PERFORMANCE EVALUATION OF DYNAMIC ROUGHING FILTRATION

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M.Sc. Thesis Report E.E. 137
August, 1994

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ACKNOWLEDGEMENTS

The author would like to thank his supervisor Prof. Dr. Ir. G.J.F.R. Alearts, and his mentors Ir. T. Ahsan and Ir. J.P. Buiteman of the International Institute for Infrastructural Hydraulic and Environmental Engineering (IHE), Delft, to their valuable advice and suggestions and continuous guidance and inspiration during the research period. He also like to extend his profound thank to Ir. J.T. Visscher of the International Water Supply Center (IRC), The Hague, who reviewed his drafts.

He is grateful to all CINARA and UNIVALLE families for their co-operation and assistance during carrying out the research project.

He is especially indebted to IHE-UNIVALLE-TU DELFT agreement for the fellowship extended to him and at the same to CINARA-IRC agreement for provided the finantial support to carried out the experimental phase in Cali, Colombia.

Finally he wishes to express deep appreciation to his wife and mother who encouraged and inspired him during the period of study.

August, 1994
Delft
The Netherlands

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Performance Evaluation of Dynamic Roughing Filtration

ABSTRACT

Dynamic Roughing Filtration (DyRF) has been proposed as a first pretreatment step before roughing filters and/or slow sand filter for turbid rivers. DyRF comprises a thin layer of fine gravel on top of a shallow bed of a coarse gravel with a system of underdrains. The influent is distributed into two directions: part of the flow (effluent) passes downward through the filter medium and then to the subsequent treatment units; and the other part (overflow) flows over the gravel bed and is normally returns to the raw water source.

Research was carried out in the Research and Technology Transfer Station of CINARA (Inter regional Center in Water Supply and Sanitation) in Cali, Colombia, where two pilot DyRF plants with declining rate filtration treated raw water from Cauca river. This research was set out to achieve a better understanding of DyRF processes. The specific objectives are:

1. To characterise the raw water quality
2. To study the particle removal process
3. To assess the impact of the surface overflow on surface particle scouring and on treated water quality
4. To suggest design guidelines

The major findings of this research are:

1. The Cauca river water has a typical characterization of untreated sewage which implies a very high sanitary risk when it is used for drinking water. Also a lot of very fine particles were found in the raw water (about 70% particles < 5 μm).

2. In DyRF sedimentation is the main particle removal process which occurs in two different locations: i) Plain sedimentation onto the exposed surface of the gravel bed. Here the removal efficiency is < 10%. ii) Sedimentation in the gravel bed.

3. The surface overflow did not have much impact on treated water quality. It may have a negative impact on the hydraulic perfomance of the DyRF.

4. The following removal efficiencies were obtained for DyRF units operating at filtration rates between 2.0 and 4.0 m/h and surface flow velocities ranging from 5 cm/s to 18 cm/s: turbidity from 50% to 52%, suspended solids from 83% to 87%, true color from 13% to 24%, total iron 55% to 84% and faecal coliform from 0.4 to 1.0 log.

5. Due to the limited and possibly negative impact of the overflow it is better to design the DyRF as a declining rate down flow roughing filter with maximum filtration rate of 4.0 m/h and gravel sizes between ϕ 6 to 13 mm in the upper layer, ϕ 13 to 19 mm in the middle and ϕ 19 to 25 mm at the bottom. The thickness of each layer can be taken as 0.20 m. It needs further research to explore if constant rate filtration would provide similar results.

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NOTATIONS

The following symbols and units are used in this thesis:

A	=	surface area of DyRF	(m^2)
d_s	=	particle diameter	($\mu m = 1x10^{-6}$ m)
d_{50}	=	median particle diameter of bed material	(m)
FCU	=	forming colony unit	
F_r	=	Froude number	(-)
f_c	=	friction factor	(-)
g	=	acceleration of gravity	(9.80 m/s^2)
h	=	water depth	(m)
I	=	hydraulic gradient	(m/m)
k_s	=	effective bed roughness	(m)
L_1, L_2, L_3	=	gravel layer thicknesses	(m)
NTU	=	nephelometric turbidity unit	
P	=	remaining concentration	(%)
p	=	porosity	(-)
Q_e	=	treated water, effluent	(l/s)
Q_i	=	raw water, influent	(l/s)
Q_o	=	overflow	(l/s)
R	=	removal ratio	(%)
R_e	=	Reynols number	(-)
S	=	overflow rate	($cm/sx10^{-3}$)
S_e	=	available surface area for sedimentation in gravel beds	(m^2)
S_l	=	surface loading	($cm/sx10^{-3}$)
SS_e	=	effluent suspended solid concentration,treated water	(mg/l)
SS_i	=	influent suspended solid concentration, raw water	(mg/l)
SS_o	=	overflow suspended solid concentration	(mg/l)
STD	=	standard deviation	
U	=	scouring velocity	(cm/s)
V	=	filtration rate	(m/h)
V_s	=	settling velocity	($cm/sx10^{-3}$)
X	=	flow velocity	(cm/s)
α	=	significance level	(%)
γ	=	kinematic viscosity	(m^2/s)
θ_{oc}	=	dimensionless critical shear stress	(-)
ρ	=	water density	(kg/m^3)
ρ_s	=	particle density	(kg/m^3)
τ_{oc}	=	critical shear stress	(N/m^2)
Ω	=	roughness coefficient	(-)

ABBREVIATIONS

APHA	=	American Public Health Association
ASCE	=	American Society of Civil Engineers
AWWA	=	American Water Works Association
CINARA	=	Inter-regional Center in Water Supply and Sanitation
CST	=	Column Settling Test
DHRF	=	Direct Horizontal-Flow Roughing Filtration
DyRF	=	Dynamic Roughing Filtration
HRF	=	Horizontal Roughing Filtration
IHE	=	International Institute for Infrastructural, Hydraulic and Environmental Engineering
IRC	=	International Water and Sanitation Centre
IRCWD	=	International Reference Center for Waste Disposal
NTU	=	Nephelometric Turbidity Unit
PCU	=	Platinum Cobalt Unit
SSF	=	Slow Sand Filtration
TU-DELF	=	Technological University of Delft
UNEP	=	United Nation Programme Development
UNIVALLE	=	Universidad del Valle
WHO	=	World Health Organization

CHAPTER 1
Introduction

1.1 Background

Experience in several countries has shown that for many communities Slow Sand Filtration (SSF) is a very appropiate alternative for drinking water treatment (Visscher et al. 1987). Nevertheless, the performance of such systems may not be sufficient to cope with the level of contamination of raw water sources. For instance, high level of faecal contamination (over 500 CFU/100 ml), Lloyd (1991), or medium to high turbidity levels (over 10 mg/l S_iO_2 for longer periods than a few weeks), Huisman and Wood (1974).
Under this situation, pretreatment is required to lower the influent turbidity and faecal contamination to the SSF to an acceptable level. Obviously the pretretment process should be such that its level of complexity is compatible with that of the SSF.

The pretreatment research carried out in Colombia on water from a highly polluted lowland river and from other Andean rivers, clearly shows the potential of combining two-stage roughing filtration with SSF, CINARA-IRC (1992).

The research reveals that roughing filtration as a pretreatment alternative helps to reduce the load in suspended solids and creates an essencial additional barrier against the transmission of disease carrying organisms and other harmful substances in the water, Galvis (1992). Also, preliminary studies by CINARA indicate that two-stage roughing filtration is a feasible alternative to lower the chemical consumption in conventional treatment plants.

In the two-stage roughing filtration, the Dynamic Roughing Filter (DyRF) provides a very good first stage in the treatment process. The results in the investigations in Cali-Colombia show that:

i) Costs are rather low (on average less than 5% of the capital investment in the treatment plant).

ii) DyRF contributes to the removal of suspended solids between 57% and 80% of average suspended solids loads in raw water in the range of 60 to 190 mg/l.

iii) Faecal coliform counts are reduced between 33% and 78% for raw water in the average range of 8476 and 73182 CFU/100 ml.

iv) Turbidity is being removed between 36% to 45% for average turbidity levels in the raw water between 25.8 and 238 NTU.

v) Iron removal is between 46% and 75%.

vi) Color removal is between 11% and 17%.

According to the available reviewed literature and taking into account the optimization done by CINARA, DyRF comprises a thin layer of fine gravel on top of a shallow bed of coarse gravel with a system of underdrains. The water entering the unit (Influent) passes over the gravel bed. Part of it is drained through the bed to the next treatment unit (Effluent) and the other part is returned to the river (Overflow). Under normal conditions the unit will gradually clog and will need to be cleaned every week or twice a week. When peak loads in suspended solids are being received clogging goes much faster. The clogging will reduce the flow to the subsequent treatment units, thus protecting the total treatment plant from peak loads, Galvis et al. (1992).

In spite of the good findings resulting from the DyRF performance, further improvements of this stage seem very well possible. Up to now DyRF systems have been basically studied as part of multi-stage filtration plants. Only initial research on DyRF itself is carried out by CINARA in collaboration with IRC and IRCWD.

A particular point for further study is the Impact of the surface overflow on the DyRF performance. Although the available reports do recognize the impact of the overflow on the behaviour of the units, there is not any systematic study of this parameter to support design or operational criteria. Data mentioned in the literature shows a wide variation in the ratio Overflow/ Effluent ranging between 0 and 10. This thus requires further research in order to achieve a better understanding of DyRF processes and also to develop adequate guidelines for design and operation and maintenance.

1.2 Objectives

The objectives of this study are:

i) To characterise the raw water quality

ii) To study the particle removal process

ii) To assess the impact of the surface overflow on surface particle scouring and on treated water quality

iv) To suggest design guidelines

1.3 Approach and Methodology

The main activities covered in the research are as follows:

i) The raw water quality of Cauca river will be analysed and results obtained from the bacteriological and phisical-chemical analysis will be compared with, existing water quality standards to assess the sanitary risk of the water.

ii) Sedimentation has been mentioned as one of the most important mechanisms in the removal of particles in roughing filtration. This process will be studied in DyRF by comparing the real efficiencies (established on the basis of the experimental data for suspended solids) and theoretical removal efficiencies established with column settling tests.

iii) The statistical comparison of removal efficiencies for each parameter and for different overflow will allow to analyse the impact of the overflow on the quality of treated water.

iv) The impact of the overflow on scouring of particles settled on the surface of the gravel will be estimated by comparing the remaining concentration of particles for different flow velocities.

v) On the basis of the results and data available from literature possible changes in the design will be presented to optimize the system.

To stablish this research pilot plants will be established in the Research Station of CINARA in Cali, Colombia, where water will be treated from Cauca river. The plants will operate under declining rate filtration and will be designed on the basis of the preliminary design criteria presented by CINARA, IRC (1993).

CHAPTER 2
Review on Pre-treatment Alternatives

2.1 General

In Europe the limitations presented in slow sand filtration application resulted in the development of pre-treatment techniques which initially were rather simple such as, long term storage and micro straining. Gradually, more complicated systems were put in place prior to slow sand filtration including coagulation, using chemicals, and flocculation followed by sedimentation and rapid sand filtration. These processes however hold little promise considering the conditions in most less developed countries. This situation has revitalized research in other pre-treatment alternatives that do not require the addition of chemicals and that are simple to operate and maintain. Most of these experiments involve rather small scale pilot plants and in fact only limited data are presented in literature and no evidence was found of comparative research of the different techniques. According to CINARA, IRC (1993) experiments reported in the literature focus particularly on the removal of suspended solids - some using kaoline suspensions - and that the performance of the systems is mostly being explained on the basis of sedimentation theory.

Pre-treatment Alternatives are categorized below and will be discussed in that order.

1. Infiltration Wells
2. Infiltration Galleries
3. Storage
4. Plain Sedimentation
5. Roughing Filters
   Dynamic Roughing Filters (DyRF)
   Downflow Coarse Sand Filters
   Downflow Roughing Filters in Series
   Upflow Roughing Filters
   Horizontal Roughing Filters (HRF)

2.2 Pre-treatment Alternatives

2.2.1 Infiltration Wells

Infiltration wells are wells dug or drilled in the banks of rivers. Depending on the water quality in the river and the soil conditions, water drawn from infiltration wells can be directly put into supply after disinfection or bringing it to a slow sand filter plant. Engels et al.(1989) have reported problems with the resuspension of iron and manganese oxides when levels of oxygen in the ground and river drop below 1 mg/l.
Another disadvantage of filtration wells is that changes may occur underground which may

result in reduction of the water flow and which can not be remedied by maintenance activities.

2.2.2 Infiltration Galleries

Infiltration galleries basically consist of perforated pipes placed in the river bed. If the natural permeability of the river bed is low, the material can be removed and replaced partially by other material such as gravel and sand. **Figure 2-1** illustrates two posibilities to install the filter material. Flow velocities applied in river bed filtration have been reported in the range between 0.25 and 1.5 m/h depending on the turbidity and the requirements to improve the water quality. Removal efficiencies for the system indicated in Figure 2-1 (b) have been reported by Salazar (1980) as 98% for turbidity removal from rivers with turbidity levels ranging from 48 to 200 NTU. A study carried out by CINARA, IRCWD (1988) in Colombia indicated however that the real efficiency of the systems is rather low and may reach only some 20%. In 1984, Nagarkar et.al. indicated that filtration galleries may not be suitable as a pretreatment method for treating raw water with colloidal turbidity greater than 100 NTU and may be considerably more costly for construction and maintenance than other systems.

The periodic blockage of the infiltration zone makes cleaning or repositioning of the material needed. In practice this maintenance is extremely complicated as the material is located under water in the rivers. Galvis et al.(1993) reported that these types of limitations have motivated the development of a modified system now being known as dynamic roughing filters.

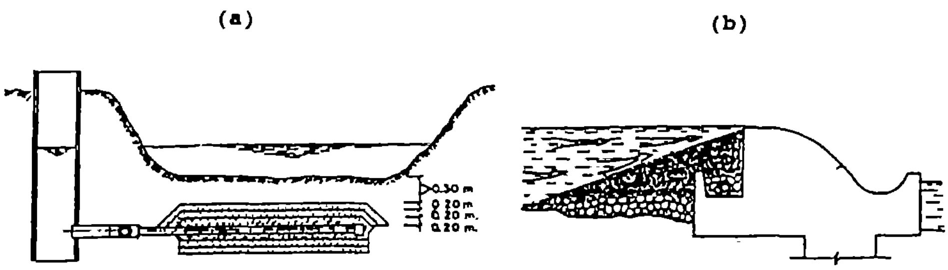


Figure 2-1 a) Infiltration Gallery without interfering in the flow gradient (Smet et. al. 1989) and (b) Infiltration Gallery Abstraction System with a weir Salazar (1980).

2.2.3 Storage

Schultz and Okun (1984) have reported that storage serves the following purposes:

1. It reduces the turbidity by natural sedimentation.
2. It prevents sudden fluctuations in raw water quality.
3. It improves the quality of water by reducing the number of pathogenic bacteria.

4. It can improve the reliability of the water supply system
5. It can be used to overcome periods of excess turbidity.

Long term storage also may have draw backs. Ellis (1985) stated that in the case of storage the problem of eutrophication may occur and monitoring of nutrient should be employed.

2.2.4 Plain Sedimentation

Plain sedimentation can very much contribute to reducing the level of suspended solids in the water source but has a limited impact for water sources with a turbidity of colloidal nature. Cleasby (1991) reported that in Cincinnati, plain sedimentation of water from the Ohio river reduced the suspended solids content from 170 to 100 mg/l, after a retention time of six days.

Two different applications can be identified: a) a system using a short retention time, less than one day, and b) a system with a very long retention time in the order of several days or weeks. For systems with a short retention time tests with sedimentation columns are recommended to explore the expected quality improvement. Cleasby (1991) experimentally recognized that these tests however are not suitable for estimating the effect of long term storage, as the process in the sedimentation column will not reflect the normal situation where other factors such as stratification because of temperature, and the influence of algae can be very important.

Long term storage is very common in England. WRC (1977) has found that in London turbidity reduction in large storage basins have reduced turbidity levels from 30 NTU to values below 4. Long term storage may also have a significant effect on the bacteriological quality. In 1974 Taylor confirmed that in the period from 1961 to 1970 the average faecal coliform counts of 6,680 per 100 ml were reduced to 249 per 100 ml. However the periodic blooms of algae made it necessary to introduce micro-screeners or rapid filters before the slow sand filters treating the stored water. The potential of long term storage for tropical countries has to be evaluated carefully before large scale application can be promoted.

Tilted plated settlers or tube settlers can be applied as sedimentation units with short retention times and may reduce the required surface area. Tilted plate settlers have been applied with good results in chemical coagulated water but hardly any experience exists in its application for non-coagulated water.

2.2.5 Roughing Filters

Roughing filters are basically boxes filled with gravel or coarse sand. The efficiency of roughing filtration is primarily based on the large surface area available in the gravel bed which facilitates the available mechanisms to remove impurities from the water. These mechanisms are of physical, chemical and biological nature. In the following section different types of roughing filters are described and are classified according to their characteristics and direction of flow.

2.2.5.1 Dynamic Roughing Filters (DyRF)

CINARA, IRC (1993) described the DyRF as a layer of fine gravel (3 to 6 mm) of some 0.2 to 0.3 m heigth placed over a layer of coarser gravel (12 to 25 mm) of some 0.2 to 0.4 m of height. In the bottom layer perforated pipes are placed as drainage system. **Figure 2-2** shows a longitudinal section.

The water flowing into the unit partly infiltrates into the gravel bed to the drainage system from where it will flow to the next unit. The other part flows over the gravel and usually returns to the raw water source. These units operate at a filtration rate which ranges from 1 to 9 m/h. Under normal operation conditions the fine gravel layer will gradually clogg as a result of the retention of suspended solids. When higher levels of suspended solids are being received the rate of clogging may be higher and - depending on characteristics of the particles - may lead to a complete blockage. Once or twice a week the gravel bed has to be cleaned by raking the fine gravel layer. Galvis et al.(1992) reported that every six to twelve months the filter material has to be removed, washed and re-installed in the unit to maintain the filtration capacity of the system.

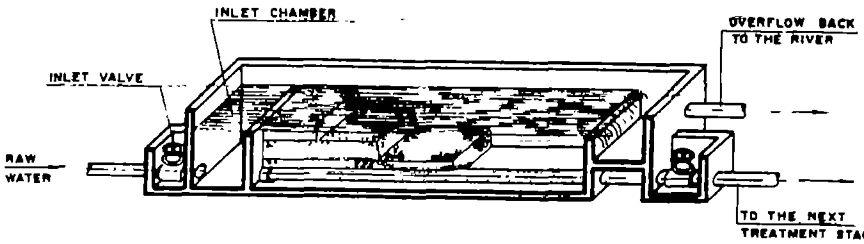


Figure 2-2 Dynamic Roughing Filter - Longitudinal Section.

2.2.5.2. Downflow Coarse Sand Filters

Since the begining of this century downflow rapid sand filters have been applied prior to slow sand filters in Europe and the U.S.A.. Cleasby (1991) reported that although these alternatives are somewhat different from conventional rapid sand filtration, as somewhat coarser sand is being used and no addition of chemical products is required, still the filters need to be backwashed frequently to clean the sand. This complicates the application of the technology and limits its use to areas where backwashing may be readily applied.

Rajapakse et al.(1989) have found another system comprising pebbles and sand called pebbled matrix filtration which is currently being investigated as pre-treatment to be combined with slow sand filtration. The stones have a size of some 50 mm and are surrounded by coarse sand. In this way rather a large quantity of suspended solids can be retained with a relative limited headloss development. An experimental pilot unit with a sand bed depth of 1.3 m operating at flow velocities between 0.5 and 1.5 m/h treating water with kaoline suspension ranging between 100 to 5000 mg/l of suspended solids produced effluents with suspended solid concentrations below 25 mg/l. On the basis of these results Rajapakse et.al.(1989) consider that this technology can be used as a pre-treatment method for surface water sources with a suspended solid content below 2000 mg/l, applying a filtration rate of 0.7 m/h.

Thus, downflow Coarse Sand Filters seem to have good potential for pre-treatment of surface water with high suspended solid content. However, its operation and maintenence requirements which include backwashing under considerable pressure may restrict its aplication.

2.2.5.3. Downflow Roughing Filters in Series

This alternative is based on the system introduced by Pueb-Chabal in Paris and other European cities in the begining of this century (Ellms 1919; cited by Cleasby 1991). In this system the water passes through three or more filter units. The first unit comprises gravel of 25 mm diameter and the following units comprise smaller gravel. Subsequently, the water was treated by a slow sand filter.

Perez et al.(1985), Pardon (1989), and Galvis et al. (1987) carried out studies in Peru and Colombia respectively. The findings stimulated new interest in this pre-treatment option. The **figure 2-3** indicates a schematic design of a Downflow Roughing Filter in Series with three units. The system has a moderate capacity to store sludge which makes periodic cleaning necessary. This is done by draining the filter with the help of a fast drainage valve connected to the drainage system.

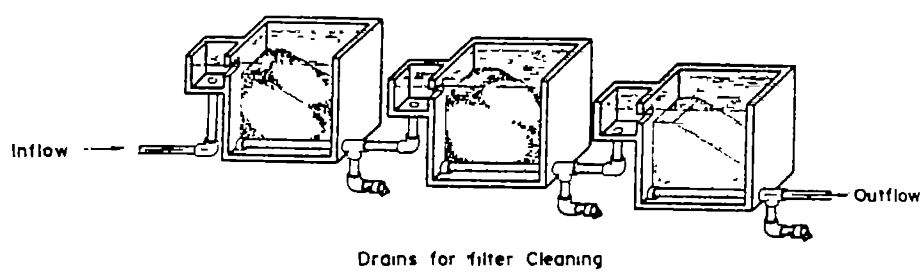


Figure 2-3 Schematic Diagram of a Downflow Roughing Filter in Series.

A study was made in Peru on a system with three units of 15 cm in diameter filled with 0.6 m of gravel. Gravel was used ranging from 50 to 12 mm diameter and flow velocities were applied ranging from 0.1 to 0.8 m/h. The turbidity of the Rimac river during the study period was approximately 50 NTU. Removal efficiencies obtained were 45% with respect to NTU for the highest filtration rate and 55% for the lowest rate. When turbidity levels were increased to 200 and 300 NTU through sludge dosing, efficiencies incresed to approximately 70% for the highest filtration rate and 90% for the lowest. On the basis of the experiments a filtration rate of 0.3 m/h was recommended to ensure an enfluent below 20 NTU, for surface water with turbidity levels below 300 NTU. It was also found that the flow velocity needed to clean the system was very high and had to be in the order of 90 m/h to transport the deposited material to the underdrain system.

In Colombia studies were also realized with pilot scale units similar to those utilized in Peru but with layers of graded gravel with grain sizes ranging from 6 to 18 mm, and filtration rates of 0.7 m/h were used. Quiroga (1988) and Galvis et al.(1989) recognized that the studies were complicated by the fact that the small diameter of the units (15 cm) and the low flow velocities made operation and maintenance of the units rather complicated. To obtain reliable results of such experiments very close monitoring is required and therefore it is better to use larger units. Quiroga (1988) and Galvis et al.(1989) reported that the first results of the studies with water from the Cauca river in Cali were obtained with raw water turbidities ranging from 20 to 100 NTU, apparent color levels ranging from 49 to 200 units, and faecal coliform levels in the order of 100,000 MPN/100 ml. The removal efficiency obtained for turbidity ranged between 50 and 90% for apparent color between 45 and 85% and for faecal coliforms between 70 and 99%. In subsequent investigations with technical scale units of 2m diameter the potential of this pre-treatment alternative was confirmed, but very little experience on full scale plants is available.

Frankel (1974) and Wolters et al. (1989) have reported that in Asia Downflow Roughing Filters were also used but instead of applying gravel other filter material was used such as coconut fiber. Raw water turbidities ranging between 25 and 130 JTU (Jackson Turbidity Units) were reduced to below 1 JTU by applying coconut fibre. However, this filter medium needs to be replaced every time when the filter needs to be cleaned which in this case was every month.

2.2.5.4 Upflow Roughing Filters

In upflow roughing filters the water flows upwards through a series of different gravel layers which are decreasing in size. Galvis et al. (1993) distinguish two alternatives namely, a) upflow gravel filtration in layers (URFL) when the gravel layers are installed in the same unit, as is shown in **Figure 2-4** and b) upflow roughing filtration in series (URFS) when the gravel layers are installed in two or three different units, as is presented in **Figure 2-5.**

Galvis et al.(1987) have expressed that the units have a moderate sludge storage capacity and therefore require periodic cleaning. This cleaning is done by draining the units by opening a fast drainage valve. The cleaning effect of draining can be increased by rapidly opening and closing the fast drainage valve.

Galvis et al.(1989) have pointed out that both alternatives of upflow roughing filtration have been evaluated in Cali, Colombia, using water from the River Cauca. For the first trials with

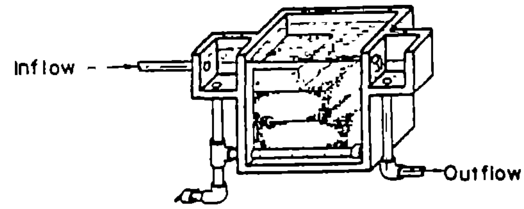


Figure 2-4 Overview of an Upflow Roughing Filter in Layers.

the URFS, filter columns were used each 15 cm in diameter filled with gravel ranging from 18 to 6 mm. The filtration rate which was applied was 0.7 m/h and raw water quality with a turbidity level ranging from 20 to 100 NTU, color from 50 to 200 TCU and faecal coliforms in the order of 100,000 MPN per 100 ml. Galvis et al. (1989) and Wolters et al.(1989) have reported that the results obtained indicated removal efficiencies for turbidity between 75 and 90% for apparent color between 50 and 70% and for faecal coliforms between 70 and 99.9%. The alternative of URFL evaluated for the same water quality showed lower removal efficiencies.

CINARA, IRC (1993) reported that between 1990 and 1993 both systems were evaluated using water from the Cauca river but using systems of 2 m diameter and gravel ranging from 25 to 1.6 mm. Filtration rates were applied ranging from 0.3 m/h to 0.75 m/h. Over the test period the raw water source had turbidity values ranging from 52 to 106 NTU, true color levels from 35 to 73 TCU, suspended solids levels ranging from 61 to 187 mg/l and faecal coliforms counts between 30,185 and 148,575 MPN per 100 ml. Removal efficiencies for the URFL were reported by CINARA, IRC (1993) ranging between 46 to 71% for turbidity, between 10 and 46% for true color, between 49 and 94% for suspended solids and between 73.3 and 98.4% for faecal coliforms. The efficiencies for the URFS ranged from 69 to 83% for turbidity, between 29 and 68% for true color, between 92 and 97% for suspended solids, and between 97.7 and 99.7% for faecal coliforms.

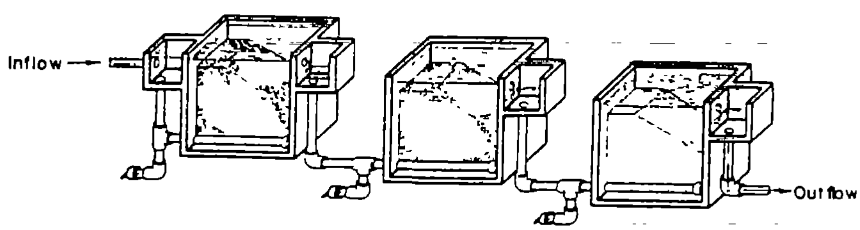


Figure 2-5 Overview of an Upflow Roughing Filter in Series.

2.2.5.5 Horizontal Roughing Filters (HRF)

Wegelin (1989) has pointed out that over the last 30 years this pre-treatment alternative has been used in combination with sand filtration for artificial recharge of ground water in countries such as Germany, Switzerland and Austria. Studies about this method have been established in Thailand as is reported by Than et.al.(1977), in Tanzania by Wegelin et al.(1981) and by Mbwette (1983).

The International Reference Center for Waste Disposal (IRCWD) has carried out laboratory studies in Switzerland with experimental pilot units using different kaoline suspensions. This study continued with a monitoring of full scale units constructed in different countries including Tanzania, Peru, Sudan and Colombia, Wegelin (1986).

CINARA, IRCWD (1988) has reported that in Colombia one experimental unit and three full scale plants have been constructed and monitored. The first experience with this alternative was obtained with an HRF of 7.14 m length which included a drainage system for cleaning purposes. The gravel size utilized ranged from 25.4 to 2mm and the filtration rate ranged from 0.3 to 0.6 m/h.

Figure 2-6 presents the schematic lay-out of a typical HRF which basically consists of the inlet, the filter bed, the outlet, and the underdrainage structure.

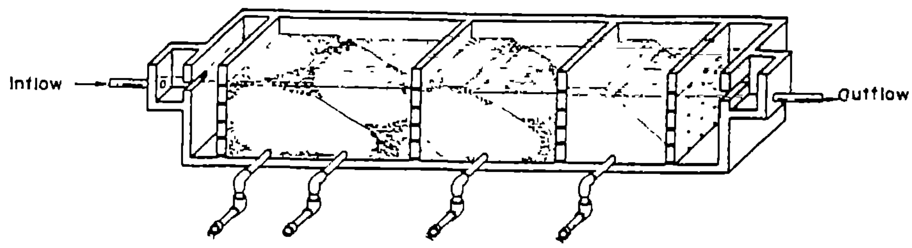


Figure 2-6 Schematic Lay-out of HRF.

Wegelin et al.(1986,1987) reported the use of a filter bed composed of gravel of different sizes varying from coarse (20 mm) to fine (5 mm) over 3 to 4 compartments in the direction of flow. The total length of the filter bed is in the magnitude of 9 to 12 m, the height is limited to 1.5 m to allow comfortable manual cleaning. The width of the filter box depends on the filter capacity and normally varies from 2 to 5 m.

More recently the International Institute for Infrastructural, Hydraulic and Environmental Engineering (IHE) based in The Netherlands carried out research to enhance the understanding of Direct Horizontal-Flow Roughing Filtration's (DHRF) behaviour and to further develop it, Ahsan et al.(1991). In DHRF coagulants are added prior to filtration. Lab scale pilot plant experiments systematically showed better removal efficiency of DHRF even at higher filtration rates (e.g. 5 m/h) when compared to HRF.

Chowdhury (1993) so far concluded that the feasibility of DHRF, proves to be an attractive and promising pre-treatment process on bench scale. Its application on full plant scale prior slow sand filtration and rapid sand filtration in semi-urban areas and the small towns of developing countries needs to be investigated and Ahsan et.al.(1991) added that the construction cost of DHRF is estimated to be about two times lower than HRF or a conventional flocculation-sedimentation system.

2.3. Dynamic Roughing Filtration

2.3.1. General

CINARA,IRC (1993) pointed out that the pre-treatment research carried out in Colombia on water from a highly polluted lowland river and from other Andean rivers, clearly revealed the potential of combining two-stage roughing filtration with slow sand filtration; Galvis (1992) reported that roughing filtration as a pre-treatment alternative helps to reduce the load in suspended solids and creates an essential additional barrier against the transmission of disease carrying organisms and other harmful substances in the water. Also, a preliminary study done by CINARA, IRC (1993) indicated that two-stage roughing filtration is a feasible alternative to lower the chemical consumption in conventional treatment plants.

In two-stage roughing filtration, the Dynamic Roughing Filter (DyRF) provides a very good first stage in the treatment process. **Figure 2-7** provides data from the evaluation brought about in Colombia, showing that the DyRF gave better results in suspended solids removal than the plain sedimentation units and the tilted plate settlers.

The results in the investigations in Colombia show that:

1. The DyRF construction costs are rather low (on average less than 5% of the capital investment in the treatment plant).

2. DyRF contributes to the removal of suspended solids between 57 and 80% of average suspended solid loads in raw water in the range of 60 to 190 mg/l.

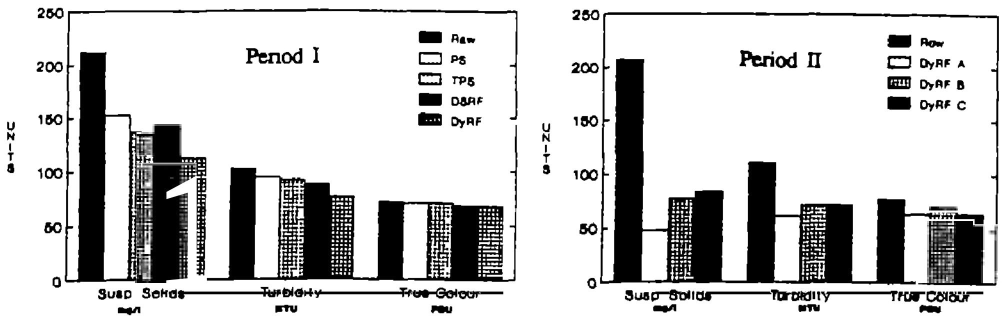
3. Faecal coliform counts are reduced between 33 and 78% for raw water in the average range of 8476 and 73182 FC/100 ml.

4. Turbidity is being removed between 36 to 45% for the average turbidity levels in the raw water between 25.8 and 238 NTU.

5. Iron removal is between 46 and 75%.

6. Manganese removal is between 52 and 60%, and

7. Color removal is between 11 and 17%.



PS = Plain sedimentation   DSRF = Downflow shallow roughing filtration
TPS = Tilted plate settlers   DyRF = Dynamic Roughing Filtration

Figure 2-7 Average levels of different contaminants in raw water and effluent of dynamic roughing filters and other conditioning processes. (CINARA-IRC, 1993)

2.3.2 Main Features

A general overview of a DyRF is presented in **Figure 2-8**. The water flow Q_t entering the filter is distributed into two flows. The first flow Q_e passes through the filter medium and on to the subsequent treatment units; the other flow Q_o normally returns to the raw water source. It is important to note that the filter medium grades from fine at top to coarse at the bottom where it is placed over the drainage system. In this way the system is designed to accumulate the suspended solids basically at the surface which very much facilitates its cleaning.

Due to the relatively coarse gravel which is being used, the headloss over the unit is very small. However, if the valve which controls the flow towards the other units of the system is not being opened, the flow through the filter will gradually reduce as a result of the small increase in headloss due to gradual clogging of the surface area. After some time too little water will flow through the units and then cleaning will be needed. In case of peak loads of suspended solids clogging will go much faster and the water flow through the other treatment units may be blocked completely. In this way the other units are being protected from excessive loads of suspended solids. Galvis et al.(1993) reported that the potential to react to increased loads in suspended solids is the reason for the name **Dynamic Roughing Filtration**.

Galvis et al.(1994) added that the filter bed is the most important element and requires special attention as it is crucial to the functioning of the system. The grading of the gravel (a layer with fine grains on the top and a layer with coarser grains at the bottom) differs from other types of roughing filters where grain size reduces and not increases in the direction of flow. Just raking the surface provokes the resuspension of the retained material which is easily carried away with the overflowing water. It is very important to keep this grading of the gravel as otherwise suspended solids will be carried deeper into the bed which would make cleaning much more difficult. A simple raking of the surface level would then not be sufficient to restore the filtration capacity.

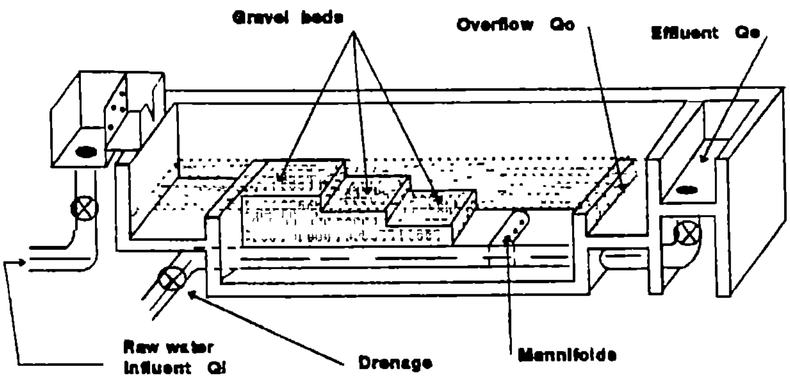


Figure 2-8 General overview of a dynamic roughing filter.

2.3.3. Design Criteria

Two types of DyRF can be designed depending on the type of raw water source. The first type of system is designed to reduce the quantity of suspended solids in the raw water, thus minimizing operation and maintenance problems in the other parts of the treatment plant. Galvis et al.(1992) state that this type of DyRF is a clear first sanitary barrier to improve the water quality because important reductions in suspended solids and also turbidity and faecal coliforms can be obtained.

A second design possibility for DyRF is used for rivers which normally transport limited quantities of suspended solids but occasionally show sharp peak loads of short duration. In this case the filter is designed to quickly block whenever the suspended solid load in the river is showing a rapid increase. This type of system can be seen as an automatic valve which blocks partially or totally the inflow to the other treatment units when the river is carrying too much suspended solids.

The velocity of the surface flow over the filter needs to be controlled because too high a velocity would carry away the fine gravel. Based on the preliminary experience Galvis et al.(1992) recommended that a surface flow between 0.05 and 0.15 m/s is a good range when the DyRF is designed to improve the water quality. A lower velocity below 0.05 m/s is proposed when the system has to protect against peak loads in suspended solids. In the second alternative the effect of sedimentation on the filter medium will help to block the surface quicly when peaks arrive at the unit, whereareas in the first alternative automatic cleaning of the surface is being strived for. A higher flow will ensure that less material will sediment on the surface of the gravel.

During a filter run the DyRF will operate as a declining rate filter and flow through the filter bed will gradually reduce with time. This reduction may be between 20 to 40% during an operation period of one week provided that no peaks in suspended solids are being received. Galvis et. al. (1993) reported that the design capacity needs to be at least 1.4 times the requiered capacity of the system.

The preliminary design criteria differentiating the two design alternatives and preliminary specifications of the filter medium are been indicated in **Tables 2-1 and 2-2** as recommended by CINARA, IRC (1993).

2.3.4. Limitations

In spite of the promising findings resulting from the DyRF perfomance, further improvements of this stage seem very well possible. Up to now DyRF systems have been basically studied as a part of multi-stage filtration plants. Only preliminary research on DyRF itself has been carried out in Colombia.

A particular point for further study is the **Impact of the Surface Overflow on the DyRF Perfomance**. Although the available reports do recognize the impact of the Overflow on the behavior of the units, there is not any systematic study of this parameter to support design or operational criteria. This thus requires further research in order to achieve a better understanding of DyRF processes and also to develop adequate guidelines for design.

Table 2-1 Preliminary Design Criteria for two DyRF design alternatives. CINARA, IRC (1993).

PARAMETER	PRINCIPAL ROLE	
	First barrier to improve quality of water	Protection against peak loads
Filtration Velocity (m/h)	0.5 - 3.0	3.0 -5.0
Range size of gravel in the upper layer (mm)	3.0 - 5.0	< 3.0
Surface flow (m/s)	0.05 - 0.15	< 0.05
Surface wash velocity (m/s)	0.2 - 0.4	0.1 - 0.3
Depth of bed (m)	0.6	0.4

Table 2-2 Specification of Filter Media for DyRF. CINARA, IRC (1993).

Position of layer	First barrier to improve qulity		Barrier against peak loads	
	Depth of layer (m)	Diameter (mm)	Depth of layer (m)	Diameter (mm)
Top	0.2	3.0 - 5.0	0.20	1.5 - 3.0
Middle	0.2	5.0 - 15.0	0.10	3.0 - 5.0
Bottom	0.2	15.0 - 25.0	0.10	5.0 - 15.0

CHAPTER 3
Review on Roughing Filtration and Sedimentation

3.1. Roughing Filtration

3.1.1. Basic Concepts

Wegelin (1991) reported that Roughing filtration using coarse and fine gravel is a simple and efficient method for the removal of suspended solids due to the large surface area available for sedimentation, absorption and biological and bio-chemical activities in the filters.

Roughing filters usually comprise filter material which gradually decreases in size from coarse in the first part of the system to relatively fine in last part. The filter media is often relatively coarse and of much larger size than material used in slow sand filtration or rapid sand filtration as is indicated in the following comparation given by Schulz and Okun (1984).

Slow sand filtration : 0.15 to 0.35 mm
Rapid sand filtration: 0.40 to 0.70 mm
Roughing filtration : > 2.0 mm

The filtration rate normally used ranges from 0.3 to 3 m/h. Schulz and Okun (1984) and Galvis (1992) reported that the applied rate will depend very much on the type of filter, the sanitary risk of the water source and the required treatment efficiency. Galvis et.al (1993) indicate that the treatment mechanisms in roughing filters can be classified as mechanisms of transport, attraction and purification.

3.1.2. Transport Mechanisms

This is the process which brings the particles into contact with the gravel and includes:

3.1.2.1. Screening

This process removes particles of larger diameter than the pores between the gravel grains. During the filtration process the diameter of the pores will gradually decrease and so smaller particles will be retained. Huisman (1984) recognized that this mechanism however, has only limited importance in roughing filtration due to the large diameter of the gravel (approximately 2mm), which corresponds with a pore of 500 μm. Wolters (1988) indicates that only at the end of the filter run when the size of the pores has decreased screening of suspended solids may be of some importance.

3.1.2.2. Sedimentation

This process removes suspended material in a similar way as in a sedimentation tank as is

reported by Ives (1975) and Wegelin (1986). The difference is that in sedimentation tank only the bottom is available as a precipitation surface whereas in roughing filters the total surface area of the grains is available.
Flocculation of smaller particles creates larger flocs which then also can precipitate on the gravel surface. Colloidal material however, is not being removed by sedimentation. Wegelin (1986) and Siripatrachai (1987) have reported that sedimentation is the main mechanism for particle removal in horizontal roughing filters. This mechanism will be discussed for DyRF according to specific data collected during the experimental phase.

Huisman (1986) reported that sedimentation removes particulate suspended matter of finer sizes than pore openings by precipitation upon the surface of the grains. A gravel bed with a pore space p, one m^3 of spherical filter grains with a diameter d_g has a gross surface area of $S_t = (6/d_g)(1 - p)\ m^2$.

Lebcir (1992) proposed that the available surface area for sedimentation in gravel beds (S_e) can be determined as:

$$S_e = \left(\frac{1}{6} x \frac{1}{2} x \frac{2}{3}\right) S_t = \frac{S_t}{18} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad (3\text{-}1)$$

In which, 1/6 = reduction factor for available upward surface, 1/2 = reduction factor for contact of adjacent grains and 2/3 = reduction factor for high flow which prevent deposition. Therefore, the equation (3-1) can be expressed for any volume of gravel bed (V_g) as follows:

$$S_e = \frac{1}{3d_g}(1-p)\, V_g \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad (3\text{-}2)$$

In gravel beds, the Surface loading (S_l) taken as quotient of the amount of water to be treated (Q_e) and the area of deposition will now be very small and hence the removal efficiency will be greater than for a conventional settling tank. For S_l the following equation can be written:

$$S_l = \frac{Q_e}{S_e} = \frac{3 d_g Q_e}{(1-p)\, V_g} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \quad (3\text{-}3)$$

3.1.2.3. Diffusion

The brown movement or molecular diffusion is caused by the collision of particles with water molecules. Huisman (1982) reported that this movement is important for the removal of colloidal material and does not affect particles above 2 μm. Removal efficiency by diffusion increases with the increase in size of the suspended particles and temperature and with the reduction of the flow velocity and the grain size.

3.1.2.4. Inertial and Centrifugal Forces

During the passage of the water through the filter the flowlines curve around the grains. Due to inertial and centrifugal forces particles may be forced to leave the flowlines and come into contact with the gravel grains. The removal efficiency increases with the increase of particle density and flow velocity and is reduced if larger grains are being used. Galvis et.al (1993) indicate that particle removal through this mechanisms is limited in roughing filtration because low filtration velocities are being applied and the gravel grains are relatively large.

3.1.2.5. Interception

Part of the particles in the water will stick to the sides of the grains and in doing so gradually reduce the diameter of the pores. Initially these particles will stick to the grains where they entered the filter but gradually part of the material deposited in this area will be transported further into the filter bed.

Yao et.al (1971) indicate that the removal efficiency through interception is independent of operational factors such as flow velocity. In 1967 O'Melia and Stum reported that the efficiency increases with increasing particle size and decreases with increasing gravel grains. Because of the large size of grains used in roughing filtration interception does not play an important role in the removal of impurities.

3.1.3. Attachment Mechanisms

The main forces that hold particles in place once they have made contact with the gravel are: electrostactic attraction, and mass attraction. A combination of these forces is frequently referred to as absorption.

3.1.3.1. Active Absorption

Mass attraction between particles (van der Waals force), is always present but very much decreases with the distance between the two. The impact of this force is therefore very limited beyond the distance of 0.01 μm. The attraction between opposite electrical charged particles (Coulomb force) is inversely proportional to the square of the distance between the particles. Like the van der Waals force it may supplement other transport mechanisms when these have brought a particle into the near vicinity of gravel grains having an opposite electical charge. As a result of the attachment of materials to the gravel the electrical charge of the gravel grains will change constantly thus attracting alternately particles with a positive or negative charge. It appears that active absorption is of limited importance in roughing filtration.

3.1.3.2. Passive Absorption

Mass attraction and electrostatic attraction although of minor importance to draw particles from the water, is considerably more important in holding the particles to the grain surface once they have been put in contact.

Particles of organic origen deposited on the surface of the gravel will quickly become the breading ground of bacteria and other micro-organisms. This will produce a stick slime layer to which particles from the raw water may easily attach. The organic material is gradually assimilated to become part of this sticky layer and may form large chains of organic material which may easily intercept smaller particles.

3.1.4. Purification Mechanisms

The purification processes whereby the trapped impurities on the filter grains are broken down are independent and therefore better described in combination than separately. The two principal processes are chemical and microbiological oxidation, but other biological processes may play a significant role as well.

3.1.4.1. Biochemical Oxidation

Through biochemical oxidation organic material is being converted into smaller particles and eventually into water, carbon dioxide and inorganic salts. Iron salts are also being transferred into several oxides which form a thick layer around the grains. The chemical and biochemical reaction only takes place on the surface of the grains where the catalic agents as well as large quantities of bacteria are present. These mechanisms only can take place after such agents have been attached to the grains. This biochemical oxidation plays an important role in the removal of color (true and apparent), and the removal of iron and manganese in roughing filters.

3.1.4.2 Bacteriological Activity

Wolters (1988), Smet et.al (1989), Wegelin (1989) among others, have recognized the importance of biological processes in roughing filtration. Through all the mechanisms indicated before, bacteria attach themselves to the surface of the grains. This concerns both bacteria which are useful for the removal processes as well as pathogene bacteria. To satisfy the energy requirement for their metabolism, bacteria oxidize part of the organic material they will encounter. Furthermore, they convert organic material into cell material for their growth. For certain types of bacteria such as faecal coliforms, the conditions in the filter are not very good, and these will gradually die off when attached to the grains.

3.2. Sedimentation

3.2.1 General

Sedimentation is the separation by gravitational settling of suspended solids from the water which carrries them. For sedimentation to take place, the density of the suspended solids must be greater than that of water (otherwise flotation would occur).

The suspended solids may already exist in the raw water and/or may result from a preliminary procces, such as hydrolysis, precipitation, etc. Generally speaking, there are two types of suspended solids: 1. discrete particles, which settle separately with a constant settling rate; and 2. more or less flocculated particles resulting from natural or induced agglomeration of the colloidal substances in suspension.

Four distinct types of sedimentation which reflect the concentration of the suspension and the flocculating properties of the particles were described by Fitch in 1958. These are: discrete, flocculant, hindered zone, and compression settling sedimentation.

In the discrete settling region, the particle moves down independently at a constant rate with a negligible amount of natural aggragation. With flocculant settling, one particle overtakes a slower settling particle and in passing collides, aggregates and consequently settles at a higher rate.

When particles are sufficiently close, interparticle forces are able to hold them in fixed positions relative to each other. As a result, the particles subside as a large mass rather than as discrete particles. This type of sedimentation is called zone settling. When the particles actually contact each other, the resulting structure of the compacting mass acts to restrict further consolidation. This is called compression.

3.2.2. Principles of Discrete Settling

3.2.2.1 Settling Theory

The terminal velocity of a settling discrete particle is given by Newton's formula and taken into account that Weber (1972) introduced the forces F_w, F_b, F_d, which are due to gravity, buoyancy, and drag respectively. The following equation can be written:

$$m.\frac{dV_s}{dt}=F_w.F_b.F_d \quad \text{. . . . .} \quad (3\text{-}4)$$

The impelling force equals the submerged weight of the particle, as: $F_w - F_b = (\rho_s - \rho).g.V_p$, in which, ρ_s and ρ are the particle and fluid density respectively, g the acceleration of gravity and V_p the particle volume.

The drag force is a function of the dynamic viscosity μ, ρ, V_s, and characteristic diameter d of the particle. Based on Newton's drag coefficient (C_D), the friction drag equals: $F_D = C_D.A_p.\rho.V_s^2 / 2$, where, A_p = projected particle area in the direction of flow and V_s = settling velocity. Incorporating dynamic behaviour of the particle into the above equation and taking into consideration that after an initial transient period, the acceleration, dV_s/dt, becomes zero and the V_s becomes constant, the general equation for the settling velocity of sphere particle of diameter d is:

$$V_s = [\frac{4}{3} \cdot \frac{g}{C_D} \cdot (\frac{\rho_s - \rho}{\rho}) \cdot d]^{0.5} \quad \ldots\ldots\ldots \quad (3\text{-}5)$$

In 1946 Camp and Fair et al. (1968) have presented the coefficient of drag C_D as a function of the Reynolds number, R_e. For laminar condition, $R_e < 1$, and the frictional resistance is only due to viscous force, $C_D = 24/R_e$ and the corresponding settling velocity is:

$$V_s = \frac{1}{18} \cdot \frac{g}{\mu} \cdot (\frac{\rho_s - \rho}{\rho}) \cdot d^2 \quad \ldots\ldots\ldots \quad (3\text{-}6)$$

which is known as Stocke's law.

Chaudhari and Tare (1993) have developed a simple regression model to predict $\mathbf{R_e}$ as a function of the non-dimensional numbers $C_D R_e^2$ (diameter term) or C_D/R_e (velocity term). Both these terms can be computed as follows:

1. For a known particle diameter (d)

$$C_D R_e^2 = (4/3)\ [g(\rho_s - \rho)/(\rho v^2)]\ d^3 \quad (3\text{-}7)$$

2. For a known velocity (V_s)

$$C_D/R_e = (4/3)\ [g v (\rho_s - \rho)]\ (1/V_s \rho) \quad \ldots\ldots \quad (3\text{-}8)$$

In which ν = kinematic viscosity.

The regression model can be expressed as follows:

$$\log R_e = -1.35873 + 0.987198(X_1) - 0.0251154(X_2)^2 - 0.00368094(X_1)^3 + 0.000273403(X_1)^4 \quad (3\text{-}9)$$

Where $X_1 = \log(C_D R_e^2)$ and $X_2 = \log(C_D/R_e)$

$$\log R_e = 0.7355 - 0.570226(X_2) + 0.025256986(X_2)^2 - 0.00239434(X_2)^3 \quad (3\text{-}10)$$

The values of R_e ($R_e = V_s d/\nu$) obtained in the above equations, can be used to calculate velocity and particle diameter respectively for raw water and overflow in DyRF.

3.2.2.2 Settling Velocity Distribution

The frequency distribution of the settling velocities for various particles is measured directly in the laboratory. The details of the experimental set up and procedure of the laboratory testing are explained in **Chapter 5** of this report. In this experiment, the samples of suspension were taken at different depths at pre-determined time intervals and were analyzed for turbidity and suspended solid concentrations.

Settling velocity is given by the quotient of the depth of the sampling point to the time elapsing from the begining of the test. A typical cumulative frequency distribution of settling velocities is shown in **Figure 3-1**.

3.2.2.3 Removal Ratio of Discrete Particles

Since 1946, Camp found that all particles in the settling zone of an ideal rectangular basin travel in a straight-line path as shown in **Figure 3-2**. They are determined by the vector sum of the settling velocity V_s of the particle and the displacement velocity V_H of the liquid.

The efficiency of the basin depends wholly on surface area (A) and the discharge (Q), which together constitute the displacement velocity. The detention time is calculated from the length of the basin (L). The equation for both the velocity and detention time is as follows:

$$X = V_H = \frac{Q}{A} = \frac{Q}{B.H} \; \ldots\ldots\ldots \; t_H = \frac{L}{V_H} = \frac{L.B.H}{Q} \quad \ldots \quad (3\text{-}11)$$

In which, B is the width of the tank. The velocity of the particle which settles a distance equal to the effective depth (H) of the tank in a travelling time (t_v) is the overflow rate ($S = V_o$) of the tank; i.e. $S = V_o = H/t_v$. All particles with a settling velocity $V_s \geq S$ are completely removed, while for particles with a lower settling velocity than S, the removal ratio amounts to: $h/H = V_s/S$. The overall removal efficiency is given by:

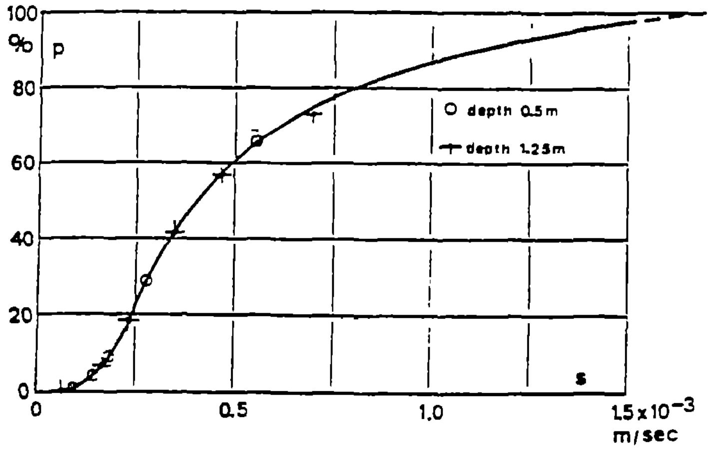


Figure 3-1 Cumulative Frequency Distribution of Settling Velocities.

$$R = (1 - p_o) + \frac{1}{S} + \int_{o}^{p_o} V_s \, . \, d \qquad (3\text{-}12)$$

which can be evaluated from the cumulative frequency distribution curve, suggested by Camp in 1946 and further by Ingersel et.al. in 1956, as ilustrated in **Figure 3-3**. The integral part represents the shaded area, which can be found graphically by drawing a horizontal line in such a way that the two shaded areas are equal.

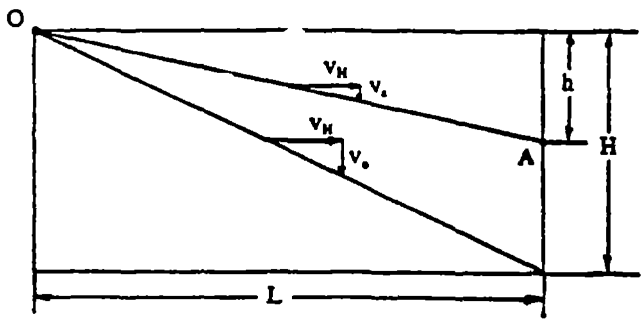


Figure 3-2 Paths Traced by Discrete Particles in an Ideal Basin.

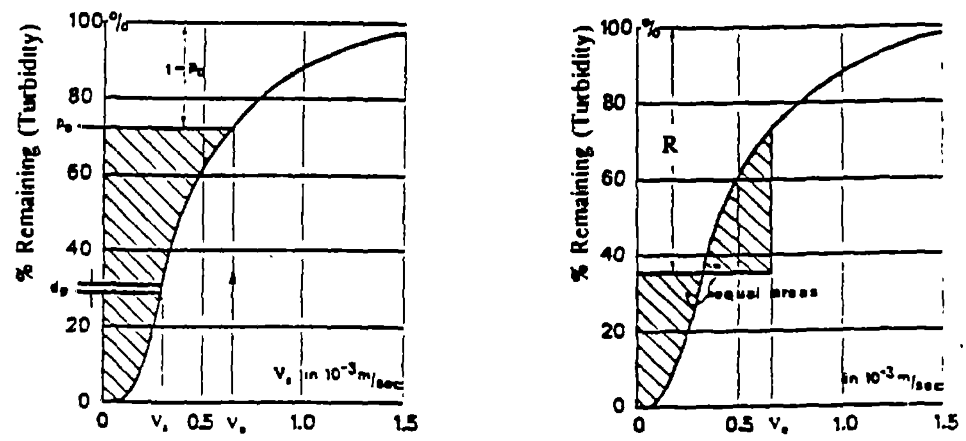


Figure 3-3 Graphical representation of removal ratio of discrete particles.

CHAPTER 4
Review on Scouring of Particles

4.1. General

Scour refers to the removal of material by running water. Galay et al.(1987) have reported that scour in rivers and canals is generally the result of secondary currents or vortices that occur in conjunction with river features such as bends, abrupt changes in flow directions, obstructions, constrictions, confluences, subsurface sills, control structures and piers of various types and sizes.

H. Chang (1988) pointed out that scour criteria are involved with physical conditions pertaining to the threshold of motion for the material. Therefore, determination of such criteria is the prerequisite for scour control and sediment transport.

4.2 Initiation of Motion

4.2.1. Introduction

Particle movement will occur when the instantaneous fluid force on a particle is just larger than the instantaneous resisting force related to the submerged particle weight and the friction coefficient.

The basic aspects that play an important role in the initiation of motion will be descibed in the following sections.

4.2.2 Critical bed-shear stress

The motion of the particle is under the interaction of two opposing forces: the applied force and the resisting force. The former is caused by the hydrodynamics of flow; the latter is associated with the submerged weight. The particle will be moved or entrained if the applied forces overcome the resistance. At the critical condition for entrainment, that is, threshold of movement, the applied forces are just balanced by the resisting force.

In a flowing stream, the forces acting on a grain of noncohesive sediment lying on the stream bed consists of the hydrodynamic drag (F_D), the hydrodynamic lift (F_L), and the submerged weight (W), as shown schematically in **Figure 4-1**. The drag F_D is in the direction of flow, and the lift F_L is normal to the flow. The lift force is not considered seperately in most analytical models because of the difficulty in determining its magnitude, Moreover, lift force is directly related to the drag. It depends on the same variables as the drag force and therefore the effect of the lift force is automatically taken into account by the empirical coefficients.

At the threshold of movement, the resultant of these two forces is along the direction of the

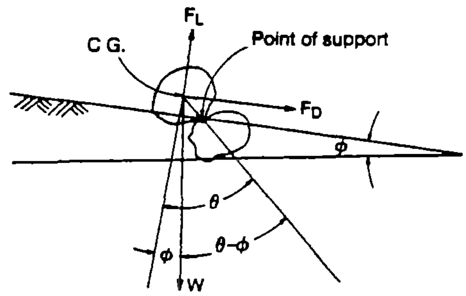


Figure 4-1 Schematic of forces on sediment grain on sloping bed.

friction angle; that is, the ratio of forces on the grain acting normal to the bed is equal to tan Θ.

$$\frac{\tau_c}{(\gamma_s-\gamma)\,d}=c\tan\theta \qquad (4\text{-}1)$$

This situation can be expressed as a function of critical shear stress τ_c. For horizontal bed, ϕ is equals to zero and the formula becomes:

$$\tan\theta=\frac{F_D+W\sin\phi}{W\cos\phi}\ . \qquad (4\text{-}2)$$

In which γ_s and γ are the specific weights for sediment and water respectively, c = constant value which must be determined experimentally and d = grain diameter.

4.2.2.1 Shields Diagram

Major variables that affect the incipient motion of uniform sediment on a level bed include τ_c, d, γ_s - γ, ρ and ν. From dimensional analysis, they may be grouped into the following dimensionless parameters

$$\frac{\tau_c}{(\gamma_s-\gamma)\,d}=F\left(\frac{U_{*c}\,d}{\nu}\right)\ .\ .\ .\ .\ .\ .\ .\ .\ . \qquad (4\text{-}3)$$

Where $U_{*c} = (\tau_c/\rho)^{1/2}$ is the critical friction velocity, and ν = kinematic viscosity. The left-hand side of this equation is the dimensionless critical shear stress and is often referred to as the critical Shields stress, τ_{*c}. The right-hand side called the critical boundary Reynolds number is denoted by R_{*c}. When any bed shear stress τ_0, other than τ_c, is used in the two quantities in **equation 4-3**, they become the Shields stress and boundary Reynolds number and are designated as τ_* and R_*, respectively.

Figure 4-2 shows the functional relationship of **equation 4-3** based on experimental data, obtained by Shields et. al. in 1936 on flumes with a flat bed. It is generally referred to as the **Shields diagram**. Each data point corresponds to the condition of incipient sediment motion or vanishing bed load.

Three different regions can be identified in Shields diagram based on Reynolds number (R_*):

1. Laminar region where R_* is less than about 2, the particle size is less than the thickness of the laminar sublayer and, hence, is enclosed in a thin laminar film. Since the boundary is hydraulically smooth, the movement is mainly caused by viscous action; the critical shear stress is inversely related to R_{*c} or τ_{*c} = C/R_{*c}, where C is a constant.

2. The transition region of intermediate boundary Reynolds numbers where the grain size has a magnitude similar to that of the laminar sublayer; therefore, the movement is partially influenced by viscosity. The critical Shields stress has a minimum value of 0.03 at the R_{*c} value of about 10.

3. The turbulent region presents large Reynolds numbers ($R_* > 400$), and the laminar sublayer is interrupted by the grain size. For this hydraulically rough boundary, the citical Shields stress has a constant value of 0.06, independent of the Reynolds number. In 1963, Zeller suggested a lower value of 0.047 in this region.

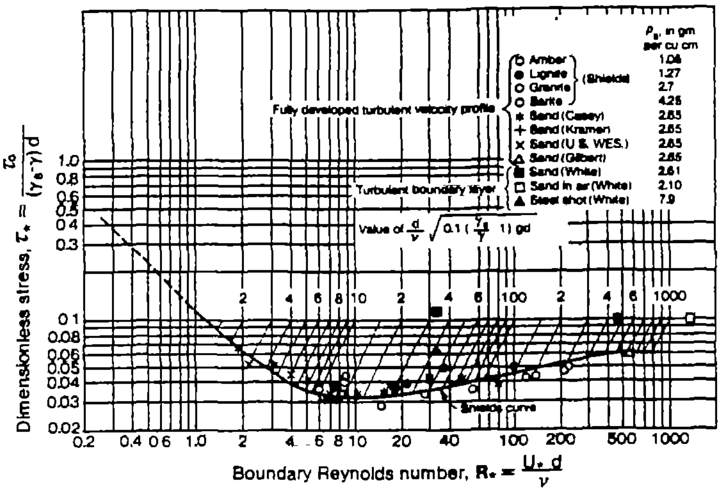


Figure 4-2 Shields diagram for incipient motion.

The Shields diagram contains the critical shear stress τ_c as an implicit variable that can not be obtained directly. To overcome this difficulty, since 1975 the American Society of Civil Engineers has published the ASCE Sedimentation Manual which utilizes a third dimensionless parameter:

$$\frac{d}{\nu}[0.1(\frac{\gamma_s}{\gamma}-1)gd]^{1/2} \quad \text{. . . . .} \quad (4-4)$$

which appears as a family of parallel lines in the diagran. From the value of the third parameter, the value of the critical Shields stress is obtained at an intersection with the Shields curve from which τ_c can be calculated.

4.2.3 Influence of Criterion

Van Rijn (1989) has reported that the complexity of defining a critical bed-shear stress for initiation of motion is mainly caused by the stochastical character of the driving fluid forces and the stabilizing resisting forces and by lack of an unambiguous definition of initiation of motion.

Since 1936, Shields found that the critical bed-shear stress is the zero transport rate after extrapolation of measured transport rates, another group of researches Neil et al.(1969) indecate that the critical bed-shear stress is the number of particles displaced per unit area and time and Delft Hydraulics (1972) experimentally recognized that it is the qualitative transport stage based on visual observation and found measurable quantities of transported particles at much smaller bed-shear stress than the Shields-values.

4.2.4 Influence of shape, gradation and size

Breusers (1988) has reported that experiments with particles of differents shapes show that the τ_{*c} parameter is not much affected by the shape of the particles when the nominal diameter (diameter that yields the same volume) is used as the characteristic parameter. Very flat particles have larger τ_{*c} values (factor 1.5 to 2).

In 1965, Egiazaroff experimentally recognized that Gradation has an effect when the size range is rather wide ($d_{90}/d_{50} > 3$), because the large particles will be more exposed, while the smaller particles are shielded by the larger particles. Armouring will occur when the bed-shear stress is not large enough to move the largest particles of the bed material. When there is no supply of smaller particles from upstream, all smaller particles will eventually erode, and the coarser particles will form an armour layer preventing further scour. Based on experimental results, the median diameter (d_{50}) of the armour layer will approximately be equal to the d_{85} of the initial bed material. This phenomenon has been observed downstream of weirs.

Mantz (1977) studied the initiation of motion of fine cohesionless flaky sediments with particle sizes (d_s) in the range of 10 to 100 μm and the **equation 4-5** represents the critical bed- shear stress (τ_{oc}):

$$\tau_{oc}=25d_s^{0.6}(\rho_s-\rho)gd_s \quad \ldots\ldots\ldots\ldots \quad (4-5)$$

4.2.5. Influence of bed forms

Bed forms are of interest in practice for several reasons:

i) Bed forms determine the roughness of a stream and a change in bed form can give changes in friction factors of 4 or more.

ii) Bed form and sediment transport have a mutual influence.

A generally accepted classification for bed forms is the following:

i) Lower flow regime: (Froude Number $F_r < 0.7$). The bed form is either ripples or dunes or some combinations of ripples and dunes all of which are triangular shape elements of irregular shape. The common mode of bed material transport is for the individual grains to move up the back of the ripple or dune and avalanche down its face.

ii) Upper flow regime: ($F_r > 0.7 \pm 0.2$). The usual bed form are plane bed or antidunes. The mode of sediment transport is for the individual grains to roll almost continuosly downstream in sheets a few diameters thick. Simons et al.(1972) believe that as soon as the sediment transport process is established, ripples and dunes are formed on the bed.

4.2.6. Influence of cohesive material

Sediment mixtures with a fraction of clay particles ($d_s \leq 4\ \mu m$) larger than about 10% have cohesive propierties because electrostatical forces comparable to or higher than the gravity forces are acting between the particles. Consequently, the sediment particles tend to stick together forming aggregates known as flocs whose size and settling velocity are much larger than those of the individual particles.

Fluid-sediment mixtures consisting of water, fine silts, clays and organic materials are generally called muds. When the bed consists of silty and muddy materials, cohesive forces between the sediment particles become important and depending on the type of clay minerals, the effect may be more or less pronounced. Biological activity at the bed may also influence the critical values for initiation of motion especially in muddy and silty environments as is reported by Van Rijn (1993).

Parchure and Trimbak (1985) found that critical shear velocities and erosion rates are greatly variable, depending upon type of mud and consolidation time. Tests with the actual sediment are necessary to obtain accurate values.

Fresh mud deposits have a very loose texture of mud flocs which already have a low density themselves. The wet bulk density of such a deposit may be within the range of 1050 to 1100 kg/m^3 of which 95% or more consists of water. In this stage the cohesive forces in the deposit are still very low and scouring can occur easily. A sand bed with small percentages of silts and clays (silty or clayed sand) already shows a distinctly increased resistance against erosion.

Van Rijn (1989) reported that most of the clay particles have a negative charge and the flocculation process for cohesive particles requires particle collisions: The three most flocculation process for cohesive particles requires particle collisions: The three most important collision mechanisms are: (1) The Brownian motions of particles (d_s < 4 μm) due to the random bombardment by the thermally agigated water moleculas, (2) turbulent mixing due to presence of velocity gradients in the fluid and (3) differential settling velocities because the larger particles have larger settling velocities and may therefore "fall" on the smaller particles.

Winterwerp et al.(1992) have recognized that the effective settling velocity (V_s) in the range of 0.001 to 1.0 mm/s decreases with increasing critical bed shear stress (τ_{oc}). Experiments carried out with cohesive sediments taken from western Scheldtwhich, the maximum τ_{oc} applied amounted to 0.2 (N/m^2), have shown the following ratio:

$$\tau_{oc}=0.3x10^{-4}V_s^{-0.588} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \quad (4-6)$$

In which V_s = effective settling velocity (m/s) and τ_{oc} = critical bed shear stress (N/m^2) .

4.2.7. Scouring velocity

Scouring velocity (U) can be expressed as a function of critical shear stress as follows:

$$U=\left(\frac{8\tau_{oc}}{\rho f_c}\right)^{0.5} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad (4-7)$$

$$f_c=0.24\left[\log\left(\frac{12\tau_{oc}}{\rho I k_s}\right)\right]^{-2} \quad (4-8)$$

In which f_c = friction factor, I = hydraulic gradient and k_s = effective roughness.

Experimental research shows that effective roughness is mainly related to the largest particles of the top layer of the bed. Van Rijn (1992) proposed that k_s can be taken as $k_s = \Omega d_{50}$, where Ω is ranging between 1 and 2 for gravel beds and d_{50} = median particle diameter of bed material.

CHAPTER 5
Materials and Methods

5.1. Introduction

The main objective of this research project is to identify the main treatment processes involved in DyRF and to assess the impact of different overflows (Q_o) on treated water quality as well as in scouring of particles and to suggest design guidelines for DyRF. The research was carried out in the Research and Technology Transfer Station of CINARA in Cali, Colombia, where two pilot plants with declining-rate filtration were arranged to treat raw water drawn from Cauca river. Two different types of experiments were conducted. First a series of tests in which the initial filtration rate in the DyRF was set at 2.0 m/h and different overflow rates were used to assess the influence on the water quality improvement. Thereafter three runs were made with higher initial filtration rates.

5.2 Description of the pilot plants

The overall lay out of the pilot plants is shown in **Figure 5-1**. The main components for each DyRF are briefly described below and a schematic presentation is illustated in **Figure 2-8**:

1. Inlet Structure: This is designed for control, measurement and flow distribution.

2. Main Box: This is the most important device in the DyRF structure and contains the filter bed and underdrain system. The dimensions were taken as: length = 1.50 m; width = 0.50 m; and depth = 0.70 m, including free bord taken as 0.10 m.
For filter beds, three different gravel layers were placed:
    Upper layer (thickness = 0.20 m): ϕ 6 to 13 mm, D_{10} = 5.1 mm, D_{30} = 6.4 mm and D_{60} = 8.6 mm.
    Middle layer (thickness = 0.20 m): ϕ 13 to 19 mm, D_{10} = 9.0 mm, D_{30} = 10.0 mm and D_{60} = 1.23 mm.
    Bottom layer (thickness = 0.20 m): ϕ 19 to 25 mm, D_{10} = 13.0 mm, D_{30} = 15.4 mm and D_{60} = 20.1 mm.
The Underdrain system was formed by manifolds ϕ 50 mm with 30 orifices ϕ 12.7 mm.

3. Outlet Structure: To facilitate the sampling and measurements activities two different chambers were constructed, one for overflow and one for treated water.
4. Drain Valve: One drain valve per each DyRF was installed in order to facilitate the operation and maintenance activities.

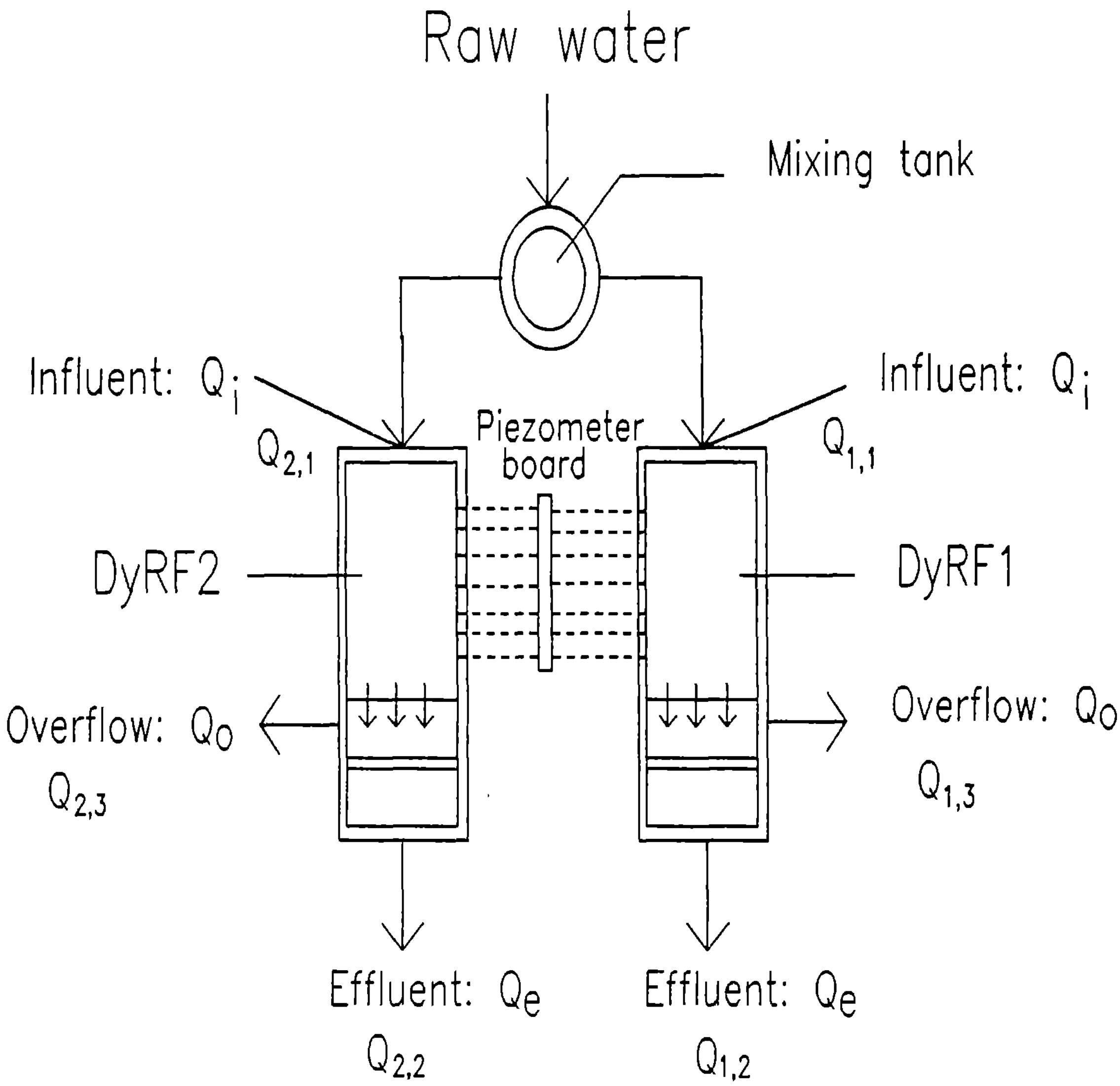


Figure 5-1 Layout of the pilot plants.

5.3 Description of the experimental set-up

Two periods can be distinguished in the research period. During the first, six nominal influent Q_i values were chosen based on: 1) DyRF guidelines for surface velocity; 2) The frequency distribution and particle diameters existing in the raw water; and 3) Hydraulic conditions in the existing infrastructure.
Based on the above criteria and in order to generate flow velocities in the range 0.05 to 0.25 m/s the initial Q_e was set at the beginning of each test at 2.0 m/h and the Q_i was set in such a way that initial overflows were obtained spread over three test runs of 0.42, 0.70, 1.0, 1.5, 2.0 and 2.65 l/s. Each test run was repeated three times.

For the second period Q_t was kept constant at 1.5 l/s and the initial filtration rates were set at 3.0, 4.0 and 5.0 m/h. Two filter runs were tested per each filtration velocity while gravel sizes and their layer thicknesses remained constant. An overview of the applied initial flow velocities is indicated in **Table 5-1**. A list of material and tools used in the research is presented in **Table 5-2**.

Table 5-1 Overview of initial flow velocities in the different research periods.

Research Period	Sub period	DyRF1	DyRF2	Initial Filtration rate V (m/h)
		Nominal Q_t		
		(l/s)	(l/s)	
January/94 to March /94	t_1 t_2 t_3	Q_3 = 1.00 Q_4 = 2.00 Q_6 = 2.65	Q_1 = 0.42 Q_5 = 1.50 Q_2 = 0.70	2.0 ; 2.0 2.0 ; 2.0 2.0 ; 2.0
March/94 to April/94	t_4 t_5	Q_4 = 1.50 ------	Q_4 = 1.50 Q_4 = 1.50	4.0 ; 3.0 ----; 5.0

Table 5-2 List of Materials, Tools and Equipments

A. For raw water feeding system:	9. Spectrophotometer (HACH DREL 2000) x 1
1. Mixing raw water constant head tank (1.2 m³ ferrocement tank) x 2	10. spectrophotometer (SHIMADZU UV-120-01) x 1
2. Distributuion system (pipes, valves,etc.)	11 pH meter (WTW PH-522) x 1
3. Electric sturrer (Siemens 0.5 Hp) x 2	12 Electrical sturrer (RW 20 DZM from TAMSON) x 1
B. DyRF Pilot Plant	13 Suction apparatus x 2
1. Rectangular filters (1.5 m length, 05 m width, 0 7 m depth) x 2	14 Whatman filter paper (934 AH 1.2 μm)
2. Gravel ϕ 6 to 13 mm approx. 0 15 m³	15. Bacteriological kit x 2
3. Gravel ϕ 13 to 19 mm approx. 0.15 m³	16. Digital cronometer x 2
4. Gravel ϕ 19 to 25 mm approx 0 15 m³	17 Drying oven, for use at 103 °C x 1
5. Mamifolds (ϕ 50 mm, 30 orifices ϕ 12.7 mm) x 2	18 Dessicator x 2
6. Piezometer board (metalic 0.70 x 0 80 m) x 2	19. Analisis balance x 1
7. Column settling apparatus (complete) x 5	
8. Turbidimeter (HACH 2100 A) x 1	

5.4 Overview of experimental parameters

The parameters and sampling frequency used in the research are summarized in **Table 5-3.** These parameters have been selected on the basis of information on roughing filtration in the literature and available equipment.

Water samples were taken at the same time at the different sampling points after which analyses were carried out. The parameters were: turbidity, suspended solids, volatile solids, true color, total iron and faecal coliform Counts. The selection criteria for physical, chemical and bacteriological parameters are supported below and their experimental tests were accomplished based on guidelines given by APHA (1986) and APHA (1989).

Turbidity: This is the universal parameter used to evaluate water treatment system. It is related with particle sizes present in the water as colloids or suspended matter which frequently is associated to high bacteriological contamination. High levels of turbidity can protect micro-organisms from the effects of disinfection, stimulate the growth of bacteria and exert a significant chlorine demand.

Suspended Solids: The amount of suspended matter removed by DyRF can be expressed in terms of suspended solids. Whatman filter paper of pore 1.2 μm was used.

True Color: Colour in drinking water may be due to the presence of coloured organic matter such as humic substances, metals such as iron and manganese, or highly coloured industrial waters.

Faecal Coliform Counter: The principal risks to human health associated with community water supplies are microbiological. Yet this comprises a wide range of bacteria and therefore faecal coliform counts are commonly used as indicator to determine the degree of bacteriological contamination.

Furthermore the following physical analyses were made: flow and head loss measurements, column settling test, mass density and mass balance. A general description of how these analyses were carried out is given below.

Flow Measurements: The volumetric procedure was accomplished and influent (Q_i), effluent (Q_e) and overflow rate (Q_o) were measured at the same time at three different points, twice a day.

Head Loss Determinations: To determine the filter resistance in different gravel layers the head loss readings of sampling from the calibrated piezometer board were recorded once a day Seven piezometers were placed at different gravel depths (0.0, 0.10, 0.20, 0.30, 0.40, 0.50 and 0.60 m). The first piezometer was installed just above surface bed and below water surface and the last one was placed in the manifold pipe ϕ 50 mm (this pipe conveyed the treated water into the outlet chamber).

Table 5-3 Overview of selected parameters and sampling frequency

Parameter	Samples/ Q_i	Runs/ Q_i	Samples/ filter run	Sampling frequency
1. Turbidity	225	3	75	every 8 hours (3)
2. Suspended solids	150	3	50	twice/day (3)
3. True color	27	3	9	three/week (2)
4. Total iron	27	3	9	three/week (2)
5. Faecal coliform	45	3	15	once/day (2)
6. Volatile solids	90	3	30	twice/day (2)
7. C S T	18	3	10	every 2 days (3)
8. Flow measurements	120	3	40	twice/day (3)
9. Head losses	-	3	-	once/day (4)(5)

(1) : one sampling point : Q_i
(2) : two sampling points: Q_i, Q_e
(3) : three sampling points: Q_i, Q_e, Q_o
(4) : Head loss readings in 7 piezometers
(5) : Suspended solids readings in 7 piezometers

. Average filter run: 5 days
. Q_i = 0.42, 0.70, 1.0, 1.5, 2.0 and 2.65 l/s
. Number of filter runs/Q_i : 3
. Mass balance : once/Q_i

Column Settling Test (CST): This indirect method was used to establish the **sedimentation diameter** and frequency distribution of settling velocities. Several testings during the research project (according to raw water quality variations and overflow rates) were accomplished and water samples to fill the CST were drawn from Q_i, Q_e, Q_o, at the same time. The quiscent settling column test apparatus used in this experiment consisted of cylindrical columns made of plexyglass of 0.30 m diameter and 1.0 m depth with four taps located at 0.26, 0.46, 0.61 and 0.71 m below the water surface.

Mass Balance: In order to know the particle removal efficiency for each DyRF run, mass balance was determined and the suspended solids variation for each sampling point (influent C_i, effluent C_e, and overflow C_o) was calculated. Moreover, the total storaged suspended solids concentration (TSSS) in each pilot plant was measured and added to mass balance according to:

$$Q_I . C_I . \Delta t = Q_e . C_e . \Delta t + Q_o . C_o . \Delta t + TSSS \quad (5\text{-}1)$$

In which Δt = filtration time. **Equation (5-1)** can be expressed as:

$$M_I = M_R + M_o + TSSS \quad (5\text{-}2)$$

or:

$$TSSS = M_I - (M_R + M_o) \quad (5\text{-}3)$$

$$TSSS = M_I - (M_R + M_o) \qquad (5-3)$$

Determination of Sediment Density: For different filter-runs three Imhoff cones were respectively filled with influent, effluent and overflow samples drawn from the DyRFs and the sedimentation process was given for three hours. After that, three different layers were observed at the Imhoff bottom. Each layer were carefully drawm and its sediment density determined. The experimental procedure is given by Van Rijn (1986).

CHAPTER 6
Presentations of Results

6.1. Introduction

The results have been organized in five blocks in line with project objectives:

1. Raw water characterization: Results of physical, chemical and bacteriological analysis of raw water during the research period are being presented. The following parameters are included: turbidity, suspended solids, true color, volatile solids, total iron and faecal coliform counts.

2. Column settling test (CST): On the basis of samples taken of Q_i, Q_e and Q_o and results of the CSTs a relation has been presented between remaining concentration (P) and settling velocity (V_s). These have been used to calculate the diameters of the particles and remaining concentrations as well as the removal ratio (R) as a function of overflow rate (S) for different Q_i.

3. Treated water characterization: Results are presented of physical, chemical and bacteriological analysis of samples taken simultaneously of Q_i and Q_e for different values of Q_i. This enables to establish the impact of different values of Q_i on the quality of treated water. The different flow velocities are indicated in **Table 5-1** (Chapter 5).

4. Scouring of particles: To analyse the impact of Q_o on the scouring of particles measurements of suspended solids concentrations in Q_i and Q_o, flow velocities, water depths and hydraulic gradients were done per each nominal Q_i.

5. Other measurements: The removal efficiency for suspended solids in a DyRF has been estimated on the basis of a mass balance. For each Q_i a mass balance was made as well as for the experiments with the different filtration rates.

Furthermore headloss was measured at different depths and flow velocities for Q_i, Q_e and Q_o were made per each Q_i.

Although during the project it was tried to keep Q_i constant this proved not possible due to operational problems not under control of the project. This has resulted sometimes in flow variations of 50%. To limit the impact of this variation only the filter runs have been taken into consideration were variations were less than 15% of the nominal Q_i value.

6.2 Presentation of results

6.2.1. Raw water Characterization

CINARA's research station receives water from the highly polluted Cauca river, which receives water from the highland rivers and untreated sewage from small and large settlements, as well as waste and runoff of agriculture lands.

During the research period the raw water in Cauca river had average turbidity levels ranging from 71 to 167 NTU with peaks ranging from 254 to 420 NTU, suspended solids ranging from 146 to 333 with peaks between 367 to 881 mg/l, volatile solids ranging from 54 to 64 with peaks between 86 and 154 mg/l, true color ranging from 60 to 109 with peaks between 102 to 157 PCU, total iron ranging from 13.37 to 15.76 with peaks between 25.60 to 57.50 mg/l, and faecal coliform counts ranging from 19440 to 64143 with peaks between 29000 and 242000 FCU/100 ml.

Figures 6-1 (a) to 6-1 (f) give an indication of the raw water quality and its variation for each of the the five test periods. More detailed descriptions are presented in **Appendix 1 Tables 1-1 a) to 1-1 l)**. This includes minimum, mean and maximum values, standard deviations and number of samples for the following parameters: turbidity, suspended solids, true color, volatile solids, total iron and faecal coliform counts. On the basis of the removal efficiency obtained in the first research period (averages ranging from 83.4% to 87.2%) suspended solids removal was selected as indicator for the experiments with different filtration velocities in the second research period.

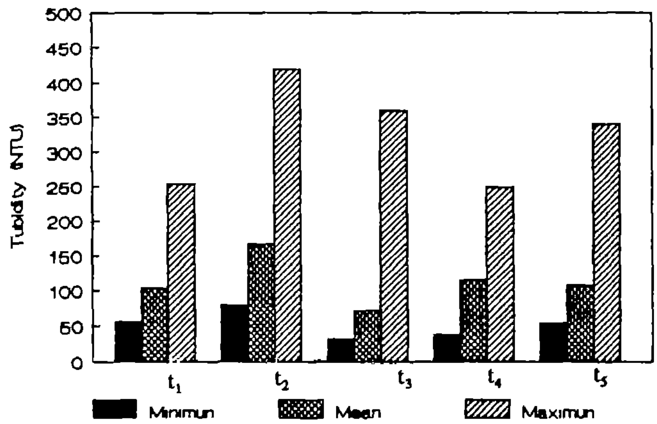


Figure 6-1 a) Raw water turbidity

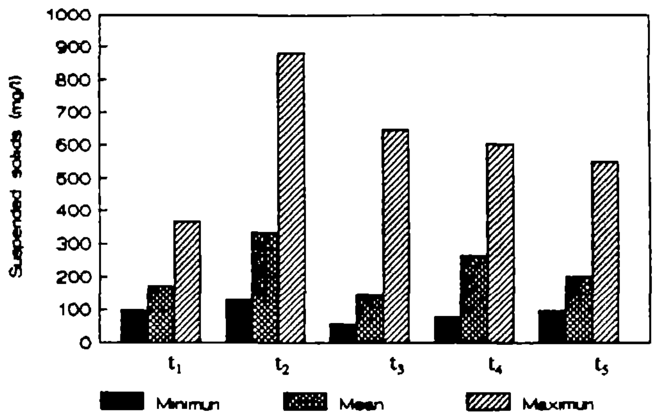


Figure 6-1 b) Raw water suspended solids.

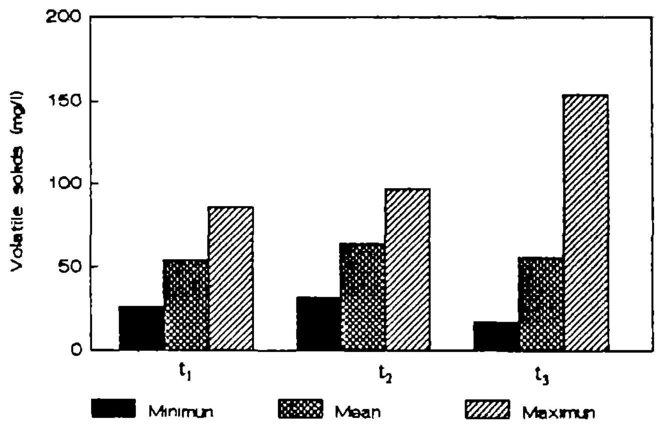


Figure 6-1 c) Raw water volatile solids.

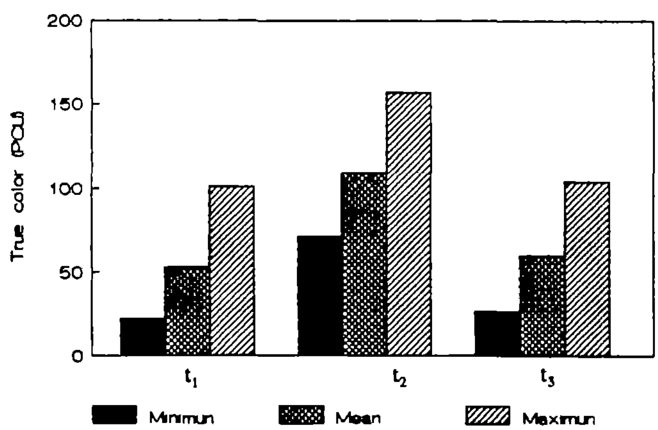


Figure 6-1 d) Raw water true color.

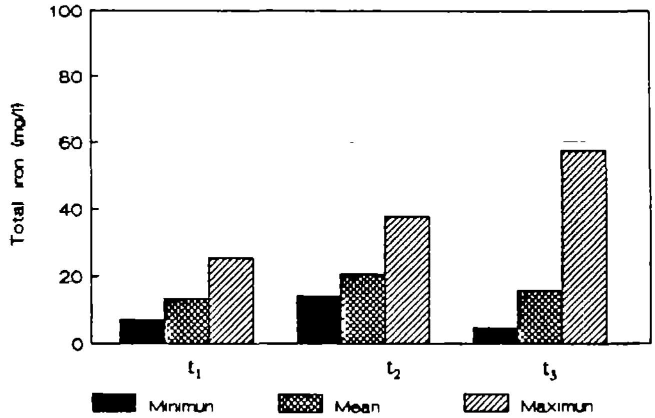


Figure 6-1 e) Raw water total iron.

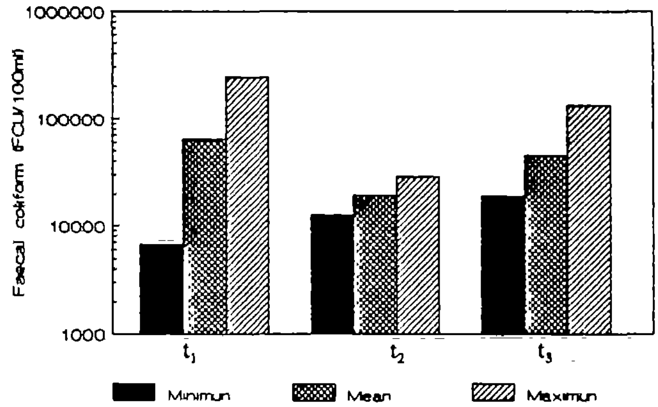


Figure 6-1 f) Raw water faecal coliform counts.

6.2.2. Column Settling Test

On the basis of the test results (**Appendix A Figures A-1 a) to A-1 c)**) a linear regression model was stablished which gives the relation between suspended solid concentration and raw water turbidity. This model is presented in **Appendix 2 Table 2-1**.

Figure 6-2 illustrates the typical pattern of the remaining concentration (P) and the settling velocity (V_s) for samples taken simultaneously from Q_i, Q_e and Q_o. The value of P is calculated as $(C_t/C_o)x100$ where C_o is the initial suspended solid concentration in Q_i at t = 0 and C_t are the measured values at different times and depths during the the CST. On the basis of Figure 6-2 it may be concluded that:

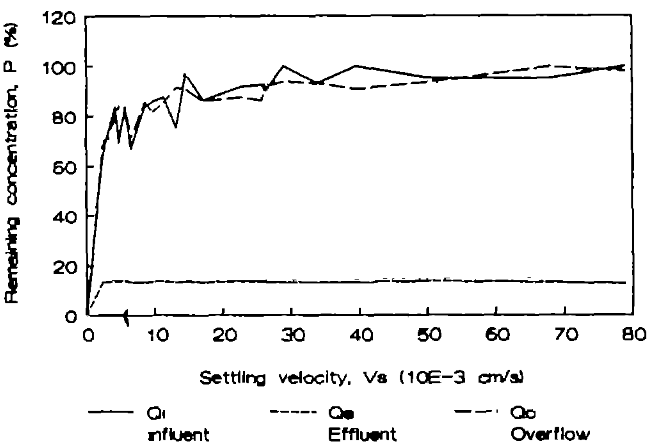


Figure 6-2 Cumulative frequency distribution of settling velocity for Q_i = 2.0 l/s and initial V = 2.0 m/h.

1. Very small differences exist between the curves for Q_i and Q_o for different settling velocities, ranging from 0% for V_s = 78.8x10^{-3} cm/s till 10.4% for V_s = 41x10^{-3} cm/s.

2. Both Q_i and Q_o show a remaining concentration P of 70% for V_s = 5.0x10^{-3} cm/s indicating that 70% of the remaining particles have a settling velocity V_s < 5.0x10^{-3} cm/s.

3. Only some 17% of the initial concentration of suspended solids in the raw water remains in the treated water thus indicating the impact of DyRF on suspended solid removal.

Laboratory analysis indicated three types of densities in the sediment of the CST. The samples were taken from Q_i and Q_o and determined at a raw water temperature of 25 ^{0}C (± 3.1 ^{0}C) using the procedure indicated in section 5.4. The following densities were found: ρ_1 = 2650 kg/m^3, ρ_2 = 2125 kg/m^3 and ρ_3= 1365 kg/m^3. Furthermore the average density of the deposit on top of the gravel in the DyRF was established as 1564 (STD = 78.2) kg/m^3.

With help of the equations presented in 3-8 and 3-10 the diameters of the particles (d_s) present in the raw water have been calculated for different densities ρ_1, ρ_2 and ρ_3 at 25 ^{0}C. Results are presented in **Table 6-1** and **Figure 6-3**. Table 6-1 also includes the values of the remaining concentration (P) for Q_i and Q_o taken from Figure 6-2. For V_s = 80x10^{-3} cm/s, P is about 100% which implies that all particles in Q_i and Q_o have a V_s < 80x10^{-3} cm/s. This indicates the presence of particles with d_s < 31, 38 and 64 μm for densities of 2650, 2125 and 1365 kg/m^3 respectively. For P of 70%, V_s is smaller than 5x10^{-3} cm/s which implies a high concentration of fine particles with d_s < 5, 7 and 13 μm respectively.

Table 6-1. Particle diameters for different sediment densities and their remaining concentrations as a function of settling velocities.

V_s x10^{-3} (cm/s)	Particle diameters d_s (μm)			P (%)	
	ρ_1	ρ_2	ρ_3	Q_i	Q_o
80	31	38	64	100	100
60	27	32	57	92	91
20	15	18	32	91	90
5	5	7	13	70	70
1	0.84	1.10	2.70	17	17

ρ_1 = 2650 kg/m^3 ρ_2 = 2125 kg/m^3 ρ_3 = 1365 kg/m^3

In **Table 6-2** the results are being presented for both research periods. This concerns:

i) Variation of Q_i, Q_e and Q_o throughout the filter runs for different initial Q_i's and for different filtration rates.
The reduction of Q_e and the increase in Q_o during the filter run can well be observed.

ii) Suspended solid concentrations in Q_i, Q_e and Q_o. These figures show a wide variation due to the variation in the raw water. Table 6-2 does not include data on suspended solids for the overflow when Q_i = 0.42 l/s because when V = 2.0 m/h Q_o = 0.

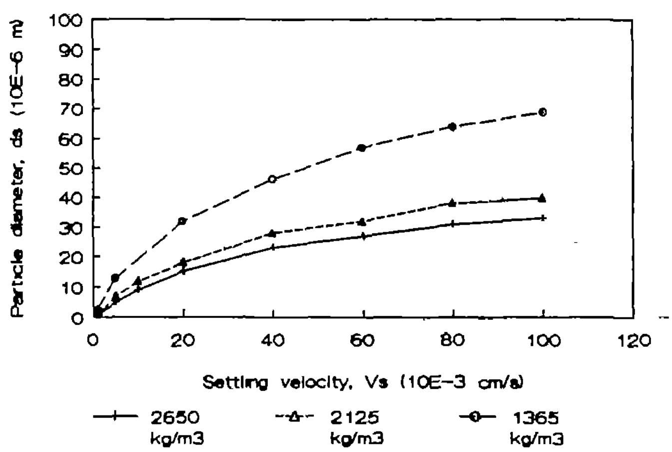


Figure 6-3 Particle diameters for different sediment densities as a function of settling velocities.

In order to get a better understanding of the results a general description of the normal performance of the DyRF during one test period is given.

Table 6-2 Experimental data for different nominal Q_i and over filter run

Time (days)	Flow variation (l/s) Q_i	Flow variation (l/s) Q_e	Flow variation (l/s) Q_o	Suspended solids (mg/l) SS_i	Suspended solids (mg/l) SS_e	Suspended solids (mg/l) SS_o
Nominal Q_i = 1 0 l/s V = 2.0 m/h run: 1						
1	1.13	0.42	0.71	133	32	94
2	1 09	0.30	0.78	246	59	226
3	1.08	0.18	0.90	169	35	143
4	1.05	0 12	0.93	143	15	125
5	1.07	0 05	1.02	251	10	228
Nominal Q_i = 2.0 l/s V = 2 0 m/h run: 2						
1	2.04	0.44	1 60	310	78	261
2	1.98	0.33	1.65	196	36	178
3	2.03	0.06	1.96	239	23	231
4	1.96	0.01	1 95	182	6	178
Nominal Q_i 2.65 lps V = 2 0 m/h run· 3						
1	2.41	0.38	2.03	79	16	63
2	2.69	0.14	2.55	325	45	362
3	2 26	0 02	2.24	79	7	80
4	2.70	0 01	2.69	107	9	104
Nominal Q_i = 1.50 l/s V = 3.0 m/h run: 1						
1	1.40	0.65	0.75	442	165	384
2	1.66	0.24	1 42	597	136	484
3	1.65	0 05	1 60	180	8	176
Nominal Q_i = 1.50 l/s V = 5.0 m/h run 1						
1	1.48	0.99	0.49	121	33	86
2	1.47	0.97	0.50	154	56	130
3	1.49	0.87	0.62	151	63	125
4	1.55	0.81	0.68	121	52	99
5	1.56	0.63	0.93	105	49	93
6	1.40	0.01	1.40	97	23	88

Time (days)	Flow variation (l/s) Q_i	Flow variation (l/s) Q_e	Flow variation (l/s) Q_o	Suspended solids (mg/l) SS_i	Suspended solids (mg/l) SS_e	Suspended solids (mg/l) SS_o
Nominal Q_i = 0.42 l/s V = 2.0 m/h run: 1						
1	0.44	0.44	0.00	66	40	-
2	0.42	0.42	0.00	223	86	-
3	0.42	0 32	0.00	155	79	-
4	0.41	0.26	0.15	129	32	-
5	0 36	0.14	0.22	231	65	-
Nominal Q_i = 1.50 l/s V = 2.0 m/h run 2						
1	1.54	0.44	1.10	310	82	256
2	1.53	0.33	1 20	185	54	131
3	1.58	0 04	1.54	251	16	231
4	1.53	0 01	1.52	170	5	164
Nominal Q_i = 0.70 l/s V = 2.0 m/h run: 3						
1	0.71	0.42	0.29	65	14	63
2	0.72	0.18	0.54	336	45	312
3	0.65	0 03	0.62	77	6	51
4	0 68	0 03	0.65	100	7	80
Nominal Q_i = 1.50 l/s V 4.0 m/h run: 1						
1	1.20	0.76	0.44	470	205	391
2	1.38	0 31	1.07	595	169	494
3	1.40	0.04	1 36	208	14	180

Under normal conditions the inflowing water Q_i is divided into two flows a horizontal flow (Q_o) and a vertical flow (Q_e).

i) **The horizontal flow**, varies along the length of the filter and is minimum at the begining of a filter run and maximum when the filter is clogged. (see Table 6-2). The typical flow variation is also shown in Figure 6-14.
During the development of the filter run, gradually a sludge deposit is formed with a variable depth between 0 at the beginning and 15 mm at the end of a filter run, with the greatest depth in the first half of the surface area.

ii) **Vertical flow**. This flow enters into the gravel bed gradually causing the clogging of the filter. Q_e varies and is maximum at the beginning and gradually reduces during the filter run. (see Table 6-2 and Figure 6-14).

For **horizontal flow** the difference between the suspended solid concentration in Q_i and Q_o determines the horizontal removal efficiency for each overflow rate ($S = Q_o/A$, A = surface area). This efficiency has been calculated for each Q_i based on experimental data shown in Table 6-2.

The theoretical removal efficiencies R (removal ratio) have been calculated for different Q_i's on the basis of sedimentation theory with help of the cumulative frequency distribution and based in the equation 3-12. **Figures 6-4 a) to 6-4 e)** show these theoretical and the measured suspended solid removal efficiencies as a function of S for different Q_i's.

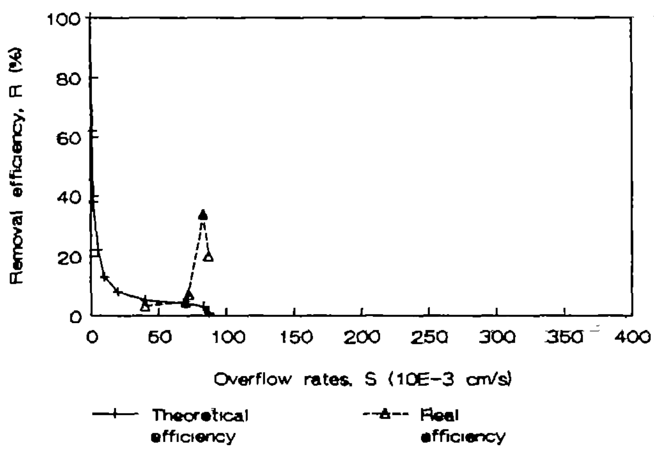


Figure 6-4 a) Removal efficiency comparation for Q_i = 0.70 l/s.

Figure 6-4 a) shows efficiencies below 5% in the first days of the run and thereafter the efficiency increases to 36% and then drops to 20%. These values are not in line with the data obtained in subsequent runs and as there is no logical explanation for this behaviour it is most likely resulting from an experimental mistake. The other Figures 6-4 b) to 6-4 e) show high efficiencies at the begining of the filter run, with a maximum of 29% for Q_i = 1.0 l/s and S = 94.7x10^{-3} cm/s, but thereafter these efficiencies drop to between 2 and 9%. The higher values clearly divert from the theoretical values and they can not be logically explained on the basis of sedimentation theory. A possible reason could be the design of the system in combination with cleaning. As a routine maintenance operation the top gravel layer of the DyRF was removed after each run and replaced after washing. It was observed that in the beginning of each run the gravel bed was slightly higher and extended some 5 to 10 mm over the overflow weir. After one day the gravel pack settled and reduced in height till the level of the weir. The initial situation makes that the water overflowing the weir is actually passing through the gravel bed whereas the second day this no longer is the case. Therefore it may be concluded that the data obtained for the second and subsequent days are more in line with normal operation conditions and show average removal efficiencies of less than 10% which is much more in line with theoretical sedimentation efficiencies.

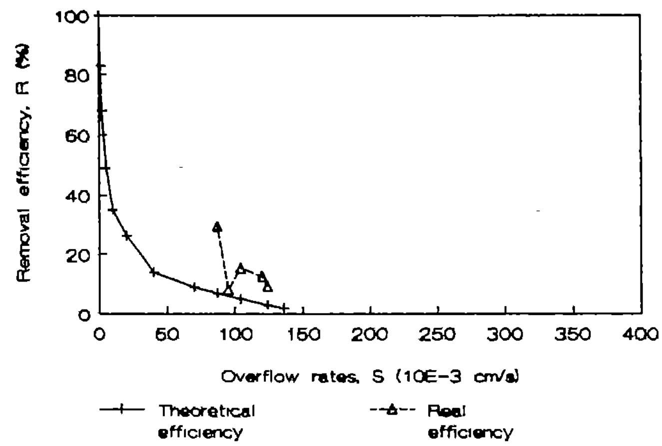


Figure 6-4 b) Removal efficiency comparation for Q_i = 1.0 l/s.

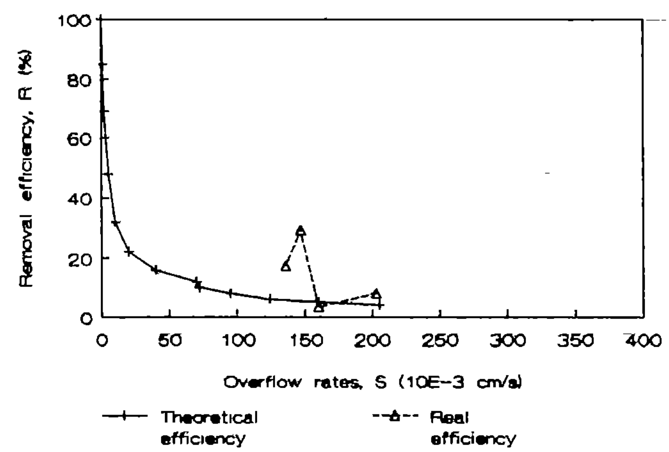


Figure 6-4 c) Removal efficiency comparation for Q_i = 1.50 l/s.

Figure 6-4 e) shows a negative efficiency for Q_i = 2.65 l/s this is a results from the high flow velocity which is able to produce scouring of deposited material.
In **Table 6-3** the theoretical and measured efficiencies are being presented for the beginning of the filter run (situation 1) and for the end (situation 2) for different Q_i's.
In situation 1 the difference in theoretical and measured efficiency is considerable with

averages of 14 and 4.7% respectively which is probably due to the filtration effect before the water passes over the weir. For situation 2 and particularly for the experiments with higher flow velocities the theoretical and measured efficiencies are much closer. The efficiency measurements of 20% for Q_i = 0.7 l/s and 29% for Q_i = 1.0 l/s have been excluded from the calculations as they are probably resulting from experimental mistakes.

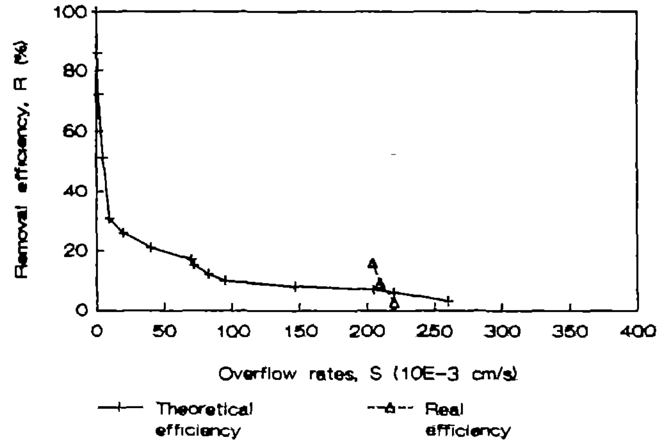


Figure 6-4 d) Removal efficiency comparation for Q_i = 2.0 l/s.

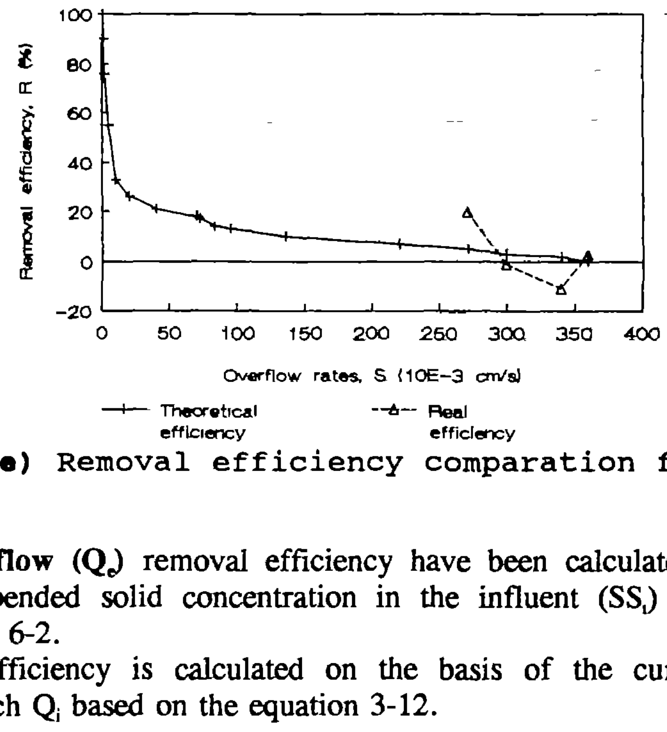


Figure 6-4 e) Removal efficiency comparation for Q_i = 2.65 l/s.

For the **vertical flow** (**Q_e**) removal efficiency have been calculated on the basis of difference in suspended solid concentration in the influent (SS_i) and effluent (SS_e) presented in Table 6-2.
The theoretical efficiency is calculated on the basis of the cumulative frequency distribution for each Q_i based on the equation 3-12.

The variation of the efficiency over the filter run is expressed in terms of surface loading (S_l), which is a function of Q_e and the available area for sedimentation (S_a). These have been calculated with help of equations 3-1, 3-2 and 3-3. **Figures 6-5 a) and 6-5 b)** show the efficiencies for different S_l values for Q_i = 0.70 and 2.65 l/s over the length of the filter run. **Table 6-4** shows the theoretical and measured efficiencies in suspended solid removal for different values of Q_i at the beginning of the filter run (condition 1) and at the end (condition 2).

Tabla 6-3. Suspended solids removal efficiencies on surface area of a DyRF due to Q_o.

Q_i (lps)	Theoretical efficiency (%)		Real efficiency (%)	
	(1)	(2)	(1)	(2)
0.70	5.0	2.0	3.0	(*)20.0
1.00	5.0	2.0	(*)29.0	9.0
1.50	4.0	2.0	17.0	4.0
2.00	5.5	2.0	16.0	2.0
2.65	4.0	1.0	20.0	3.0
Mean	4.7	1.8	14.0	4.5
STD	0.7	0.5	7.3	2.7
95% Confidence	0.6	0.4	6.4	3.1

(1): Beginning filter run (2): Ending filter run STD: Standard deviation
(*) not included in average and STD calculations.

Table 6-4 Suspended solids removal efficiencies in gravel beds due to vertical flow (Q_e).

Nominal Q_i (l/s)	Theoretical Efficiency (%)		Real Efficiency (%)	
	(1)	(2)	(1)	(2)
0.42	60.3	64.0	29.4	61.9
0.70	59.2	79.3	68.5	83.0
1.00	60.8	76.0	65.9	86.0
1.50	61.1	80.0	63.5	87.0
2.00	60.5	77.9	64.8	86.7
2.65	60.3	80.1	69.7	81.6
Mean	60.4	78.7	66.5	84.9
STD	0.65	1.54	2.3	2.2
95% Confidence	0.7	1.6	2.3	4.4

(1) Beginning filter run (2) Ending filter run

The Figures 6-5 a) and b) and Table 6-4 indicate that the measured efficiencies are slightly higher 6.1% for condition 1 and 6.2% for condition 2, than the theoretical efficiencies.

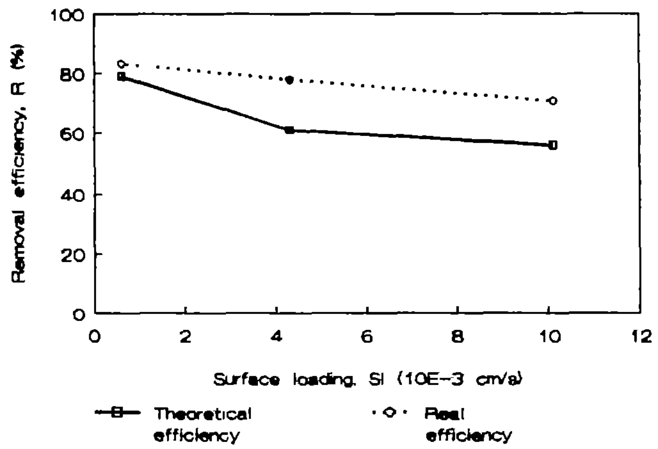


Figure 6-5 a) Comparation of suspended solid removal efficiencies for treated water under Q_1 = 0.70 l/s.

A slight improvement in removal efficiency is observed in the course of the filter run. This is in line with the sedimentation theory as the value of S_l decreases due to the reduction in flow resulting from gradual clogging shown in Figures 6-14 and 6-15 .

Although the values obtained for Q_1 = 0.42 l/s have been included in Table 6-4 these have not been taken into account in the calculations because of irregularities in the performance of the DyRF. The efficiency for Q_1 = 0.42 l/s varies strongly during the filter run as a result of the hydraulic performance of the system. In the beginning no overflow is produced and part of the DyRF remains dry at the surface. Gradually this pattern changes until the total bed is under water. This irregular water distribution will no doubt have an impact on removal efficiency. This different behaviour is also shown in **Figure 6-6** were the relation between suspended solids removal efficiencies and surface loading S_l is given for different Q_i's. For the other values of Q_i behaviour is very much the same with average efficiencies of 76% (STD = 2.3%) with S_l = 10.5x10^{-3} cm/s at the beginning of the runs and 95% (STD = 2.4%) with S_l = 0.105x10^{-3} cm/s ending filter run. The average efficiency for the total filter run for different Q_i's was above 83.3% (STD = 9%).

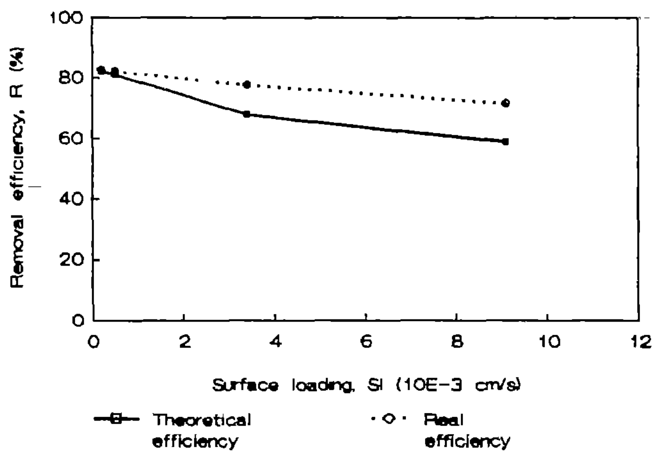


Figure 6-5 b) Comparation of suspended solid removal efficiencies for treated water under Q_1 = 2.65 l/s.

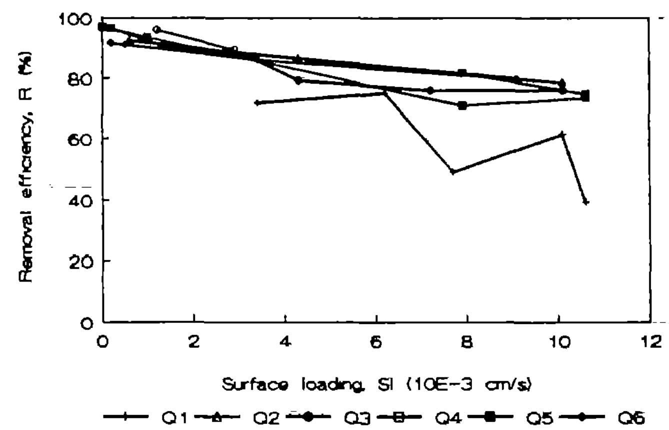


Figure 6-6 Real suspended solid removal efficiencies for treated water under different Q_i and V = 2.0 m/h.

6.2.3. Treated water quality

The impact of the overflow on the quality of treated water is shown for different parameters in **Figures 6-7 a) to 6-7 f)** as function of the efficiency over the filter run. Day 1 corresponds with the start of the filter run whereas the last days depends on the reduction in the effluent flow Q_e. When Q_e dropped below 0.05 l/s the filter run was stopped. More details including statistical information is presented in **Appendix 1 Table 1-1 a) to 1-1 l) and Appendix 3, table 3-1.**

The efficiencies concern:

. Turbidity from 50% (STD = 11.9%) to 52% (STD = 13.1%)
. Suspended solids from 83% (STD = 9.1%) to 87%(STD = 5.5%)
. Volatile solids from 25% (STD = 11.6%) to 32% (STD = 12.5%)
. True color from 13% (STD = 15.9%) to 24% (STD = 18.4%)
. Total iron from 55% (STD = 11.8%) to 84% (STD = 7.8%)
. Faecal coliform 0.45 log (STD = 0.54) to 1.0 log (STD = 0.61).

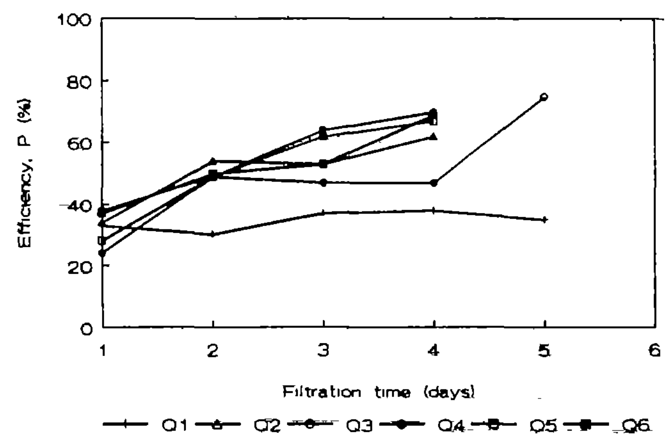


Figure 6-7 a) Treated water turbidity.

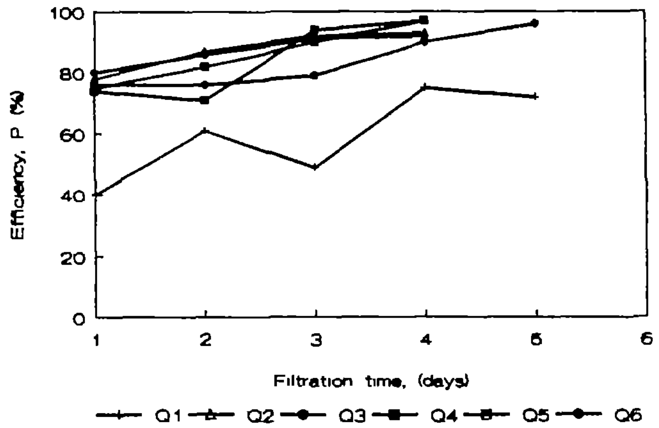


Figure 6-7 b) Treated water suspended solids.

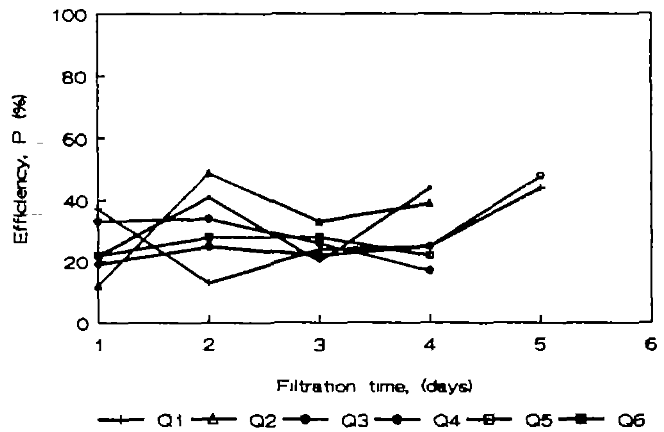


Figure 6-7 c) Treated water volatile solids.

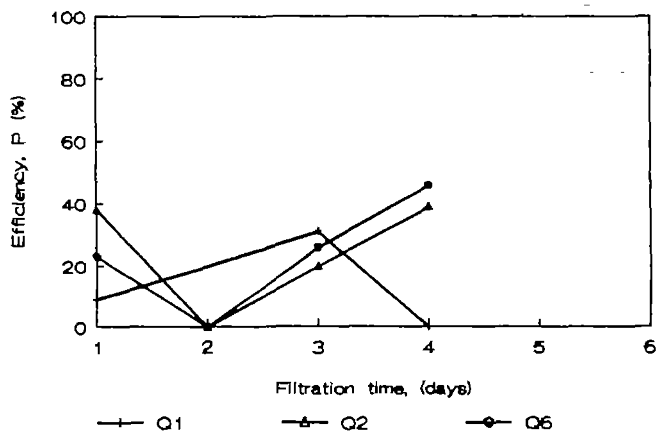


Figure 6-7 d) Treated water true color.

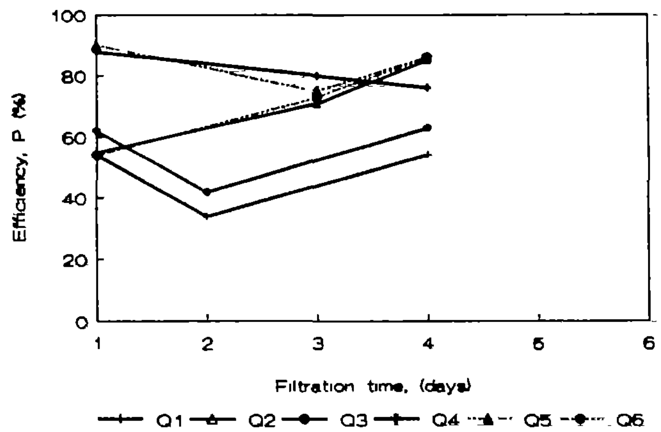


Figure 6-7 e) Treated water total iron

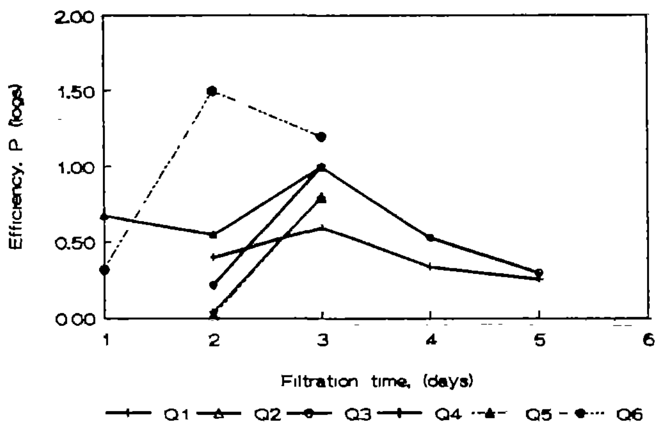


Figure 6-7 f) Treated water faecal coliform counts.

The efficiencies showed a small increase in the course of the filter run.

In the second research period the effect of the higher filtration rates (V = 3.0, 4.0 and 5.0 m/h) with a constant Q_t of 1.5 l/s were evaluated and compared with the results of the second run in the first period Q_t = 1.5 l/s and V = 2.0 m/h. **Figure 6-8** shows the removal efficiencies for suspended solids for the second period. **Table 6-5** shows the main statistical parameters when suspended solid removal efficiencies were evaluated under different filtration rates and constant Q_t = 1.50 l/s. It may be observed that at the start of the run efficiencies are close to 60% for $S_l = 23.3 \times 10^{-3}$ cm/s and increase towards the end to 95% for $S_l < 1 \times 10^{-3}$ cm/s.

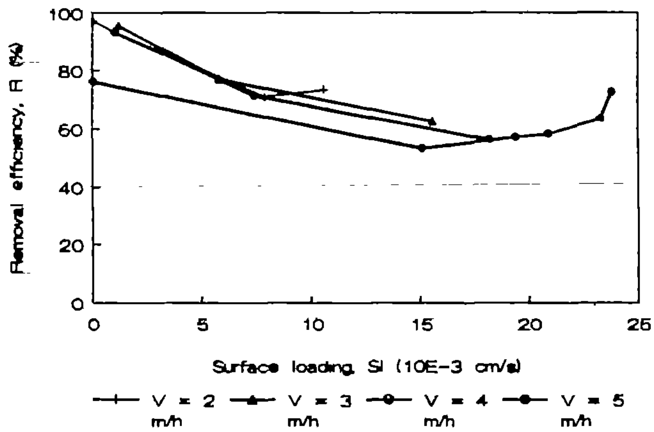


Figure 6-8 Suspended solid removal efficiencies under different filtration rates and constant Q_i = 1.50 l/s.

Results for V = 2.0, 3.0 and 4.0 m/h are rather similar with effective removal efficiencies of 73.8% (STD = 18.5%) for V = 4.0 m/h and 83.8% (STD = 13.4%) for V = 2.0 m/h. For V = 4.0 m/h a low removal efficiency was obtained in the beginning, but this is likely due to insufficient cleaning of the gravel. With a filtration rate V of 5.0 m/h efficiencies are considerably lower and vary between 58% at the start to 76% at the end with an average of 63.6% (STD = 9%) over the filter run.

Tabla 6-5 Main statistical parameters for different filtration rates and constant Q_i = 1.50 l/s.

Statistics	Filtration rates V (m/h)			
	2 m/h	3 m/h	4 m/h	5 m/h
Mean	83.8	78.5	73.8	63.6
Maximum	97.0	95.6	93.3	76.3
Minimum	71.0	62.7	56.4	53.3
STD	13.4	16.5	18.5	9.2
Standard error	6.7	9.5	10.7	3.7
95% confidence	13.2	18.7	21.0	7.3
99% confidence	17.3	24.6	27.6	9.6
Size	4	3	3	6

6.2.4. Scouring of particles

Table 6-6 shows the hydraulic parameters which have been measured at the end of the filter runs. Overflow rates have varied between 86.7×10^{-3} and 358×10^{-3} cm/s and flow velocities between 10 to 18 cm/s for Q_i = 0.7 and 2.65 l/s respectively. In a similar way the Reynolds numbers have differed between R_e = 1074 for Q_i = 0.70 l/s and laminar flow conditions and R_e 3415 for Q_i = 2.65 l/s and turbulent flow. The average flow velocities have been estimated as 0.75 times the velocity of the flow at the surface as is reported by Yalin (1977).

The flow at the surface has been measured at the beginning during and at the end of each filter run. Flow variation was rather small with velocities starting with 9 cm/s for Q_i = 0.70 l/s and ending with 10 cm/s and starts with 15.5 cm/s for Q_i = 2.65 l/s and ending with 18 cm/s.

Table 6-6 Main hydraulic parameters at the end of filter runs for different nominal Q_i

Nominal Q_i (l/s)	Overflow Q_o (l/s)	Overflow rate S (cm/s)x10^{3}	Water depth (m)x10^{-3}	Flow Velocity X (cm/s)	Hydraulic Gradient Ix10^{-3} (m/m)	Reynolds Number R_e	Froude Number F_r
0.70	0.65	86.7	10.0	10.0	1.32	1074	0.106
1.00	1.02	136.0	10.5	11.5	1.96	1283	0.133
1.50	1.52	202.7	11.5	12.0	2.54	1473	0.135
2.00	1.95	260.0	14.0	17.0	2.61	2466	0.227
2.65	2.69	358.7	18.5	18.0	3.27	3415	0.194

A theoretical relation has been stablished for the calculation of the scouring velocity (U) as function of particle diameter (d_s). For the three densities the scouring velocity has been calculated using the equations 4-5, 4-7 and 4-8 for non cohesive sediments. This theoretical expression is based on initiation of motion of fine cohesionless flaky sediments with particle diameters between 10 to 100 μm.

In **Figure 6-9** is shown the theoretical relationship between noncohesive particles with diameters d_s and scouring velocities (U) for different sediment densities: 2650, 2125 and 1365 kg/m^3.

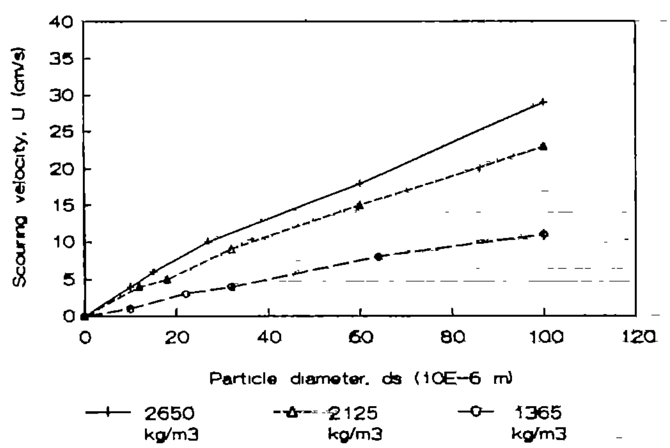


Figure 6-9 Theoretical relationship between noncohesive particles and scouring velocities.

Based on equation 4-6 to 4-8 and for cohesive sediments, U was calculated as a function of V_s and critical shear stess (τ_{oc}). In **Figure 6-10** is shown the U variation versus d_s for ρ_1 = 2650 kg/m^3.

As shown in Table 6-6 for Q_o = 0.65 l/s the minimum flow velocity X of 10 cm/s is obtained. Figure 6-9 shows that non cohesive particles with d_s < 27, 32 and 80 μm (for

2650, 2125 and 1365 kg/m^3) can be scoured. These particles are approximately 91% of the particles in Q_i and Q_o. The same behaviour may be expected for cohesive sediments with d_s >3.2 μm (2650 kg/m^3).
For maximum X = 18 cm/s non cohesive particles with d_s < 57, 82 and 176 μm and cohesive particles with d_s > 0.5 μm will be scored. Under this maximum X, almost 100% of existing particles in Q_i and Q_o can be scoured.

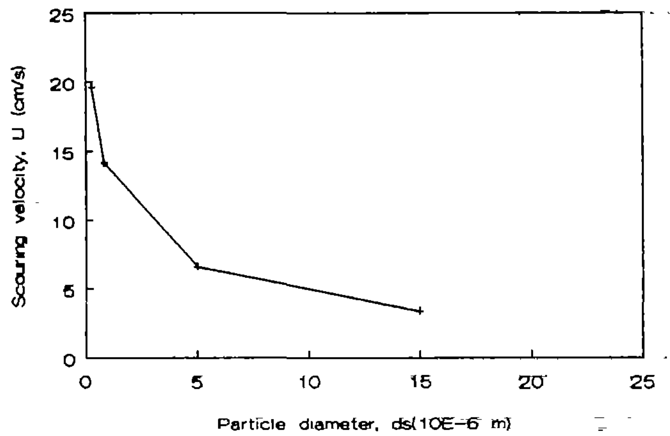


Figure 6-10 Theoretical relationship between cohesive particles and scouring velocities.

6.2.5. Other measurements

6.2.5.1. Mass balance

Figure 6-11 indicates the sludge accumulation distribution in the different layers of the DyRF for different Q_i's with V = 2.0 m/h. It is clear that the main accumulation takes place in the upper layer (L_1) with an average of 80.5% (STD = 9.5%). In the second layer (L_2) 11% (STD = 4.2%) is retained and in the bottom layer (L_3) 8.5% (STD = 6.2%). For Q_i = 0.42 l/s a different pattern is found due to the different hydraulic behaviour as explained in 6.1.

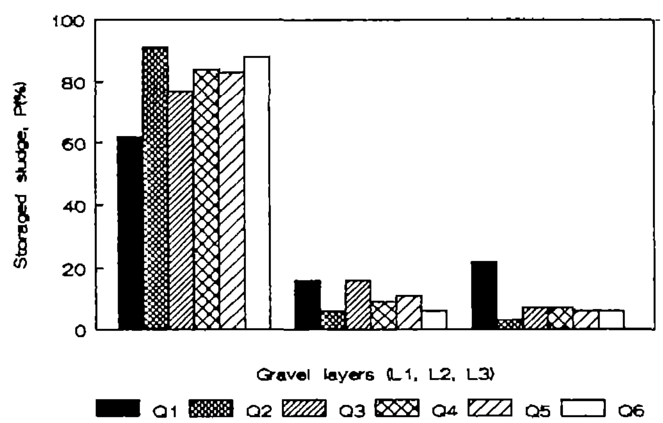


Figure 6-11 Storaged sludge per layer under V = 2.0 m/h and different Q_i.

Figure 6-12 shows the sludge retention distribution for the filter runs with respectively V = 2.0, 3.0, 4.0 and 5.0 m/h and Q_i = 1.5 l/s. The averages are for L_1 = 75.8% (STD = 14.9%), L_2 = 13.5% (STD = 6.7%) and L_3 = 10.8% (STD = 8.3%). A different behaviour was found for V = 5.0 m/h were the average was 50% for L_1, 25% for L_2 and 25% for L_3.

Figure 6.13 shows the sludge storage per square meter per day for V = 2.0, 3.0, 4.0 and 5.0 m/h. The figure shows an increase in sludge storage for V $\leq$ 3.0 m/h due to the increase in Q_e. For V $\geq$ 4.0 m/h the stored quantity reduces due to the high flow rate which causes resuspension of sediment material.

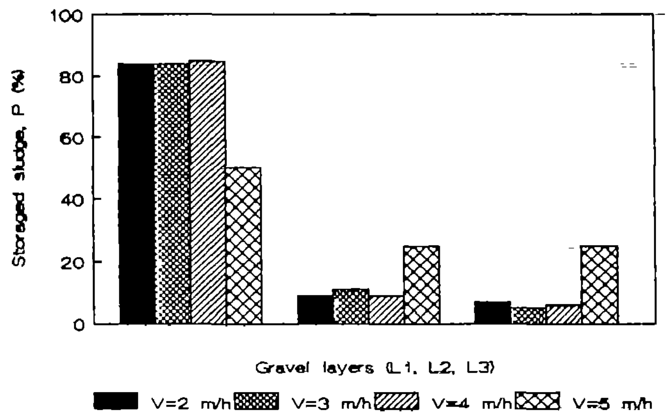


Figure 6-12 Storage sludge per layer under different filtration rates and constant Q_i = 1.50 l/s.

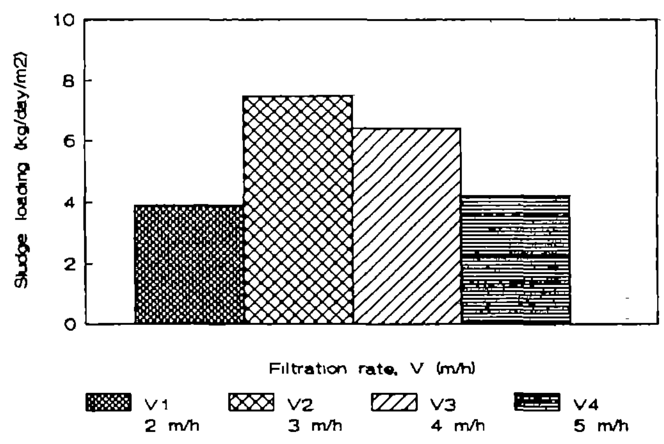


Figure 6-13 Sludge load for different filtration rates.

6.2.5.2. Flow measurement

Figure 6-14 shows the typical variation in water flows Q_i, Q_e and Q_o over the filter run. Q_e varies between 0.42 l/s (V = 2 m/h) to almost 0 l/s after 4 days of filter run. The reduction of Q_e implies an automatic increase in Q_o until almost reaching the value of Q_i.

In Figure 6-14 day 1 corresponds with the start of the filter run and the last day will depend on the length of the run. In general the filter is stopped when $Q_e < 0.05$ l/s. A similar behaviour was found for V = 3.0, 4.0 and 5.0 m/h as shown in **Appendix B Figures B-1 a) to B-1 f)**.

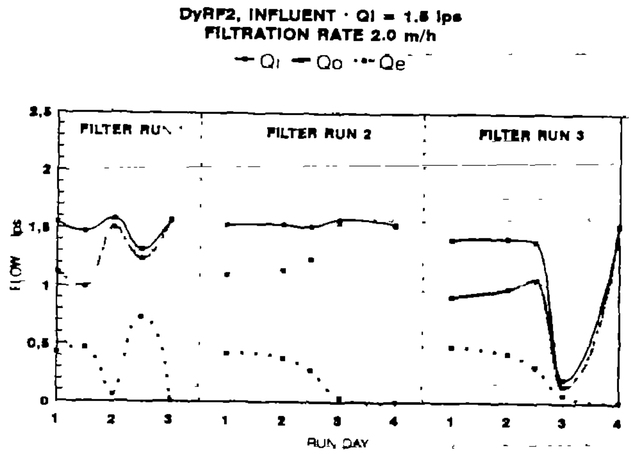


Figure 6-14 Typical flow variation over the filter run in DyRF.

6.2.5.3 Headloss

The difference in headloss over the gravel bed is presented in **Figure 6-15**. It can be observed that the main headloss is concentrated in the first 20 cm corresponding with the finest gravel layer. In **Appendix C Figures C-1 a) to C-1 f)** are shown the typical headloss variation over the filter run for different Q_i and V = 2.0 m/h.

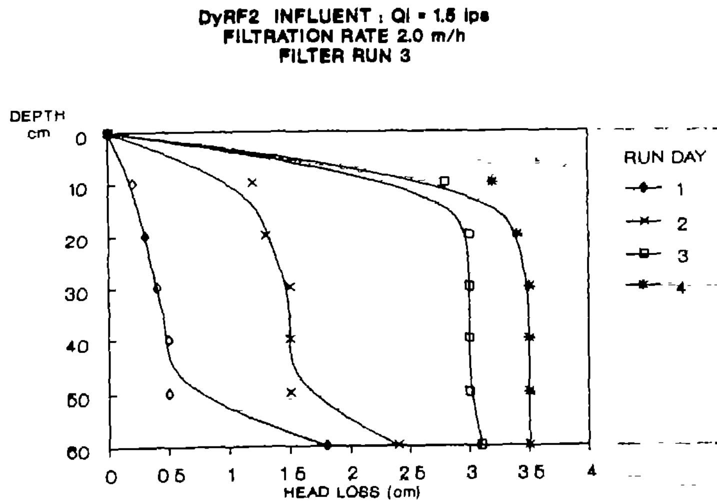


Figure 6-15 Typical headloss variation over the gravel bed throughout filter run.

CHAPTER 7
Discussion of Results

7.1 Raw water characterization

A large part of the diseases in developing countries are water borne. Many of these are directly related to faecal contamination of the water sources. The Cauca river which is the raw water source for several million colombian inhabitants falls in the worst category (E) (coliform counts/100 ml > 1000) proposed by Lloyd et al. to WHO and UNEP. During the research period the average faecal coliform count ranged between 19440 and 64143 with peaks between 29000 and 242000. Implying a very high sanitary risk.

Also the physical-chemical quality is very poor. With values for several parameters will above limitations suggested for the application of Slow Sand Filtration (SSF) as a treatment alternative. Cleasby (1991), Spencer et al.(1991) and Di Bernardo (1991) have for example proposed upper limits for SSF application for turbidity between 5 to 10 NTU, true color between 15 and 25 TCU, total iron between 0.3 and 2.0 mg/l. The raw water quality of the Cauca river is well above these values thus clearly indicating the need for pre-treatment alternatives.

A further complicating factor is the composition of the raw water. According to the research findings 91% of the particles have a diameter d_s < 27, 32 and 57 μm and 70% has a diameter < 5, 7 and 13 μm for densities of 2650, 2125 and 1365 kg/m^3 respectively. A large portion of fine material may affect the performance of slow sand filters (Galvis et al. 1993) but may also influence the behaviour of DyRF because of the large pore size usually above 500 μm for gravel sizes above 2mm (Huisman 1986). In view of the rather special water quality of Cauca river findings for the behaviour of the DyRF treating this water quality may not be the same for other water qualities.

7.2 Particle removal process

Sedimentation is the most important process in DyRF as clearly shown by the results obtained in the research as presented in 6.2.2.
On the basis of the operating conditions two sedimentation areas have been distinguished:

i) Plain sedimentation on the exposed surface of the gravel bed.

ii) Sedimentation in the gravel bed (as part of filtration process).

The impact of the sedimentation process is analysed by comparing the theoretical and measured suspended solid removal efficiencies for each of the two sedimentation areas. The calculation of the measured efficiency permits the elimination of the effect of the variation in suspended solid concentration in the influent. The theoretical efficiencies are based on the removal rate R established with column settling tests (CST).

7.2.1. Plain sedimentation on the exposed surface of the gravel bed

This area can be considered as a plain sedimentation tank where (R) depends on two factors: i) the frequency distribution for settling velocities of the suspended particles and ii) overflow rate (S). Only the last factor can be influenced by the design or operating conditions of the DyRF. The efficiency in this area is independ of the depth of the basin and of the detention time.

In comparing the efficiencies at the beginning of the filter run considerable differences were found for a significance level α of 5%, the higher measured efficiency however is probably the result of a the water flowing through the expanded gravel bed to the weir. The higher efficiency at the start of the filter run is favoured by the cleanliness of the filter and the low value of Q_o, which permits horizontal flocculation to take place in the upper part of the gravel bed (0 - 10 mm) such as the formation of the bed forms ripples and dunes. The latter is favoured by the low value of the Froude number ($F_r < 0.7$).

In the beginning of each filter run X, R_e and F_r have the lowest values due to Q_c being highest and Q_t being approximately constant. The flocculation effect can be stimulated by the high concentration of total iron (average ranging from 13.37 to 15.76 mg/l with peaks between 25.60 and 57.50 mg/l). This effect is very much reduced after the first day when the gravel bed is settled and horizontal filtration no longer takes place.

If the results of the first day are excluded, no significant differences exist for $\alpha = 5\%$ between the theoretical and measured efficiencies except for the value of 20% obtained for $Q_t = 0.70$ l/s and reported as a likely experimental mistake.

Excluding the effect of horizontal filtration it may be concluded that the removal efficiency in the horizontal flow is mainly due to plain sedimentation and is presenting values below 10%.

The low efficiency in sedimentation is the result of a combination of factors including:

i) High values of S ranging between 86.7×10^{-3} ($Q_t = 0.70$ l/s) to 358.7×10^{-3} cm/s ($Q_t = 2.65$ l/s).

ii) Low settling velocities $V_s < 80 \times 10^{-3}$ cm/s.

iii) High values of the R_e ranging from 1074 ($Q_t = 0.70$ l/s) to 3415 ($Q_t = 2.65$ l/s), which results in flow conditions between laminar and turbulent. Laminar flow will occur for R_e between 580 and 2000.

iv) Negative effects resulting from the formation of a sludge layer on the surface causing the development of horizontal and vertical flow lines which reduce sedimentation efficiency.

v) Limitations in the hydraulic design of the system which does not permit a good flow distribution in the inlet structure or a good abstraction at the outlet.

The efficiency of plain sedimentation can also be influenced by the bed forms such as ripples and dunes which may be formed when $F_r < 0.7$ and have the common mode of bed material transport for the individual grains to move up the back of the ripple or dune and avalanche down its face. Another aspect to take into account is the effect of the gradation because the large particles will be more exposed while the smaller particles are shielded by the larger particles.

For a 95% confidence interval it has been established that the horizontal removal efficiency for Q_i values between 0.70 l/s and 2.65 l/s does not depend on the value of Q_i.

7.2.2. Sedimentation in the gravel bed

In the gravel the efficiency depends on the surface loading $S_l = Q_e/A_l$, which Q_e reducing during the filter run generating a reduction in S_l and thus an increase in efficiency. In **Table 6-4** average theoretical efficiencies range from 60.4% (STD = 0.65%) at the start of the filter run and 78.7% (STD = 1.54%) at the end. Nevertheless measured efficiencies are higher and range from 66.5% (STD = 2.3%) to 84.9% (STD = 2.16%) respectively. For a 95% confidence interval differences between theoretical and measured efficiencies are not significant which implies that reported efficiencies do not depend on Q_i for values ranging from 0.70 and 2.65 l/s.

Different factors can justify the high efficiency of sedimentation in gravel beds:

i) High surface area for sedimentation

ii) Presence of other mechanisms such as screening and flocculation. The former may be important at the end of the filter run when the pore sizes have decreased and the latest, due to the fact that the packed bed of gravel provides ideal conditions for the formation of compact settleable flocs because of continuous recontacts provided by the sinuous flow of water through the interticies formed by the gravel. Moreover high total iron concentration in raw water can help to floc formations. According to experimental conditions and based on the formula presented by Schultz and Okun (1984) for gravel- bed flocculators the velocity gradient G can get a value of about 40 s^{-1}.

iii) Reduction of Q_e during the filter run. This process of declining rate filtration reduces S_l and increases the sedimentation efficiency.

On the basis of the high removal efficiencies and the fact that no significant differences exist at a 95% confidence level it can be said that sedimentation is the main particle removal mechanism in DyRF with average efficiencies ranging from 66.5% to 84.9%.

Based on the achieved results, **Figure 7-1** illustrates the main particle removal mechanisms in DyRF. Zone 1 corresponds with plain sedimentation on the exposed surface of the gravel bed with an efficiency below 10% and possibly having a negative influence on the hydraulic behaviour of the DyRF. Zone 2 is the area of major efficiency in particle removal in DyRF where sedimentation takes place, but also other mechanisms such a screening and flocculation may contribute to the removal efficiency.

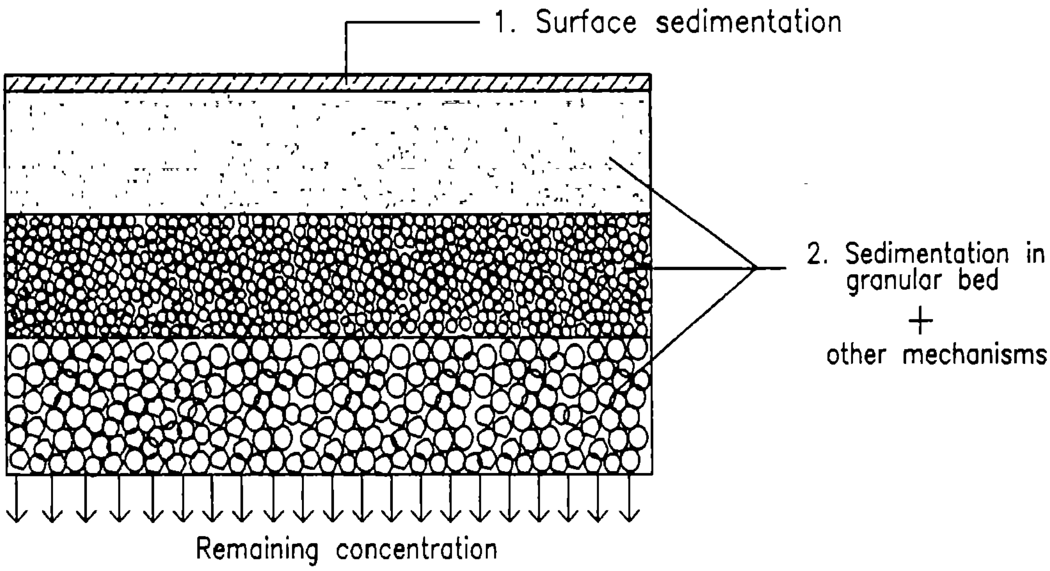


Figure 7-1 Main particle removal mechanisms in DyRF.

7.3. Impact of overflow on treated water quality

The effect of the overflow on the treated water quality has been analyzed according to the design of the experiment indicated in Table 5-2. Although the concentrations of the different parameters vary over time this has been solved by calculating removal efficiencies $[(C_{in} - C_{out}) / C_{in}]x100$ with C_{in} = influent concentration (raw water) and C_{out} = treated water concentration.

As Q_i is the controlled variable and V varies over the filter run due to the process of declining rate filtration, a relative simple analysis can be made of the established information. The statistical data (average, STD, standard error, and confidence level) have been calculated with the programme Slide.

The analyses has gone through the following steps:

i) Calculation of real efficiencies for each Q_i and for each parameter under evaluation based on the results of the experiments.

ii) Calculations of the statistical data on the basis of the data collected under i) and help of the programmne Slide.

iii) Analysis of significant differences for each of the parameters for a 95% confidence interval. This interval has been selected because of the small sample size n < 30.

On the basis of the statistical data presented in Figures 6-6, 6-7 a) to 6-7 f) and 6-8 it can be established that:

i) The removal efficiencies for suspended solids in DyRF for V = 2.0 m/h do not depend on Q_i for values of Q_i = 0.70 l/s and 2.65 l/s. The average efficiency over the total filter run is over 83.3% (STD = 9%) except for Q_i = 0.42 l/s which had an efficiency of 59.38% (STD = 15.1%) due to poor hydraulic performance as discussed earlier.

ii) No significant differences exist in the removal efficiencies of turbidity, suspended solids, true color, volatile solids and faecal coliform, when evaluated during the filter runs for different Q_i's and V = 2.0 m/h.

iii) At a 95% confidence level significant differences have been established for the removal efficiency of total iron for different Q_i's. This implies that the removal of iron depends on the value of Q_i. This can be explained as resulting from the horizontal filtration process which may stimulate floc formation. Furthermore the high level of iron in the raw water may be an important factor.

iv) Removal efficiencies for suspended solids for filtration rates V = 2.0, 3.0 and 4.0 m/h do not present significant differences at a 95% confidence level. For V = 5.0 m/h however significant lower efficiencies of 63.6% (STD = 9.2%) were obtained. This can be explained by the increase in the flow velocity which implies an increase in the shear stress and thus the scouring capacity.

The removal efficiencies obtained in the evaluation of the impact of the overflow on the quality of the treated water are justified primarily on the basis of sedimentation and the effect of declining rate filtration. Other mechanisms such as screening and flocculation also may contribute to an increase in efficiency, but its effect has not been quantified.

7.4 Impact of overflow on scouring of surface particles

The analysis of the impact of the overflow on scouring of surface particles is based on the following considerations:

i) The raw water of Cauca river has a high concentration of fine material: 70% of the particles in Q_i and Q_o have diameters d_s < 5, 7 and 13 μm respectively for particle densities of 2650, 2125 and 1365 kg/m^3 and 91% has a d_s < 27, 32 and 57 μm respectively. Moreover the effective weight of the particle (ρ_s - ρ) acts downwards and mechanical friction is a function of the size of the deposited particles, the resistance to scouring is minimal.

ii) The remaining concentration of suspended solids in Q_i is approximately the same as in Q_o for different values of Q_i. This may imply that particles have been sedimented and simultaneously others have been removed by scouring or that the particles remain in suspension indicating low efficiency in sedimentation. The latter was clearly observed in the experiment with removal efficiencies for suspended solids below 9% at the end of the filter run.

iii) Minimum resistance against displacement appears to exist in the deposited material on the exposed filter bed, which has an average density of 1564 kg/m^3 (STD = 78.2). This behaviour is indicated in **Figure 6-11** for cohesive material where particles with diameter > 3.2 μm can be scoured with X = 10 cm/s and with d_s > 0.5 μm with X = 18 cm/s. The cohesive sediments tend to stick together forming aggregates where the electro-chemical forces are acting between the particles and are increasing their scouring velocities than those of the individual particles.

iv) The critical condition for scouring of surface particles per each Q_t is taken place ending each filter run when the real flow velocity (X) is taken the maximum value. Therefore, the analises of scouring was based on the maximum velocities X for Q_t = 0.7 l/s and 2.65 l/s which correspond with 0.10 and 0.18 cm/s respectively. For these velocities the Reynolds number varies between 1074 and 3415 which generates flow conditions between laminar and turbulent.

The experimental data show that with a minimum value X of 10 cm/s, 91% of the particles in the sediment can be scoured and for X = 18 cm/s all particles can be scoured. The drag force of the fluid on the sediments due to hydraulic shear acts in the direction of the motion of the fluid and hydraulic shear is a function of the fluid velocity therefore high flow velocities mean high drag forces. Scouring starts when the hydraulic shear stress (τ_{oc}) between the flowing water and the sludge deposits exceeds the mechanical friction or attractive forces between layers of deposits or between deposits and the grain surface. If τ_{oc} is very much greater than mechanical friction, scouring is not important and the particles will be in suspension in the water and therefore scouring is not important aspect for DyRF.

v) The particle size for cohesive sediments were analysed for a density ρ_1 = 2650 kg/m^3, which corresponds with an average value for clay. Specific test to determine cohesive sediments have not been made but in view of the diversity of sediments transported by Cauca river as supported by literature they are present. The literature refers to cohesive material such as a mixture of sediments with clay (d_s < 4 μm) and densities about 2650 kg/m^3. In this research this value was to determine particle diameters and the calculation of U.

The high flow velocities and the low density of the deposited material (1564 kg/m^3), contribute to the justification of the low efficiency of plain sedimentation. The deposited material can easily be removed because of the high drag force and only large non cohesive particles can remain or small cohesive particles. In practice we see that quite some material remains on the surface of the gravel bed even at the highest value for X and particularly in the first half of the bed. This may be caused by either lower flow velocities in the beginning of the bed due to a greater height of the water level causing a different flow distribution over the heigh. This can be confirmed by a more detailed measurement of the water level and flow velocities in the DyRF.

Another possibility is that the deposited material is strongly influenced by other forces involved in absorption and therefore is not removed. Nevertheless even if all particles would be removed scouring only affects the material which is deposited on the surface of the grains and which is shown to be less than 10% of the total material removed by a

DyRF.

7.5. Other measurements

7.5.1. Mass balance

The sludge accumulation in the different layers of a DyRF does not depend on the value of Q_i for values between 0.7 to 2.65 l/s. This is showing again that the impact of the horizontal flow over the DyRF is very small. At a 95% confidence level it was established that there was no significant difference in sludge retainment in the different layers for different Q_i except for Q_i = 0.42 l/s which caused a poor hydraulic performance, with dead zones at the surface and part of the bed at the start of the filter run.

The obtained averages for the retained sludge in each layer for V = 2.0 m/h corresponds with L_1 = 80.5% (STD = 9.5%), L_2 = 11% (STD = 4.2%) and L_3 = 8.5% (STD = 6.2%). This clearly indicates the efficiency and importance of the first layer which composes of the finest gravel.

The sludge accumulation for the higher filtration velocities V = 3.0 and 4.0 m/h show a similar pattern as for V = 2.0 m/h but differs at a 95% confidence level for V = 5.0 m/h when more material was transported into the deeper layers. This can be explained by the increase in flow velocity causing higher shear stress, taking deposited material deeper into the bed.

For $V \leq 4.0$ m/h sludge accumulation distribution over L_1, L_2 and L_3 does not depend on V with average accumulation levels for L_1 = 84%, L_2 = 9.7% and L_3 = 6%, whereas for V = 5.0 m/h the distribution was L_1 = 50%, L_2 = 25% and L_3 = 25% showing clearly a deeper penetration of the sludge into the bed.

8. Conclusions and Recommendations

8.1. Conclusions

The major findings of this research are:

1. Characterization of raw water:

The Cauca river water has a typical characterization of untreated sewage which implies a very high sanitary risk when it is used for drinking water. Also very fine particles were found in raw water (about 70% particles < 5 μm). This water quality should better be rejected as a source of drinking water and if used will need different treatment steps to reduce the sanitary risk.

2. Particle removal process:

In DyRF **Sedimentation** is the main particle removal process which occurs in two different locations:

i) Plain sedimentation onto the exposed surface of the gravel bed. Here the removal efficiency is < 10%.

ii) Sedimentation (as part of filtration process) into the gravel bed. The removal efficiency ranged from 65% to 85% and is considered as the most important process in DyRF.

3. Effect of surface overflow:

The surface overflow did not have any impact on scouring of surface particles and on treated water quality.

4. DyRF perfomance:

The following average removal efficiencies were obtained for DyRF units operating at filtration rates between 2.0 and 4.0 m/h with average surface flow velocities between 5 cm/s and 18 cm/s:

. Turbidity from 50% (STD = 11.9%) to 52% (STD = 13.1%)
. Suspended solids from 83% (STD = 9.1%) to 87%(STD = 5.5%)
. Volatile solids from 25% (STD = 11.6%) to 32% (STD = 12.5%)
. True color from 13% (STD = 15.9%) to 24% (STD = 18.4%)
. Total iron from 55% (STD = 11.8%) to 84% (STD = 7.8%)
. Faecal coliform 0.45 log (STD = 0.54) to 1.0 log (STD = 0.61).

5. Design guidelines:

The impact of the surface flow in DyRF is limited and may even result in uneven distribution of water over the filter. Therefore it is better to design DyRF whithout surface flow. The gravel sizes can be kept, although it may be explored to reduce to two sizes. Preliminary design criteria for such a system are:

. Maximum filtration rate of 4.0 m/h
. Gravel sizes between ϕ 6 to 13 mm in the upper layer, ϕ 13 to 19 mm in the middle and ϕ 19 to 25 mm at the bottom.
. Thickness per layer: 0.20 m.

8.2. Recommendations

i) To eliminate the overflow as a normal hydraulic element in DyRF operation.

ii) To keep the weir in the out let structure as "tapon valve" in order to protect the water treatment plant against suddenly raw water quality changes. This weir is also useful for maintenance activities.

iii) To evaluate downflow roughing filter with declining rate filtration and constant rate filtration in order to identify the best alternative and compare this with a DyRF with overflow.
This evaluation preferably is carried out both in the Research Station and in another location with different water quality.

REFERENCES

Ahsan, T., Alaerts, G.J. and Buiteman, J.P. (1991). "Direct Horizontal-Flow Roughing Filtration: An improved pretreatment process for highly turbid water." Proc. of the Filtration Society Conference (Karlsruhi - Germany), October.

APHA, (1986). "Standard Methods for Examination of Water and Wastewater." 16th Edition. American Public Health Association. Washington.

APHA, (1989). "Standad Methods for Examination of Water and Wastewater." 17th Edition. American Public Health Association. Washington.

Arboleda, J. V.,Giraldo, R., and Snel, H. (1985). "Hydraulic Behavior of Declining-Rate Filtration." Journal of the American Water Works Association. 67-74. December.

Breusers, H. N. C. (1988). "Sediment Transport 1." Lecture note, Delft, IHE.

Chang, H. H. (1988). "Fluvial Processes in River Engineering." John Wiley & Sons, New York.

Chaudhari, S. and Tare, V. (1993). "Comment on an Integrated Expression for Settling Velocity of Particles in Water." Wat. Res. Vol. 27, No.8 pp. 1393 - 1394.

Chowdhury, J. M. (1993). "Direct Horizontal - flow Roughing Filtration: Modelling of Particle Separation in the Medium Size Grains." M. Sc. Thesis, E.E.11, Delft, IHE.

CINARA, IRCWD/EAWAG/ETH. (1988). "Estudio Bocatomas de Lecho Filtrante y Filtros Dinamicos Gruesos." Reporte de Progreso. Cali, Julio.

CINARA, IRC. (1993). "Proyecto Integrado de Investigacion y Demostracion de Metodos de Pretratamiento para Sistemas de Abastecimiento de Agua." Informe Final, Cali.

Cleasby, J. L.and Di Bernardo, L. (1980). "Hydraulic Considerations in Declining-Rate Filtration." Journal of the Environmental Engineering Division. ASCE, 106: EE6: 1043. December.

Cleasby, J. L.and Di Bernardo, L. (1980). "Declining-Rate Versus Constant-Rate Filtration." Journal of the Environmental Engineering Division. ASCE, 106: EE6: 1023. December.

Cleasby, J. L. and Hilmoe, J. D..(1986). "Comparing Constant_Rate and Declining-Rate Direct Filtration of a Surface Water." Journal of the American Water Works Association. 26-34. December.

Cleasby, J. L. (1991). "Source Water Quality and Pretreatments options for Slow Sand Filtation." Chapter 3 in: Task Committe on Slow Sand Filtration. New York.

Delft Hydraulics. (1972). "Systematic Investigation of Two - Dimensional and Three - Dimensional Scour." Report M648/M863, Delft.

Ellis, K. V. (1985). "Slow Sand Filtration. C. R. C. Critical Reviews in Environmental Control." Department of Civil Engineering, University of Technology. United Kingdom. Vol. 15.

Engels, C. and Poggenburg, W. (1989). "Riverbank Filtration." Chapter 2, Pretreatment Methods for Community Water Supply. The Hague. IRC.

Frankel, R. J. (1974). "Series Filtration Using Local Filter Media." JAWWA, 66, 2, 124 -127.

Galay, V. J., Yaremko, E. K., and Quazi, M. E. (1987). "River Bed Scour and Construction of Stone Riprap Protection." Sediment Transport in Gravel-Bed Rivers, Ed. by Thorne, C. R., Bathurst, J. C., and Hey, R. D., John Wiley & Sons, New York.

Galvis, G. and Visscher, J. T. (1987). "Participacion y Perspectivas de la Universidad de Valle en el Desarrollo del Proyecto de Filtracion Lenta en Arena." Proceedings of the International Seminar on Simple Water Treatment Technology. ACODAL, Cali, August.

Galvis, G. and Visscher, J. T. (1989). " Proyecto Integrado de Investigacion y demostracion en Filtracion Lenta en Arena." Informe Final. Version Resumida. CINARA, Cali / IRC, The Hague.

Galvis, G., Fernandez, J., and Visscher, J. T. (1992). "Comparative Study of Different Pretreatment Alternatives. Roughing Filters for Water Treatment." Workshop in Zurich.

Galvis, G., Visscher, J. T., Fernandez, J., and Beron, F. (1993). "Pre-treatment Alternatives for Drinking Water Supply Systems." Occasional Paper 22, IRC. The Hague.

Hudson, H. E. Jr. (1981). "Water Clarification Processes. Practical Design and Evaluation." Van Nostrand Reinhold Environmental Engineering Series.

Huisman, L. (1986). "Sedimentation and Flotation." Lecture Note, Delft, IHE.

Huisman, L. (1986). "Rapid Filtration." Lecture Note, Delft, IHE.

Ives, K.J. (1975). "Capture Mechanisms in Filtration, in the Scientific Basis of Filtration." Ed.Ives K.J., Noorrdhoff International, Leyden.

Lebcir, Rabia. (1992). "Factors Controlling the Perfomance of Horizontal Flow Roughing Filters." PhD thesis, The University of Newcastle upon Tyne, Department of Civil Engineering. April.

Mantz, P. A. (1977). "Incipient Transport of Fine Grains and Flakes by Fluids - Extended Shields Diagram." Journal of the Hydraulics Division, ASCE, Vol. 103, HY6.

Miller, M. C., McCave, I. N., and Komar, P. D. (1977). "Threshold of Sediment Motion under Unidirectional Current Sedimentology." Vol. 24, p. 507 - 527.

Neil, C. R. and Yalin, M. S., (1969). "Quantitative Definition of Begining of Bed Movement." Journal of Hydraulics Division, ASCE, Vol. 95 HY1.

Perez, J., Pardon, M., Lavado, I. and Vargas, C. (1985). "Informe Preliminar de la Investigacion sobre Prefiltros de Grava." CEPIS/OPS/OMS. Lima.

Quiroga, E. L. (1988). "Estudio Preliminar sobre Pretratamientos." Reporte Final. Univalle -CIR. Cali.

Rajapakse, J. P. and Ives, K. J. (1987). "Final Report on Pebble Metric Filtration." Department of Civil and Municipal Engineering. University Collegs, London.

Richard, Y. and Capon, B. (1985). "Developments in Water Treatment - 1 ." Edited by W.M. Lewis. Developments Series.

Salazar, L. G. (1980). "Agua Potable para la Zona Rural Colombiana." ACODAL No. 96, Bogota.

Schultz, C. R. and Okun, D. A. (1984). "Surface Water Treatment for Communities in Developing Countries." Wiley - Interscience Publication. Great Britain.

Simons, D. B., Richardson, E. V. and Nordin, C. F. (1972). "Sedimentary Structures Generated by Flow in Alluvial Channels." River Morphology, Ed. by Schumm, S. A., Dowden, Hutchinson & Ross, Inc. Pennsylvania.

Smet, J.E. and Visscher, J.T. (1989). "Pre-treatment Methods for Community Water Supply: an overview of techniques and present experiences. The Hague, Netherlands, International Reference Centre for Community Water Supply and Sanitation.

Thanh, N. C. and Hettiaratchi, J.P.A., (1982). "Surface Water Filtration for Rural Areas: Guidelines for Design, Construction, Operation and Maintenance." ENSIC. Asian Institute

of Technology, Bangkok.

UNIVALLE, (1986). "Estudio Morfologico del rio Cauca - Fase 2." Departamento Mecanica de Fluidos y Ciencias Termicas, Cali.
Van Rijn, L. C. (1986). "Manual Sediment Transport Measurements." Delft Hydraulics Laboratory, March 1986, Delft.

Van Rijn, L. C. (1989). "Handbook Sediment Transport by Currents and Waves." Delft Hydraulics, Report H461, June, Delft.

Van Rijn, L. C. (1993). "Principles of Sediment Transport in Rivers, Estuaries, Coastal Seas and Oceans." Lecture Note, Delft, IHE.

Weber, W.M. (1972). "Physicochemical Processes for Water Quality Control." John Wiley & Sons, Inc.

Wegelin, M.. (1986). "Horizontal Flow Roughing Filtration." International Reference Center for Waste Disposal. IRCWD. Report No. 06/86. Duebendorf, Swtzerland.

Winterwerp, J. C., Cornelisse, J. M., and Kuijper, C. (1992). "Erosion of Natural Sediments from The Netherlands, Analysis of Laboratory Experiments." Cohesive Sediments Report, No. 38, March, Delft Hydraulics.

Wolters, H., Smet, J. E. and Galvis, G. (1989). "Up - Flow Roughing Filtration." Chapter 8. In: Treatment Methods for Community Water Supply. The Hague, IRC.

Yalin, M. S. (1977). "Mechanics of Sediment Transport." Queens University Ontario. Second edition, Pergamon Press.

APPENDIX 1

Table 1-1 a)

Descriptive Statistics - Mean values for three filters runs
DyRF1: $Q_1 = 1.0$ l/s and DyRF2: $Q_2 = 0.42$ l/s; runs 1,2 and 3

PARAMETERS		DyRF1			DyRF2		
		Qi	Qo	Qe	Qi	Qo	Qe
	MEAN	109	100	53	104		67
TURBIDITY	STD DEV	54	49	17	49		36
(NTU)	MINIMUM	59	57	30	56	*	34
	MAXIMUM	270	240	92	254		104
	TOTAL SAMPLE	37	36	37	37		37
	MEAN	109	144	31	171		60
SUSPENDED	STD DEV	73	57	18	60		31
SOLIDS	MINIMUM	108	17	6.4	97	*	30
(mg/l)	MAXIMUM	376	300	73	347		145
	TOTAL SAMPLE	27	26	27	27		27
	MEAN	53		39	54		41
VOLATILE	STD DEV	16		14	16		16
SOLIDS	MINIMUM	25		14	26	*	15
(mg/l)	MAXIMUM	86		77	86		82
	TOTAL SAMPLE	27		27	27		27
	MEAN	46	42	45	33		44
TRUE COLOR	STD DEV	29	10	14	23		14
(PCU)	MINIMUM	7	32	22	22	*	20
	MAXIMUM	102	52	64	101		63
	TOTAL SAMPLE	8	7	8	8		8
	MEAN	64143		30979	64143		24764
FAECAL	STD DEV	71301		49444	68785		29467
COLIFORM	MINIMUM	6700		500	6700	*	4500
(FCU/100ml)	MAXIMUM	242000		186000	242000		97000
	TOTAL SAMPLE	14		14	14		14
	MEAN	13.38		4.16	13.37		5.03
TOTAL IRON	STD DEV	6.77		1.83	6.40		2.10
(mg/l)	MINIMUM	7.30		2.70	7.20	*	3.00
	MAXIMUM	25.60		7.90	25.60		9.00
	TOTAL SAMPLE	9		9	9		9

Table 1-1 b) Descriptive Statistics run 1: 1.0 and 0.42 l/s

PARAMETERS		DyRF1			DyRF2		
		Qi	Qo	Qe	Qi	Qo	Qe
	MEAN	116	104	55	107		71
TURBIDITY	STD DEV	42	35	15	37		25
(NTU)	MINIMUM	59	59	30	56		38
	MAXIMUM	210	186	88	185		118
	TOTAL SAMPLE	14	14	14	14		14
	MEAN	108	156	30	170		60
SUSPENDED	STD DEV	55	54	18	56		26
SOLIDS	MINIMUM	129	94	10	99		31
(mg/l)	MAXIMUM	269	245	66	264		110
	TOTAL SAMPLE	10	9	10	19		10
	MEAN	61		45	61		46
VOLATILE	STD DEV	18		17	18		17
SOLIDS	MINIMUM	31		22	31		23
(mg/l)	MAXIMUM	86		77	86		82
	TOTAL SAMPLE	10		10	10		10
	MEAN	31	42	31	34		31
TRUE COLOR	STD DEV	13	10	11	17		8
(PCU)	MINIMUM	22	32	22	22		28
	MAXIMUM	46	52	47	54		37
	TOTAL SAMPLE	3	2	3	3		3
	MEAN	44025		15825	44025		19375
FAECAL	STD DEV	37675		17432	37675		21975
COLIFORM	MINIMUM	11200		5000	11200		4500
(FCU/100ml)	MAXIMUM	92000		46000	92000		51000
	TOTAL SAMPLE	4		4	4		4
	MEAN	10.63		4.90	10.60		5.80
TOTAL IRON	STD DEV	2.70		2.14	2.80		2.80
(mg/l)	MINIMUM	8.40		3.10	8.40		3.90
	MAXIMUM	13.70		7.90	13.70		9.00
	TOTAL SAMPLE	3		3	3		3

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Table 1-1 c) Descriptive Statistics run 2: 1.0 and 0.42 l/s

PARAMETERS		ByRF1			ByRF2		
		θi	θo	θe	θi	θo	θe
	MEAN	151	142	65	144		95
TURBIDITY	STD DEV	68	66	16	61		49
(NTU)	MINIMUM	78	74	45	92		50
	MAXIMUM	270	240	92	254		104
	TOTAL SAMPLE	10	9	10	10		10
	MEAN	230	150	38	216		83
SUSPENDED	STD DEV	69	77	23	66		38
SOLIDS	MINIMUM	104	17	6.4	160		44
(mg/l)	MAXIMUM	376	300	73	367		145
	TOTAL SAMPLE	8	8	8	8		8
	MEAN	50		34	50		37
VOLATILE	STD DEV	14		9	13		10
SOLIDS	MINIMUM	25		23	26		21
(mg/l)	MAXIMUM	60		47	60		52
	TOTAL SAMPLE	8		8	8		8
	MEAN	60		46	67		47
TRUE COLOR	STD DEV	25		7	24		9
(PCU)	MINIMUM	45		40	46		37
	MAXIMUM	102		56	101		55
	TOTAL SAMPLE	3		3	3		3
	MEAN	63520		42040	63520		29040
FAECAL	STD DEV	100289		72130	89702		38657
COLIFORM	MINIMUM	9400		500	9400		6600
(FCU/100ml)	MAXIMUM	24200		186000	242000		97000
	TOTAL SAMPLE	5		5	5		5
	MEAN	19.67		4.67	19.67		5.90
TOTAL IRON	STD DEV	8.93		1.79	7.29		2.00
(mg/l)	MINIMUM	9.40		2.95	9.40		4.40
	MAXIMUM	25.60		7.10	25.60		8.10
	TOTAL SAMPLE	3		3	3		3

Table 1-1 d) Descriptive Statistics run 3: 1.0 and 0.42 l/s

PARAMETERS		ByRF1			ByRF2		
		θi	θo	θe	θi	θo	θe
	MEAN	70	66	43	68		42
TURBIDITY	STD DEV	9.1	6.9	14	10		5.9
(NTU)	MINIMUM	61	57	30	56		34
	MAXIMUM	96	86	84	92		52
	TOTAL SAMPLE	13	13	13	13		13
	MEAN	129	119	3	134		39
SUSPENDED	STD DEV	15	17	5.5	27		7.2
SOLIDS	MINIMUM	100	100	12	97		30
(mg/l)	MAXIMUM	152	156	30	106		52
	TOTAL SAMPLE	9	9	9	9		9
	MEAN	40		37	40		39
VOLATILE	STD DEV	14		13	14		17
SOLIDS	MINIMUM	26		14	26		15
(mg/l)	MAXIMUM	65		55	65		61
	TOTAL SAMPLE	9		9	9		9
	MEAN	65		62	60		60
TRUE COLOR	STD DEV	5		4	1		4
(PCU)	MINIMUM	61		59	59		57
	MAXIMUM	68		64	61		63
	TOTAL SAMPLE	2		2	2		2
	MEAN	80860		32040	80860		25780
FAECAL	STD DEV	68430		36955	68429		30624
COLIFORM	MINIMUM	6700		4200	6700		5000
(FCU/100ml)	MAXIMUM	165000		94000	165000		9000
	TOTAL SAMPLE	5		5	5		5
	MEAN	9.83		3.00	9.80		3.40
TOTAL IRON	STD DEV	2.60		0.30	2.65		0.40
(mg/l)	MINIMUM	7.30		2.70	7.30		3.00
	MAXIMUM	12.50		3.20	12.50		3.80
	TOTAL SAMPLE	3		3	3		3

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Table 1-1 e)

Descriptive Statistics - Mean values for three filters runs
DyRF1: Q_1 = 2.0 l/s and DyRF2: Q_2 = 1.50 l/s; runs 1,2 and 3

PARAMETERS		DyRF1			DyRF2		
		Qi	Qo	Qe	Qi	Qo	Qe
	MEAN	167	159	91	160	162	96
TURBIDITY	STD DEV	86	76	57	90	82	69
(NTU)	MINIMUM	79	76	26	78	74	27
	MAXIMUM	420	335	225	420	380	336
	TOTAL SAMPLE	24	24	24	24	24	24
	MEAN	333	313	64	329	304	79
SUSPENDED	STD DEV	190	170	67	182	181	78
SOLIDS	MINIMUM	129	94	3	112	74	5
(mg/l)	MAXIMUM	881	808	286	868	780	314
	TOTAL SAMPLE	18	18	18	18	18	18
	MEAN	64		46	64		47
VOLATILE	STD DEV	19		16	19		15
SOLIDS	MINIMUM	32		23	32		21
(mg/l)	MAXIMUM	97		76	97		71
	TOTAL SAMPLE	17		17	17		17
	MEAN	109			109		
TRUE COLOR	STD DEV	32			32		
(PCU)	MINIMUM	71			71		
	MAXIMUM	157			157		
	TOTAL SAMPLE	9			9		
	MEAN	19440		10446	19440		7666
FAECAL	STD DEV	5455		8500	5455		3910
COLIFORM	MINIMUM	12600		800	12600		2600
(FCU/100ml)	MAXIMUM	29000		25900	29000		11400
	TOTAL SAMPLE	5		5	5		5
	MEAN	20.16		4.48	29.06		4.10
TOTAL IRON	STD DEV	7.29		2.86	7.40		1.70
(mg/l)	MINIMUM	14.00		1.60	13.90		1.80
	MAXIMUM	37.90		11.90	38.00		6.90
	TOTAL SAMPLE	10		10	10		10

Table 1-1 f) Descriptive Statistics run 1: 2.0 and 1.50 l/s

PARAMETERS		DyRF1			DyRF2		
		Qi	Qo	Qe	Qi	Qo	Qe
	MEAN	258	238	131	262	250	147
TURBIDITY	STD DEV	90	65	49	96	75	92
(NTU)	MINIMUM	108	180	34	174	184	44
	MAXIMUM	420	335	225	420	380	336
	TOTAL SAMPLE	7	7	7	7	7	7
	MEAN	597	547	122	553	531	142
SUSPENDED	STD DEV	209	158	110	199	159	116
SOLIDS	MINIMUM	296	383	26	330	341	31
(mg/l)	MAXIMUM	881	808	286	868	780	314
	TOTAL SAMPLE	5	5	5	5	5	5
	MEAN	74		54	74		54
VOLATILE	STD DEV	20		14	20		13
SOLIDS	MINIMUM	50		31	50		40
(mg/l)	MAXIMUM	97		76	97		71
	TOTAL SAMPLE	5		5	5		5
	MEAN	137			137		
TRUE COLOR	STD DEV	23			23		
(PCU)	MINIMUM	112			117		
	MAXIMUM	157			157		
	TOTAL SAMPLE	3			3		
	MEAN	21633		12367	21633		7966
FAECAL	STD DEV	6615		12135.	6615 3		4683
COLIFORM	MINIMUM	16200		800	16200		2600
(FCU/100ml)	MAXIMUM	29000		25600	29000		10900
	TOTAL SAMPLE	3		3	3		3
	MEAN	27 73		6.37	27.77		4.90
TOTAL IRON	STD DEV	9 46		4.92	9.51		1 90
(mg/l)	MINIMUM	19 20		2.30	19.20		3 20
	MAXIMUM	37 90		11.90	38.06		6 90
	TOTAL SAMPLE	3		3	3		3

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Table 1-1 g) Descriptive Statistics run 2: 2.0 and 1.5 l/s

PARAMETERS		DyRF1			DyRF2		
		Oi	Oo	Oe	Oi	Oo	Oe
	MEAN	117	113	60	117	112	59
TURBIDITY	STD DEV	29	29	28	30	30	32
(NTU)	MINIMUM	82	77	29	79	74	27
	MAXIMUM	150	150	11	150	140	110
	TOTAL SAMPLE	9	9	9	9	9	9
	MEAN	239	217	40	238	200	56
SUSPENDED	STD DEV	59	50	28	62	64	52
SOLIDS	MINIMUM	324	145	6.0	138	74	5 0
(mg/l)	MAXIMUM	158	295	81	323	277	148
	TOTAL SAMPLE	7	7	7	7	7	7
	MEAN	54		41	54		38
VOLATILE	STD DEV	24		19	24		16
SOLIDS	MINIMUM	32		23	32		21
(mg/l)	MAXIMUM	97		76	97		65
	TOTAL SAMPLE	6		6	6		6
	MEAN	82			82		
TRUE COLOR	STD DEV	17			17		
(PCU)	MINIMUM	71			71		
	MAXIMUM	102			102		
	TOTAL SAMPLE	3			3		
	MEAN	16150		7550	16150		7300
FAECAL	STD DEV	5020 4		6293 2	5020 4		5798
COLIFORM	MINIMUM	12600		3100	12600		3700
(FCU/100ml)	MAXIMUM	19700		12000	19700		11400
	TOTAL SAMPLE	7		2	2		2
	MEAN	18 17		3.22	18 15		3.38
TOTAL IRON	STD DEV	5.43		1 15	5 45		0 20
(mg/l)	MINIMUM	14 20		0 01	14 20		3 10
	MAXIMUM	26.10		0 13	26 10		3.50
	TOTAL SAMPLE	4		4	4		4

Table 1-1 h) Descriptive Statistics run 3: 2.0 and 1.5 l/s

PARAMETERS		DyRF1			DyRF2		
		Oi	Oo	Oe	Oi	Oo	Oe
	MEAN	144	142	89	145	141	92
TURBIDITY	STD DEV	67	69	53	71	67	52
(NTU)	MINIMUM	79	76	26	78	75	31
	MAXIMUM	243	258	171	270	246	165
	TOTAL SAMPLE	8	8	8	8	8	8
	MEAN	244	230	43	240	237	53
SUSPENDED	STD DEV	113	123	29	116	120	35
SOLIDS	MINIMUM	129	94	3 0,	112	83	7 0
(mg/l)	MAXIMUM	447	451	77	449	300	117
	TOTAL SAMPLE	6	6	6	6	6	6
	MEAN	66		64	65		56
VOLATILE	STD DEV	13		14	13		12
SOLIDS	MINIMUM	50		31	79		41
(mg/l)	MAXIMUM	79		63	69		69
	TOTAL SAMPLE	6		6	6		6
	MEAN	107			107		
TRUE COLOR	STD DEV	32			32		
(PCU)	MINIMUM	83			83		
	MAXIMUM	143			143		
	TOTAL SAMPLE	3			3		
	MEAN	15 2		4.3	14 9		4 3
TOTAL IRON	STD DEV	1 43		2.51	1 65		2.60
(mg/l)	MINIMUM	14 00		1 60	13 90		1 80
	MAXIMUM	16.80		6 60	16.80		6.90
	TOTAL SAMPLE	3.00		3.00	3.00		3.00

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Table 1-1 i)

Descriptive Statistics - Mean values for three filters runs
DyRF1: Q_1 = 2.65 l/s and DyRF2: Q_2 = 0.70 l/s; runs 1,2 and 3

PARAMETERS		DyRF1			DyRF2		
		θi	θo	θe	θi	θo	θe
	MEAN	71	33	67	70	32	64
TURBIDITY	STD DEV	64	20	55	64	17	54
(NTU)	MINIMUM	32	16	32	32	16	32
	MAXIMUM	360	120	290	380	100	700
	TOTAL SAMPLE	34	34	34	35	35	35
	MEAN	146	146	19	140	127	20
SUSPENDED	STD DEV	138	151	18	154	148	17
SOLIDS	MINIMUM	58	63	6.0	65	50	5.0
(mg/l)	MAXIMUM	645	665	80	760	702	77
	TOTAL SAMPLE	23	23	23	24	24	24
	MEAN	56		40	60		42
VOLATILE	STD DEV	30		17	29		20
SOLIDS	MINIMUM	17		13	16		3
(mg/l)	MAXIMUM	154		70	154		82
	TOTAL SAMPLE	22		22	23		24
	MEAN	60		48	60		50
TRUE COLOR	STD DEV	23		26	22		28
(PCU)	MINIMUM	26		21	26		20
	MAXIMUM	104		112	104		126
	TOTAL SAMPLE	12		12	13		13
	MEAN	45327		17409	40742		14717
FAECAL	STD DEV	36583		10439	36993		10623
COLIFORM	MINIMUM	19000		2800	4300		500
(FCU/100ml)	MAXIMUM	133000		63400	133600		35000
	TOTAL SAMPLE	11		11	12		12
	MEAN	15.67		2.55	11.61		2.71
TOTAL IRON	STD DEV	18.31		0.77	16.24		1.41
(mg/l)	MINIMUM	4.70		1.30	3.50		1.40
	MAXIMUM	57.50		3.70	57.50		6.40
	TOTAL SAMPLE	9		9	10		10

Table 1-1 j) Descriptive Statistics run 1: 2.65 and 0.70 l/s

PARAMETERS		DyRF1			DyRF2		
		θi	θo	θe	θi	θo	θe
	MEAN	59	32	58	58	32	54
TURBIDITY	STD DEV	6.0	5.79	4.56	6.53	4.82	4.53
(NTU)	MINIMUM	49	25	50	46	25	45
	MAXIMUM	70	40	65	71	44	60
	TOTAL SAMPLE	12	12	17	13	13	13
	MEAN	104	99	18	57		52
SUSPENDED	STD DEV	15	14	10	20		21
SOLIDS	MINIMUM	97	85	6.0	32		28
(mg/l)	MAXIMUM	139	129	34	84		82
	TOTAL SAMPLE	8	8	8	9		9
	MEAN	50		44	575		52
VOLATILE	STD DEV	15		12	20		21
SOLIDS	MINIMUM	32		32	32		28
(mg/l)	MAXIMUM	72		61	84		82
	TOTAL SAMPLE	7		7	9		9
	MEAN	38		30	46		37
TRUE COLOR	STD DEV	19		11	22		18
(PCU)	MINIMUM	26		21	26		20
	MAXIMUM	60		42	68		60
	TOTAL SAMPLE	3		3	4		4
	MEAN	22533		10233	24650		14175
FAECAL	STD DEV	3075		1966	4922		7283
COLIFORM	MINIMUM	19000		8000	19000		7000
(FCU/100ml)	MAXIMUM	24600		11700	31000		24000
	TOTAL SAMPLE	3		3	4		4
	MEAN	16.07		3.32	5.83		3.25
TOTAL IRON	STD DEV	16.45		4.38	1.08		1.78
(mg/l)	MINIMUM	5.30		2.95	3.50		2.60
	MAXIMUM	35.00		3.70	7.90		6.40
	TOTAL SAMPLE	3		3	4		4

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Table 1-1 k) Descriptive Statistics run 2: 2.65 and 0.70 l/s

PARAMETERS		DyRF1			DyRF2		
		Ci	Co	Ce	Ci	Co	Ce
TURBIDITY (NTU)	MEAN	76	31	70	70	29	66
	STD DEV	88	13	70	93	13	67
	MINIMUM	32	16	32	32	16	32
	MAXIMUM	340	70	290	380	65	280
	TOTAL SAMPLE	13	13	13	13	13	13
SUSPENDED SOLIDS (mg/l)	MEAN	169	168	20	183	150	20
	STD DEV	185	192	17	221	210	17
	MINIMUM	58	6.0	6.0	66	52	5.0
	MAXIMUM	645	665	59	760	702	56
	TOTAL SAMPLE	9	9	9	9	9	9
VOLATILE SOLIDS (mg/l)	MEAN	60		40	64		35
	STD DEV	44		24	46		23
	MINIMUM	17		13	16		3
	MAXIMUM	154		70	154		67
	TOTAL SAMPLE	9		9	8		9
TRUE COLOR (PCU)	MEAN	67		53	67		52
	STD DEV	21		18	21		24
	MINIMUM	54		40	54		27
	MAXIMUM	104		85	104		92
	TOTAL SAMPLE	5		5	5		5
FAECAL COLIFORM (FCU/100ml)	MEAN	31200		18260	30800		16940
	STD DEV	7694		13288	7367		12565
	MINIMUM	22000		3000	22006		500
	MAXIMUM	41000		35600	41000		35000
	TOTAL SAMPLE	5		5	5		5
TOTAL IRON (mg/l)	MEAN	24 27		2 63	24 27		2 28
	STD DEV	28 85		0 32	28 85		0.51
	MINIMUM	5 70		2 45	5.70		1 70
	MAXIMUM	57.50		3 00	57 50		2.65
	TOTAL SAMPLE	3		3	3		3

Table 1-1 l) Descriptive Statistics run 3: 2.65 and 0.70 l/s

PARAMETERS		DyRF1			DyRF2		
		Ci	Co	Ce	Ci	Co	Ce
TURBIDITY (NTU)	MEAN	77	38	76	77	36	74
	STD DEV	70	35	71	70	29	71
	MINIMUM	37	17	34	35	20	32
	MAXIMUM	252	120	255	252	108	253
	TOTAL SAMPLE	9	9	9	9	9	9
SUSPENDED SOLIDS (mg/l)	MEAN	166	175	22	165	145	21
	STD DEV	156	189	29	154	158	20
	MINIMUM	74	63	6	65	58	5 0
	MAXIMUM	476	552	80	458	455	77
	TOTAL SAMPLE	6	6	6	6	6	6
VOLATILE SOLIDS (mg/l)	MEAN	58		35	58		38
	STD DEV	14		5	14		6
	MINIMUM	40		29	40		30
	MAXIMUM	80		41	84		45
	TOTAL SAMPLE	6		6	6		6
TRUE COLOR (PCU)	MEAN	67		56	67		59
	STD DEV	23		38	23		41
	MINIMUM	50		30	50		34
	MAXIMUM	100		112	100		120
	TOTAL SAMPLE	4		4	4		4
FAECAL COLIFORM (FCU/100ml)	MEAN	91667		23167	78767		11733
	STD DEV	45446		34844	66693		14110
	MINIMUM	4300		2800	4300		2800
	MAXIMUM	133000		63400	133000		28000
	TOTAL SAMPLE	3		3	3		3
TOTAL IRON (mg/l)	MEAN	6 67		1 70	6.67		1 75
	STD DEV	2 30		0 34	2.30		0 35
	MINIMUM	4.70		1.30	4.70		1 40
	MAXIMUM	9.20		2 15	9 20		2 10
	TOTAL SAMPLE	3		3	3		3

APPENDIX 2

Table 2-1

Linear Regression Model between Suspended Solids and Raw Water Turbidity Based on Column Settling Test Results.

Turbidity Range (NTU)	Descriptive Statistics for Suspended Solids measured in the laboratory	Regression Model	R^2	Descriptive Statistics for Suspended solids estimated by the Regression Model
42-210	Mean 128 STD 67 Minimum 32 Maximum 355 sample size 219	$Y^{(1)} = 1.2929X^{(2)}$	0.9758	Mean 130 STD 57 Minimum 54 Maximum 272 sample size 219
42-50	Mean 55 STD 9 Minimum 32 Maximum 67 sample size 39	$Y^{(1)} = 1.1570X^{(2)}$	0.9794	Mean 55 STD 4 Minimum 49 Maximum 58 sample size 39
51-100	Mean 88 STD 23 Minimum 49 Maximum 158 sample size 69	$Y^{(1)} = 1.2311X^{(2)}$	0.9692	Mean 87 STD 21 Minimum 64 Maximum 123 sample size 69
101-150	Mean 154 STD 37 Minimum 52 Maximum 227 sample size 78	$Y^{(1)} = 1.2257X^{(2)}$	0.9778	Mean 155 STD 21 Minimum 125 Maximum 184 sample size 78
151-210	Mean 236 STD 46 Minimum 185 Maximum 355 sample size 33	$Y^{(1)} = 1.4220X^{(2)}$	0.9883	Mean 237 STD 23 Minimum 215 Maximum 299 sample size 33

(1) Suspended Solid Concentration (mg/l) (2) Turbidity (NTU)

APPENDIX 3

Tabla 3-1

Real removal efficiency variations over the filter runs for different Q_i's and V = 2.0 m/h. Descriptive Statistics for physical, chemical and bacteriological parameters.

STATISTICS	Nominal Q_i (l/s)					
	0 42	0.70	1.00	1.50	2.00	2.65
TURBIDITY						
Minimum	30.0	34 0	38.0	24.0	28 0	37.0
Mean	34 6	50 7	51.2	51.8	51.5	52.2
Maximum	38.0	62.0	75.0	70 0	67.0	69.0
STD	3.2	11 9	14 0	20.5	17 4	13.1
Standard error	1.4	5.9	6 2	10.2	8.7	6.6
95% confidence	2.8	11 6	12.2	20.2	17.1	12.9
99% confidence	3 7	15.3	16.2	26 4	22.5	17.0
Sample size	5	4	5	4	4	4
SUSPENDED SOLIDS						
Minimum	39 4	78.5	75 9	71.0	74 8	79.7
Mean	59.38	87.5	83.3	83 8	85.9	87.1
Maximum	75.2	92.5	96.0	97 0	96.7	91.6
STD	15.1	7 0	9.0	13 4	9.6	5.5
Standard error	6 8	4 1	4 0	6.7	4.8	2.8
95% confidence	13.3	8.0	7 9	13.2	9.4	5.4
99% confidence	17.5	10.5	10.4	17.3	12.4	7 1
Sample size	5	3	5	4	4	4
VOLATILE SOLIDS						
Minimum	13.0	12 0	19 0	17.0	22.0	20.0
Mean	28 6	33 3	27 8	27.5	25.0	31.7
Maximum	44 0	49 0	48 0	34.0	28.0	44.0
STD	12 1	15.6	11.6	7.9	3.5	12.5
Standard error	5.4	7 8	5.2	3.9	1.7	6.2
95% confidence	10 6	15.3	10.1	7.7	3.4	12.2
99% confidence	14 0	20 2	13.3	10 1	4.5	16.1
Sample size	5	4	5	4	4	4
TRUE COLOR						
Minimum	0 0	0 0	-	-	-	0.0
Mean	13.3	24.2	-	-	-	23.8
Maximum	31 0	39 0	-	-	-	46.0
STD	15 9	18.4	-	-	-	18 8
Standard error	9 2	9 2	-	-	-	9.4
95% confidence	18.0	18 0	-	-	-	18.0
99% confidence	23.7	23.7	-	-	-	24.3
Sample size	3	4	-	-	-	4
TOTAL IRON						
Minimum	34 0	55.0	42 0	76.0	75 0	54.0
Mean	47.3	70 3	55 6	81.3	83 7	71.0
Maximum	54.0	85.0	63.0	88 0	90 0	86.0
STD	11.5	15 0	11.8	6.1	7 8	16 1
Standard error	6.7	8 7	6 8	3.5	4.5	9.3
95% confidence	13.1	17 0	13 4	6 9	8 8	18.2
99% confidence	17.2	22.4	17.6	9.1	11.6	24.0
Sample size	3	3	3	3	3	3
FAECAL COLIFORM COUNTS						
Minimum	0.26	0 55	0.22	0 04	0.02	0.32
Mean	0 4	0.7	0.5	0 4	0 41	1.0
Maximum	0.60	1 00	1.00	0.80	0.80	1.50
STD	0 15	0.23	0.35	0.54	0.55	0.61
Standard error	0.07	0 13	0 18	0.38	0.39	0.35
95% confidence	0 14	0.26	0.34	0.74	0.76	0.69
99% confidence	0 19	0.34	0 45	0.98	1 00	0.91
Sample size	4	3	4	2	2	3

(-)samples were not taken.

APPENDIX A

Cumulative frequency distribution of settling velocities for different Q_i.

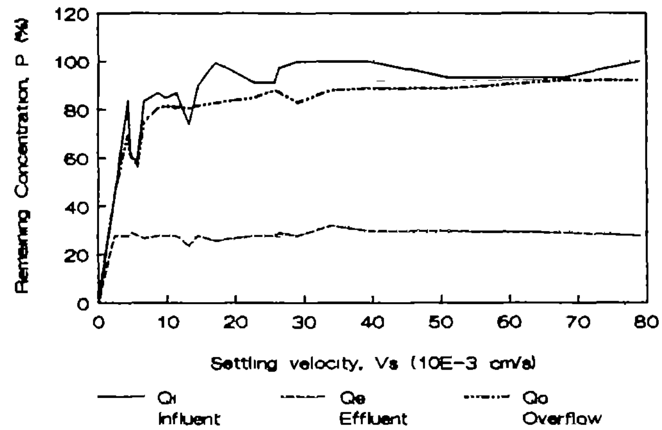

Figure A-1 a) Nominal Q_i = 1.0 l/s

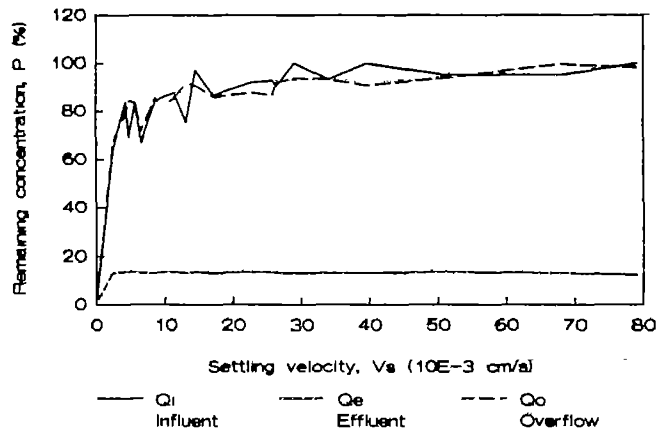

Figure A-1 b) Nominal Q_i = 2.0 l/s.

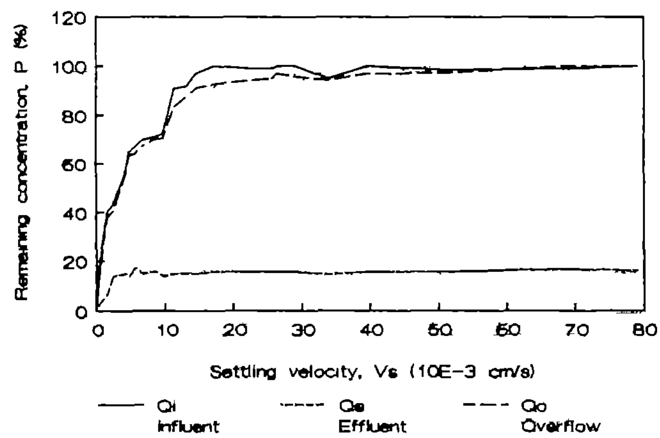

Figure A-1 c) Nominal Q_i = 2.65 l/s

APPENDIX B

Typical variation in water flows Q_i, Q_e and Q_o over the filter runs.

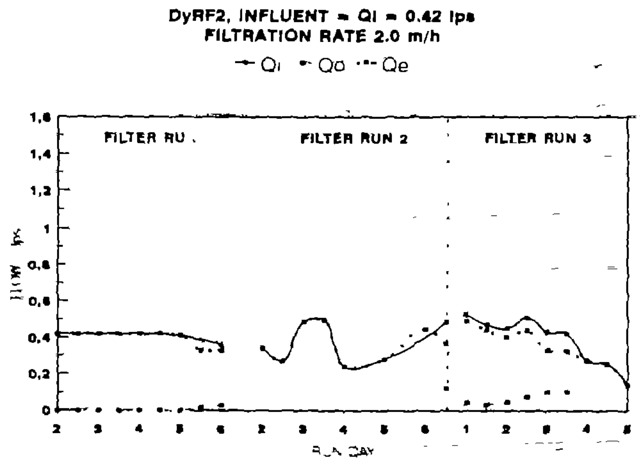

Figure B-1 a) Nominal Q_i = 0.42 l/s.

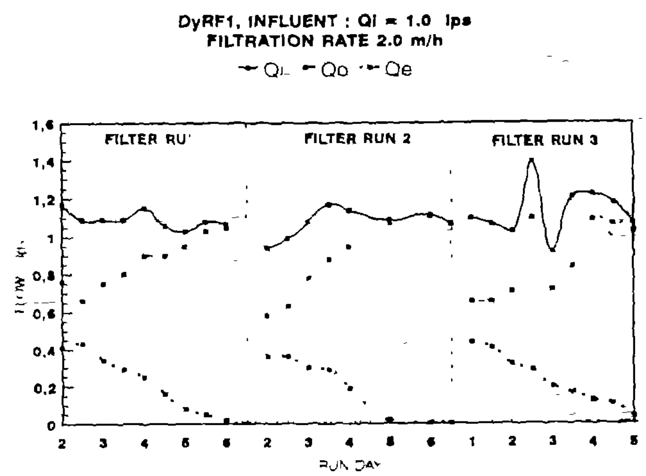

Figure B-1 b) Nominal Q_i = 1.0 l/s.

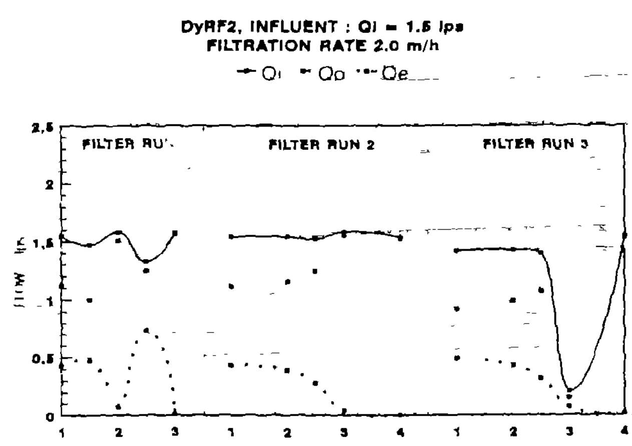

Figure B-1 c) Nominal Q_i = 1.50 l/s.

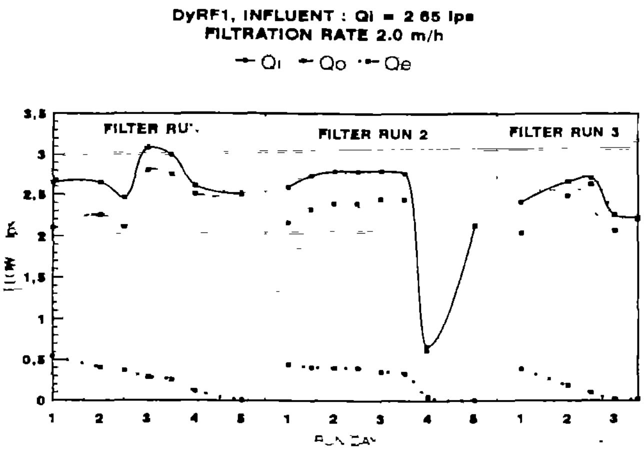

Figure B-1 d) Nominal Q_i = 2.65 l/s and V = 2.0 m/h.

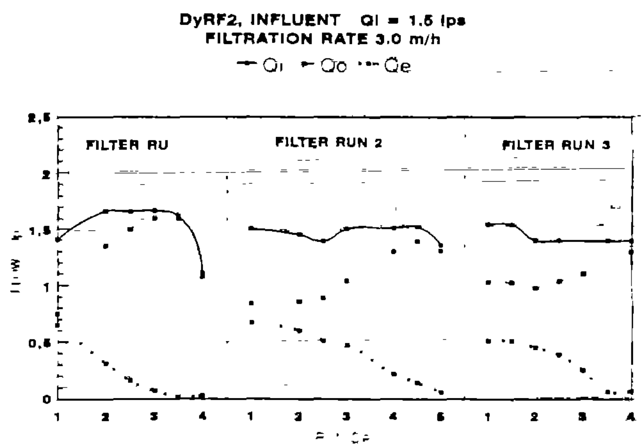

Figure B-1 e) Nominal Q_i = 1.50 l/s and V = 3.0 m/h.

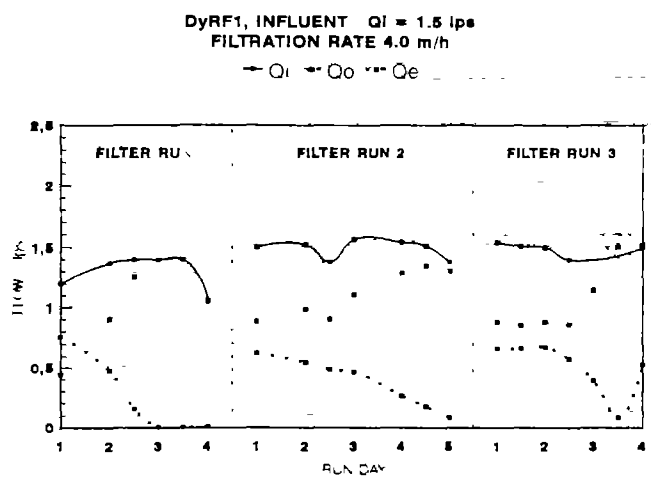

Figure B-1 f) Nominal Q_i = 1.50 l/s and V = 4.0 m/h.

APPENDIX C

Typical variation of headloss for different Q_i and $V = 2.0$ m/h.

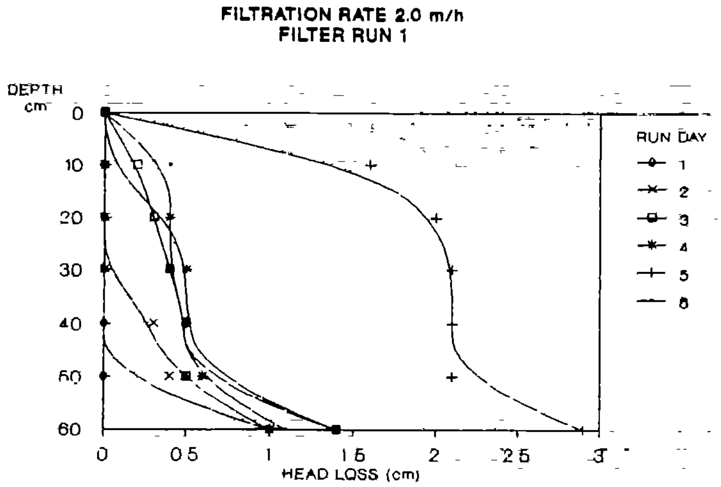

Figure C-1 a) Nominal Q_i = 0.42 l/s.

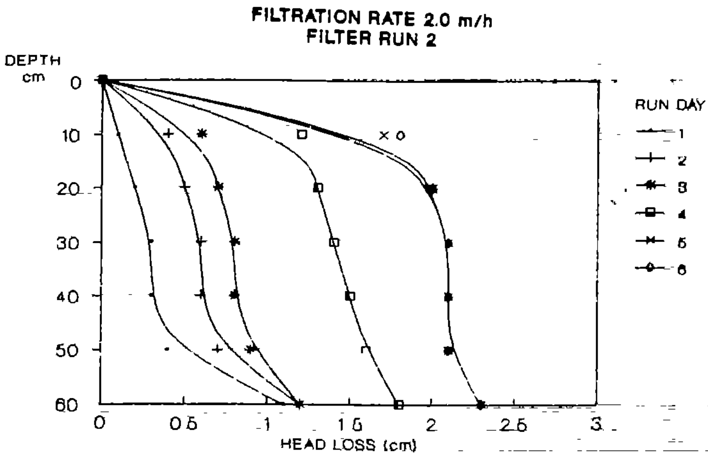

Figure C-1 b) Nominal Q_i = 0.70 l/s.

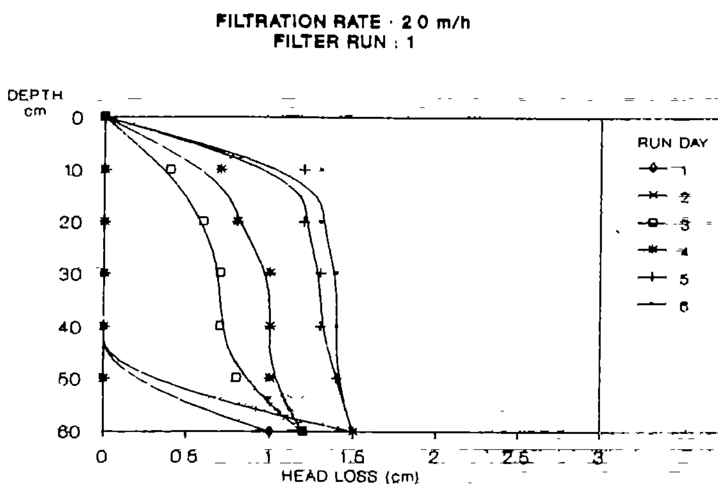

Figure C-1 c) Nominal Q_i = 1.00 l/s.

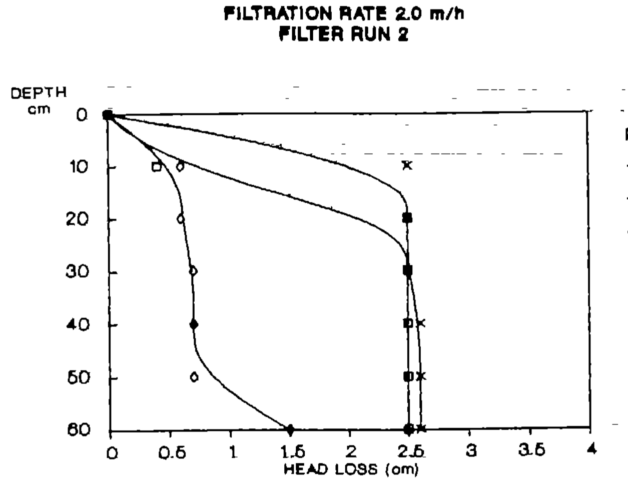

Figure C-1 d) Nominal Q_i = 1.50 l/s.

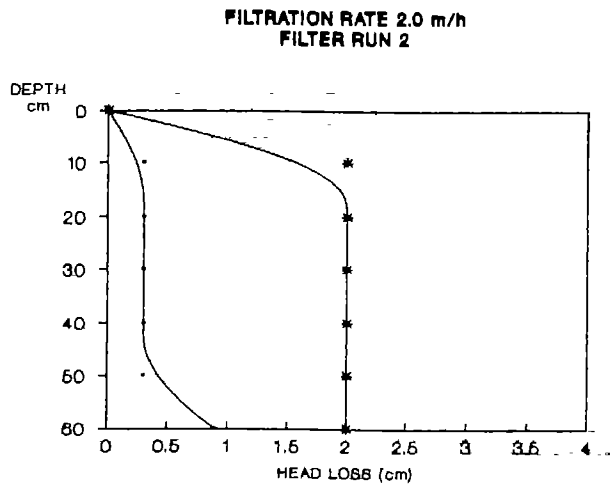

Figure C-1 e) Nominal Q_i = 2.0 l/s.

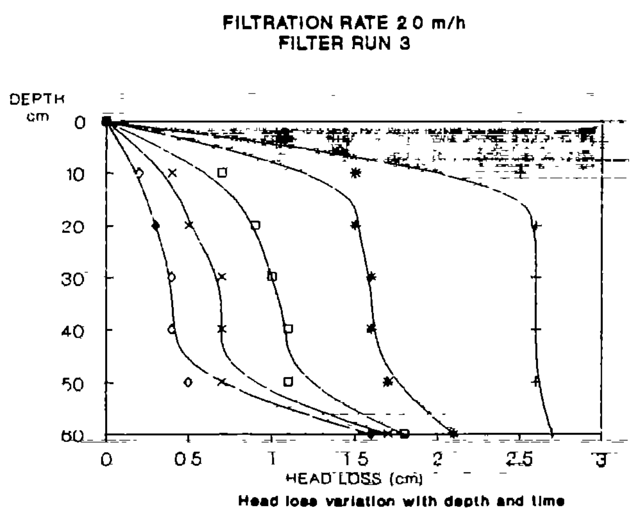

Figure C-1 f) Nominal Q_i = 2.65 l/s.