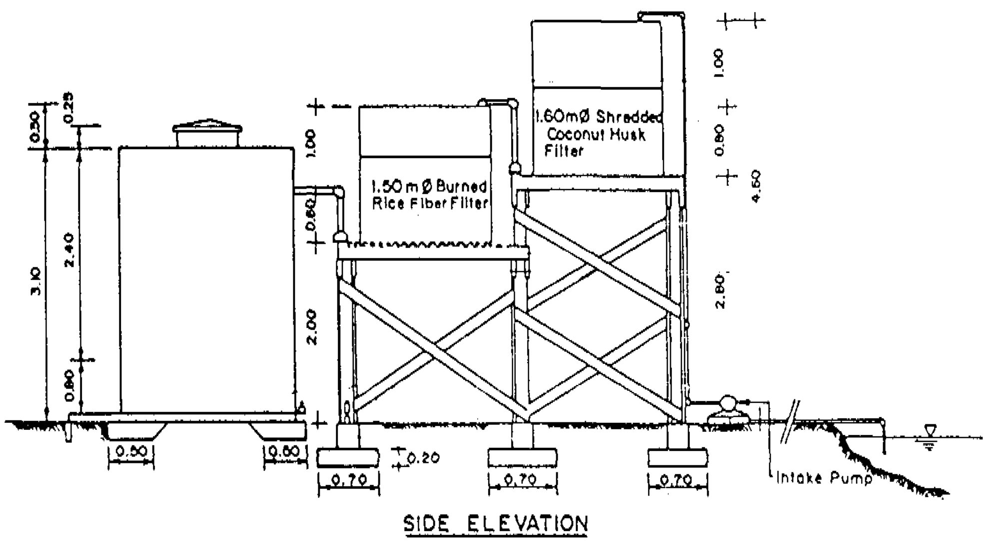


Figure 7. Schematic diagram of Two-Stage Filter (Adapted from ENSIC, 1983)

In Nong Khaem (Frankel, 1981) a filter was operated for 6-10 hours per day. Bacteria removal was 90% on the average but effluent values were erratic due to intermittent operation. Filter media needed replacement only after 4-5 months. A filter unit in San Francisco, Canaman, The Philippines, draws water from a shallow well with a high iron content. Levels of iron were reduced from 50 mg/l to less than 0.2 mg/l in the effluent (Frankel, 1981).

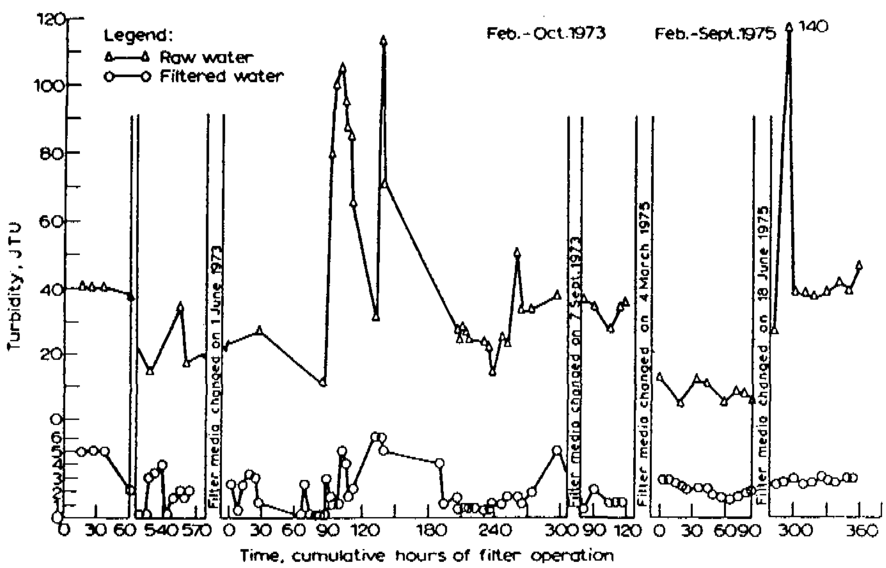


Figure 8. Turbidity removal efficiency of series filter, Ban Som, Changwat Korat, Thailand (Source: Frankel, 1981)

4.2 Gravel filters

In the rural village of San Vicente de Azpitia, Peru, a water treatment plant comprising downflow roughing filters in series followed by a slow sand filter has been operating since 1985.

The plant capacity is 35 m^3/d, and the filtration rate 0.30 m/h. The bed height for all filters is 0.6 m and the gravel sizes are 40-25, 25-12 and 12-6 mm. Evaluation showed promising results on both efficiency and effectiveness of the prefilters (Pardón, 1987). Turbidity removal efficiency depended on influent turbidity levels, with an overall removal efficiency of 78% (see Table 1).

Table 1. Turbidity removal of the downflow roughing filters of the San Vicente de Azpitia treatment plant, Peru

Influent turbidity (NTU)	Average removal efficiency (%)	Effluent turbidity (NTU)
20 - 100	63	7 - 37
100 - 300	79	21 - 63
> 300	92	> 24

Source: Pardón, 1987

The prefilters showed maximum efficiency of faecal coliform removal of 90% (average about 66%). In two wet seasons with average influent turbidities of 217 and 178 NTU (range 6-5 400 and 17- 1 000 NTU respectively), slow sand filter cleaning was not required more than once a month, while prefilter runs of 3 to 7 weeks were reported, but no head loss development was detected.

The plant was cleaned only when turbidity removal declined to consistently below 80% and effluent turbidity was consistently higher than 20 NTU (Lloyd et al., 1986).

5. DESIGN CRITERIA

Selection criteria for filter materials are filtering efficiency, service life, abundance, ease of preparation, storage ability, and low cost. Parameters such as granular size of filter media, depth of medium bed, filtration rate and backwash rate depend largely on raw water characteristics and the variation in the quality.

Raw water characteristics should be monitored over a period of preferably 12 months to include the seasonal variations. Data on suspended solid load, and proportion of stable and destabilized suspensions are needed for optimum filter design. The initial design of the pilot filter should be based on measurements of these parameters.

5.1 Coconut fibre roughing filters

A filter bed of coconut fibres is usually 60-80 cm deep. No backwashing arrangements are required as the filter material is discarded when the maximum headloss (usually 1.2 m) is reached. The head loss rate can be considered to be the main parameter determining the length of filter runs (Thanh et al., 1976). Filter runs vary from 3 to 4 months, depending on filtration rate and raw water characteristics. Most common filtration rate ranges between 1.25 and 1.5 m/h (Frankel, 1981).
The underdrain system can be the same as for other pretreatment systems. Attention must be given to uniform hydrodynamic flow conditions.

5.2 Gravel filters

Because of the limited research and field experience with downflow gravel roughing filtration only preliminary design criteria are presented in Table 2.

Table 2. Preliminary design criteria for downflow roughing filtration.

Period of operation	24 h/d
Filtration rate	0.3 - 1.2 m/h
Number of pre-filter units	minimal 2
Number of compartments	3
Height of filter bed	0.50 - 0.80 m
Gravel size	
first compartment	50 - 25 mm
second compartment	25 - 12 mm
third compartment	12 - 6 mm
Height of underdrains, including gravel layer (d_g = 50 - 60 mm)	0.15 - 0.30 m
Filtration velocity during cleaning	90 - 120 m/h 1)
Additional volume of water required for cleaning	1.1 - 1.5 m^3/m 1)

1) Based on laboratory experiments.

Based on: Perez, 1986 and Quiroga, 1988

6. CONSTRUCTION METHODS AND MATERIALS

Filter box

Downflow roughing filters may be either circular or rectangular in shape. Rectangular filters have the advantage of a common wall, thus reducing construction costs. Filters boxes can be constructed of reinforced concrete, mass concrete, masonry blocks, bricks, quarry stones, or ferrocement. A combination of masonry and ferrocement is also possible. Rendering the inside of the filter box is advisable.

Whether the filter box is located above or partly below groundlevel depends on hydraulic, structural and topographic considerations. It is advisable to locate the filter above the groundwater level. Firm foundation are essential to prevent cracks occurring in the construction at a later stage. The structure should be checked for watertightness before underdrains are constructed and the filter is commissioned.

Inlet and outlet structures

The inlet and outlet structures have to ensure even distribution of the raw water and abstraction of the treated water, flow regulation and water level control. For inlet-controlled operation, a V-notch weir should be built. The weir at the outlet may be rectangular since this should only fix the level of filtered water.

For gravel downflow roughing filtration consideration needs to be given to gravel bed and underdrain system.

Gravel bed

The gravel size and bed height must be selected, to optimize storage capacity, removal efficiency and ease of cleaning. Coarse gravel combines a large storage capacity with a relatively low removal efficiency and ease of cleaning. Fine gravel, on the other hand, provide a smaller storage capacity, higher removal efficiency and makes cleaning more difficult.

Underdrain system

Drainage facilities such as perforated pipes, drainage troughs or culverts permit hydraulic cleaning of the filter medium. The spacing between the drains should be approximately 1 - 2 m. The hydraulic capacity should permit drainage at 90 - 120 m/h for effective cleaning (Pardon, 1987). Valves, slide gates or flexible hose pipes can be used to operate the drainage system. Each drain should discharge into an open channel to allow visual inspection of the drainage operation. Facilities for safe washwater disposal are necessary to prevent erosion and water ponding.

7. OPERATION AND MAINTENANCE

7.1 Coconut husk fibre filters

Coconut fibers need preparatory treatment before use as filter medium. Soaking in water is required for about 3 days until the fibre no longer imparts more colour to the water (Frankel, 1981).
When the maximum head loss is reached filter material has to be discarded. The fibre can be removed manually and replaced with new material.

Since backwashing is not required high installation and running cost of these facilities are not incurred.

7.2 Gravel filters

Flow control

Downflow roughing filtration features an inlet-controlled flow and a fixed weir at the outlet. In gravity schemes, constant feeding is maintained by fixing the position of the valve or slide gate in the supply pipe or channel.

Filter cleaning

Head loss will increase as suspended solids removed from the raw water slowly accumulate in the filter. At the end of the filter run complete clogging or a constant breakthrough of impurities will occur. However, filter efficiency can be restored to some extent by periodical hydraulic cleaning. Most of the deposits can be flushed out by opening the underdrain system completely. Routine cleaning is required when turbidity removal declines consistently to less than 80% and effluent turbidity is higher than 20 NTU (Lloyd et al., 1986). Drainage of the gravel bed was not enough to restore filter efficiency of the plant of San Vicente de Azpitia. The removal efficiency of retained suspended solids ranged between 7-21%. This was due to the unfavourable hydraulic characteristics of the underdrain system. Providing pilot plant filters with additional water (1.1 to 1.5 m^3/m^2) and using high filtration velocities (90 to 120 m/h), increased the cleaning efficiency (60-70 %) considerably (Pardón, 1987). There is at present no experience with this cleaning method under field conditions.

After several years service, periodic hydraulic cleaning of the filter media may not be sufficient to re-establish filter efficiency. This may occur if large volume of silt and other suspended solids have penetrated deep into the gravel bed, causing either partial or complete blockage, or continuous breakthrough of retained suspended solids during filtration. The filter material must then be removed, washed and replaced.

8. COSTS

8.1 Capital costs

Coconut fibre roughing filters

Only total cost data are available for the series filtration system using shredded coconut husks as roughing filter and burnt rice husks as polishing filter material (Frankel, 1981). Capital costs including diesel operated pump, range from US$ 0.3 to 6.5 per capita, depending on location and durability of construction. In the Philippines the units had to withstand typhoons for example.

Gravel roughing filters

The construction cost of downflow roughing filter is mainly determined by the cost of cement, building sand, filter gravel, reinforced steel, pipes and valves. The costs of unskilled labour and land required are generally low. Construction of common walls in rectangular filters reduces construction costs considerably.

In the San Vicente de Azpitia case the construction of the prefilters cost US$ 30/m^3/d or US$ 1.4 per capita, of which 55% was for material and equipment and 45% for skilled and unskilled labour. The cost of the downflow roughing filtration units represents about 8.5% of the total construction cost of the water supply system (Pardón, 1987).

8.2 Running costs

General

The operation costs largely depend on labour and energy when pumping is required. While prefilters need more regular cleaning than slow sand filters, the hydraulic cleaning is less labour intensive. Additional labour cost will be incurred when the filter medium has to be removed and washed after several years of filter operation. When constructed of durable materials, maintenance costs are generally low for minor repairs to the filter and, if necessary, replacement of the few moving parts.

Coconut husk fibre filters

Operational costs (including fuel, pump repairs, medium replacement and operator's wage) ranged from US$ 0.15 to 0.50 to per capita per month (Frankel, 1981).
Costs are reduced if hand pumps are installed instead of diesel pumps, as has been the case in some demonstration projects in the Philippines.

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 Coconut husk fibre roughing filters

Two-stage filters (coconut fibre and burnt rice husks) are very simple to construct, operate and maintain. High filtration rates (up to 1.5 m/h) can be achieved, but the effluent quality is not as high as that attained by well operated slow sand filters, although significant removal of suspended solids and bacteria have been achieved. Continuous operation, which was not applied in the demonstration units, would definitely improve the filter efficiency. Due to biodegradability of the organic filter material, the filter effluent has a bad odour towards the end of the filter run, and the medium needs to be replaced. This type of filtration could be seen as a first stage in improving the rural water supplies prior to slow sand filtration. It is an affordable investment for the local communities.

9.2 Gravel roughing filters

Evaluation of both pilot plants and full-scale operation has demonstrated the effectiveness of downflow roughing filters as pretreatment prior to slow sand filtration.

Further investigation is needed of the effect of gravel size, gravel bed height and filtration rate as well as the removal mechanisms present in downflow roughing filters.

Given the moderate silt storage capacity of downflow roughing filters, an efficient hydraulic cleaning process is required to overcome the need for frequent removal and washing of the filter material, and thus make the technology a viable alternative. High filtration rates during cleaning and provisions for additional drainwater have produced promising results in laboratory experiments. However, this improved cleaning method still has to be demonstrate under field conditions.

Only very low filtration rates have been used under field conditions. In fact, they are only slightly higher than the ones used in slow sand filtration. These low filtration rates imply large filter bed areas and high construction costs. Studies should aim at the use of higher filtration rates (0.75 -1.50 m/h). Performance of the downflow roughing filter needs to be assessed under high turbidity loads over extended periods and extremely high turbidity levels for short periods using these higher filtration rates.

10. REFERENCES

Azevedo-Netto, J.M. (1977), Tecnica de Asbastecimento e Tratamiento de Aqua. 2nd Edition. CETESB, Sao Paulo, Brazil. 777-796.

Babenkov, E.D.,(1982) The Role of Structure Formation in the Process of Filtration Clarification of Water, Khimiya i Tekhnologiya Vody, 4, 1, 35-39.

Delagua (1986). The rehabilitation of the water treatment system of the rural community of Cocharcas, Huancayo/Junin. A pilot project report concerning horizontal roughing filtration. Lima, Peru, DITESA/Min Salud.

Frankel, R.J. (1974a), Series Filtration Using Local Filter Media, Journal of the American Water Works Association, 66, 2, 124-127).

Frankel, R.J. (1974b), Evaluation of low cost water filters in rural communities of the lower Mekong basin. Bangkok, Thailand, Asian Institute of Technology.

Frankel, R.J. (1981), Design, Construction and Operation of a New Filter Approach for Treatment of Surface Waters in Southeast Asia, Journal of Hydrology, 51, pp.319-328

Jaksirinont, N. (1972). Development of a series filtration water treatment methods for small communities of Asia. A thesis report No.509. Bangkok, Thailand, Asian Institute of Technology.

Kardile, J.N. (1981), Development of Simple and Economic Filtration Methods for Rural Water Supplies, Aqua, no.1, pp. 226-229

Lloyd, B., Pardón, M. and Wheeler, D. (1986). Final report on the development, evaluation and field trials of a small scale, multi-stage, modular filtration system for the treatment of rural water supplies. Rural water treatment package plant. Lima, Peru, Delagua.

Pardón, M. (1987). Consideraciones, desarollo y evaluación de un sistema de tratamiento que implementa la filtración gruesa de flujo vertical en grava. Lima, Peru, Delagua.

Perez, J., Pardón, M., Lavado, I. and de Mayo, C.V. (1985). Informe preliminar de la investigación sobre prefiltros de grava. Lima, Peru, GTZ-CEPIS/OPS.

Quiroga, E. (1988). Estudio preliminar sobre pretratamientos. Reporte final. Cali, Colombia, Univalle/CIR.

Sevilla, A.S. (1971). A study of filtration methods for providing inexpensive potable water to small communities in Asia. Thesis report No.442. Bangkok, Thailand, Asian Institute of Technology.

Thanh, N.C. and Pescod M.B. (1976). Application of slow sand filtration for surface water treatment in tropical developing countries. Final Report No.65. Bangkok, Thailand, Asian Institute of Technology.

Wegelin, M. (1986). Horizontal-flow roughing filtration: A design, construction and operation manual. Dübendorf, Switzerland, IRCWD.

Wheeler, D., Pardón, M., Lloyd, B. and Symonds, C. (1985). Aspects on prefiltration concerned with the application of small scale slow sand filtration in rural communities. Guilford, England, University of Surrey.

CHAPTER 8

UPFLOW ROUGHING FILTRATION

H. Wolters, J.E.M. Smet and G. Galvis

1. INTRODUCTION

Upflow roughing filtration can be used in the pre-treatment of surface waters prior to slow sand filtration to reduce the turbidity levels. In this pre-treatment method raw water flows at low velocity through a coarse medium in an upward direction. The filter bed is composed of material decreasing in size in the direction of the flow.

The Universidad del Valle, Cali, Colombia, carried out investigations on performance and behaviour of upflow roughing filters under field conditions. There are two main types of upflow roughing filtration, using gravel as filter medium: filtration in series and filtration in layers. In filtration in series different gravel size fractions are distributed over two or three compartments while in filtration in layers these fractions are placed on top of one other, in one compartment.

2. PRINCIPLES

2.1 Basic components

The basic components of an upflow roughing filter are:

- filter box containing one or several gravel bed compartments
- gravel bed
- underdrain system
- collecting drain system
- inlet and outlet construction
- set of filter control and regulation devices.

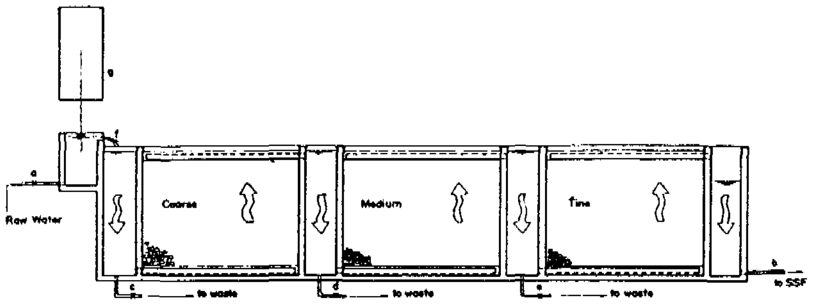
An inlet-controlled upflow roughing filter in series is shown in Figure 1 and a filter in layers in Figure 2.

Filter box

The height of the filter box is determined by the height of the gravel bed, height of the support layer of the underdrain system, supernatant water level, the extra volume of water required for adequate cleaning, and the freeboard. Most filter boxes are 1.5 to 2.0 m in height. The surrounding walls may be either vertical or partially inclined (angle of 60°). To ensure watertightness most filter boxes have been constructed of reinforced concrete, although ferrocement or masonry can also be used.

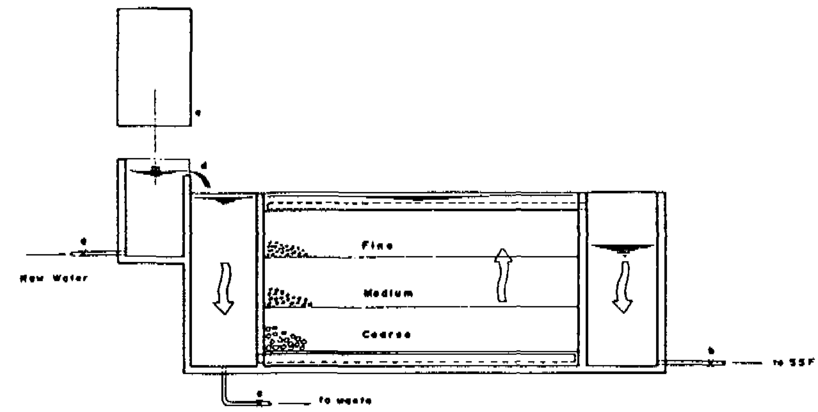
Gravel bed

The gravel bed consists of three gravel size fractions. In an upflow roughing filter in series each compartment contains a specific gravel size fraction, while in an upflow filter in layers these fractions are placed on top of one other in one compartment.
The gravel size fractions must be carefully selected to optimize storage capacity, removal efficiency and ease of cleaning.



a. valve for raw water inlet and regulation of filtration rate
b. effluent outlet valve
c. valve for drainage of first gravel bed compartment
d. valve for drainage of second gravel bed compartment
e. valve for drainage of third gravel bed compartment
f. inlet weir
g. calibrated flow indicator

Figure 1. Inlet-controlled upflow roughing filter in series



a. valve for raw water inlet and regulation of filtration rate
b. effluent outlet valve
c. valve for drainage of gavel bed
d. inlet weir
e. calibrated flow indicator

Figure 2. Inlet-controlled upflow roughing filter in layers

Inlet and outlet structures

The inlet-controlled upflow roughing filter has a V-notch weir for flow measurement (Figure 3). Water passes from the inlet chamber to the underdrain system. As filter bed resistance increases, the water level in the inlet chamber also rises. The outlet structure consists of a small chamber, which serves as a filtered water reservoir from where the water flows to the slow sand filters or to the next gravel bed compartment.

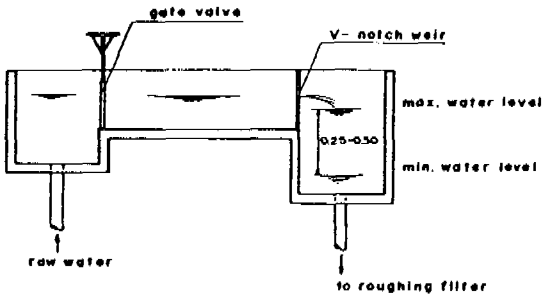


Figure 3. Inlet structure

Underdrain system

The underdrain system serves three purposes: to promote even distribution of the raw water over the gravel bed area, to support the gravel bed, and to assist even abstraction of water during filter cleaning when suspended solids deposits are washed out of the gravel bed by fast drainage. The underdrain system thus needs to be designed carefully as both flow direction and velocity will vary.

Collecting drain system

A main and lateral drain system may be used for uniform collection of treated water. The collecting drains also serve as a filtered water level control, as this water level compensates the entry losses the orifices and head loss along the main and lateral system. The main collector should discharge freely into the outlet chamber.
To facilitate manual cleaning of the filter bed surface, it is advisable to install a disconnectable drain system. This usually consists of a main and lateral drain system of perforated PVC pipes (diameter 80-150 mm).

2.2 Treatment process

Mechanical, physical,biological and chemical processes all play an role in upflow roughing filtration. Experience to date is limited and little is known about the mechanisms responsible for removal of suspended and colloidal material.

The removal efficiency is attributed largely to sedimentation but clearly this is not the only mechanism. Sedimentation of the larger suspended particles is expected to take place on the coarse gravel fractions and mainly in the lower part of the gravel bed, near the underdrain system. The deposit is well distributed over the depth of the coarse gravel filter when compared to rapid sand filtration. Small colloidal material will partly be retained by the coarse gravel fraction, while the remaining particles will be retained by subsequent, finer gravel fractions.

Theoretically, sedimentation may occur at two places (Figure 4). Direct sedimentation may take place on the granule surface perpendicular to the flow direction, not in contact with other granules and not exposed to scour. If the current lines do not completely follow the granule surface, (because of laminar/turbulent or complete turbulent flow conditions), a hydrodynamic shadow zone will be created at the upper side of the filter granule and sedimentation may occur.

The retention efficiency for non-inert suspended material, such as organic matter and algae, is determined by the ratio of effective diameter of the media and the height of the filter bed (d: D ratio) and the hydro-dynamic characteristics of flow (Babenkov, 1982). Non-inert suspended matter accumulates predominantly on the upperside of the granules, in the hydrodynamic shadow zone. The captured, non-inert material forms growing chains along the current lines. These organic chains may enhance capture (passive adsorption) of small suspended particles present in the passing water (Babenkov, 1982). Excessive adsorption of organic material has to be prevented because this is difficult to be removed by hydraulic cleaning.

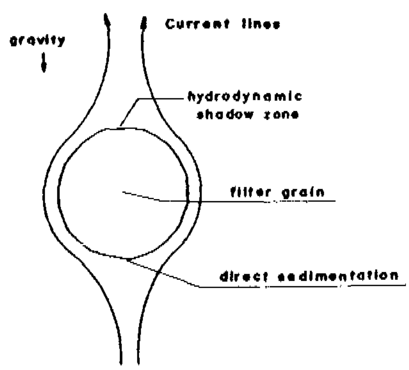


Figure 4. Sedimentation mechanisms in upflow filtration

Considerable reduction in (true and apparent) colour and total iron content has been reported (Quiroga, 1988). Through biochemical activity soluble ferrous compounds are transferred into unsoluble ferric oxide-hydrates, and degradable organic matter, such as humic acids, is gradually broken down into simpler organic material and finally converted into inorganic salts.

Biochemical and microbiological activity are responsible for the reported total and faecal coliform reduction (Quiroga, 1988).

Filtration efficiency is determined by filtration rate, filter material, and raw water characteristics. In general, suspended solids removal is higher at lower filtration rates, with smaller media size, deeper filter bed and larger size of suspended solids.

Continuous sedimentation and adsorption of suspended particles will gradually increase filter resistance and simultaneously decreases filter efficiency. Filter efficiency can be restored by cleaning the filter by rapid drainage.

3. RESEARCH

3.1 Pilot plants

In Colombia, upflow roughing filtration has been investigated at three sites by the working group 'Area de Abastecimiento y Remoción de Agua' of the Universidad del Valle, Cali (Quiroga, 1988). At Puerto Mallarino raw water from the Cauca river was used. Portable PVC pipe models (150 mm diameter) were installed as upflow roughing filter units and the compartments filled with gravel of 18, 12, and 6 mm respectively (Figure 5). The filters were operated at a filtration rate of 0.70 m/h.

The results were promising. Raw water turbidities under 100 NTU were reduced to less than 20 NTU. Removal efficiencies between 50 and 70% were recorded for influent values above 100 NTU. Apparent colour removal efficiency ranged from 50 to 80%. Total and faecal coliforms removal ranged from 70 to 99.9%.

Upflow roughing filtration in layers showed considerable lower removal efficiencies. This may be due to the shorter total retention time of the filters in the pilot plant and other factors. This pretreatment method might therefore be considered less suitable for heavily polluted surface waters.

The effectiveness of fine media upflow filtration in removing suspended solids in tropical regions is summarized in Table 1 (Gregory et al.,1983).

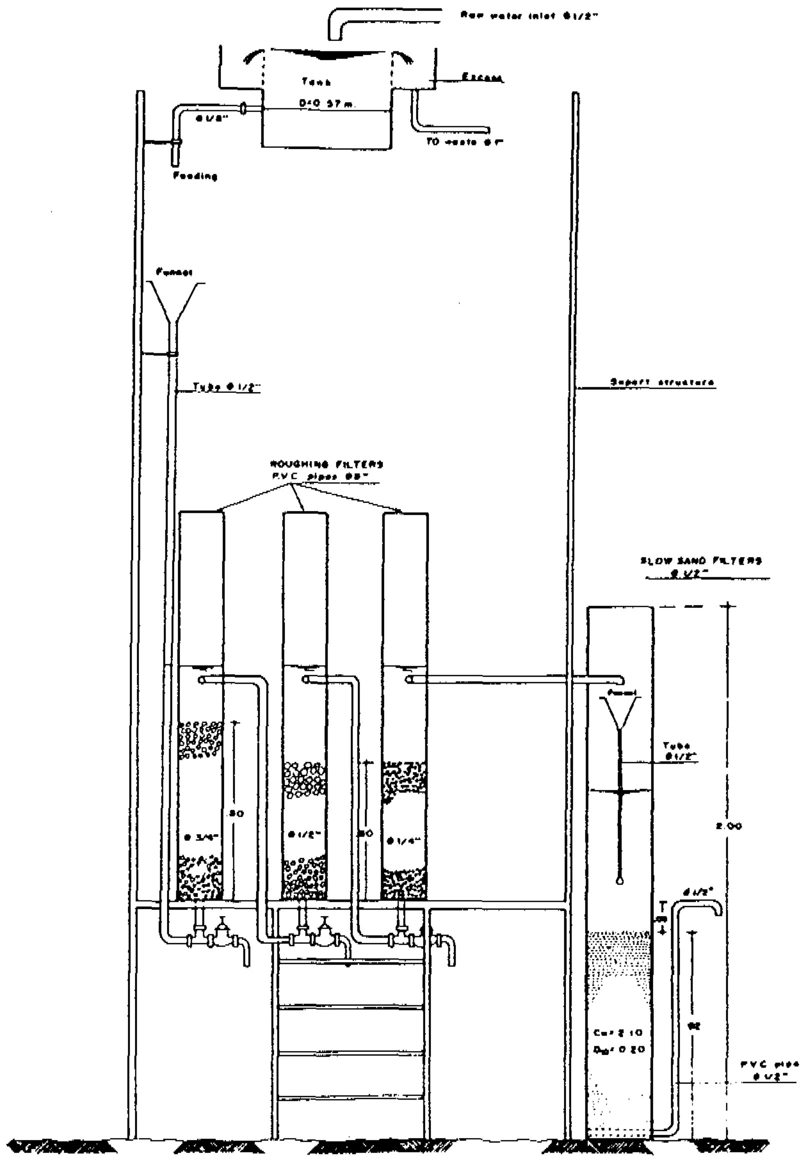


Figure 5. Pilot plant upflow roughing filtration
(Puerto Mallarino, Cali, Colombia)

Table 1: Effectiveness of fine media upflow filtration in removing suspended matter

Suspended matter in tropical waters	Effect of upflow filtration
a. Coarse particulate organic matter including organic debris, algae, large swimming organisms may all be present in tropical lakes and rivers in concentrations up to and exceeding 250mg	Very substantial removal with high filtration rates and a coarse filter media. Downwash is very easy.
b. Coarse inorganic soil solids are common in rivers and streams but not in lake waters. Concentrations can be relatively low at normal flow rates but can be very high following rains.	Very substantial removal, as (a), but generally advantageous to pretreat by sedimentation in order to extend filter run length.
c. Bacteria, parasite eggs and other organisms of faecal origin are common in rivers and lakes because of runoff. They may also be significant in wells and boreholes because of runoff and airborne pollution.	Up to 50% removal can be achieved without difficulty using lower filtration rates and a finer medium than for (a) and (b). This in conjunction with the removal of other organic matter will reduce the chlorination load.
d. Colloidal and semi-colloidal soil particles mostly derived from runoff can exceed concentrations of 2000 mg/l in rivers after rain. Some boreholes and springs in E.Africa have upto 160 mg/l without flocculation semi-colloidal SiO_2 present.	Colloidal matter is very difficult to remove by filtration alone. But, if coagulant is dosed immediately before the filter without flocculation tank, filtration can be in the same range as rapid gravity filtration. Backwashing needs careful attention.
e. Small motile organisms, commonly motile algae and certain dangerous parasites such as cercariae of bilharzia.	Motile organisms are difficult to retain in granular filters. Bilharzia control must include slow sand filtration and chlorination with adequate contact time or very long retention.
f. Precipitated iron compounds, which can be in surface and groundwaters in concentrations up to about 15 mg/l.	After cascade or trickling filter most of the precipitated iron oxides can be removed by filtration. However, precipitation is not successful in some cases.

Source: Gregory et al., 1983

3.2 Field Experiences

Operational problems with chemical flocculation and sedimentation led to the rehabilitation of the conventional plant El Retiro, near Cali, Colombia. This involved the installation of upflow roughing filters in layers together with slow sand filters. Use was made of the existing structures to incorporate a sedimentation channel into the scheme.

Effluent turbidity of the roughing filters was under 5 NTU in 90% of cases observed. Average removal efficiency was 52% for turbidity, 45% for true colour, 62% for total iron, and 82.4 and 89.4% for total and faecal coliform respectively. From these observations it may be concluded that in addition to mechanical and physical processes, chemical and biological action also take place in the roughing filters.

Removal of impurities gradually increased the resistance of the roughing filters at the El Retiro plant (Figure 6). Maximum filter resistance (0.30 m) was reached within one or two weeks. Filter resistance was restored to initial values of 5 - 7 cm by cleaning the filter bed by rapid drainage. By quickly opening the drainage valve on the underdrain system, accumulated material near the drain pipes, was washed out of the filter bed (Figure 7). Concentration of suspended solids in the washwater remained constant after the valve had been open for more than one minute. After rapid closing and re-opening the valve, more suspended solids peaks in the drained water were detected (Figure 8). Shock loading, destabilizes small amounts of settled material on the filter granules and facilitates their removal by the downwash. It can be concluded that shock loading is essential for adequate cleaning of the filter bed.

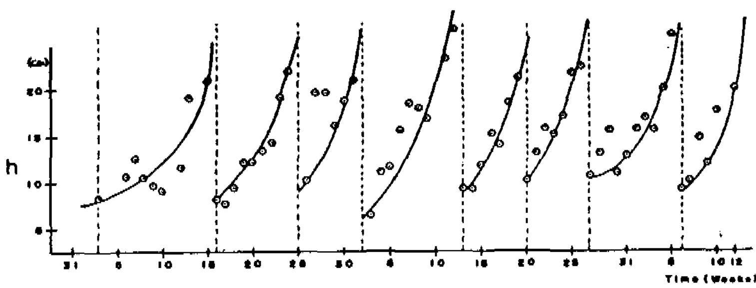


Figure 6. Development of resistance in the filter bed of a upflow roughing filter in layers (El Retiro, Cali, Colombia)

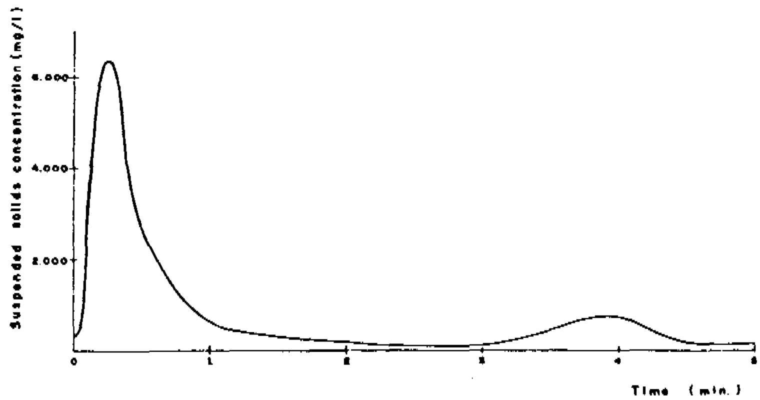


Figure 7. Suspended solids concentrations during filter cleaning
(El Retiro, Cali, Colombia)

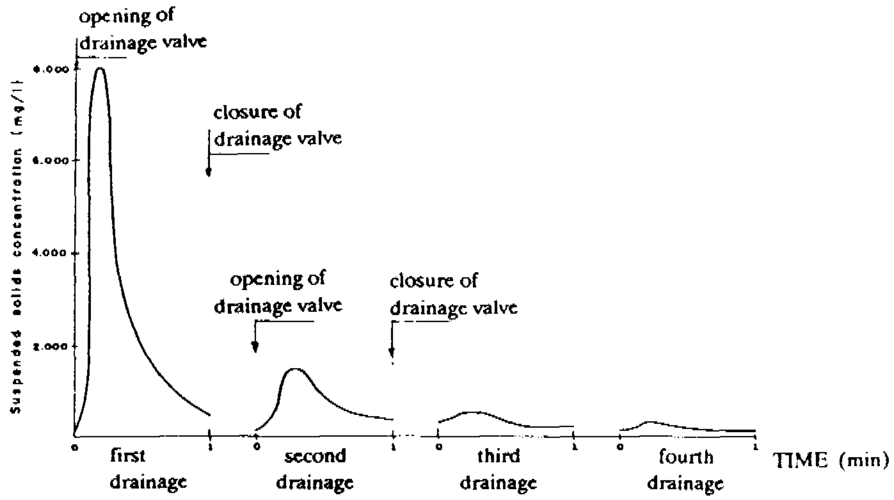


Figure 8. Suspended solids concentration during filter cleaning: effects of shock loading (El Retiro, Cali, Colombia)

4. DESIGN CRITERIA

Preliminary design criteria based on experience of the working group 'Area de Abastecimiento y Remoción de Agua' of the Universidad del Valle, Cali, Colombia, in the design and the performance of upflow roughing filtration, are presented in table 2.

Table 2: Preliminary design criteria for upflow roughing filtration

Period of operation	24 h/d
Filtration rate	0.5 - 1.0 m/h 1)
Number of filter units	minimal 2
Number of filter bed compartments	1 to 3 2)
Gravel size fractions	
coarse	24 - 18 mm
medium	18 - 12 mm
fine	12 - 6 mm
Height of gravel size fraction	0.30 - 0.80 m 3)
Height of underdrains, including gravel layer (30 - 24 mm)	0.10 - 0.20 m
Filter bed area	15 - 25 m² per compartment
Filtration velocity during cleaning	4 - 6 m/h 4)

1) Approach velocity to filter bed.
2) One filter bed compartment refers to upflow filters in layers.
3) Minimum value (0.30 m) refers to upflow filters in layers.
4) Approach velocity to filter bed. Based on test results.

The choice of either a filter in series or in layers should be based on the physical/chemical characteristics of the raw water and socio-economic factors. As the total filter bed height of a filter is series (1.8 - 2.4 m) is two to three times higher than that of a filter in layers (0.60 - 0.80 m), removal efficiencies will be higher. Hence, upflow roughing filters in series are more suitable for heavily polluted raw waters. But more land is required, and construction costs as well as costs of pipes and valves are higher.

5. CONSTRUCTION METHODS AND MATERIALS

Filter box

Upflow roughing filters are usually rectangular in shape with either vertical or sloping walls. Rectangular filters offer the possibility of common wall structures, reducing construction costs. They can be constructed of reinforced concrete or a combination of reinforced concrete, mass concrete and masonry, or ferrocement. The filter may be located either above or partly under ground level, depending on hydraulic and structural conditions.

Preferably the filter should be placed above groundwater level.

Prior to construction of the underdrain system and commissioning the filter, the structure should be checked for watertightness and physical integrity.

Gravel bed

The gravel size fractions must be carefully selected, to obtain an optimum balance between storage capacity, removal efficiency and ease of cleaning. It should be done on the basis of results from a pilot plant obtained over a period of at least one year to include seasonal variations in the raw water quality. On the one hand, coarse gravel sizes combine a large storage capacity with ease of cleaning but have a relatively low removal efficiency. Filters with small gravel sizes feature higher removal efficiencies, smaller storage capacity, accelerated filter resistance development and greater cleaning difficulty. Further, gravel of small diameters seems to contribute most to the biochemical removal of total iron.

The gravel size fractions can be separated by sieving. To remove all impurities from the granule surface, the aggregate should be washed thoroughly before being placed in the filter box. Attention should be paid to relocation of the various gravel size fractions in a filter in layers.

If possible, size diameters and bed height of the gravel should be determined in pilot plant experiments. Physical and chemical characteristics of the raw water together with the findings of these pilot plant experiments should form the basis for selection of either a one or multi-stage upflow roughing filter. Socio-economic factors, such as land required, construction costs, must also be taken into account. The height of each gravel layer will vary between 0.30 m for roughing filters in layers, and 0.80 m for filters in series.

Underdrain system

The underdrain system should be carefully designed and constructed because it is subject to varying hydraulic conditions as both flow direction and flow velocity will vary greatly. During filter operation the underdrain pipes should distribute the raw water evenly over the filter bed area while during filter cleaning they should permit rapid drainage of the filter bed. There is a quick acting valve (Figure 9) connected to the underdrain system for filter cleaning. To increase pressure head the valve should be located below the floor of the filter bed (0.80 - 1.20 m below). Closing the valve quickly, will generate pressure waves in the underdrain system and the filter bed. The inlet pipe should be closed to prevent loss of these pressure waves (Figure 10).

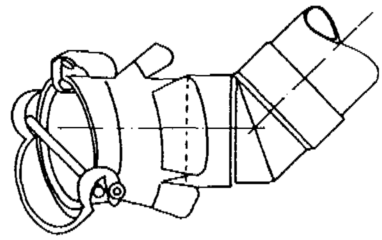


Figure 9. Rapid opening drainage valves

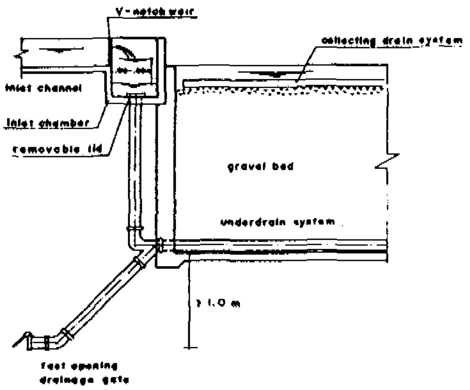


Figure 10. Drainage system for roughing filtration

The underdrain system consists of a main and lateral drain system of perforated PVC pipes (diameter 80 to 150 mm) and covered with a small layer of coarse gravel. Generally a layer of gravel of granule size 20 - 30 mm and 0.10 - 0.20 m in height prevents blockage of the underdrains.
To avoid accumulation of fine particles in the drain pipes and subsequently blockage of the pipe during filtration, the flow velocity in the drain should be between 0.1 and 0.5 m/s. Higher velocities cause friction losses in the pipe, leading to uneven distribution of the water over the length of the pipe. Interspacing between the lateral pipes should be limited to 1.00 - 1.50 m.
The orifices in the underdrain pipes need to be 8-10 mm in diameter and are distributed in pairs along the lateral pipes. To ensure a maximum variation of ten percent of the distribution of the water along the laterals, the total cross sectional area of orifices per lateral should be 0.44 times the cross sectional area of the lateral (Castilla et al., 1985).

The orifices form an angle of 45° with the horizontal diameter of the drain pipe (Figure 11).

Interspacing between the orifices should be between 0.40 - 0.50 m. To limit head loss in the orifices, the flow velocity should be less than 1.0 m/s, but preferably under 0.5 m/s.

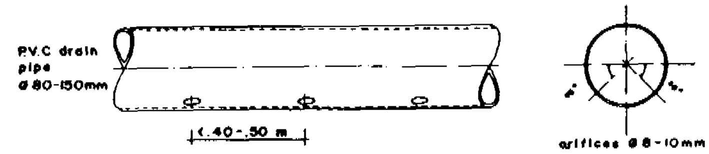


Figure 11. Orifices in laterals of distributing drain system

Collecting drain system

A main and lateral drain system is used for uniform abstraction and collection of treated water. This system is placed on top of the gravel bed or just below the bed surface. PVC pipes of 80, 100 or 150 mm diameter are commonly used.

Flow velocity in the drain pipes should be between 0.1 and 0.5 m/s. Interspacing between the lateral pipes should be limited to 1.00 - 1.50 m. The orifices of the collecting drain pipes should be 6-8 mm in diameter (Figure 12). To ensure a maximum variation of ten per cent of the abstraction of the water along the laterals, the ratio between the total cross sectional area of the orifices and the area of the collecting lateral should be equal to 0.144 (Castilla et al., 1985).

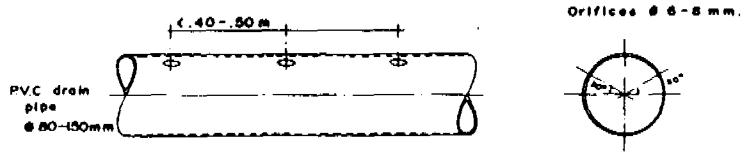


Figure 12. Orifices in laterals of collecting drain system

To facilitate removal of the filter material the collecting drain should preferably be disconnectable.

6. OPERATION AND MAINTENANCE

Flow control

Intermittent operation disturbes the biochemical and bacteriological activity and influences filter efficiency negatively.

Hydraulically an upflow roughing filter is operated by manual rate control at the inlet, while the filtered water level is kept constant by the collecting drain system.

Filter cleaning

As large quantities of suspended solids accumulate in the gravel bed with time, filter efficiency will gradually decrease. Filter efficiency can be restored by removing the accumulated material by fast drainage. Besides reducing filter efficiency, filter resistance will increase gradually during operation. Initial filter resistance will be low (5 - 7 cm). Maximum head loss of the roughing filter may be set at 0.30 m (Figure 10) resulting in filter run lengths of one or two weeks, depending on raw water characteristics. Head losses of the filter may partly or entirely be recovered through rapid hydraulic drainage.

Before starting to drain the filter, the inlet valve should be closed and also the valve in the inlet pipe. The presence of water reservoirs with a free water level in direct contact with the underdrain system will initilty the effects of shock loading (Figure 13).

Fast drainage is carried out by quickly opening the drainage valve on the underdrains. This prompt opening will create high velocities in the gravel bed and lead to high dragging forces on the accumulated suspended solids, the unstable heaps of retained material and the biological film on the filter media. Since most of the accumulated material settles in the lower part of the gravel bed, near the underdrain system, high cleaning efficiencies can be expected.

Drainage may be enhanced by shock loading during filter drainage. Prompt closure of the outlet valve generates pressure waves in the gravel bed and induces 'whirl-effects' on the settled matter, thus destabilizing the small heaps of settled material on the filter granules.
As probably not all the accumulated material will be washed out of the gravel bed by this method, the filter material may need to be removed, washed and replaced after a number of years of operation.

Backwashing in the flow direction as practised in fine media upflow filters to fluidize the bed, is not necessary for coarse media upflow filters and difficult to accomplish because of the heavy coarse granules. On the other hand, problems commonly experienced in upflow filtration with finer media, that is the burst of solids due to sudden or local fluidization, do not occur.

The main characteristics of the cleaning procedure are the introduction of high velocities in the filter bed and the generation of pressure waves (shock loading). The cleaning efficiency is influenced by:

- the arrangement of the underdrain system (distance of the drainage valves with respect to the floor of the filter box, interspacings and cross sectional area of laterals and orifices);
- the characteristics of the filter material (gravel diameter, porosity, pore size distribution);
- the characteristics and amount of wash water (viscosity, cohesion to filter material).

Experience to date with the cleaning procedure is limited and still little is known about the factors effecting it.

Filter operation

Routine operation and maintenance can be done by a trained local caretaker. Additional labour is required only when the entire gravel bed has to be cleaned. Involvement of the local community can reduce these incidental labour costs. Maintenance activities are in general limited to relatively minor repairs to the filter, and to simple, regular maintenance such as greasing the moving parts.

7. COSTS

7.1 Capital costs

Construction cost of an upflow roughing filter is mainly determined by the cost of cement, building sand, gravel granules, reinforcement steel, piping and valves. Land and labour costs are generally lower. More land is required for upflow filters in series. Because several compartment have to be built, construction costs will be relatively high for filters in series. However, these costs could be reduced by using rectangular filter bed areas and common wall structures. Upflow roughing filters in series need more pipes and valves than filters in layers.

Construction costs in Colombia ranged from US$ 20 to 40 /m^3/d, representing 25 to 45 % of the total construction costs of a treatment plant, consisting of roughing filters and slow sand filters.

7.2 Operating costs

The operating costs of an upflow roughing filter depend largely on the local costs of labour and energy (if pumping is needed). Conversion of the conventional treatment plant El Retiro, Cali, Colombia (using chemical coagulation, flocculation and sedimentation) into a treatment plant comprising upflow roughing filtration in layers together with slow sand filtration, reduced running costs by a factor of five (Galvis; et al.,1987).

Running costs are low because chemicals do not need to be applied and operation and maintenance activities are limited. When constructed of durable materials, maintenance costs will in general be low for minor repairs to the filter and replacement of the few moving parts. Regular maintenance, such as greasing of the moving parts, will reduce wear.

8. CONCLUSIONS AND RECOMMENDATIONS

Upflow roughing filtration has been demonstrated to be viable under field conditions. The pretreatment method showed promising results in turbidity reduction, in total iron, true and apparent colour removal, and in the reduction of total and faecal coliforms. Furthermore, it is very effective in protecting slow sand filters against high turbidity loads. At the El Retiro plant in Cali, Colombia, effluent turbidities of the roughing filters were in 90% of the cases below 5 NTU.

Upflow roughing filtration must be considered as an integral part of a treatment plant, since a combination of mechanical, physical, chemical and biological processes take place in the filter.

Upflow roughing filtration offers great design flexibility. The gravel layers can be divided over one, two or three compartments. However, guidelines have to be developed for the selection of the type of upflow roughing filtration, taking factors into account such as raw water characteristics, required effluent quality, land requirement, construction costs and ease of operation.

Hydraulic cleaning of the filters still needs further investigation. Since the bulk of the suspended solids settles in the lower parts of the gravel bed near the underdrain system, high cleaning efficiencies can be expected when fast drainage is used. Experiments carried out under field conditions demonstrated the importance of shock loading.

9. REFERENCES

Azevedo-Netto, J.M. (1977), Tecnica de Asbastecimento e Tratamento de Aqua. 2nd edition, Sao Paulo, Brazil, CETESB.

Babenkov, E.D. (1982), The Role of Structure Formation in the Process of Filtration Clarification of Water, Khimiya i Tekhnologiya Vody, 4, 1. 35-39.

Castilla, A. and Galvis, G. (1985). Diseño de multiples. Curso de abasto de agua para poblaciones. Cali, Colombia, Univalle.

Documentos preliminares. Informes. Convenio Univalle-CIR. No.35861. Cali, Colombia, Univalle/CIR

Frankel, R.J. (1974). Evaluation of low cost water filters in rural communities of the lower Mekong basin. Bangkok, Thailand, Asian Institute of Technology.

Galvis, G. and Visscher, J.T. (1987). Participation y perspectivas de la Universidad del Valle en el desarrollo del Proyecto de Filtración Lenta en Arena. In: Filtración Lenta en Arena y Pretramiento. Cali, Colombia, UNIVALLE/IRC/Ministro de Salud.

Gregory, R., Mann, H.T. and Zabel, T.F. (1983). The Roles of Upflow Filtration and Hydraulic Flocculation in Low Cost Water Technology, Water Supply Vol. 1., pp. 97-112.

Kardile, J.N. (1987), Simple Methods in Water Purification. Nasik, India.

Quiroga, E. (1988) Estudio preliminar sobre pretratamientos. Reporte final. Area de Abastecimiento y Remoción de Aguas, Universidad del Valle. Cali, Colombia, Univalle/CIR.

Restrepo, I., Visscher, J.T. and Galvis, G. (Editors) (1987). Segundo Taller Nacional para la Promoción de Filtración Lenta en Arena. Cali, Colombia, ACODAL/UNIVALLE/IRC.

Thanh, N.C. and Pescod M.B. (1976). Application of slow sand filtration for surface water treatment in tropical developing countries. Final Report No.65. Bangkok, Thailand, Asian Institute of Technology.

HORIZONTAL-FLOW ROUGHING FILTRATION

by M. Wegelin and T.S.A. Mbwette

To this contribution recent experimental results and new developments in horizontal roughing filtration as given in IRCWD's "Filter News" No. 2 have been appended.

1. INTRODUCTION

Horizontal-flow Roughing Filtration (HRF) as an Alternative Pretreatment

Since gravel and sand layers of aquifers significantly improve the water quality of infiltrated surface water, why ignore such an excellent process just because nature has not provided the specific hydrogeological conditions at the site? An artificial aquifer might act in the same way and produce a hygienically safe drinking water.

Horizontal-flow Roughing Filtration copies nature. The main characteristic of this process is its horizontal flow direction along which the filter material is graded in decreasing size. This specific flow direction enables to construct a shallow and structurally simple filter of unrestricted length. Three or four subsequent fractions ranging from coarse to fine material effect a gradual removal of the solids from the water. The coarse filter material, contained in the first part of the filter, retains all the larger particles and some of the finer matter, while the last filter compartment with the finest material has to cope with the remaining particles of colloidal size. Thus, the effluent of a HRF is virtually free from any solids.

Since this filter technique makes use of natural purification processes, no chemicals are required to assist the treatment process. The installation of such a filter requires only local resources such as construction material and manpower. Furthermore, no mechanical parts are necessary to operate or clean the filter.

Historial Background

Last century already, coarse media filters were used for raw water pretreatment prior to Slow Sand Filter (SSF) in England and France. For the past 25 years, gravel prefilters have been used in combination with sand beds for artificial groundwater recharge in Germany, Switzerland and Austria (Ref. 1). More recently, investigations on coarse media prefilters were carried out at the Asian Institute of Technology in Bangkok/Thailand (Ref. 2,3) and at the

University of Dar es Salaam (Ref. 4,5) Tanzania, to examine the treatment efficiency of these filters with highly turbid water. In 1982, the International Reference Centre for Waste Disposal (IRCWD), attached to the Swiss Federal Institute for Water Resources and Water Pollution Control (EAWAG), in Duebendorf, Switzerland, started extensive laboratory investigations on HRF (Ref. 6). The investigations were followed up by a demonstration project involving construction and monitoring of village schemes and sponsored by the Swiss Development Cooperation.

2. DESCRIPTION OF THE TREATMENT PROCESS

Main Features and Lay-out of HRF

HRF basically comprises three main parts as illustrated in Fig. 1: the inlet structure, the filter bed and the outlet structure. The inlet structure is composed of a weir which leads the raw water into the inlet chamber. The outlet structure consists of a chamber located upstream of the outlet weir. In and outlet structures are flow control installations required to maintain a certain water level and flow along the filter, as well as to establish an even flow distribution across the filter. The filter bed is composed of three or four sections which are filled with filter media of different sizes arranged from coarse to fine in the direction of the water flow. The coarsest material should be around 15-25 mm and the finest gravel not smaller than 4 mm. Each fraction is usually separated by perforated well segments or an open-jointed masonry wall to avoid mixing during cleaning. The filter bed is usually also provided with an underdrainage system to enable hydraulic sludge extraction to be carried out after a certain running period.

The raw water falls over a weir into an inlet chamber where coarse solids settle and floating material is retained by a separation wall. The water passes through the perforated separation wall and flows in horizontal direction through a sequence of coarse, medium and fine filter material. The pretreated water is collected at the filter end by an outlet chamber and then discharged over a weir.

The bulk of solid matter in the raw water will be retained by the first coarse filter pack. The last filter compartment should act as polisher and remove the last traces of solid matter in the raw water. Each gravel pack becomes gradually loaded with retained solid matter until its respective filter efficiency is exhausted.

In HRF, sedimentation is the main process responsible for the separation of the solid matter from the water as observed in laboratory tests conducted at EAWAG (Ref. 6). The filter acts as a multi-store sedimentation basin, thus providing a large surface area for the accumulation of settleable solids. The solids accumulate on top of the collectors and grow into dome-shaped aggre-

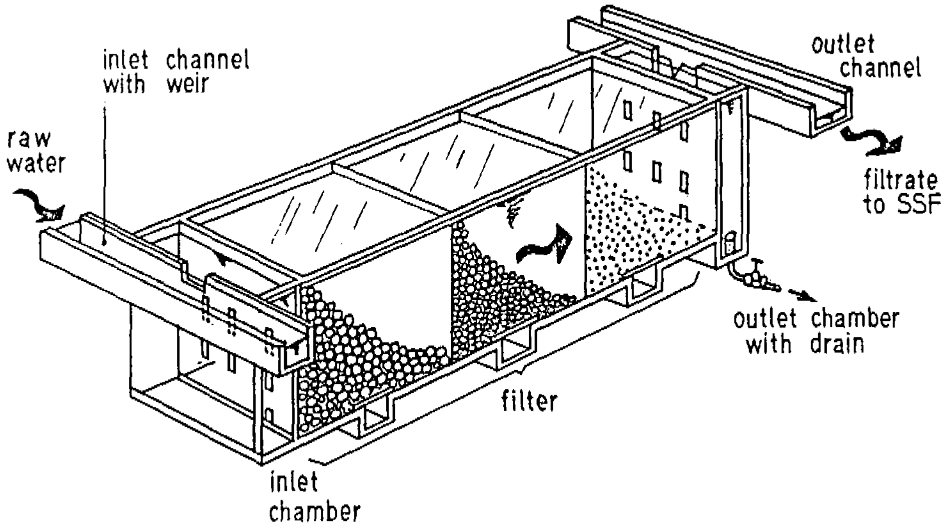


Fig. 1 Main Features of a Horizontal-flow Roughing Filter

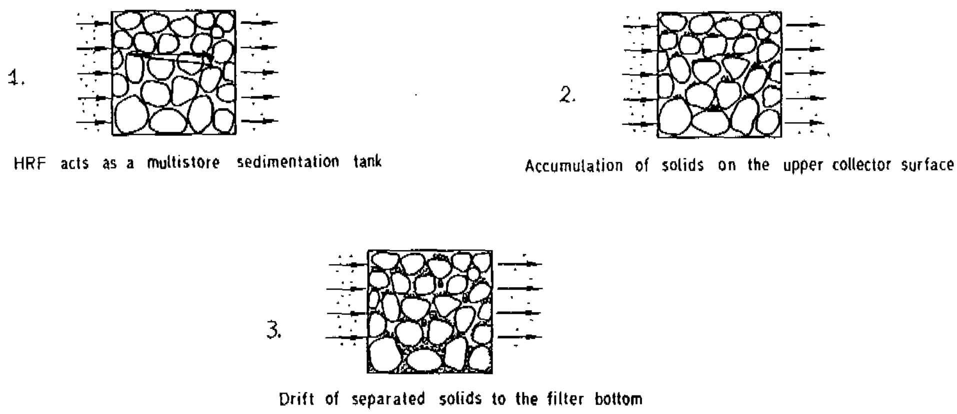


Fig. 2 Mechanisms of Horizontal-flow Roughing Filtration

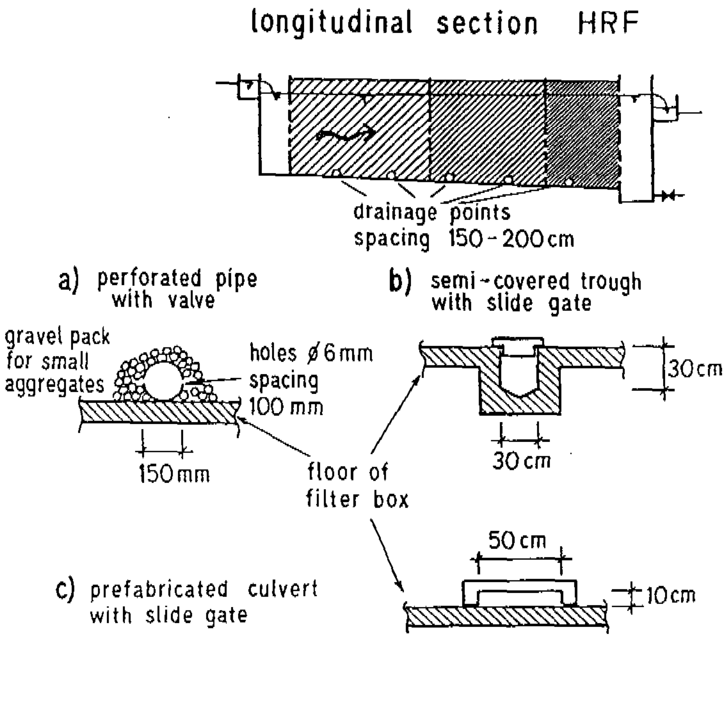


Fig. 3 Drainage Systems

gates with advanced filtration time. Part of the small heaps drift towards the filter bottom once the heap reaches instability. The drift regenerates efficiency of the upper gravel layers and enables the accumulation of a considerable amount of retained material. Fig. 2 schematizes the different mechanisms of sedimentation taking place in a HRF. Depending on the organic characteristics of the raw water, other processes such as biological oxidation or adsorption of solid matter at the slimy filter surface might also occur.

Filter efficiency can be restored to a certain extent by filter drainage installations illustrated in Fig. 3. After a filtering period of some years, depending on the raw water quality and filter lay-out, the accumulated solid matter has to be removed. Filter cleaning is carried out manually by excavating the filter material, washing the media and refilling the filter box. This operation can easily be carried out with village participation or by casual labourers, thereby requiring no mechanical equipment.

3. EXPERIENCE WITH THE TREATMENT PROCESS

3.1 Laboratory scale experiments

Experience in Thailand

In 1977 AIT ran at its premises laboratory and pilot scale filtration tests with HRF and SSF (Ref 2). Surface water ranging from 20 to 120 NTU in turbidity was used from a canal near the laboratory. A preliminary test was carried out with a prefilter of 1.5 m length comprising gravel packs of 0.30 m length and filter material of an effective size varying from 9.1 to 2.8 mm. The prefilter was operated at a filtration rate of 0.6 m/h and produced a filtrate of about 15 NTU. The subsequent SSF run at 0.15 m/h clogged after 44 days. In another test, a prefilter of 5 m length containing 7 filter packs and effective filter sizes ranging from 15.7 to 3.4 mm was installed. Operated at 0.6 m/h, turbidity of the prefiltered water varied from 10 to 20 NTU. The following SSF run at 0.15 m/h revealed a headloss of 57 cm after 55 days of continuous operation.

Experience in Tanzania

At the University of Dar es Salaam, laboratory tests with a water mixture of tap water and water-borne sludge originating from domestic tanks of a 60 NTU turbidity revealed that sedimentation alone could not meet the standards required by SSF (Ref. 4). Experiments with vertical filter columns of 1 m depth filled with gravel varying from 1 to 64 mm in size and filtration rates from 0.5 to 8 m/h, confirmed the sensitivity of the filter performance with respect to the flow conditions in these filters. Remarkable turbidity reduction was experienced with flow velocities of less than 2 m/h. Due to their small silt storage capacity and consequently short filter runs, frequent cleaning is necessary. A shift of the flow direction from vertical to horizontal can eliminate these disadvantages. Therefore, an open channel of 15 m length was filled with 3 gravel fractions of 16-32, 8-16 and 4-8 mm in size. Filtration rate varied from 0.5 to 8 m/h, whereas turbidity of the water mixture was maintained at 60 NTU. Under the given conditions, the tested HRF produced an effluent quality of less than 10 NTU at filtration rates of 0.5 and 1 m/h.

Laboratory investigations in Switzerland

At IRCWD, the filtration tests were carried out with a model suspension of kaolin (Ref. 6). Kaolin is a clay mineral commonly found in tropical weathered soils. The clay particles used for the filtration tests were smaller than 20 µm and of an average size of about 1.75 µm to simulate a suspension of extremely fine particles. Presettled river waters usually exhibit similar fractions of particulate matter.

Transparent tubes and filter channels allowed observation of the mechanisms governing HRF. It became obvious that the predominant process in HRF is sedimentation. The horizontal movement of the suspension through the pore system of the filter is combined with a gravitational downward drift of the particles. The solids settle on top of the media grains and form loose agglomerates of several mm height. The shape of these small heaps is controlled by their slope stability. Once the deposits exceed this stability, small lumps of settled matter will move downwards within the coarse filter media. However, their movement will be hindered if filter material of less than 4 mm is used. This process presents great advantages since the retention capacity in the upper part of the filter is restored to a certain degree. At the same time, the filter bed will be gradually filled from bottom to top with retained matter. This mechanism endows the HRF with great silt storage capacities and consequently long filter runs. However, it must be noted that the predominance of sedimentation is confined to the settleable particles. Other capture mechanisms like those due to hydrodynamic retardation, diffusion and vander Waal's forces enhance the removal of colloidals.
Some results of the parameter tests carried out in Switzerland are presented in the following. The relationship of the total filter efficiency η_{tot} (%/cm), defined as the percentage of removed particles per cm of filter length with the filter load σ(g/l), defined as dry weight of accumulated solids (in g) per unit of filter volume (in l) for different filter materials is presented in Fig. 4. Glass spheres, quartz, pumice and charcoal possess very different surface characteristics. However, under the tested conditions, all four filter media behave similarly with respect to filter efficiency and filter load. This observation is of great value since most of the locally available gravel either found in a river bed or obtained from a quarry can be

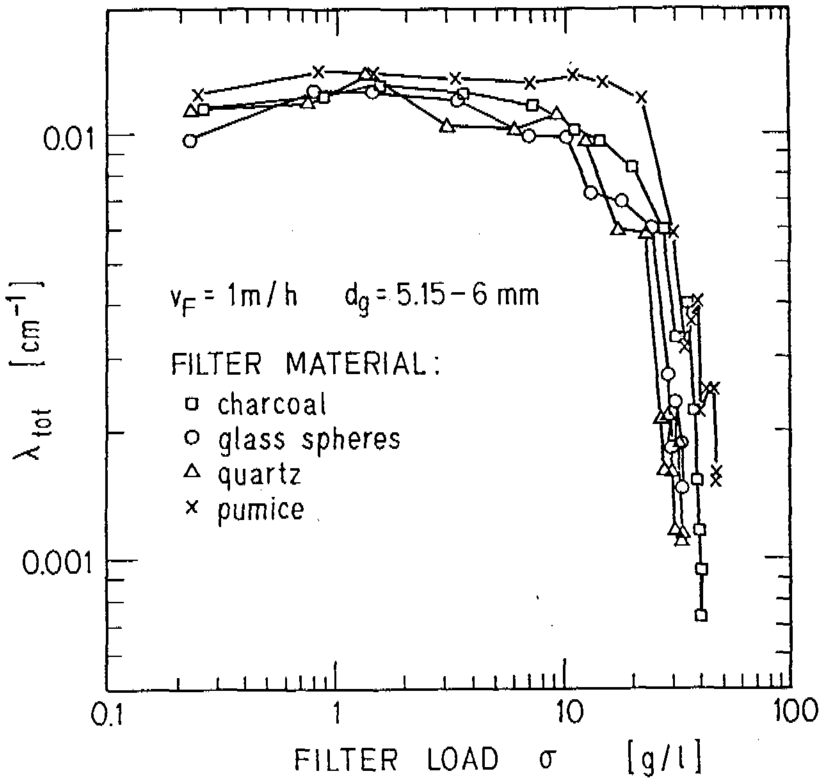


Fig. 4 Filter Efficiency in Reltationship to Filter Load for Different Filter Material

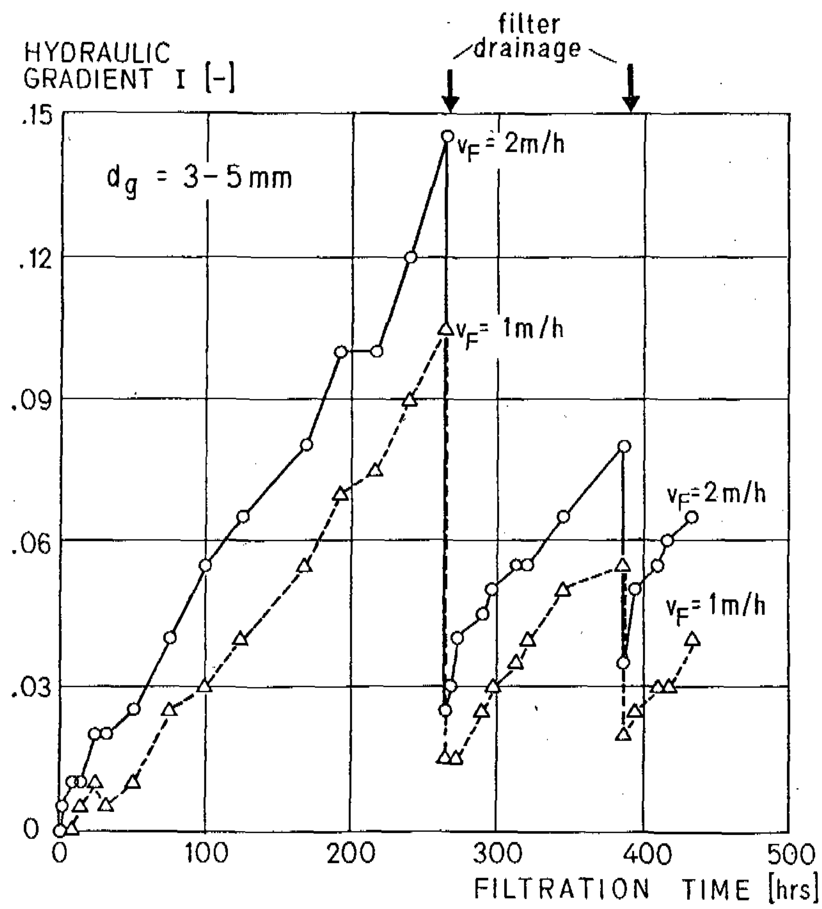


Fig. 5 Filter Resistance Development and Filter Regeneration by Drainage

used. Gravel might even be replaced by any other inert material like broken burnt bricks. The graphs in Figs. 5 and 6 illustrate another phenomenon observed during filtration tests, namely regeneration of the filter efficiency by drainage. The downward drift of the deposits can be enhanced by filter drainage. If the water table in the filter is lowered, the loose agglomerates accumulated on top of the filter grains will collapse and get flushed to the filter bottom. Besides, filter resistance which is the second decisive parameter in filter design is advantageously reduced by such a drainage procedure. Fig. 5 plots the headloss development as hydraulic gradient (pressure drop per filter length) for two different filters. After 270 hours of operation, the filters were drained and filtration reassumed. Headloss dropped significantly after this drainage and reincreased with progressive filtration time. A second drainage of the filter had a similar effect. Fig. 6 shows how filter efficiency η_i was restored almost to its initial value after the two drainages. This observation is also of great practical value. Occasional HRF drainages will improve filter performance and will even flush a substantial amount accumulated matter out of the filter and thus prolong total filter run.

A semi-empirical filtration model of HRF was developed by IRCWD on the basis of the results obtained from these laboratory tests. The developed filtration model can be used to simulate HRF filter runs. With the help of model calculations, the number of filtration tests can be limited and the lay-out economically optimised. A scientific paper describing the filtration model in detail has been published in AQUA (Ref. 5).

3.2 Pilot Plant Experiments

Field tests at three different sites in Tanzania were carried out to verify the laboratory results. At Handeni and Wanging'ombe, portable PVC pipe models of 250 mm diameter were installed as HRF and SSF units, whereas in Iringa, a large-scale pilot plant was constructed with a HRF (cross-section 1.6 x 1.0 - m, filter length 10 m) and a SSF (4 m^2 filter area) in addition to different pipe models. Fig. 7 summarises the filter resistance of different SSF fed either with untreated river water or with water pretreated by HRF. From the graphs it is obvious that SSF cannot be applied without pretreatment of the river waters examined. The respective filter runs last from a few days up to 2-3 weeks but reasonable operation times for SSF are in the range of 2-3 months of more. With properly cleaned SSF filter beds and adequately pretreated raw water, the filter resistance of different SSF at 0.2 m/h filtration rate was less than 20 cm after 1 months of continuous operation. Hence, running times of 2-3 months could easily be achieved. Most filtration tests were run as short-term experiments to investigate the influence of filtration rate and gravel size on filter efficiency. However, a HRF pipe model (Ø 250 mm; gravel sizes 32-16, 16-8, 8-4, 4-2 mm; gravel packs of 40 cm length each) was used for several long-term filtration tests to determine the silt storage capacity. Filter loads of 35 to 50 g/l were recorded and thus confirm the outstanding silt storage capacity of HRF (Ref. 4).

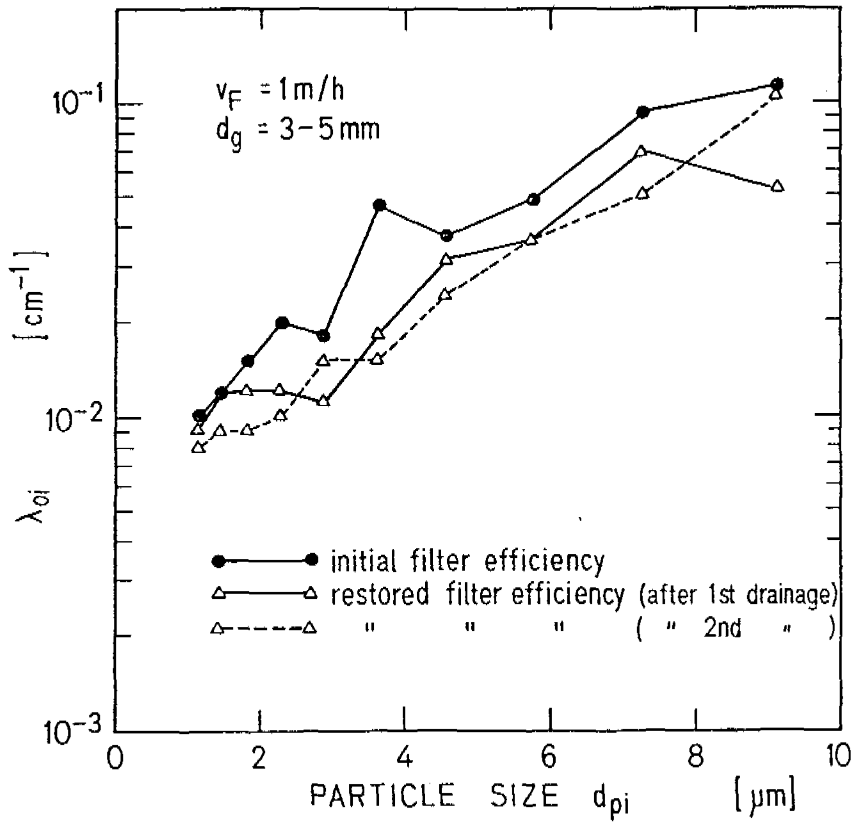


Fig. 6 Initial and Restored Filter Efficiency by Drainage
in Relationship to the Suspended Solids Particle Size

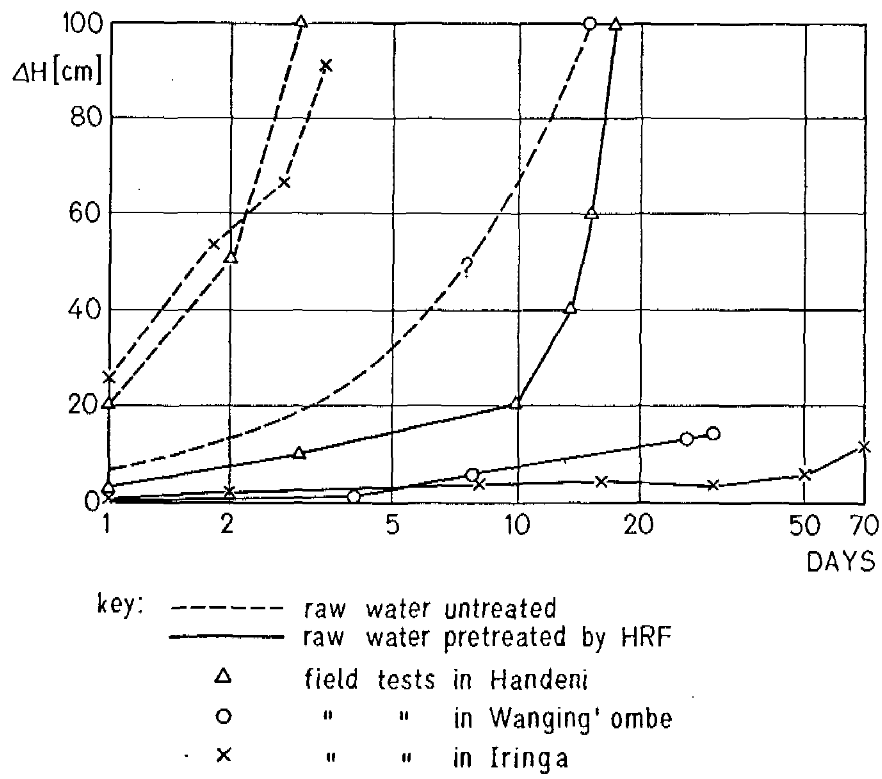


Fig. 7 Filter Resistance Development for
Different Slow Sand Filters in Tanzania

Filtration tests with a semi-technical pilot plant in Switzerland were carried out to verify the elaborated mathematical model. Four different gravel fractions were filled in three channels, each 4 m long and of a 0.2 x - 0.5 m cross-section. A parallel test was run at 0.5, 1 and 2 m/h filtration rates, whereby the qualitative and hydraulic gradients were carefully recorded. A fairly good correlation was observed between model calculation and actual performance of the filter.

3.3 Full-scale operation

Three demonstration schemes using HRF were constructed in Thailand between 1977-80. The full-scale schemes were set up in the villages of Jedee-Thong, Ban Bangloa and Ban Thadindam. Surface waters of turbidities up to 275 NTU are pretreated with HRF of approx. 5 m filter length and comprising 6 gravel packs with effective grain sizes ranging from 15 to 2.6 mm. The prefilters operate at filtration rates between 0.5 and 1 m/h, whereas the SSFs run at a rate of 0.15 m/h. The available records report good HRF performance, thus enabling SSF filter runs of several months. Apart from turbidity, several other water quality parameters were recorded. The good bacteriological quality of the water produced by the filters is mentioned explicitly. The projects were supported by IRC The Hague and IDRC (International Development Research Centre, Ottawa, Canada).

With the financial support of the Swiss Development Cooperation, the practical phase of the HRF project was initiated by IRCWD in 1984. The HRF technology is expected to be introduced by the construction and monitoring demonstration schemes in different developing countries. The salient figures of 3 schemes of this programme are presented in Table 1.

Kasote was set up by the local authorities with the assistance of NORAD. The high specific construction costs are partly caused by the transportation costs required for carrying the construction material to the remote village located near the shore of Lake Tanganyika. Fig. 8 illustrates the construction phase of the water treatment plant which also includes 2 SSF units. The first practical experience is promising. The filter resistance in the SSF was recorded at 60 cm after the first rainy period, and a filter running time of 4 months. None of the filters had to be cleaned until then.

Cocharcas rehabilitated its water treatment plant with the assistance of Del Agua and CARE. The work was carried out with extensive community participation, and therefore the specific construction costs for the HRF could be kept small. More details on the lay-out and the structure of the 2 filter units are given in Fig. 9.

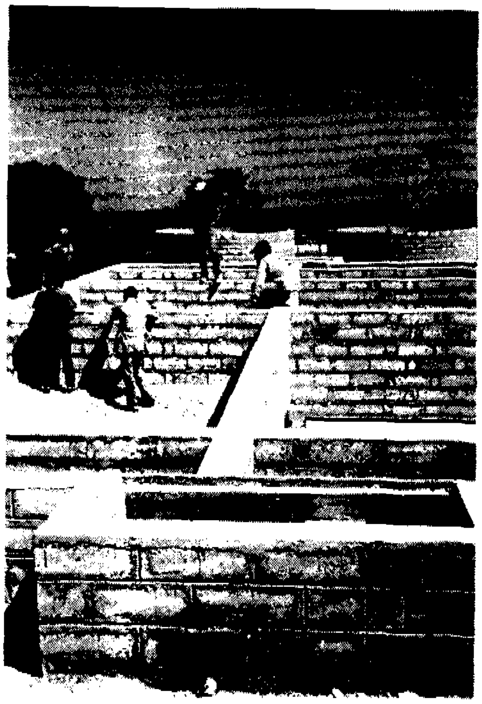
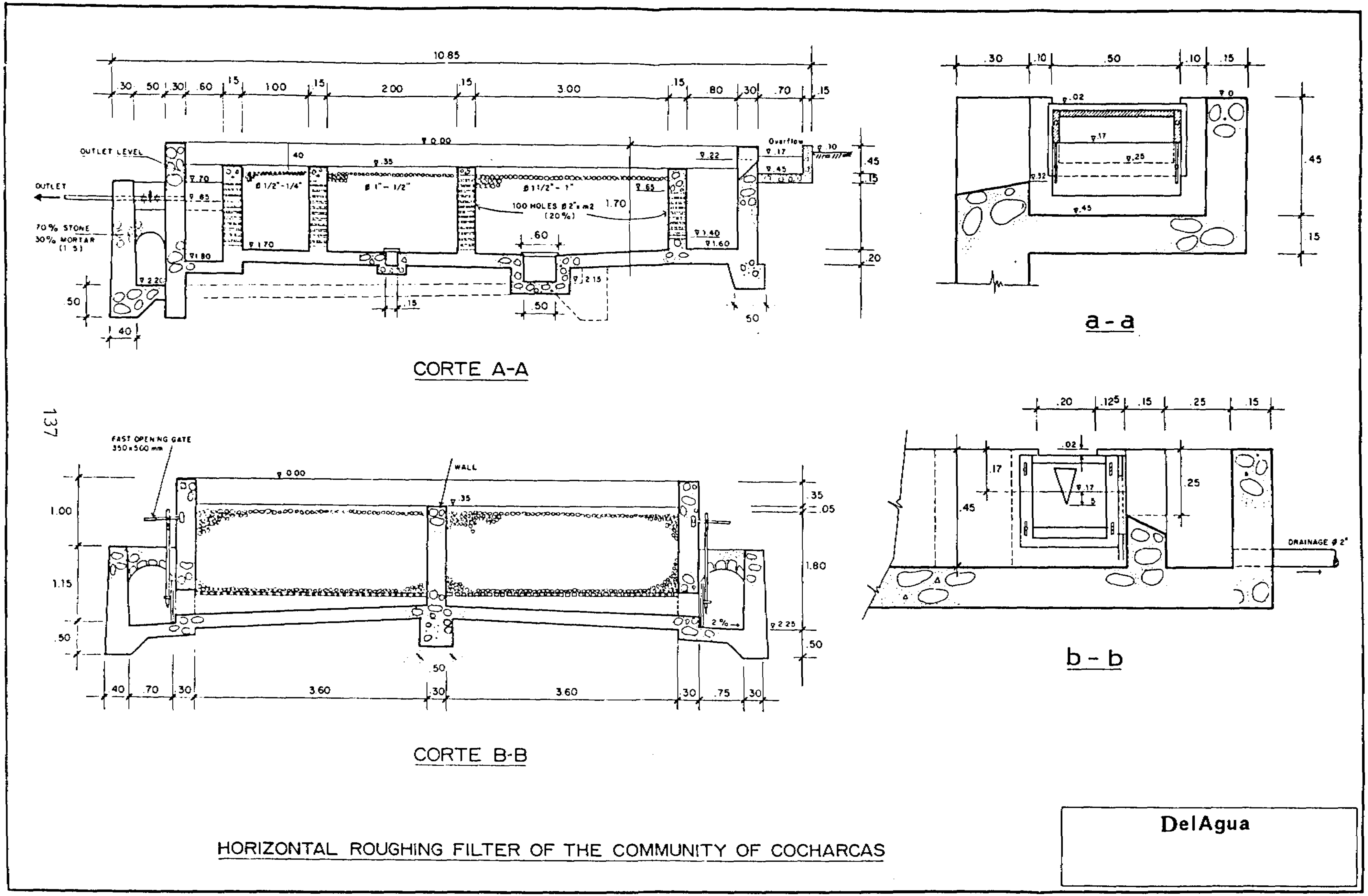


Fig. 8 HRF under construction at Kasote, Tanzania

Fig. 10 Excavation of the HRF earth basin
(picture by Swiss Disaster Relief Unit)

Fig. 9 Horizontal-flow Roughing Filters constructed at Cocharcas, Peru



FAU 5 is a refugee camp set up by the Swiss Disaster Relief Unit. The water treatment plant comprises 2 sedimentation tanks and 4 HRF units. The treated water is desinfected before distribution. Earth basins with inclined walls and earth dams constructed with bags filled with the excavated soil were installed. Therefore, the filter boxes were coated with a plastic lining and filled with gravel. Fig. 10 illustrates the construction of a HRF earth basin. The first practical experience with the treatment plant revealed that the raw water turbidity of 2000 NTU could be reduced to 1000 NTU by the sedimentation tanks, and to 5-20 NTU by the HRF.

Table 1:

name/country of the schemes		Kasote, Tanzania	Cocharcas, Peru	FAU 5, Sudan
design capacity	(m^3/d)	196	103	240
present population		3'000	650	20'000*
raw water source		river	irrigation canal	irrigation canal
max. turbidity	(NTU)	200	500	2'000
HRF:				
- number of units		2	2	4
- filtration rate	(m/h)	1	0.6	0.75
- filter length	(m)			
• coarse gravel		6	3	4
• medium gravel		4	2	4
• fine gravel		2	1	2
- filter width	(m)	3.7	3.6	2.0
- water depth	(m)	1.1	1.0	1.2
construction period		May '84 - Nov '85	Dec '85 - April'86	Aug '85 - Sept'85
specific construction costs HRF	($/$m^3$/d)	130	41	130

*) planned number of refugees

4. DESIGN CRITERIA

The design of a HRF is dependent on its ability to reduce the suspended solids concentration in the raw water to levels which are acceptable for SSF operation (usually less than 5 ppm) throughout the year. The characteristics of the raw water determine filter lay-out and its operation, whereas the required capacity only determines the cross-section area of the filter bed.

The following four design criteria have to be considered for HRF design:

1) the required effluent quality for a specific raw water quality in terms of separated suspended solids concentration ΔC in mg/l
2) the required daily output Q in terms of m^3/d
3) the required filter run period T_r in terms of weeks
4) the maximum allowable filter resistance ΔH in terms of cm.

The following four design variables determine the HRF lay-out:

a) the filtration rate v_F in m/h, which is the hydraulic load in m^3/h on the filter's cross-section area in m^2
b) the individual sizes $d_{g,i}$ of filter material in mm
c) the individual lengths $l_{f,i}$ of each filter material in m
d) the cross-section area A of the filter in m^2.

Design Aspects

HRF has to be dimensioned for extreme situations, i.e. for maximum suspended solids concentration in the raw water. As the filter efficiency decreases with increasing filter load σ (g/l), peak loads in the raw water should preferably be treated with a recently cleaned filter. The annual operational plan should therefore consider seasonal quality fluctuations of the raw water.

In order to guarantee an economic lay-out of HRF, moderately higher effluent concentrations of suspended solids might be permitted during extreme situations. Furthermore, the filtration velocity v_F can normally be increased when the mean suspended solids concentrations of the river water is moderate or low. Higher filtration rates permit smaller filter cross-sections A although the filter length l_f may, as a consequence, have to be increased. The three design variables v_F, A and l_f are interrelated. An economic optimization of the filter bed volume by a variation of these three variables is possible within narrow limits only. The detention time of the water in the filter is one major operational filter characteristic since sedimentation remains the main process in HRF. However, a reduction of the filter run period T_r will not only reduce the required filter length l_f but also the filter bed volume. Thus, economic filter optimization can more readily be achieved by a variation of T_r and l_f.

With respect to the grain size d_g of the filter medium, one would primarily tend to use finer material as coarser filter aggregates have a lower efficiency. However, besides efficiency in the separation of suspended solids, other criteria such as final headloss ΔH, filter run period T_r and filter cleaning aspects have to be considered. With the use of only one fine filter material, it might be possible to pretreat the raw water sufficiently but at the expense of high head losses, short filter runs and difficulties in filter cleaning. Such problems arise with filter material of less than 4 mm in size. A graded filter bed with differently sized fractions overcomes the aforementioned difficulties.

Design Guidelines

The suspended solids concentration in the raw water and the particle size distribution of these solids mainly determine acceptable ranges of filtration rates over known filter media fractions used for the pretreatment of a SSF influent. It can therefore be assumed that the suspended solids levels in the effluent will be less or equal to 5 ppm. In practice, however, the determination of suspended solids concentration is time consuming. Hence, the best approach is to produce a reasonable correlation between turbidity and suspended solids for the raw water to be treated. Turbidity measurements can be used to depict levels of suspended solids concentration from the graph. It is worthwhile mentioning that the validity of the correlation between turbidity and suspended solids concentration is limited since, in some cases, it is subjected to great seasonal fluctuations, especially if periodic algal growths or colour increase are pronounced.

Table 2: Tentative Design Guidelines

Approximate suspended solids concentration in raw water	C_o (mg/l)	High: > 150	Low: < 100-150
Filtration rate	V_F (m/h)	0.5-0.75	0.75-1.5
Individual filter lengths for dg_i:			
25-15 mm		3 - 5	3 - 4
15-10 mm	$Lf,_i$	2 - 4	2 - 3
10-5 mm	(m)	1 - 3	1 - 2
suspended solids concentration in the effluent	C_e (mg/l)	Less than 5	

Height and Width

Neither the height H nor the width W of a HRF are dependent on the raw water characteristics but are influenced by structural and operational criteria. The following aspects were taken into account for the recommendation of the respective dimensions:

- although the efficiency of a HRF can partly be restored by intermittent drainage, the filter media has to be taken out and cleaned manually to remove the sticky sludge which will have accumulated in the lower part of the filter after longer operational periods. Therefore, a convenient side walls height will allow for easier removal and refilling of gravel from and into the filter box respectively. In addition, a shallow height will also enable the construction of non-reinforced side walls and thus effectively reduce construction costs. On the other hand, too shallow structures require extensive land. Therefore, the height of the side walls should lie between 1.0 and 1.5 m.

  $H_{max.}$ = 1.5 m
  $H_{recommended}$ = 1.0 - 1.5 m

- the width W of a HRF depends on the required capacity of the treatment plant. In general, at least 2 HRF units should be provided in order to allow for treatment continuity during maintenance of any one unit. For hydraulic and structural reasons and in order to limit the interruption period necessary for manual filter cleaning, the maximum width should not exceed 5 m, whereas the minimum width should be at least 1 m.

$$W_{max} = 5 \text{ m}$$
$$W_{recommended} = 1 - 4 \text{ m}$$

Filter Control

The hydraulic conditions in a HRF are determined by the hydraulic load and by the water depth in the filter. These conditions are controlled by certain installations such as weirs and valves. Filter control is essential to maintain specific flow conditions and to detect leakages.

A distributor box or channel divides the flow into equal parts to the different HRF units. The simplest control device is a V-notch weir. Maximum flow through the treatment plant can be limited by an overflow located in the distributor box or channel.

The water level in the HRF is influenced by the outlet control system. The simplest option here is also the installation of a weir which maintains the effluent water level at a fixed height. The maximum flow level of water should be kept 5-10 cm below the top of filter media so as to avoid algal growth problems.

The filter resistance increases with progressive filter operation. Since the water flows through coarse material at low velocities, the final headloss in a HRF will usually be in the range of 10 to 20 cm, but should not exceed 30 cm. The headloss variation in the filter is accommodated in the top part of the filter material. Therefore, filter material should be filled to approx. 30 to 40 cm above the effluent's weir level. Fig. 11 illustrates the general HRF lay-out.

5. CONSTRUCTION METHODS AND MATERIALS

HRF Construction

As a matter of principle, locally available material, manpower and community participation should be used whenever possible in the construction of any water supply scheme. The installations should be simple, sturdy and of good finish, as well as maintainable with local means. The lay-out should facilitate both operation and maintenance.

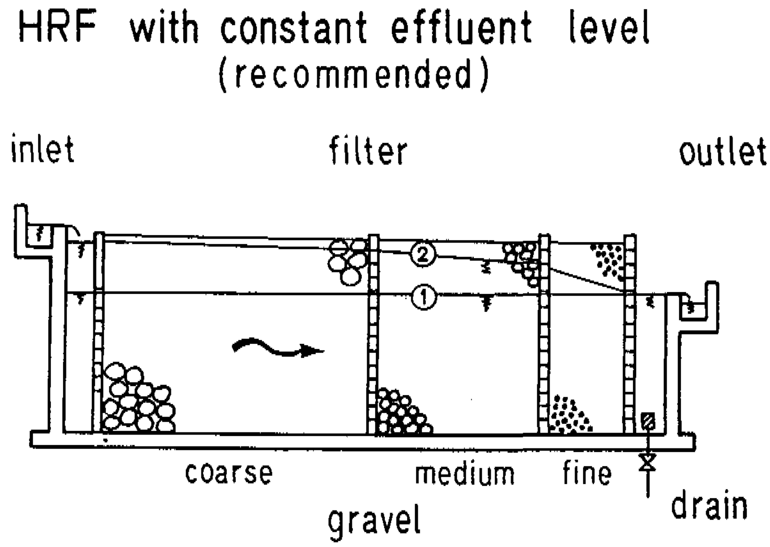


Fig. 11 Fixed Effluent Water Level Filter Control

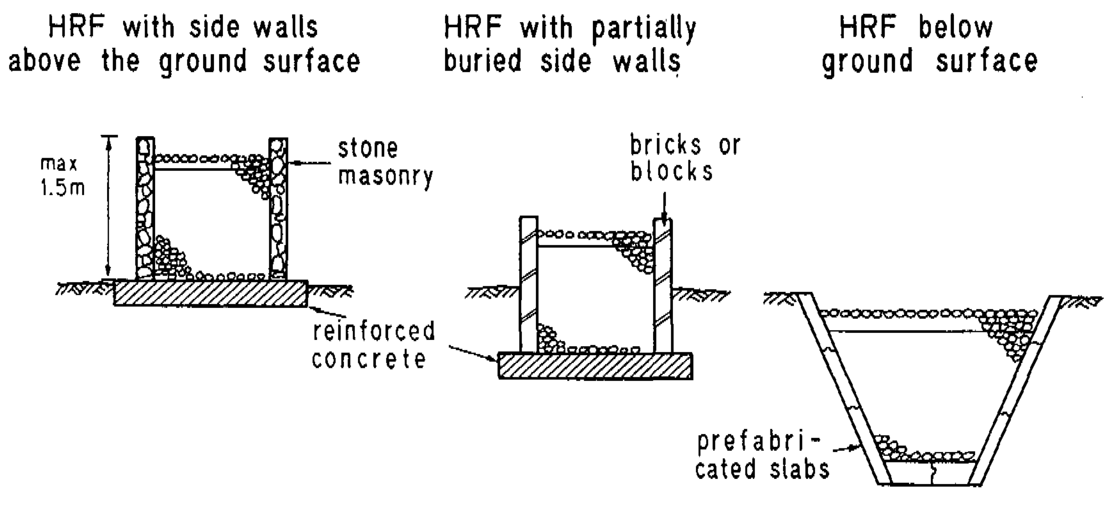


Fig. 12 Location and Material of Filter Box

Filter Box Location

HRF can generally be located below or above ground level as illustrated in Fig. 12. The choice of a HRF structure depends on the hydraulic profile, soil characteristics and available construction material. In a flat topography, gravity flow often requires the structures to be placed below ground level. A partially buried HRF has the advantages of requiring less excavation work, of providing support to the side walls by the back-filled soil and of presenting greater protection against dust and sand.

Filter Box Material

The most modest box consists of a trench excavated in an impervious soil such as clay, silt or laterite. The ditch has inclined side walls not exceeding the slope stability of the water-saturated soil (slope approx. 1:1).

Lining of the base and the side walls of such a basin prevents the mixing of clean filter material with the soil. The type of lining depends on the degree of soil impermeability and stability to be investigated in the design phase. Prefabricated slabs, in-situ applied coatings (concrete lining, ferrocement, lime mortar) or prefabricated plastic material in emergency cases (e.g. refugee camps) can be used as lining material. The horizontal:vertical slope of the side walls is controlled by slope stability properties of the water-saturated soil (a typical slope ranges from 1:1.5 to 1:3).

A watertight box has to be constructed if the underground is permeable or if the filter is installed above the ground surface. In such cases, vertical side walls are recommended. Burnt clay bricks with a cement mortar lining, concrete bricks or reinforced concrete are the filter box's building materials. A washing slab sloping away from the HRF side walls and ending into a main wastewater drain channel should be provided on one side of the HRF. This will keep the surroundings and the filter clean during the washing out of the sticky material deposited on the filter media. The bottom/foundation slab should be built from either plain or mesh-reinforced concrete cast onto a hardcore layer of about 20 cm thickness underlain by a loam-free soil.

In order to avoid cracks in the box resulting from uneven settling of the soil, construction of the foundation and the floor of the box require special attention. Finally, dilatable joints will eventually be necessary in long filter boxes constructed with material prone to shrinking or the HRF can alternatively be divided into two interconnecting compartments.

Another alternative to reduce the total length of the filter box is the design of a U-shaped unit. In and outlet are on the same filter side and the filter box is divided by a longitudinal separation wall into two equal parts.

The filter box should be tested for watertightness, preferably before it is filled with filter material, as leakages are more easily detected and repaired in empty structures.

Filter Material

The filter material should have a large specific surface in order to enhance the sedimentation process taking place in the HRF. Furthermore, it should provide high porosity necessary for the accumulation of the separated solids. Generally speaking, any inert, clean, insoluble, and mechanically resistant material fulfilling the above two criteria can be used as filter medium. Filtration tests revealed that neither the surface roughness nor the shape or structure of the filter material have an appreciable influence on filter efficiency.

The following filter material can for instance be used:

- gravel from a river bed or present in soils
- broken stones or rocks from a quarry
- broken burnt bricks made of clay
- plastic material either as chips or modules, e.g. used in trickling filters (self-reliance as regards the use of locally-available material is no longer considered here; attention should be paid to the uplift forces of the water)
- possibly burnt charcoal (risk of disintegration when cleaning the filter material)
- possibly coconut fibre (risk of odour nuisance during longer filter operation periods)

Practical aspects such as e.g. the availability of specifically-sized material from a quarry are also an important criteria in the choice of the filter material size. In order to remove all loose and dirty material from the surface of the filter, the aggregates should be washed thoroughly. If this recommendation is not observed, the HRF's initial effluent quality will be poor and result in a rapid clogging of the SSF.

Separation Walls

The different filter fractions should be separated from each other in order to avoid mixing of the aggregates during manual cleaning of the filter. Media separation walls (or elements) help to separate the different media fractions so that they do not get mixed up during the cleaning process. Therefore, they have to be structurally strong enough to support the media, especially when the neighbouring compartment is empty. In addition to structural requirements, they have to perform the following hydraulic functions:

- They should also allow an even inflow and outflow of water across the fractions,
- and negligibly contribute to headloss across the HRF during the flow of water.

Burnt brick or cement block walls with open vertical joints are best suited for such a separation. The total area of the open joints should idealy cover 20% of the total filter cross-section area and be equally distributed over the entire cross-section in order to maintain even flow through the HRF. Prefabricated perforated bricks or blocks (e.g. holes Ø 3 cm, spacing 5 x 5 cm) or loose rubbles could be installed as an alternative to the open joints. Finally, wooden boards might be used for separation of the different gravel fractions.

In and Outlet Structures

Even distribution of the raw water and abstraction of the treated water, flow regulation and water level control are the objectives of the in and outlet structures.

The in and outlet structures are preferably equipped with V-notch weirs for flow control if weirs of approx. 30 cm headloss can be installed in the hydraulic profile of the treatment plant. The V-notch weir at the outlet can be replaced by a fixed effluent pipe in treatment plants with minimal available hydraulic heads.

Even distribution of the flow through the full filter's cross-section area is achieved by an inlet chamber. The separation wall between this compartment and the first filter package should contain openings in its middle part as shown in Fig. 13. A solid wall at the bottom and at the top respectively, hinders penetration of coarse settled solids or floating matter into the filter. The minimum width of the inlet chamber should not be smaller than 80 cm to ease cleaning.

A similar outlet chamber is installed at the effluent side. However, the openings in the separation wall located after the last filter package are distributed all over the filter's cross-section.

A weir or an effluent pipe maintains the water table of the filter outlet zone at a specific level. The progressively increasing filter resistance must be accommodated within the filter bed. For this reason and to avoid mosquito breeding, and algae growth it is necessary to fill filter material up to approx. 30 to 40 cm above the weir's level.

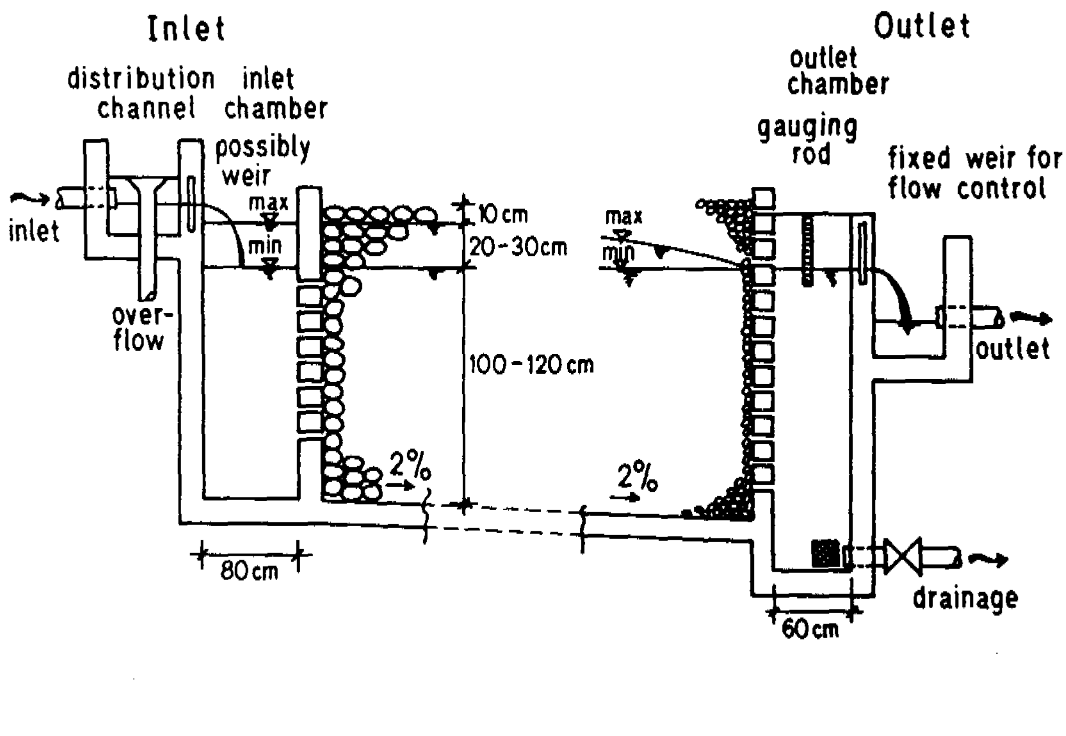


Fig. 13 In and Outlet Structures of HRF

Drainage System

Drainage facilities such as perforated pipes, drainage troughs or culverts enable hydraulic cleaning of the filter medium as illustrated by Fig. 3. The system is placed perpendicular to the direction of flow at the filter bottom. The spacing between the drains should be approx. 1-2 m. The hydraulic capacity of these installations should permit an initial filter drainage velocity of 60-90 m/h necessary for efficient cleaning. Valves, slide gates or flexible hose pipes can be used to operate the drainage system. Each drain should discharge into an open channel to allow visual supervision of any drainage operation. Facilities for safe washwater disposal are necessary to prevent erosion and water ponding.

For manual cleaning of the filter medium, a drain is placed in the outlet chamber to enable complete drainage of the filter box. The filter bottom should therefore be slightly inclined by 1 to 2 % in the direction of flow. A side effect of this proposed slope is the saving of some filter material.

6. OPERATION AND MAINTENANCE

HRF can easily be operated and maintained by trained local caretakers. They do not depend on external inputs, provided the necessary materials and tools are available. The daily activities of the caretaker are preferably supported by occasional visits of a supervisor attached to the operation and maintenance section of the governmental or public institution responsible for the water supply system. Important maintenance work should be carried out at the time when village participation can be involved. This is of particular importance as regards manual cleaning of the HRF.

Commissioning of the Filter

Filter operation should only start when construction work is totally completed. The efficiency of a HRF filled only partially with gravel will be poor as the unit will not act as a filter but as an inadequate sedimentation tank. Emphasis should therefore be placed on a good finish of the construction work including the installation of proper flow control and drainage facilities as well as a full supply of filter material.
Before starting filter operation, it is recommended to wash the installed filter material by drainage. The filter unit should be filled with water up to the effluent's weir level at low flow rates of 0.5-1 m/h. Thereafter, the water should be drained off through the first drainage installation located next to the inlet. Any dust on the surface of the filter material is rinsed to the filter bottom. The impurities accumulated around the drainage system will be flushed out of the filter. This procedure should be repeated if necessary 2 to 3 times by changing the point of drainage from filter inlet to filter outlet side.

Flow Pattern

A continuous filter operation makes the best use of the installations, provides maximum production and a constant flow pattern. However, full gravity flow will be required for such an ideal situation. If pumping is necessary, the treatment plant can be staffed for 8 or 16 hours a day, depending whether 1 or 2 shifts are available.

Unlike SSF, HRF acts as a physical filter and therefore does not depend like SSF on a continuous supply of nutrients. Hence, intermittent operation is possible without a marked deterioration of the filtrate, provided smooth restarting of filter operation is observed. Due to the relatively small water volume stored in the HRF, it is not reasonable to operate the HRF at a declining filtration rate to enable SSF operation at a constant filtration rate.

It can be concluded that for operational and economic reasons it is recommended to continuously operate a HRF-SSF plant at constant filtration rates for 24 hrs/day. In case of a pumped scheme, a raw water balancing tank is required. Removal of the coarse solids is a positive side effect of such a tank.

Flow Control

HRF is hydraulically controlled by a flow control device at the inlet and by a fixed weir at the outlet. In gravity schemes, constant feeding is maintained by a more or less fixed position of the valve in the supply pipe and a subsequent overflow in the distributor box. In pumped schemes with a raw water tank, the flow to the HRF is regulated by a mechanical flow rate device as illustrated in Fig. 14.

Filter Cleaning

Filter efficiency decreases with progressive accumulation of solid matter in the filter. Hence, periodic removal of this accumulated matter restores filter efficiency and keeps the filter in good running condition. A HRF can be cleaned in two ways, either hydraulically or manually.

Hydraulic cleaning assists the mechanisms of self-regeneration already discussed and illustrated in Fig. 2. The natural drift of accumulated matter towards the filter bottom can be enhanced by filter drainage. The retained solids are washed down when the water level in the filter is lowered. The upper part of the filter bed is thereby cleaned and regenerated while an additional accumulation of solid matter takes place at the filter bottom. These solids can be flushed out of the filter by an adequate drainage system at initial drainage velocities ranging preferably between 60 and 90 m/h.

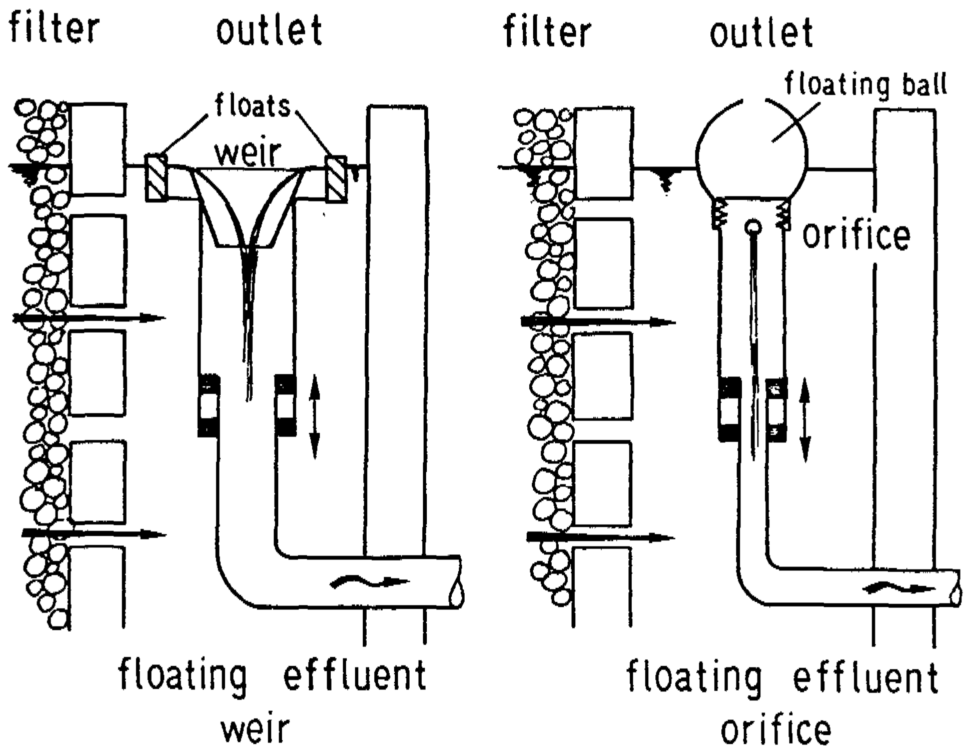


Fig. 14 Mechanical Flow Rate Devices

It is very important to start the cleaning procedure at the inlet side as most of the solids are retained in this part of the filter. An initial vigorous drainage of the finest fraction of the filter material can wash the bulk of solids towards the used drainage point and increase the risk of clogging of that fraction.

Furthermore, full drainage of the HRF at individual points in turn is equally important as it flushes out the accumulated matter in the vicinity of the drainage point. The drained HRF is thereafter refilled with water and re-drained at the same drainage point if the solids have not been completely washed out in this filter part. This can be noted by observing the turbidity decrease of the drained water. At low washwater turbidities, the next point should be drained using the same procedure.

When refilling the HRF, attention must also be paid not to drag to the fine filter fraction the accumulated solids at the filter bottom. Moderate flow rates must therefore be applied and may be increased during refilling. The HRF can be filled only partially with water if an efficient drainage system is available for complete wash-out of solids at low water demand, since most of the solid matter will be rinsed towards the filter bottom after 2 or 3 full drainages.

If a special drainage system is absent, partial filter efficiency regeneration can still be achieved if the ordinary drain, preferably at the inlet chamber, is used. If only a single drain is provided at the HRF outlet, lower drainage velocities in the range of 10 to 20 m/h should be observed to prevent blockage of the fine filter material.

The annual hydraulic cleaning schedule has to be adapted to the annual fluctuation of the raw water quality. High turbidity loads are preferably treated by relatively clean filters to prevent a breakthrough of the solid matter which would otherwise affect SSF operation. It is therefore recommended to thoroughly clean the HRF before peak loads (e.g. before the start of the rainy season). Hydraulic cleaning can be handled by the caretaker and does not normally require external assistance (e.g. community participation). Therefore, the annual working plan of the community does not influence the planning of the hydraulic cleaning schedule.

Manual cleaning must be applied when the solids accumulated at the filter bottom or at worst, all over the filter, can no longer be removed by hydraulic cleaning. This occurs if a drainage system is absent under the filter bed, if proper hydraulic cleaning has been neglected or if solid matter has cohered to the filter material or at the bottom. A slimy layer might cover the filter material if there is biological activity in the filter caused by high loads of dissolved organic matter in the water. This biological layer will most probably increase the filter's efficiency at the beginning, but will subsequently hinder the drift of deposited matter towards the filter bottom. Accumulated cohesive matter might also hinder self-regeneration of the filter.

Finally, retained material in silted, but drained filter beds will also dry up and form a skin around the filter material. Thus, HRF should never be kept dry unless the filters are properly cleaned in advance.

Manual cleaning mainly consists in excavation, washing and reinstallation of the filter material. The filter material is excavated from a drained filter. The coarsest filter material is normally cleaned first. The filter box compartment must be washed and the cleaned filter material can be refilled thereafter.

The washing of the filter material is best achieved by mechanical stirring of the aggregates in a washwater basin as mechanical friction rubs the impurities off the aggregates' surface. Washwater can be saved and a good efficiency achieved if small filter material loads are stirred with a shovel in a first tank to remove gross impurities before they are transferred to a second tank for final washing. However, centralized cleaning involves transportation of the filter material. Use of the open drainage channel located along the HRF is an alternative to the washing place which requires less efforts as regards gravel movement.

Resieving of the filter material is necessary if mixing of the different fractions occurred or if the filter medium has been broken up into smaller pieces due to excavation and mechanical cleaning. A well specified, uniform size for each filter fraction is essential to maintain high porosity of the filter bed. In this context, it is obviously advantageous to install a mechanically-resistant filter material right at the beginning.

Manual filter cleaning involves a great deal of manual work which is often beyond the caretaker's capacity. Additional manpower must be mobilized either by contracting local casual labourers or by involving the community. Careful planning and organization is necessary when manual filter cleaning is carried out with village participation. The cleaning schedule should for instance not coincide with a period of intensive agricultural work.

Adequate material and tools must be provided to enable efficient filter cleaning, otherwise maintenance work will become too tedious and might never be done. Manual filter cleaning requires shovels, sieves, preferably 2-3 sturdy wheelbarrows, some wooden boards and buckets. The same material already used for construction should therefore remain at the treatment plant or in the care of the local operator at the end of construction.

Filter Maintenance

HRF maintenance is not very demanding as the filter does not contain any mechanical parts. Nevertheless, maintenance should aim at maintaining the plant in good condition right from the beginning. External assistance for maintenance work can usually be avoided if the following work is carried out properly by the local caretaker:

- periodic upkeep of the treatment plant's premise (grass cutting; removal of small bushes and trees which could impair the structures by their roots; removal of refuse)
- soil protection against erosion (especially surface water intake structures, the wastewater drainage channels and surface runoff)
- repairing fissures in the walls of the different structures and replacing the chipped plastering
- application of anti-corrosive agents to exposed metal parts (V-notch weirs, gauging rods, pipes)
- checking the different valves and drainage systems and occasionally lubricating their moving parts
- weeding out the filter material
- skimming off floating material from the free water surfaces
- washing out coarse settled material (distribution box, HRF inlet)
- controlling the ancillaries and replacing defective parts (tools and test equipment)

7. COSTS

HRF Construction Cost Structure and Specific Costs

A construction cost structure evaluation of different HRF projects with a design capacity ranging from 70 to 750 m^3/d and located in Tanzania, Kenya, Indonesia and Australia revealed rather similar results:

- earthwork and structure	approx. 70%	of total costs
- filter medium	approx. 20%	" " "
- piping and accessories	approx. 10%	" " "

Topography, soil conditions (required excavation work and type of foundation) and type of structure (reinforced concrete or brickwork) are cost decisive factors for earthwork and structure. Local availability of filter material in the required sizes strongly influences the purchase price, i.e. the supply. These first two cost components have only a little economy of scale, however, the relative costs for piping and accessories will decrease with increasing plant size.

The specific HRF construction costs per daily m^3 water output depend on the filter length and the applied filtration rate. For an assumed total filter length of 10 m and a filtration rate of 1 m/h for 24 h/d, the specific costs per daily capacity are the following:

- total construction costs	approx. 60-80 US $\$/m^3/d$
- material costs only (e.g. in self-help projects)	approx. 30-40 US $\$/m^3/d$

Operational Costs of a HRF

A HRF is operated without the use of chemicals. The costs for filter cleaning are the only operational costs of a HRF. Hydraulic cleaning of the filters can be carried out by the caretaker, and therefore does not create additional costs. Manual cleaning, however, usually requires additional labour.

Since only labour is involved in the use of a HRF, any community with a strong interest in treated water can afford the operation of these filters. The running costs can be reduced to a minimum if the community participates in filter cleaning. This fairly self-reliant treatment process therefore does not depend on any external financial and technical support. Hence, large operation and maintenance expenditures, often not sufficiently available, can be reduced to an absolute minimum by the installation of self-reliant treatment processes such as HRF and SSF. This is one criteria for long-term operation of any water supply scheme.

Local and Foreign Currency Cost Component

HRF is essentially a self-reliant technology largely reproducible with local means. According to the construction cost structure, 90% of these investment costs are expenditures for construction material such as gravel, sand, cement, bricks and stones, and for labour, both readily available in the country. The remaining 10% are costs for the purchase of pipes, valves and accessories (V-notch weirs, gauging rods) which may partly have to be imported. Hence, none or a very small amount of the construction costs require foreign currency.

HRF operation and maintenance basically require manpower but no additional material. HRF is a system which is operated on village-level, and thereby run and maintained entirely by the local community. Hence, the absolute self-reliant process demands local input only.

8. CONCLUSION AND RECOMMENDATION

The construction, operation and maintenance simplicity and self-reliant nature of the filter combination HRF and SSF make this treatment technology suitable for application in rural and small urban water supplies in developing countries. The process has proven its viability under laboratory, field and practical conditions. Pretreatment of turbid water with HRF will solve operational problems experienced with so many SSF. Hence, this filter type will play a major role in the rehabilitation of water treatment plants using SSF, and will commonly enhance a broader implementation of SSF. Once tested under local conditions and introduced through demonstration schemes, this technology is reproducible with the use of local resources only. HRF combined with SSF will therefore have a potential future in the water technology of developing countries.

Note:

The part of Horizontal-flow Roughing Filtration is mainly an extract of the HRF Manual (Ref. 7) published by IRCWD, Ueberlandstr. 133, 8600 Duebendorf, Switzerland.

References

(1) Horizontal-flow Roughing Filtration: An Appropriate Pretreatment for Slow Sand Filters in Developing Countries, M. Wegelin, IRCWD News No. 20

(2) Horizontal-flow Coarse-Material Prefiltration, N.C. Thanh and E.A.R. Ouano, AIT, Research Report No 70, 1977

(3) Surface Water Filtration for Rural Areas, N. Thanh and J. Hettiaratchi, Ensic, Bangkok/Thailand, 1982

(4) Slow Sand Filter Research Report No. 1, 2 and 3, M. Wegelin and T.S.A. Mbwette, University of Dar es Salaam, 1980 and 1981

(5) Horizontal-flow Roughing Filters for Rural Water Treatment in Tanzania, T.S.A. Mbwette, M.Sc. Thesis, University of Dar es Salaam/Tanzania, 1983

(6) Particle Removal by Horizontal-flow Roughing Filtration, M. Wegelin, M. Boller and R. Schertenleib, AQUA, 3/1986

(7) Horizontal-flow Roughing Filtration; A Design, Construction and Operation Manual, M. Wegelin, IRCWD, 1986

HRF DESIGN -
move from a conservative to an economical lay-out

Introduction

Originally, HRF design was geared towards the provision of a large silt storage capacity. This would permit filter runs of several months also at periods of high suspended solids concentrations in the raw water. Relatively large filter lengths of 9 to 12 m were the consequence of this initial design approach which propagated the advantages of an unlimited filter length use - conversely to vertical filters where filter beds are limited to approx. 1 m due to structural contraints. Manual filter cleaning was also given priority. Therefore, the height of the HRF side walls was recommended to be in the range of 1-1.5 m in order to ease manual filter cleaning. However, the importance and the possible benefits of hydraulic filter cleaning were recognized in the meantime. New design practice tends to reduced filter lengths of 4-7 m and to install efficient hydraulic cleaning facilities. Frequent hydraulic cleaning should cope with the reduced filter length. Or in terms of costs, capital cost savings are replaced by possibly higher operation costs.

The type and the gradation of the filter material is another HRF design aspect. Sometimes, gravel is locally not available and if carried from far away will increase the respective unit costs. Hence, reduced filter length is also in the line of gravel sawing. Gravel is sometimes also substituted by locally available other inert filter material such as broken burnt bricks or coconut fibres.

Finally, the preliminary design guidelines recommend the use of 3-4 different gravel fractions to prevent the development of a large filter resistance. The installation of 3 gravel fractions has become general practice. However, some HRF plants were designed with only 1 or 2 differently sized gravels in order to simplify filter material handling.

In the following, the implications of the 3 introduced design parameters

filter length, filter material and filter gradation

will be discussed with regard to filter efficiency and operation.

General Considerations

Figures.2 and 3 show graphically some results of the filtration tests run with a kaolin suspension at EAWAG/IRCWD's laboratory carried out some years ago (1). The diagramms indicate the following relationships:

- the filter efficiency depends on the applied filtration rate v_F, the used gravel size d_g and the actual filter load σ (Fig. 2)
- the filter efficiency does not depend on the surface characteristic of the filter material (Fig. 3)
- the filter efficiency decreases considerably once the filter load exceeds a value of approx. 10 g/l (Figs. 2 and 3).

The information of these graphs was also used for the calculation of the E.value (filter efficiency) summarized in Table 7, Appendix 1 of the HRF Manual (2). These E-values were determined for a relatively clean filter (filter load σ < 10 g/l).

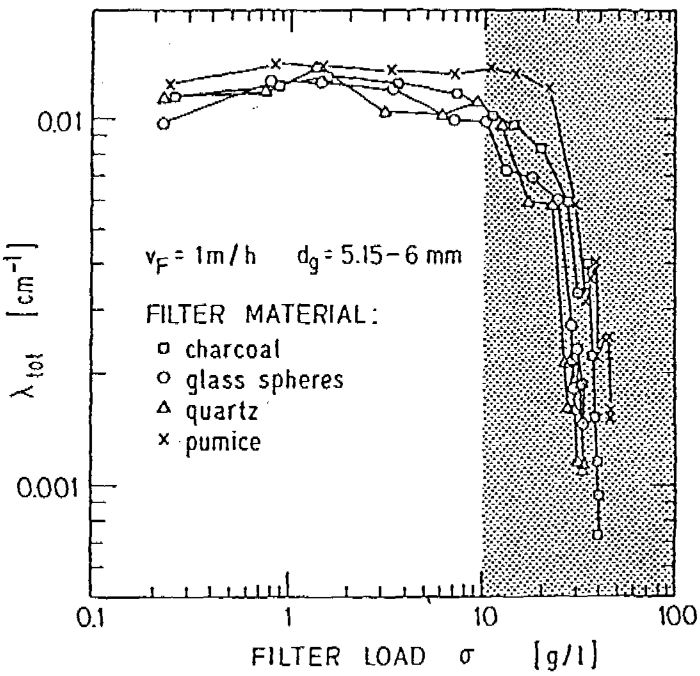


Fig. 2 Filter coefficient for different filter material in correlation to filter load

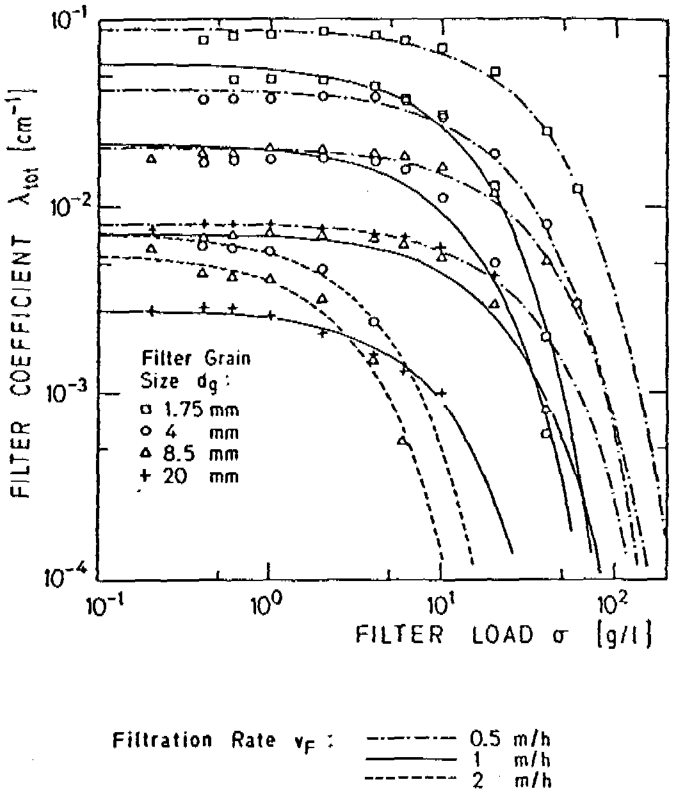


Fig. 3 Filter coefficient in correlation to filtration rate, grain size and filter load

Filter Length

The tentative design guidelines with respect to HRF filter lengths presented in Table 2 of the HRF Manual (2) aim at relatively long filter runs. Prolonged filter operation will generate high filter loads. This silt accumulation, however, is not evenly distributed over the filter length. Highest filter loads are experienced at the entrance zone of each gravel fraction. The front of the accumulated silt will gradually penetrate into the filter with progressing filter operation. On the other hand, filter efficiency will rapidly decrease with increased filter load as expressed by Figs. 2 & 3, i.e. filter efficiency will be reduced quickly by a factor of 10 or more, once a filter load of 10 g/l is exceeded or in other terms, 0.5 m of a relatively clean filter will have the same efficiency as 5 m of a silt-loaded filter.

Hence, silted filter zones contribute little to a further reduction of suspended solids. Therefore, filter lengths can be minimized to a design which still has to produce an effluent suitable for SSF application (according to (3) < 20-30 NTU) also at periods of high raw water turbidity. The HRF filter length can possibly be further reduced by adequate installations at the river intake (e.g. intake or dynamic filters) as also suggested in (4) or in front of HRFs (e.g. by a sedimentation tank).

Filter Material

Based on the laboratory results presented in Fig. 3 it was concluded that "any inert, clean, insoluble and mechanically resistant material can be used as a filter medium" (on page 39 in (2)). Practical field experience and further laboratory investigations tend to confirm this statement. Gravel was replaced by broken burnt bricks in HRF constructed by the Blue Nile Health Project (BNHP) in the Sudan (5), by palm fibre called "ijuk" in a HRF project in Indonesia (6) and by plastic material in laboratory tests at the University of Newcastle in England (7).

Table 1 shows that bricks and plastic as filter material have a similar filter efficiency as gravel with respect to turbidity reduction. The filter filled with palm fibre has a better respective performance compared to the gravel filter. The higher porosity (92% versus 37%) which also reduces the effective flow velocity is certainly an explanation for this observation. However, a considerable drop of the dissolved oxygen concentration occurred in the palm fibre filter which could create odour and taste problems. Therefore, the long-term applicability of palm fibre needs further investigations.

	Turbidity reduction (size of filter material)	
Project	Gravel	alternative filter material
burnt bricks, BNHP/Sudan	87 % (20-30, 15-20 and 5-10 mm)	77 % (bricks 30-50, 15-20, 5-10 mm)
palm fibre* Plumbon/Indonesia	39 % (16-25 mm)	67 % (fibre)
plastic material, University of Newcastle	92 % (broken bricks, 30-50 mm; gravel 14-18 and 5-9 mm)	94 % (rings ø 38 mm pipes ø 30 mm caps width 5 mm)

*only filled in first filter compartment

Table 1 Relative filter performance with different filter material (parallel tests)

Filter gradation

Filter efficiency increases with decreasing filter material size as illustrated also by Fig. 2. Therefore, one could tend to use the smallest possible filter material or even to omit the larger filter material and to install only one - the finest - filter medium as for instance done for a HRF in South Africa (8). There, a 12 m long HRF was filled with gravel of only 1.2 mm size after a 1 m long inlet zone filled with pebbles of 20-50 mm size. Williams reported that "the HRF gradually blocked up from the inlet with dirt, and after 4 months of operation the water level at this end was at the surface of the gravel. From then on, some of the influent water to the HRF flowed over the surface of the gravel bed until it reached cleaner gravel. After 15 months operation of the HRF, water was flowing over the first 5 m of the gravel bed. However, despite this shortened length of active HRF, there was still no sign of turbidity break-through in the HRF effluent".

This is a clear example of an inappropriate HRF design. The large filter volume is only partly used for suspended solids removal at the begin (outlet filter zone not active) and at the end (inlet filter zone passive due to blockage) of the filter run. No hydraulic drainage system has been installed and after 15 months of operation a large volume of filter material had to be cleaned with the help of a concrete mixer.

The HRF technology requires the use of coarse filter material as described in the name: **Horizontal Roughing** Filter. The filter material should not be smaller than approx. 4 mm in order to ease hydraulic filter cleaning. Too coarse filter material will, however, increase the required filter length. Hence, the use of at least 2 or generally 3 different filter material sizes will result in an economic filter design and adequate filter operation. Fig. 4 illustrates the general relationships between size of filter medium, filter length and filter performance.

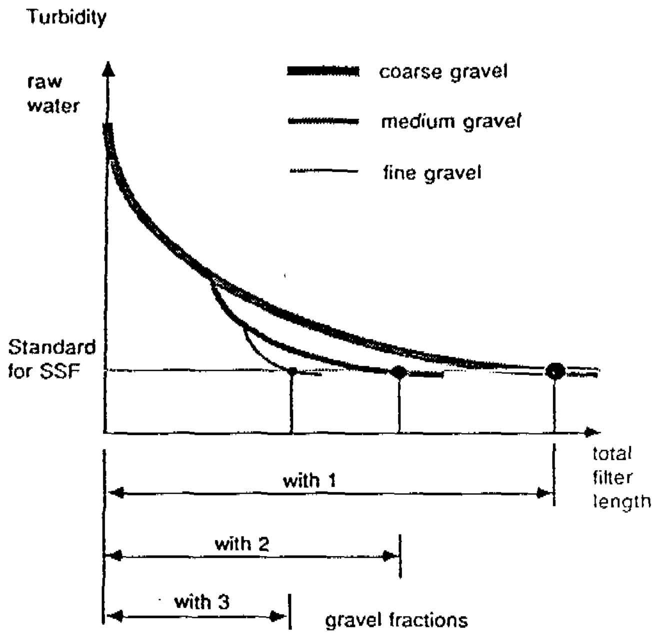


Fig. 4 Turbidity reduction along HRF filter length for differently sized filter material

General Recommendations

Based on the presented general considerations and on the experiences made in the laboratory and in the field, the following recommendations are made:

1. Filter design criteria with respect to filter length should be based on the provision of a sufficiently clean filter bed efficiency for periods of high raw water turbidity and not on the provision of a large silt storage capacity enabling filter runs of excessive duration.

2. HRF lay-outs meeting peak raw water turbidities can prove to be uneconomic since full filter lengths are utilized during short periods only. Turbidity peaks should be reduced through preliminary raw water treatment (i.e. intake and dynamic filtration, sedimentation).

3. Broken burnt bricks and plastic material prove to be an alternative filter medium to gravel, broken stones or rocks. Material with a high specific surface area and porosity increases filter efficiency and thus the filter volume can be reduced.

4. Economic HRF design and adequate filter operation call for filter lay-outs with 3, at the minimum with 2 different gravel fractions.

References

(1) Particle Removal by HRF, M. Wegelin, M. Boller and R. Schertenleib, Aqua No 2/1987
(2) HRF Manual, M. Wegelin, IRCWD Report No 06/86
(3) SSF for Community Water Supply, J.T. Visscher, R. Paramasivam, A. Raman and H.A. Heijnen, IRC Technical Paper No 24/1987
(4) Evaluation of the Water Treatment Plants of Cocharcas, La Cuesta and Compin, M. Pardon, DelAgua/CEPIS, Final Report May 1988
(5) SSF for the Blue Nile Health Project, S.E. Basit and D. Brown, Waterlines Vol. 5, No 1/1985
(6) Pilot Tests on a HRF, W.J. Fellinga, Diploma Report, Delft University of Technology, Oct./1988
(7) HRF as an appropriate Pretreatment before small SSF in Developing Countries, D. Brown, M.Sc. Report, University of Newcastle, Sept./1988
(8) Study of a HRF for Pretreatment prior to SSF, P.G. Williams, Seminar on water supply and sanitation, Kwa Zulu, June/1988

WINDOW -

Your meeting place for discussions and information exchange

The following reactions to Filter News No 1 were received:

- During a meeting in early January 1989 the HRF Project was reviewed by staff members of IRCWD and EAWAG, the Swiss Centre for Appropriate Technology (SKAT) and guests from Ghana. During the discussion, the use of light filter material easy to lift and to rinse was proposed as an alternative to hydraulic filter cleaning. Cubical plastic packs (Fig. 5) or loose plastic material (Fig. 6) filled in nets could be used for that purpose. Ideally, the pore system of such a plastic filter medium should have a graded size between 6 and 1 mm for HRF application. The filter medium surface area should be horizontal as far as possible which, however, will hinder the self-cleaning mechanisms of HRF described on page 13 in (2). In addition, a high porosity and a large specific surface area of the plastic filter medium are of advantage. Filter cleaning would have to be carried out by a lift-and-rinse procedure illustrated in Fig. 7.

IRCWD invites the cooperation partners to field test this idea with locally available plastic material which, however, has to be UV and heat resistant.

Fig. 5 Plastic packs used in trickling filters for waste water treatment

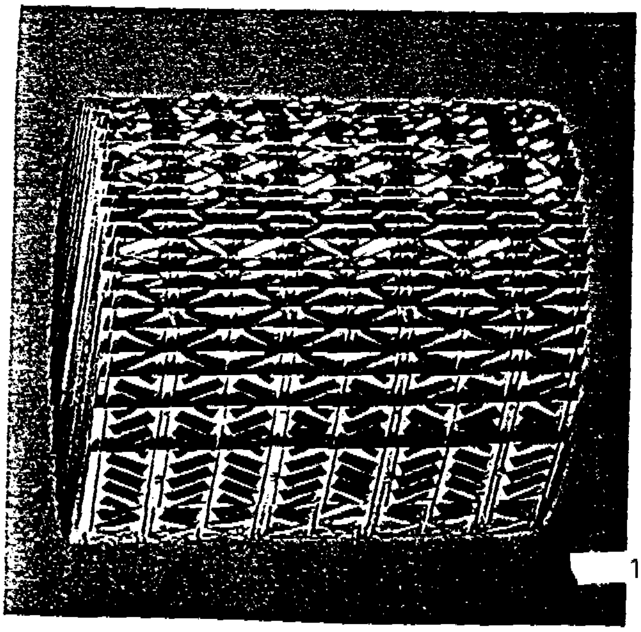


Fig. 6 Example of loose plastic material possibly suitable as HRF filter medium

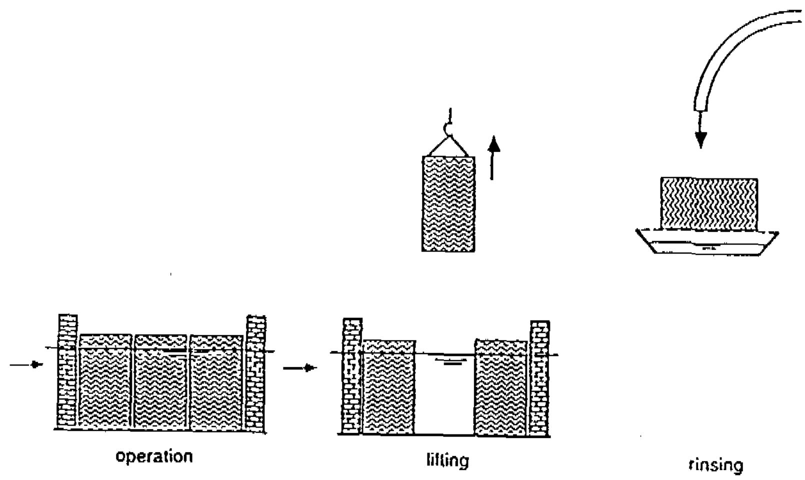


Fig. 7 Possible lift-and-rinse procedure for cleaning plastic HRF filter medium

- From IRC in the Netherlands I received the following note:

  In Colombia a Research and Demonstration Project on Pretreatment is being implemented by the working group "Area de Abastecimiento y Remoción de aguas" of the "Universidad del Valle", Cali, Colombia in collaboration with IRC International Water and Sanitation Centre. In this project preliminary experiments were carried out by Mr. Gerardo Galvis, Mr. Hielke Wolters and Mr. Luis Alfonso Hurtado to explore functioning and possible optimization of hydralic cleaning of three upflow roughing filters in layers (with a total filter bed area of 56 m^2) at the "El Retiro" water treatment plant. The drainage system of these roughing filters consists of main and lateral drain pipes. A fast opening valve (a modified milk can cover) is placed on the main drain pipe, one metre below the bottom of the filter bed in order to increase hydrostatic pressure acting upon it.
  Two different cleaning procedures were followed: a standard drainage and a drainage in combination with shock loading. During the standards drainage procedure, when the drainage valve was left open for an extended period, high suspended solids concentration of the wash water (up to 13,000 mg/l) were detected 10 to 20 seconds after opening of the drainage valve. Thereafter suspended solids concentration dropped rapidly to remain constant from one minute onwards.
  However, the introduction of shock loading (the prompt closure and opening of the drainage valve) resulted in subsequent peak values of up to 1,500 mg/l. A drainage procedure of four consecutive drainage periods of one minute each (with a total wash water volume of 4.5 to 7 m^3) removed 3 to 7 kg of sludge from each filter compartment and restored head-losses in the bed to its original value.
  The working group plans to proceed research on hydraulic cleaning at both full-scale installations and pilot plants. At the pilot plant "Puerto Mallarino" drainage procedures will be tested simultaneously at four different pretreatment methods.

IRCWD is grateful for this piece of information which helps to formulate the field test programme for hydraulic cleaning of the Upflow Roughing Filters (URF) in Aesch/Switzerland.

PEBBLE MATRIX FILTRATION (PMF)

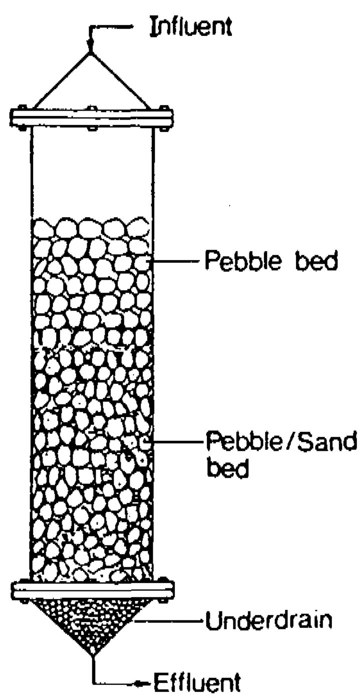
by: J.P. Rajapakse and Prof. K.J. Ives*

1. INTRODUCTION

At University College London (UCL) the Pebble Matrix Filter operating at (0.7 - 1.2) m/h appeared to meet the requirements for the pre-filtration of high turbidity flood waters in tropical conditions, as a pre-treatment technique for slow sand filtration. Tests up to 5000 mg/l are currently showing promise, producing filtrates below 25 mg/l. One particular advantage of this technique is that it does not require any chemicals for its operation, even at extremely high turbidities in the raw water. The cleaning of the filter has been kept at a simple level (no air scouring) because of its intended application in semi-rural conditions in tropical countries.

This filter, first known as Skeleton-fill filter, has been developed in Tashkent, USSR, at an experimental scale for tertiary treatment of sewage, with a maximum suspended solids load of 20 mg/l. (Khabirov et al 1983).

In principle the filters consist of a matrix of large pebbles about 50mm in size (the "Skeleton") which is infilled for part of its depth with sand, as shown in fig. 1.



PEBBLE MATRIX FILTER

Fig. 1.

* Department of Civil and Municipal Engineering, University College London, U.K.

The flow direction is downward through the media. Therefore, the suspension approaching the filter first passes through a layer of large pebbles, and then through a layer of mixed pebbles and sand. This creates a crude two layer filter where the pebbles alone have a pre-filtering effect (see fig. 2), and the filter can retain very large quantities of deposit without a great loss of permeability (as shown in photo 1 below).

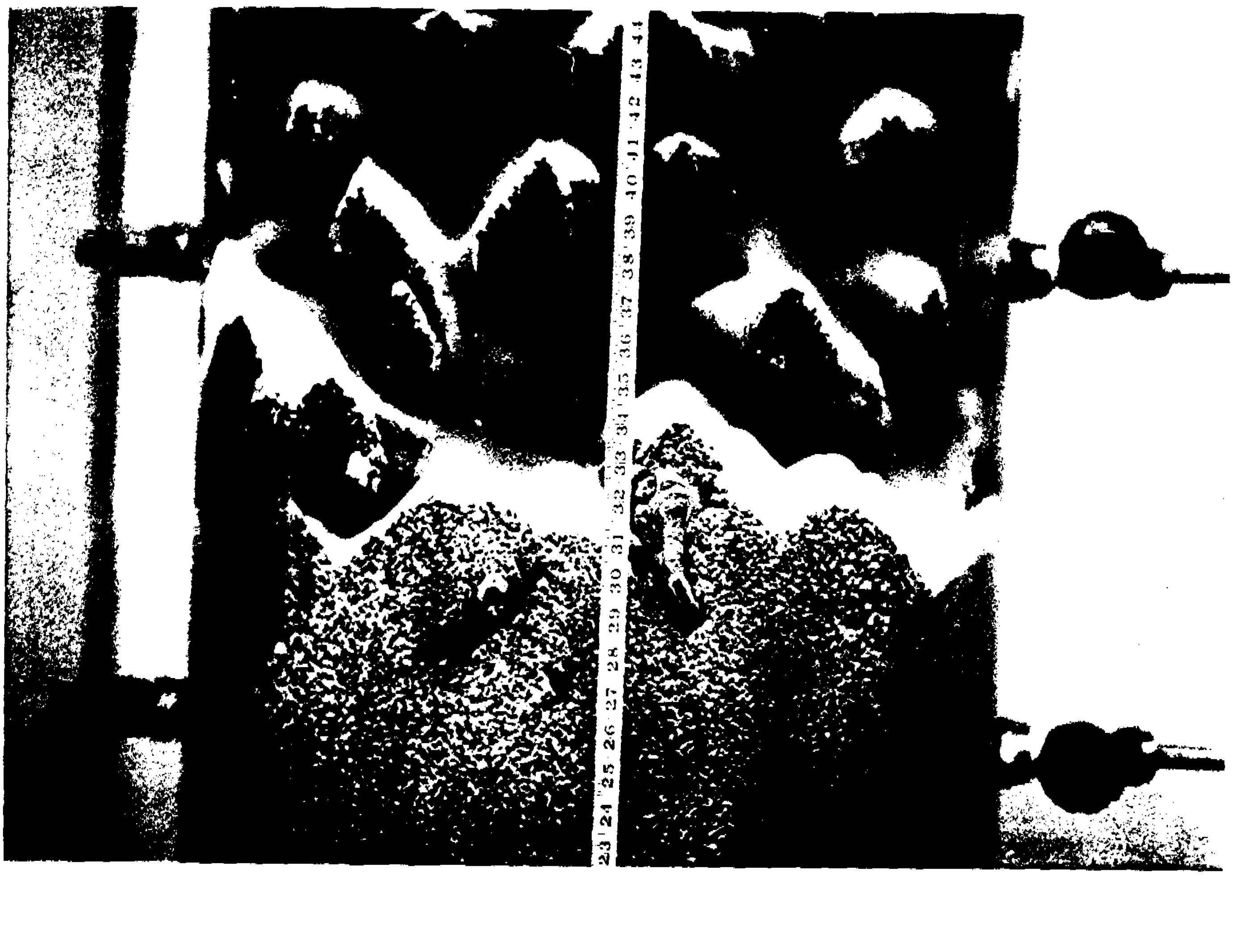


Photo 1

2. LABORATORY INVESTIGATIONS AT UCL

The experimental investigations were carried out in the Chadwick Public Health laboratory employing a model - Pebble Matrix filter which was constructed for this study. The Unit composed of a 245mm ID, 1.30m long perspex column with the top and bottom ends covered with two cones made of fibreglass. Filtration velocities of 0.5m/h to 1.5 m/h with suspended solids in the raw water from 100 mg/l to 5000 mg/l were tested. The filtrate quality was monitored continuously using a HACH-Ratio turbidimeter and pen-recorder. This general arrangement is shown below in photo 2.

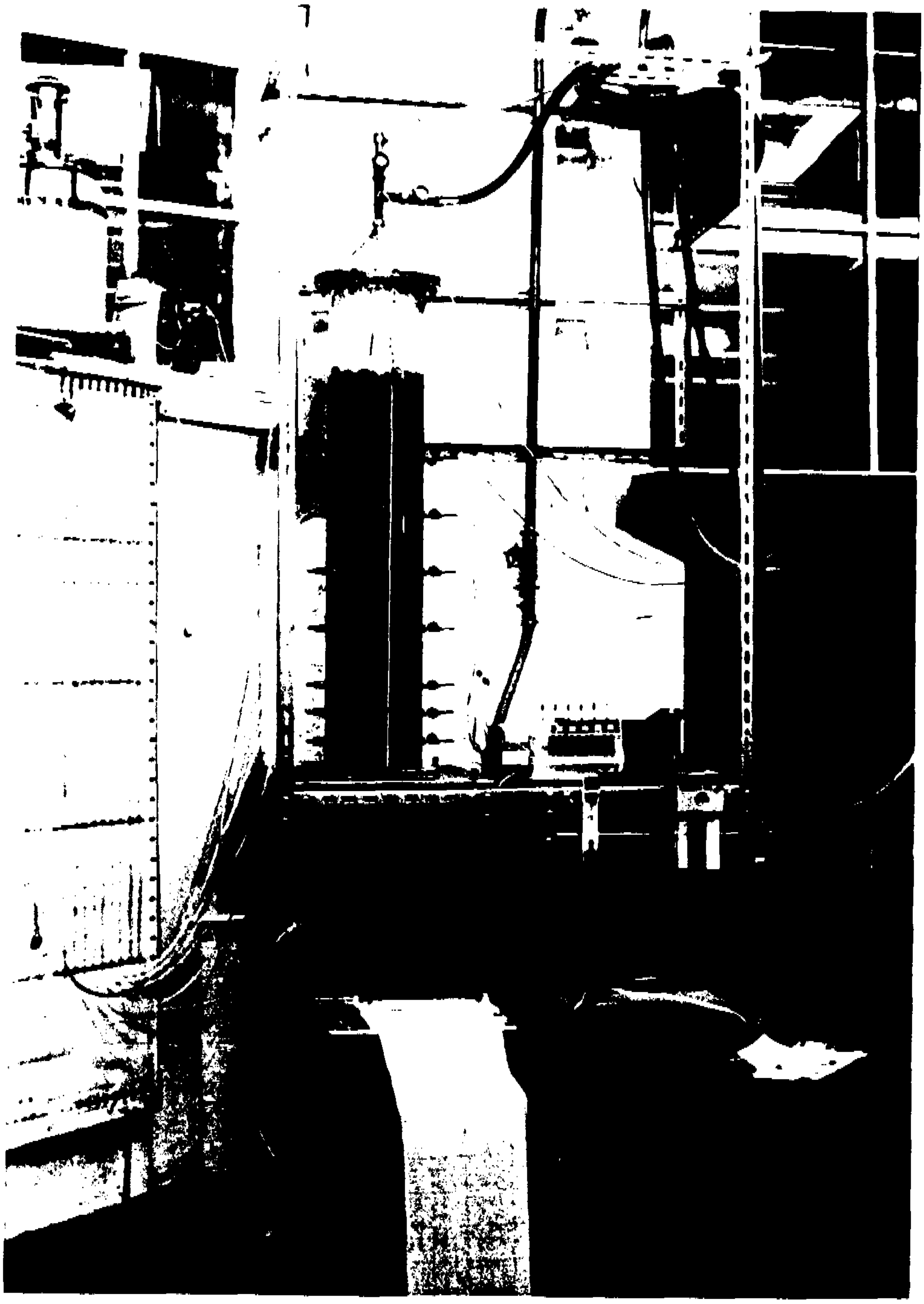
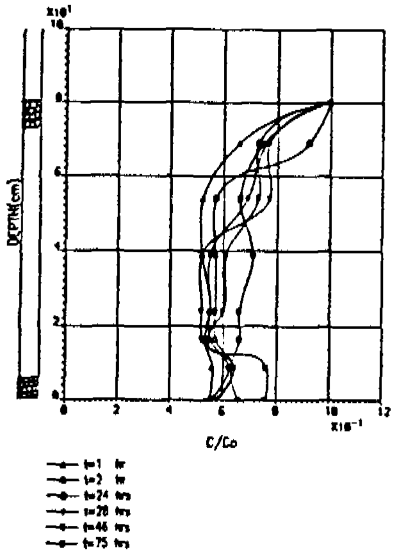


photo 2

At a filtration velocity of 0.72 m/h with fine sand (de=0.38mm) the filter produces an effluent of below 1 mg/l (suspended solids) even as high as 1000-5000 mg/l concentrations at the inlet. At all filtration rates mentioned earlier an effluent quality of less than 25 mg/l can be obtained. A typical graph showing the variation of effluent quality with time (for sand de=0.56mm) is given in fig. 3.

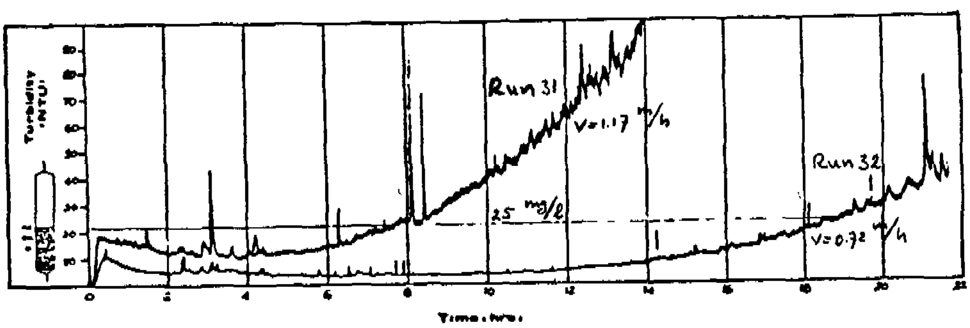


C_o = 500mg/l
V_a = 0.72m/h

SUSPENDED SOLIDS REMOVAL IN PEBBLES ALONE -RUN63

Fig. 2

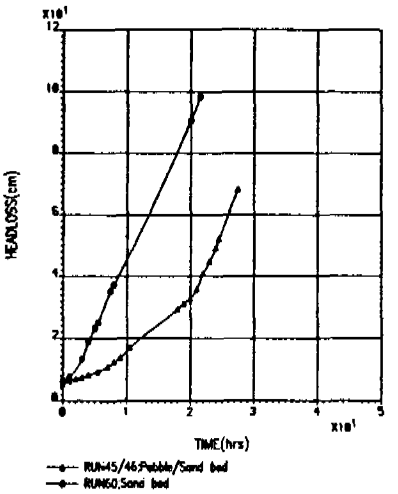
d_e = 0.56mm
Pebble/sand depth = 34cm.



VARIATION OF EFFLUENT TURBIDITY vs TIME (inlet 500 mg/l)

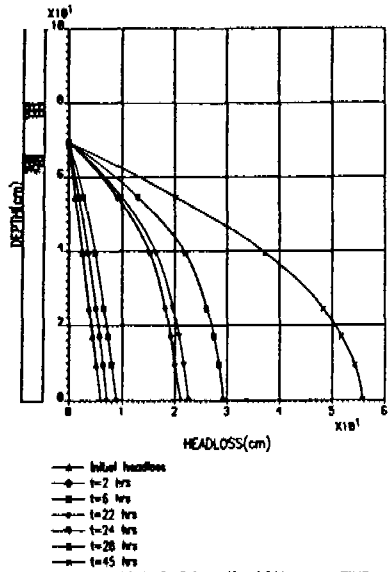
Fig. 3

Comparative studies with ordinary sand filters showed that the pebble matrix filter has a remarkably low pressure drop due to its high permeability caused by the presence of pebbles in sand (Fig. 4). Consequently the silt storage capacity of the bed is also considerably higher (30 - 70 $kg/m^2/run$) than of the conventional sand filters. The headloss development and the pressure distribution within the bed are shown graphically in Figs 5 and 6 respectively.



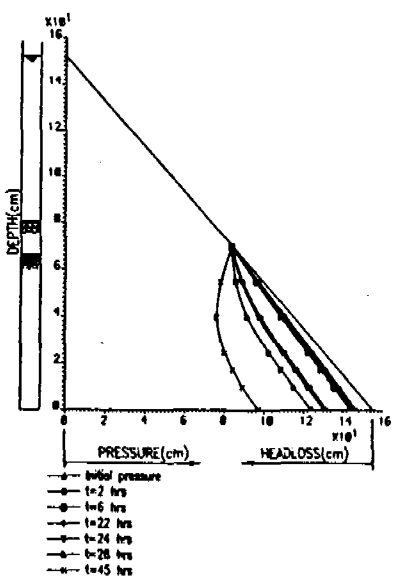
COMPARISON OF TOTAL HEADLOSS VARIATION WITH TIME FOR PEBBLE/SAND AND SAND BED;
C=1000mg/l;Vo(Sand)=1.8m/h;BED DEPTH=62.0cm

Fig. 4



HEADLOSS VARIATION WITH DEPTH AND TIME

Fig. 5



PRESSURE PROFILES FOR PEBBLE MATRIX FILTER

Fig. 6

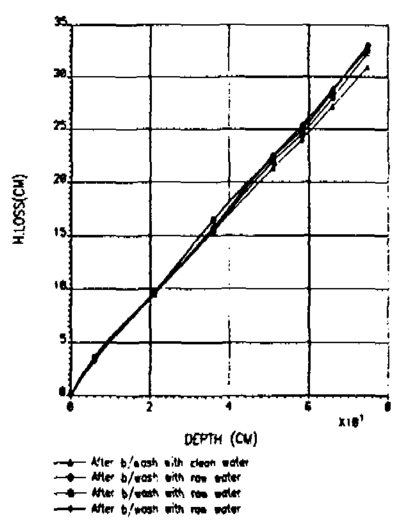
2.1 Filter cleaning

For efficient operation the filter must be maintained in good condition by removing the deposited material from the bed at the end of a filter run. Several cleaning processes have been investigated satisfactorily.

1. By draining down the filter and refilling it with raw water, and draining down again, the majority of the deposit can be removed ($>70\%$). By reverse flow washing with clean water the sand was fluidised to occupy all the spaces between pebbles.

2. The draining procedure was the same as in (1) but washing was first accomplished with raw water followed by clean water. This reduces the clean water consumption during back-washing by about 50%.

3. After draining down the filter it was back-washed using raw water only. This was sufficient to clean the filter of the accumulated clay so that when filtration was re-started the initial headloss was very similar to the one attained after washing with clean water (see Fig. 7). Several consecutive runs incorporating this cleaning method produced similar headlosses, effluent quality against time graphs and runtimes. One advantage in this method is that raw water would be plentiful during a monsoon period.



Fine Sand
d_e=0. 38mm
V_a (gross) = 1.2m/h

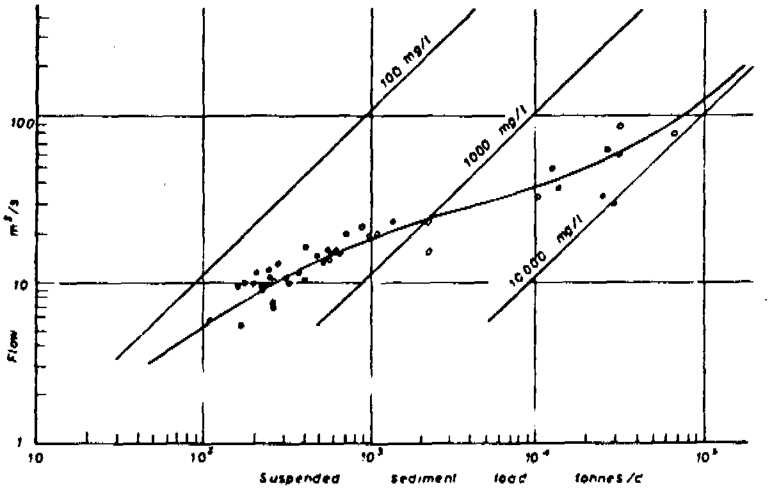
INITIAL H.LOSS AFTER B/WASH WITH DIRTY WATER

Fig. 7

3. DESIGN CRITERIA (PRELIMINARY)

When designing the PMF as in any pre-treatment process one important factor that has to be taken into consideration is the maximum suspended solids concentration in the river that would occur during a monsoon period. Suspended solids concentration would be expected to vary significantly with river flow as shown in Fig. 8 (Ives, 1983). However, these monsoon turbidity/suspended solids values for rivers are not readily available in most parts of the developing countries and therefore the appropriate authorities should be encouraged to collect this data in general and more particularly during a preliminary investigation of a pre-treatment project.

As in all sample analyses, such data should be obtained over a period to allow for seasonal changes, alterations in river flows, etc.



Dependence of suspended solids concentration on river flow

Sabaki River, Kenya

Fig. 8

The raw water quality determines the choice of treatment process. However, it is the gross approach velocity (V_a) that determines the plan area of the filters, which is a major contribution to the capital cost of the plant. Therefore choosing the right approach velocity is also an important decision in the design stage. A guideline to choose the gross approach velocity for the Pebble Matrix Filtration is given in the diagram below (Fig. 9).

GUIDE FOR THE SELECTION OF APPROACH VELOCITY (V_a) FOR THE PMF

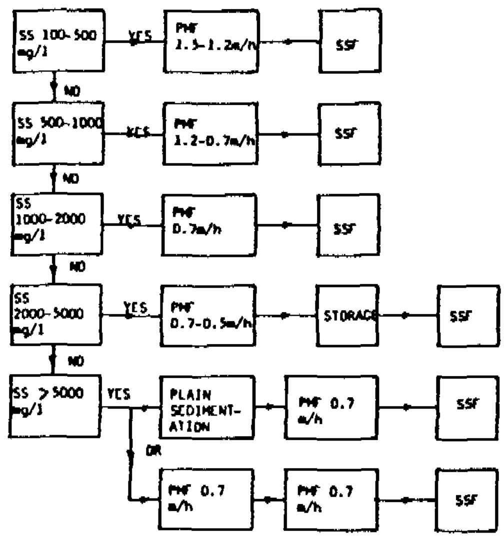


Fig. 9

3.1 Filter Area and the number of Units

Having decided the approach velocity V_a the area A can be calculated to be equal to or greater than Q/V_a where Q is the total hourly volume of water to be treated by the slow sand filters.

The number of filter units should preferably be at least 2 or more. The formula $n = \frac{1}{4}\sqrt{Q}$ suggested by Huisman & Wood (1974) can be used as a first approximation, when n is never less than 2 and Q is expressed in m^3/h.

3.2 Constituent Parts

In addition to the above it is necessary to give consideration to the following constituent parts of the PMF during its design stage:

1. supernatant water reservoir
2. free pebble bed
3. pebble/sand bed
4. underdrainage system
5. flow control devices
6. filter box

Since there is virtually no headloss within the pebble bed alone, the depth of the supernatant water reservoir can be fixed so that the head of water above the pebble/sand bed is sufficient to overcome the resistance of the pebble/sand bed and allow a downward flow through the bed.

Too short depths would create a negative head within the bed and consequently shorter filter runs. A value of between 1.0m and 1.3m would be a suitable figure to give a reasonable length of run time. Two factors should be borne in mind when determining the depth of the free pebble bed:

a) suspended solid removal in pebbles alone (Fig. 2)

b) sufficient depth to allow the sand bed to expand up into the free pebble matrix during backwashing.

If rounded pabbles are not available some other media such as road materials or broken bricks may be used. As for the sand bed both finer and coarser materials have been found to work satisfactorily under laboratory conditions, but the selection will actually have to depend on the locally available materials and pilot plant observations as in the case of pebble bed. Effective diameters of (d_e) between 0.35mm and 1.00mm have produced filtrates of extremely good quality as mentioned earlier. However, in addition to the removal efficiency other factors such as filter run time (T), maximum headloss (H_l), bed depth (L) and approach velocity V_a have to be considered as a whole.

Underdrains serve the purpose of collecting the filtered water and distributing the washwater during cleaning. The simplest form is a system of perforated lateral pipes packed round with gravel.

A floating effluent weir on the outlet is a suitable device to maintain the constant flow through the filter against the rising headloss due to clogging.

The filter box can be built either by reinforced concrete or brickwork.

A suggested plant layout is given in the diagram below (Fig. 10) :

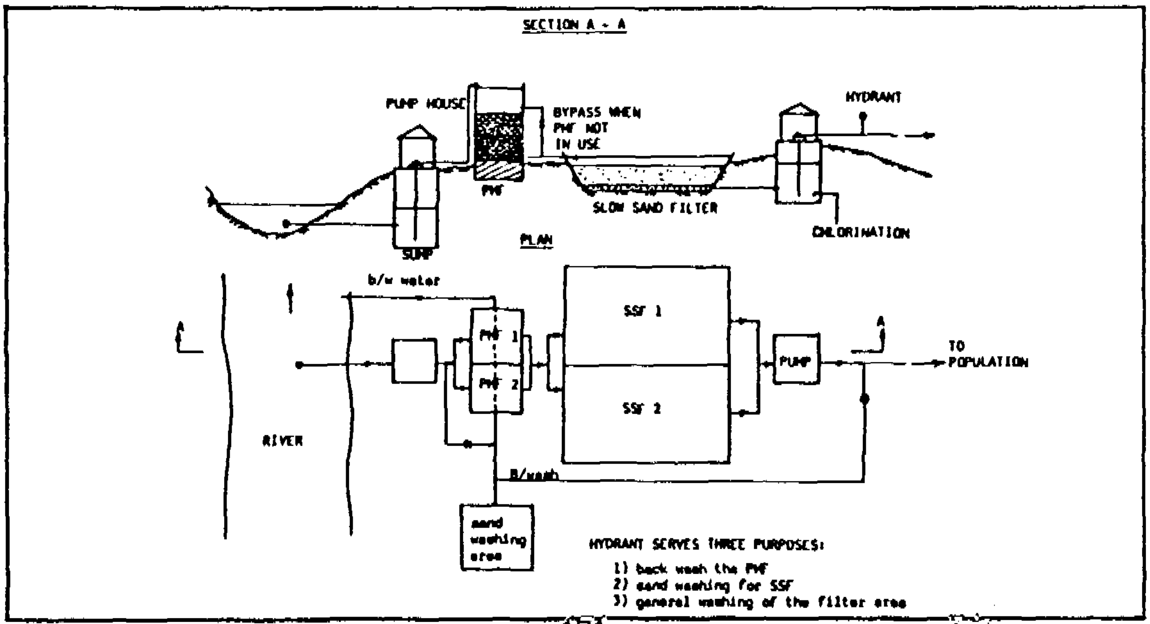


Fig. 10

CONCLUSIONS AND RECOMMENDATIONS

1. Pebble Matrix Filter is an appropriate pre-treatment method for slow sand filtration, suitable for semi-rural conditions in the developing countries where monsoon turbidities reach several thousand mg/l.

2. Final design, construction and operation criteria has to be decided upon completion of pilot plant studies.

3. It would be a worthwhile effort to encourage water authorities in the developing countries to prepare a record of turbidity/suspended solids and particle size analysis for rivers during monsoon periods.

LITERATURE

1. HUISMAN, L. and WOOD, W.E.
Slow Sand Filtration, 1974, WHO Geneva.

2. IVES, K J
Treatability of Water
Journal of IWES, 37 (2), 151, 1983.

3. KHABIROV, R.S., SLAVINSKI, A.S., KRAVTSOVA, N.V.
Optimization of the technological and design parameters of Skeleton-fill filters (in Russian)
Khimiya i Tekhnologiya Vody
Vol. 5 (4) 352-353, 1983.

ACKNOWLEDGEMENTS

Funds received by the U.K. Overseas Development Administration and University College London are gratefully acknowledged.

Thanks are also due to Mr I Sturtevant for his expertise in constructing the experimental apparatus.