IRC

International Water and Sanitation Centre
WHO Collaborating Centre

[redacted]

Pre-treatment Alternatives for Drinking Water Supply Systems

Selection, Design, Operation and Maintenance

Universidad del Valle

Ministerio de Salud de Colombia

ORGANIZACION MUNDIAL DE LA SALUD
Oficina Sanitaria Panamericana

EMCALI

DNP
Departamento Nacional de Planeación
Colombia

CINARA
CENTRO INTER-REGIONAL DE ABASTECIMIENTO Y REMOCION DE AGUA

CINARA, is a non-profit foundation, involved in research development and organized technology transfer in drinking water supply and sanitation. Its work covers the rural sector, small and medium towns and peri-urban communities - traditionally areas with lowest coverage in public services.

The staff of the foundation comprises in engineers, social scientists and basic scientists. They work closely together with and in support of professionals and technicians of cooperation institutions, water sector agencies and communities towards the overall goal of the improvement of the sanitary conditions and the protection of the environment.

CINARA maintains an Agreement on Academic and Research Cooperation with the Universidad del Valle and develops projects through agreements and contracts with local, national and international organizations involved in the water sector taking into account the crucial role of water projects in development and in the improvement of the quality of life.

For more information, address to:
CINARA
A.A. 25360
Cali, Colombia (Sur América)

IRC INTERNATIONAL WATER AND SANITATION CENTRE

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

The centre facilitates the availability and use of appropriate knowledge and information in the water, sanitation and related environment sectors in developing countries.

Activities include capacity development for information management, exchange of available knowledge and information, and development and transfer of new knowledge on priority issues. All activities take place in partnership with organizations in developing countries, United Nations organizations, bilateral donors, development banks, and non-governmental organizations.

Emphasis in programme activities is on community-based approaches including rural and low-income urban water supply and sanitation systems, community participation and hygiene education, gender issues, sustainable systems, rehabilitation and environmental management.

The multi-disciplinary staff provides the support through development and demonstration projects, training and education, publications, documentation services, general information dissemination as well as through advisory services and evaluation.

For further information:
IRC
P.O. Box 93190
2509 AD The Hague
The Netherlands

Telephone:	+31 - (0)70-33 141 33
Telefax:	+31 - (0)70-38 140 34
Telex:	33296 irc nl
Cable:	Worldwater, The Hague

Pre-treatment Alternatives for Drinking Water Supply Systems

Selection, Design, Operation and Maintenance

prepared by

Gerardo Galvis, Jan Teun Visscher, Javier Fernández
and Fabiola Berón

with support from

Antonio Castilla, María Mercedes Hincapié, Viviana Vargas
and other CINARA staff members

Occasional Paper 22

IRC International Water and Sanitation Centre
The Hague, The Netherlands

December 1993

This publication was developed in the context of the International Research, Development and Transfer Project on Pre-treatment systems for drinking water supply. It is available both in English and Spanish. It was prepared with financial support from the Research and Development Department of the Netherlands Ministry of Foreign Affairs.

Table of Contents

List of Tables

List of Figures

Preface

This document comprises information about the selection, design, operation and maintenance of different pre-treatment alternatives all using coarse media filtration followed by slow sand filtration. It also includes some general concepts and consideration on drinking water treatment for domestic use. It has been developed to cater for the need for information about pre-treatment technologies which can be combined with slow sand filters. The concepts discussed in the document very much enhance the possibility to provide reliable water supply treatment particularly in rural areas and small and medium sized towns. It has particular relevance for staff involved in planning, design and supervision of operation and maintenance of water supply schemes in developing countries and countries with increasing water pollution problems. Part of the information is still very much under development and therefore a somewhat conservative approach is still needed until detailed confirmation is being obtained.

The present document has been developed in the context of the integrated research, development and transfer project on pre-treatment systems for drinking water supply. This project was developed on the basis of earlier investigations in Colombia and India in the context of the International research and demonstration project on slow sand filtration. The first phase of the pre-treatment project has been carried out in Colombia in the period 1989 - 1993. It was implemented by CINARA, Centro Inter-regional de Abastecimiento y Remoción de Agua and IRC International Water and Sanitation Centre.

Different pre-treatment systems have been compared in this project both in a research station in Cali, Colombia and in full scale plants. The research has clearly demonstrated the vast potential of the combination of two-stage roughing filtration and slow sand filtration. It has proven the feasibility of applying a multi-barrier concept towards water treatment without using chemical coagulants leaving low dose disinfection as a possible final safety barrier. Although further technical and economical optimization is required, two-stage roughing filtration and slow sand filtration is a very reliable and effective water treatment which can be managed even by small communities. The project and project findings are discussed in more detail in annex 3.

The project received financial support from the Research and Development Department of the Netherlands Ministry of Foreign Affairs. Inputs were also provided by Colombian institutions including: the National Planning Department, the Ministry of Health, Universidad del Valle, Empresas Municipal de Cali EMCALI, and Comité Cafeteros and by international organizations including: WHO, PAHO, CEPIS, IDRC, IRCWD, University of Surrey, the School of Engineering of Sao Carlos and the University of Sao Paolo.

This publication was developed by a group of people including Mr. Gerardo Galvis, Mr. Jan Teun Visscher, Mr. Javier Fernández, Ms. Fabiola Berón, Mr. Antonio Castilla, Ms. María Mercedes Hincapié and Ms. Viviana Vargas. This core group received support from different social scientists, basic scientist, designers and support staff. CINARA and IRC would like to thank all who have contributed to the project and this publication.

The research is being continued as the second phase of the pre-treatment project has been approved. We therefore very much look forward to your comments and reactions, which will contribute to the preparation of a more elaborated publication.

1. Introduction

Good drinking water supply and basic sanitation are essential to improve the health and well being of the population and enable socio-economic development, particularly in countries where a significant part of the population is effected by water-related diseases. This type of diseases are amongst the most important reasons for morbidity and mortality in the world, effecting in principle the human population living in communities with limitations in the sanitary infrastructure. This limits the possibility of a healthy life and reduces significantly the productivity of the population.

Enhanced coverage of systems continuously producing water in sufficient quantity and of sufficient quality is very important to reduce the incidence of water-related diseases. Water quality improvement needs to ensure a low risk in transmission of disease-carrying organisms and an end product which is attractive to the consumer. Furthermore, it should permit the normal functioning of all components of the water supply system.

Countries with limitations in the sanitary infrastructure have to give priority to the selection, development and transfer of technology for water supply and sanitation systems which matches their specific local conditions. To enhance long-term sustainability the selected technology needs to match the management capacity of the beneficiary communities, to ensure that it can be operated and maintained with minimum external support and at acceptable costs to the users.

In the Latin American region and the Caribbean urban water supply coverage increased from 186 million (84%) in 1980 to 256 million persons (88%) in 1988. Coverage with sewerage systems increased from 100 million (40%) in 1980 to 142 million (49%) in 1988. In the same period coverage with water supply systems in rural areas comprising house connections increased from 49 million (40%) to 68 million (55%) and rural sanitation maintained a coverage at 32% (WHO, 1990). Many of these systems, however, are not functioning adequately and produce water of poor quality because of problems in operation and maintenance and with efficient water use. Thus the huge investment in these type of systems does have a very limited impact on the improvement of the health conditions.

Most water sources are not well managed and are not protected from domestic and industrial pollution. This not only creates problems in allocation and use of the water source but also results in considerable water contamination. This is particularly the case for surface water, which for example is the main water source for over 50% of the communities in Latin America and the Caribbean. In some areas, such as the valley of the Cauca river and the Andes region in Colombia, this may even be the source for over 80% of the municipalities and over 60% of the rural centres. These water sources face quickly increasing levels of suspended solids, turbidity and faecal coliforms, due to erosion and discharge of untreated waste of over 90% of the municipalities. The hygienic risk associated with these water sources is increasing and getting more out of line with the established norms for water sources which can be used for producing drinking water.

Improving the management and protection of water sources is urgently needed but often will not be sufficient to provide good quality drinking water. Then suitable and efficient treatment of the water will be required as discussed in chapters 2 and 3. Chapter 4 provides information on slow sand filtration summarizing the more extensive information provided in IRC's Technical Paper 24 "Slow Sand Filtration for Community Water Supply"; planning, design, construction, operation and maintenance, Cali, Colombia.

Slow sand filtration is indeed a water treatment technology with enormous potential particularly for rural areas and small and medium towns. Its utilization, however, has been limited because it could not cope with the high level of contamination actually present in an increasing number of surface water sources, due to erosion problems and increase of human activities in the water catchment areas. Pre-treatment using gravel filtration has been identified as a good possibility to overcome this limitation and enhances the possible use of slow sand filtration.

The other chapters of this publication therefore focus on different pre-treatment technologies using gravel as filter medium. Chapter 5 provides an overview of different alternatives, followed by a discussion on the treatment mechanisms involved in chapter 6 and a selection process in chapter 7. Chapters 8, 9, 10 and 11, describe in more detail dynamic roughing filtration, upflow roughing filtration, downflow roughing filtration and horizontal roughing filtration. Chapter 12 presents general considerations about the construction of pre-treatment systems and chapter 13 provides general guidance on operation and maintenance of such systems.

2. *Basic Concepts About Water Supply Treatment*

2.1 Key factors

A number of factors need to be taken into account to ensure the long-term sustainability of water supply systems. These factors can be summarized as follows: (adapted from earlier work in the pre-treatment project, as presented in Lloyd et al. 1987).

- *Coverage*

With many countries in the world not having good and reliable water supply it is essential that coverage is increased as quickly as possible. This implies the construction of new systems which match the need of the population and provide a service level which they appreciate and are willing and able to pay for, possibly with some financial support of the Government. Equity in the distribution of water needs to be taken into account. This is particularly important in mountain zones where the population living in the higher areas needs to get the same share of water as those living in lower areas, which may require the introduction of valves and pressure break chambers. Furthermore, an adequate operation and maintenance system is required to sustain the functioning of the systems. This needs to include good monitoring, possibly with help of the community, and promotion of efficient water use.

- *Continuity*

Ensuring continuous water supply is essential to avoid contamination in the distribution network and may require specific attention in the design phase. It may be needed to provide alternative energy sources in case of potential breakdown of the prime source, or provide additional treatment lines in case there is a risk that a specific treatment system will not perform continously well.

- *Quantity*

Water quantity is very important for the reduction of water borne diseases. Water has to be provided for drinking, cooking and personal hygiene. However, other uses may have to be catered for as well such as, watering of domestic animals and small scale irrigation. The quantity of water to be provided per capita per day therefore will depend on a social-economic and cultural analysis of the potential water use, and an intensive discussion with the recipient community. The indiscriminate use of norms and guideline values for water supply needs to be avoided to prevent over or under design of projects.

- *Quality*

The water which is being provided to the users needs to be free of chemical substances and micro-organisms which could cause diseases. Furthermore, the quality needs to be acceptable for the users and should not damage the distribution system. It is important to try to identify the most suitable source which is available for the water supply. In this respect the limited information available on water quality maybe an important limitation for the selection of water sources particularly in rural areas. Evaluation of the water sources is essential and should include a sanitary inspection to analyze the risk of contamination prevalent in the catchment area. A sanitary inspection will provide more reliable data then a single water quality analysis, and will very much help to determine the required type of treatment and number of treatment steps.

- *Cost*

The cost of the water supply needs to be within reach of the users of the system. It is important that the tariff covers all aspects including operation and maintenance, as well as recuperation of the initial investment, and leaves room for future extension of the system. It may be possible to obtain an initial subsidy from the Government, but it may not be expected that such a subsidy can be provided for a longer period of time.

- *Management capacity*

The management capacity required for a specific water supply system is an important selection criteria. Increasingly, managerial tasks are being delegated to local levels. This implies that in the design full account needs to be taken of the organizational capacities at the local level to operate, maintain and manage the water supply system with minimum support from outside.

- *Water culture*

The water culture of a community will very much determine what type of system can be put in place, and what type of design criteria needs to be taken into account. The water culture includes the attitude of the community towards the environment and in particular the water source, but also the way in which community members handle water in the household. These aspects need to be identified in ample discussions with the community at large.

2.2 Technology selection

To overcome the hygienic problems presented in a given water source an adequate technology needs to be selected which is in harmony with the local conditions. The selection will depend on a variety of factors, such as the reliability of local infrastructure, the organizational capacities of the different actors involved and the possibilities for external support to be provided to the local organizations. A very important first step in such a selection process is a review of the different technologies which have already been in operation in the region. An in-depth evaluation of these technologies often will give a good indication of potential constraints and the possibilities and necessities to modify available technologies. Technology selection thus will depend on the one hand, on the characteristics of the hygienic problems in the water source, and on the other hand on the external conditions present in the region.

Evaluation of the hygiene risks

The pollution problems in a water catchment area will depend on the characteristics of the area itself and the diversity of the activities which are taken place in the area: agriculture, industry, etc. Waste products of these activities may create considerable pollution in the water source. In general, a sanitary survey of the catchment area, will provide very useful information to estimate the hygienic risks associated with the water source. This sanitary survey is a systematic analysis of the water catchment area to identify potential sources of contamination and the possible behavior of the catchment area. It needs to be supported with a series of water quality analysis to obtain a good indication of the biological, micro-biological and physical quality of the water source. The overall result of the analysis needs to be carefully assessed as test results may not be reliable, and may not be representative to the differences in conditions throughout the year. If properly interpreted, the sanitary survey supported by some water quality analysis will provide a reasonable reliable assessment of the level of contamination one may expect throughout the year in the given water source. Figure 2.1 presents an overview of three surface water sources with different hygienic risk levels.

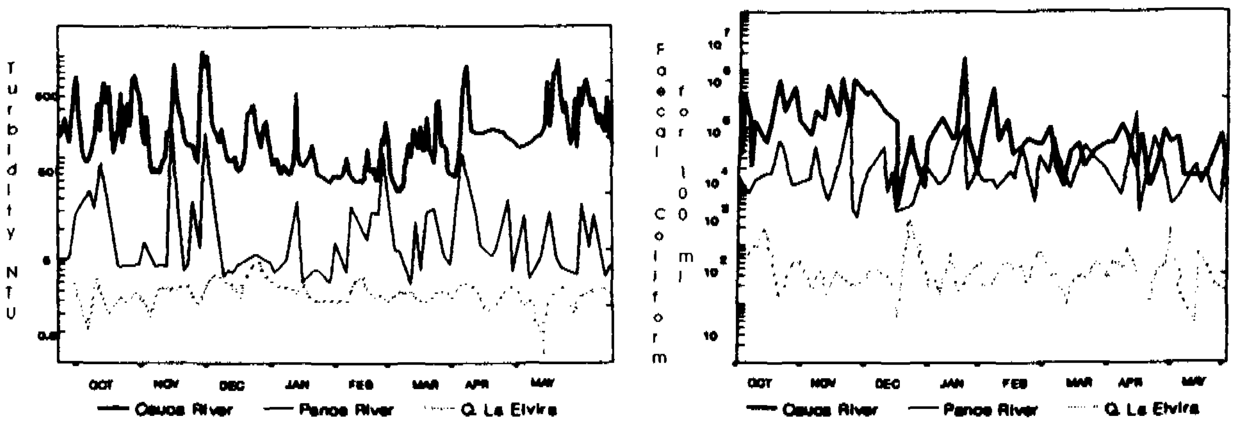


Figure 2.1 Levels of hygienic risks associated with three surface water sources in the department of Valle del Cauca, Colombia, Period: October 1990 to May 1991.

The incidence of water-related diseases in communities can be another indicator of the sanitary risk of their water source or their hygiene habits. Health statistics, however, are not always reliable and therefore water resources may even present a high sanitary risk in communities with relatively low reported disease incidents. The international experiences supports the need to introduce water treatment technology in communities which rely on contaminated water sources. When introducing water treatment, however, it is equally important to review and discuss the way in which the community handles the water (water culture) which may be an important cause of re-contamination.

Identification of the water treatment technology

Knowledge about the nature and level of contamination in the water source permits to identify the combination of water treatment technologies required. In general, two principal concepts have to be taken into account: the multi-barrier treatment concept and the concept of integrated treatment.

- *Multi-barrier treatment concept*

This concept has a long history and has evolved gradually from water supply treatment practices and experiences (Craun, 1988). Under this concept, reliance is placed on more than one stage of treatment to produce safe water for the consumers. Together these stages progressively remove the contaminants from the raw water and consistently produce a safe and wholesome drinking water. Ideally, water low in sanitary risk should be obtained before the final treatment stage, which then would be considered as a safety barrier (Lloyd, 1974; Galvis et al, 1992). Disinfection is normally the last line of defense or final barrier, and will only be effective if the preceding barriers virtually remove all harmful micro-organisms and other substances which may interfere with the disinfection process. This is particularly important in conditions were disinfection has shown to be still very vulnerable. According to the Pan American Health Association over 75% of the disinfection systems in Latin America are not functioning well (Reiff, 1988).

Whereas the multi-barrier concept developed gradually on the basis of field experiences under the pressure of increasingly strict regulations, we can turn now the tables. New systems can be designed on the basis of deliberate decisions about the different barriers which can be included. The first barrier is selecting and protecting the best available source. This is far more economical and effective than allowing developments to occur in the water shed area and rely subsequently on advanced treatment (Okun, 1991).

Watershed management in Europe and the USA is guided by regulations but still not sufficiently well established as for instance shown in the current debate about the nitrate problem. Adequate watershed management in developing countries is even more in its infancy, with insufficient legislation and very limited trained staff to oversee its implementation. It is essential that more attention is paid to this issue which, particularly in rural areas, will have to rely heavily on close collaboration between communities and water agencies. It may, for example, be required to regulate tree felling in remote areas, which can often only be achieved by social control. Use of pesticides may have to be restricted in certain areas, which brings up the question of compensation for the farmers in the watershed, in regions which have just embarked on a cash economy.

Basic water treatment concepts indicate the convenience to first separate the heavier or larger material from the water and gradually separate or inactivate smaller particles including, colloidal material and micro-organisms. Surface water sources with impurities of different natures and sizes need a larger number of different barriers than groundwater to assure good quality effluent with low level of sanitary risk. Preferably in those regions where this is viable, final disinfection is being applied as last safety barrier.

- *Concept of integrated treatment*

Under this concept the possibilities and limitations of each barrier or treatment stage to remove different contaminants are being quantified and treatment processes are combined in such a way that all impurities can be removed effectively (Lloyd et al, 1991). The combination of different treatment steps needs to be selected in such a way that a satisfactory effluent quality is being produced against minimum investment and operation and maintenance costs, without jeopardizing the reliability of the system (Figure 2.2).

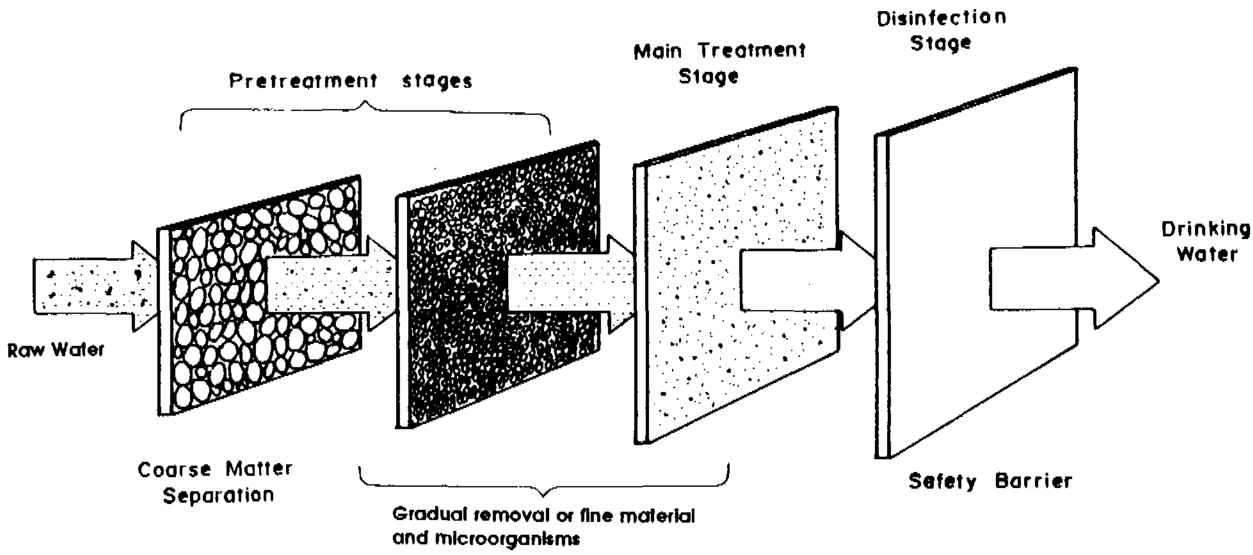


Figure 2.2 Treatment combination which permits the application of final disinfection as safety barrier.

Community acceptance and support

To ensure that the technical solution is viable the multi barrier concept and the concept of integrated treatment has to be adopted and the solution needs to be accepted and supported by the community or their legal representation. The community therefore, has to participate in the selection process and the solution has to be in harmony with their culture, their capacity and willingness to pay, and their capability to operate and maintain the system. Working with the community implies a change in the role for government institutions. The developments in the sector and the results of the Water Decade clearly show that water sector agencies have to modify their role from provider to facilitator, advisor and promotor.

3. *General Considerations about Water Treatment Projects*

3.1 Location

The project location has a very important bearing on the cost of the system. Projects close to the communities are most convenient and should be located, preferably in flat areas to avoid large and costly excavations. In selecting the site for the intake of the water system, often different possibilities exist. It may for example, be possible to construct the intake at a larger distance from the community but at a higher elevation. This would permit a smaller pipe diameter but would require a longer length of pipe. The alternative may be to locate the intake closer to the community but at a lower elevation, thus requiring a shorter distance of pipe but of a larger diameter to reduce the resistance of the pipeline. Another element which is important to take into account in selecting the site for the intake is the accessibility of operation and maintenance staff.

3.2 System capacity

The most important factor to be determined when designing a water supply system is the capacity of the system, that is the total quantity of water required per day. This daily water demand depends on the design period, the number of users and the quantity of water to be provided per person per day. The design period is the length of time in which the system is expected to provide a community with good quality water in sufficient quantities. This period should be neither to short, not less than 10 years, nor too long, because of economic reasons and the difficulty of predicting future water demand. A suitable design period for systems including pre-treatment technologies and slow sand filters is usually between 10 and 15 years.

The design population can be determined after the design period has been selected. The projected population growth during that time may be derived from available demographic data. Socio-economic factors should be taking into account when estimating the expected population growth. The water consumption per capita per day has a particular impact on the cost of the system and therefore needs to be assessed carefully and discussed in detail with the recipient community. This is particularly so when treatment is to be included in systems which already exist. Many systems operating by gravity supply and not including treatment are providing high quantities of water to the consumers. If the same quantities would be provided with treatment included, the cost of treatment would be excessive. A detailed discussion with the consumers will therefore be required to agree upon the service level which the new system will provide and the associated cost. It may be necessary to introduce the concept of efficient water use in the community and help them to change their water culture, in order to avoid water wastage to the maximum possible. Another important factor is the determination of the water consumption pattern over the day. The peak flows which the system needs to be able to cope with may be considerable. The peak factor is the ratio between the maximum hourly demand and the average hourly demand. A high peak factor implies the need for larger diameter pipelines to transport the water to the consumers. Other important points for attention in the design, include the flow velocity in the different parts of the system, the hydraulic gradients in the different pipelines and treatment units and the avoidance of possible cross connections.

In the interpretation of the design criteria, the designer has to visualize the role and tasks of the operator. It is essential that the design is made in such a way that the future work of the operator is facilitated to the maximum extent possible. This may imply for example, the provision of a larger number of smaller units to make cleaning easier and the provision of storage facilities close to the location where materials are required.

The water quality is the other dominating factor in the design of a treatment system. The best possible solution always needs to be identified, and in the preparatory stage different alternatives need to be analyzed including biological as well as chemical treatment, such as, direct filtration. The designer has the task to come up with the best possible alternative at lowest cost, which has the highest potential for sustainable performance. In the ultimate design also the esthetic features of the different structures should not be overlooked as these may have an important impact on the authorities and recipient communities.

3.3 Operation and maintenance

Many water supply systems are facing problems because operation and maintenance tasks are being neglected and this, in practice, jeopardizes the whole investment in these systems. It is essential to establish good maintenance programmes and accompany these programmes with adequate monitoring and backstopping. All daily, weekly, monthly and other operation and maintenance tasks need to be identified and specified and discussed with the operator as well as its supervisors. Monitoring programmes need to include key data such as, water quantity available in the water source, consumption pattern and water quality. Monitoring programmes should be developed in such a way that they provide a useful tool for the people who are collecting the data. Operators not only have to collect the data but also have to know what to do if the data are not in line with the expected values. Monitoring is an essential tool to really manage a water supply system.

3.4 Communication and administration

A good communication system between the different partners involved in the provision of good quality water is essential. This does not only include the relation between the operator, the water committee, the local authorities and the water company but also the relation with the consumers. A good relation between these partners is the key to obtaining and sustaining an efficient water supply system.

A good administration of the system is required and should include financial and managerial control. Full accountability of the administrative organization towards the community is needed to ensure that the users trust and are willing to support the organization. The administration may seek support from outside the community for specific tasks, such as, the specification of a new contract for construction which needs to be very clear and complete, to ensure the legal guarantees required.

3.5 Protection of the water catchment area and the water supply system

Protection of the water catchment area is one of the most important activities which need to be guaranteed to ensure sustainability of the system. Monitoring of the catchment area is therefore a key task for the operator. It is crucial that the community at large understands the relation between a well protected catchment area and a good functioning water supply. Only then will they be sufficiently motivated to actively support catchment area protection.

4. *Development of Slow Sand Filtration Technology*

During the 1970s and part of the 1980s important improvements have been made particularly in Latin America with rapid sand filtration of coagulated water. This helped to reduce the complexity of operation and maintenance of this technology and to lower the investment cost. However, the need to purchase, transport, store and dose chemicals has limited the effective utilization of this technology. Particularly in municipalities and rural centres, existing infrastructure and socio-economic conditions are not sufficient to support the application of this technology.

Slow sand filtration however, has been recognized as an excellent alternative for water treatment which often is economically competitive and has only limited requirements for operation and maintenance (Huisman, et al, 1974; WHO, 1987). This technology has already a long history in water quality improvement. The first indication of its use dates back to the previous century and it gradually became clear that filtration of water strongly contributed to the reduction in the transmission of cholera and tyfoid. At the end of the 19th century, it was therefore recommended to filtrate the water for human consumption.

According to the literature, the first application of slow sand filtration dates back to 1804 in Paisley, Scotland. In England it was applied as of 1829 and in Berlin as of 1856, followed by Altona in 1860 and Zurich in 1871. In 1892 an important event underscored the importance of slow sand filtration. In Hamburg, more than 7500 people died from an epidemic of cholera, whereas in the neighboring city of Altona only a few victims were counted. (Huisman, et al, 1974). Both cities used the Elba river as their water source but only in the case of Altona this was filtrated before distribution. Hamburg therefore immediately initiated the construction of a filtration plant, in 1893 when it began operation. (Hazen, 1913 sighted by Bellamy et al 1985). At the end of the 19th Century in Europe about 55 m^3 of water was filtered before putting it into distribution (Hazen 1913).

Slow sand filtration continues to be an important component of water treatment systems in Europe including those of large cities such as Amsterdam, London, and Zurich. In the United States the application of this technology has not been so wide spread. The short filter runs associated with high turbidity in the raw water sources, encountered in the eastern and central regions of this country, stimulated the interest in rapid filtration of chemically coagulated water as developed in the end of the last Century (Bellamy et al 1985). The few systems using slow sand filters were drawing water from sources of relative good quality and basically serving smaller communities. In total 71 slow sand filtration treatment plants had been identified in the U.S.A., in a search reported by Sims et al in 1991. Based on the information from Slezak, et al, 1984 and Sims et al, 1991, covering 47 plants, the following picture arises. The majority (76%) of the systems provided water to communities below 10,000 inhabitants, and 21% provided water to those between 10,000 and 100,000, and only 3% provided water to a population over 100,000. Over 54% used small rivers as the raw water source and 41% lakes, whereas only 5% used ground water. These water sources had an overall turbidity of 2 NTU with peak values of 15 NTU. Over 88% of the plants produce water below a turbidity level of 1 NTU when inflow turbidities were below 10 NTU. And in 95% of the cases a water turbidity below 1 NTU was continuously obtained as required by the standards in the U.S.A. About 80% of the water sources had coliform counts over 100 per 100 ml and over

70% of the plants produced water with less than 1 coliform per 100 ml.

In Latin America, slow sand filtration has had limited application and was introduced without adequate transfer of the technology and good adaptation to the conditions in the region. In countries such as Brazil (Hespanhol, 1969) and Peru (Canepa, 1982) where a larger number of plants have been constructed, most plants have not performed well.

Presently, a renewed interest in the application of slow sand filtration exists. Different factors can be identified to explain this including the activities realized in the context of the Water Decade promoted by the UN Organizations and the "Development and Demonstration Programme on Slow Sand Filtration" which was carried out with financial support from the Government of The Netherlands by organizations in different developing countries including Colombia, in collaboration with IRC, (Galvis et al 1989). These factors also include the recent recognition of the technology in the U.S.A. where the construction of over 100 filtration plants is now foreseen in the context of the health improvement programme (Longsdon et al, 1988). It is also supported by the decision made by different countries including Colombia to enhance the coverage with good water supply systems in areas where technologies are required with low technical, economical and organizational requirements to ensure their sustainability (DNP, 1991; ACODAL, 1992).

It is obvious that the technology has gained a large space in water treatment and its major users such as the Thames Water Authority in England indicate that its potential needs new developments in the next century (Rachwal et al, 1988). Countries in the Andean regions and in the tropics in general, do not sufficiently benefit from this potential because the raw water sources in these regions may transport large quantities of suspended solids and have high levels of bacteriological contamination. Therefore, it is very important to develop adequate pre-treatment alternatives which can be used to condition the water before it is being passed on to the slow sand filter units. These pre-treatment alternatives have to have similar levels of simplicity as the SSF technology and therefore should not be complex nor have high costs involved in their utilization and maintenance.

4.1 The slow sand filtration process

Basically, a slow sand filter consists of a tank constructed in reinforced concrete, ferrocement or stone/brick masonry, containing: (Figure 4.1)

- a supernatant layer of raw water;
- a bed of fine sand;
- a system of underdrains;
- an inlet and outlet structure;
- a set of filter regulation and control devices.

The supernatant water on top of the sand layer provides the hydraulic energy to transport the water through the sand. The improvement of the water quality in the filter is the result of a combination of physical, chemical and biological processes (Huisman et al, 1974), which still are not fully understood (Haarhoff et al, 1991). In a mature filter a thin layer forms on the surface of the bed. This filter skin (Schmutzdecke) consists of retained organic and inorganic material and a great variety of biologically active micro-organisms which break down organic matter. To keep these micro-organisms active and to prevent anaerobic conditions a continuous water flow through the filterskin, is needed. This will provide the oxygen and nutrients required to sustain the treatment process.

When after several weeks or months the filter skin gets clogged, the filtration capacity can be restored by cleaning the filter, i.e. by scraping off the top few centimeters of the filter bed including the filter skin. Periodic scraping can be continued until a minimum height of the sand bed of 50 cm has been reached. Thereafter the filter has to be resanded. This implies that new sand is placed underneath the remaining sand layer to avoid that material accumulates in the bottom layer of the filterbed. The length of filter run, being the period between two cleanings, has important cost implications. An analysis of 25 plants in the U.S.A. showed average filterruns of some 40 days in spring and 60 days in winter (Sims et al 1991). Another study on five plants treating lake water with turbidities levels of 1 to 3 NTU indicates filter runs of 3 to 6 months and a plant treating water with a turbidity of 9 NTU and peaks of 20 to 40 NTU had on average a filter run of 1 month and sometimes only of a few days (Letterman et al 1985).

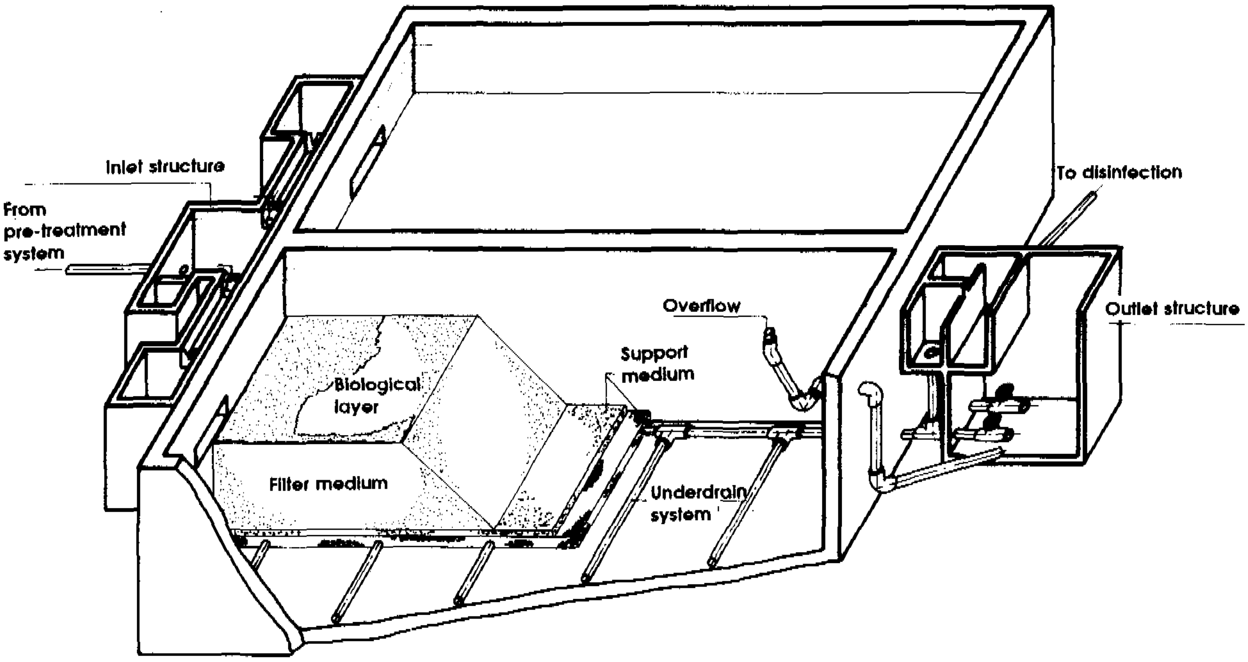


Figure 4.1 Basic Components of an inlet controlled slow sand filter

The water flow may be controlled at the outlet or at the inlet of the filter. The first alternative makes it necessary to open the outlet valve, daily or every two days a little further to compensate for the increase in resistance in the filter skin. This causes a slight variation in the rate of filtration and forces the operator to visit the plant at least every day, otherwise the output will fall.

In an <u>inlet-controlled filter</u>, the rate of filtration is set by the inlet valve. Once the desired rate is set, no further manipulation of the valve is required. At first the water level over the filter will be low but gradually it will rise to compensate for the increasing resistance of the filter skin. Once the level has reached the scum outlet, the filter has to be taken out for cleaning.

Inlet control reduces the amount of work which has to be done in: removing algae and floating material and cleaning the filter. The rate of filtration will always be the same as the rate of delivery of water into the filter, and the build-up of resistance is directly visible.

Design criteria as presented by different authors based on different experiences and circumstances are indicated in table 4.1. Those recommended by Visscher et al 1987, are being considered particularly appropriate for small systems in developing countries as well as in countries such as the U.S.A. (Pyper et al, 1991).

Table 4.1 General design guidelines for slow sand filters in rural water supply.

DESIGN CRITERIA	RECOMMENDED LEVEL		
	Ten States Standards USA (1987)	Huisman and Wood (1974)	Visscher et al (1987)
Design period	no data	no data	10-15 years
Period of Operation	no data	24 h/d	24 h/d
Filtration rate (m^3/h)	0.08 - 0.24	0.1 - 0.4	0.1 - 0.2
Height of filter bed (m)			
-initial	0.8	1.2	0.9
-minimum		0.7	0.5
Specification of sand			
-effective size	0.30 - 0.45	0.15 - 0.35	0.15 - 0.30
-uniformity coefficient			
-acceptable	≤ 2.5	< 3	< 5
-preferred		< 2	< 3
Height of underdrain system -including gravel layer (m)	0.4 - 0.6	no data	0.3 - 0.5
Height of supernatant water level (m)	≥ 0.9	1 - 1.5	1
Free board (m)	no data	0.2 - 0.3	0.1

Application of adequate pre-treatment may have an important influence on the design criteria. If water of relative good quality is being pre-treated it may well be possible to further relax the design criteria and apply for example a higher rate of filtration or a lower filter box. However, if insufficient experience is available with the technology for treating a specific water source it is better to go for a more conservative design to avoid problems at a later stage.

4.2 Water quality limitations for slow sand filtration

The application of slow sand filtration should be carefully evaluated when designing a water supply scheme. When surface water is more readily available than groundwater, slow sand filtration will frequently prove to be the simplest, most economical and reliable method of preparing safe drinking water. The slow sand filtration process has the enormous advantage that it can produce an effluent low turbidity, free from offensive dissolved impurities and more important, virtually free from harmful entero-bacteria, entero-viruses and protozoan cysts. However, slow sand filtration is not a panacea to solve all water quality problems. This process when applied as single treatment will meet with problems when:

- the treatment efficiency is not sufficient to reduce the level of harmful substances in the water to produce a filtrate which meets current drinking water quality standards;
- the raw water contains substances which reduce or inhibit the purification processes in the slow sand filters;
- filterruns are becoming very short;

Pollution levels exceeding the treatment efficiency

Slow sand filters have a high efficiency for removing a wide variety of substances, but do not necessarily remove all harmful substances to the extent required. Table 4.2 presents treatment efficiencies reported in literature for filter units operated at filtration rates ranging from 0.04 to 0.2 m/h at temperatures above 5°C, with a sandbed depth greater than 0.50 m and filled with sand with an effective size between 0.15 and 0.30 mm.

The efficiencies indicated in Table 4.2, however cannot always be fully realized and much will depend on the level of contamination and the characteristics of the substances in the water.

- *High level of micro-biological contamination*

In some communities, the only sources available for water supply may be so heavily contaminated with harmful micro-organisms that slow sand filters alone will not be able to produce a good quality effluent. In such case other treatment barriers will have to be added to ensure that a water low in hygienic risk can be produced.

- *Excessive load in organic material*

The literature indicates several cases where SSF treatment has not been sufficient to remove organic material and humic acids. Removal efficiency for true color resulting from humic acids for example is reported to be approximately 25 to 30% (Cleasby et al, 1984; Ellis, 1985; Collins et al, 1991). Earlier, this was not considered a major problem as color at best was considered an aesthetic problem. At present, this is quite different with the discovery that chlorine components react with organic material forming sub-products which may be harmful (Rook, 1974) and the recognition that color is an indicator of the presence of humic acids, and thus of organic material in surface water sources.

Table 4.2 Typical treatment efficiencies of slow sand filters (Ref: Bellamy et al., 1985, Ellis, 1985; Huck, 1987; Hrubec et al., 1991; Haarhoff, 1991).

PARAMETER	TYPICAL REDUCTION (%)
Entero-bacteria	90 - 99.9%, however coliform removal efficiency is reduced under low temperature conditions, increased hydraulic rate, use of coarse filter sand, shallow depth of sandbed, decreased contaminant concentration and just after removal of the biological filter skin;
Protozoan cysts	99 - 99.99% removal even after filter scraping
Cercariae of schistosomiasis	Virtually complete removal
Turbidity	Generally reduced to less than 1 NTU if the influent is below 10 NTU. The removal efficiency can be affected by particle size and distribution
Color	30 - 90% with 30% being mentioned as the most usual efficiency
Organic matter	COD 30 - 70%; TOC 15-30%. Organic matter such as humic acids, detergents, phenols, and some pesticides and herbicides are being removed from 5 to over 90%
Iron, manganese	Considerable removal
Heavy metals	30 - 90% or even higher

This has motivated recommendations to reduce the level of true color in the affluent of slow sand filters to some 15 to 25 PCU (Collins et al 1989).

The figures on removal of true color are consistent with the reductions which are generally reported for total organic carbon (TOC), chemical oxygen demand (COD) or dissolved organic carbon (DOC) with values ranging between 10 and 25%. (Fox et al 1984, Williams 1985). Other studies report values which are significantly higher and range between 50 and 68% (Joshi et al., 1982). These values were not affected by changes in the filtration rate which was ranging between 0.1 and 0.3 meters per hour. The discrepancy in the removal efficiencies reported in literature may result from the nature of the different organic components and the indirect measurement by using parameters such as TOC and COD. This is confirmed by a study in Germany (Haberer et al, 1984) which shows a wide difference in the removal of 6 different types of organic components. The lowest efficiencies he measured were only in the order of 4% for volatile hydrocarbon halogenes, and the highest efficiencies he found in the order of 86% for aromatic policyclics.

- *High quantity of colloidal material*

Turbidity is usually largely removed even if caused by colloidal material, particularly if intensive biological activity takes place in the filterbed. Nevertheless, low removal efficiencies may present itself if water is treated flowing from clay-bearing catchment areas having a turbidity made up of colloidal material and very small particles below 0.5 μm.

Bellamy reports for river water of this nature and an average turbidity below 10 NTU a removal efficiency ranging from only 27 to 39%. The same study showed that these removal efficiencies could be increased if nutrients were added to the raw water.

Also other contaminants such as heavy metals may not be sufficiently removed although high removal efficiencies are being obtained (Cleasby, 1991). Hence this may also be a reason to introduce pre-treatment processes.

Conditions reducing process efficiency

Several circumstances may interfere with the purification process in the slow sand filter and may result in normal efficiencies not being reached. Three main conditions can be mentioned: low temperatures, low nutrient level and low level of dissolved oxygen.

- *Low temperature*

Low temperature increases the viscosity of the water and reduces the biological activity in the filter. Huisman reports that E coli removal will be reduced from a normally achieved 99% at 20°C to 50% at 2°C. This conclusion is also supported by the findings of Toms that filters run at 0.3 m/h at temperatures below 4°C, produced a filtrate with faecal coliform counts above 50 CFU/100 ml. The strategy followed in the Netherlands and Switzerland to cope with low temperatures has been to cover the filters or construct them in the ground. Applying a lower filtration rate may also be a way to cope with low temperatures as can be derived from the information presented by Toms which indicates that older works in London operated at a filtration rate of less than 0.20 m/h usually produced a filtrate with a faecal coliform count of less than 10 CFU/100 ml, even at temperatures below 4°C.

- *Nutrient deficiency*

The micro organisms in the filter require carbon, nitrogen, and sulfates for their metabolism and growth. The humic acids are rich in carbon but only comprise traces of nitrogen, phosphate and sulfate (Spencer et al 1991). This would explain the limitations of SSF units to remove color from water coming from well protected water sources with low nutrient levels. This explanation is supported by the findings of Bellamy et al, 1985, which indicate that the addition of nutrients permits the increase in biological activities in the filter and increases the removal efficiency for colloidal turbidity and for indicators for the microbiological quality.

- *Low level of dissolved oxygen*

When the filtration rate is low, the level of dissolved oxygen in the water can be strongly reduced because of the long contact time with the micro organisms which are consuming the oxygen (Joshi et al 1982). Anaerobic conditions in the filter need to be avoided as they may result in important water quality problems such as taste and smell problems and the resuspension of heavy metals and increase the chlorine demand of the filtered water (Ellis, 1985).

Reduction of filter runs

Taking into account criteria such as maintenance cost and acceptance of the technology by the operators who are having limited mechanical equipment to maintain the filters, it has been suggested that the minimum filter run has to be in the order of 30 days (Hendricks et al, 1991; Collins et al 1991). The filter runs may be strongly affected by high turbidity levels and the presence of algae as will be discussed in the next section.

- *Turbidity level*

The inability of slow sand filters to cope with high turbidity levels is well documented. It is indicated as the major reason for the limited application of slow sand filtration and for the development of rapid filtration techniques in the U.S.A.

To avoid the risk of producing an affluent with high turbidity values or to avoid the premature obstraction of the filter Huisman et al, 1974, has recommended that the water to be treated needs to have an average turbidity level below 10 mg/l S_iO_2 yet permitting occasional turbidity level ranging from 50 to 200 mg/l S_iO_2, with durations of a few days. In a wide revision of the literature by Ellis 1985, the majority of the references indicate an upper limit for turbidity in the range of 10 to 50 NTU and accept values ranging from 50 to 120 NTU if of a duration of less than two days. However, with water quality norms for human consumption becoming more demanding in recent times, authors such as Cleasby et al (1984) recommend that the turbidity of the water to be treated by slow sand filters should be below 5 NTU.

Although turbidity is the parameter most frequently used to indicate in an indirect way the amount of particles in the water, it is being recognized that this has a serious limitation to reflect the load of suspended solids which are coming to the filters, particularly when it concerns clay or silica particles. Also the presence of algae or iron flocks can influence and complicate the relation between turbidity (NTU) and suspended solids (mg/l). Particles size distribution therefore requires further attention in research about the filtration processes in slow sand filters.

- *Presence of algae*

Algae may be present in the raw water sources or grow in the supernatant water layer above the filter, if nutrient conditions are favorable and solar radiation is available. Growth of algae in the supernatant water can be prevented by covering the filters to avoid solar radiation to enter into the water. Covering in such situation may be beneficial for the length of the filter run. If algae are present in moderate quantities then they are usually considered beneficial for the functioning of the filter. Algae are largely retained by slow sand filters although occasional breakthrough of unicellular green algae have been reported by Toms et al 1988. Furthermore, algae blooms may result in filter runs of the normal filter run (Ives, 1957). These blooms may create severe problems such as premature blocking of the filters, production of taste and odor in the water, increase in the concentration of soluble and biodegradable organics in the water and increased difficulties associated with precipitation of calcium carbonate in the filter.

As a result of the photosynthetic activity algae may reduce the natural buffering capacity of the water and as a result the pH may rise considerably, even above 10 or 11. As a result magnesium hydroxide and calcium hydroxide will be precipitated on to the sand grains (Ives, 1957). This contributes to the blocking of the filter and increases the effective diameter of the sand which may have repercussions for the treatment efficiency. On the basis of their experiments Cleasby et al (1984) consider that the availability of algae can be a good indicator for the length of filter run and they recommend an upper limit of 5 mg/m3 of chlorophyll-a as an indirect parameter to control the level of algae in the supernatant water.

- *Iron and manganese*

In the filter bed, bacteria may be present which can assist in the oxidation of iron and manganese present in the water. Small deposits of iron improve the removal capacity for organic components (Collins et al, 1985). However, concentration of iron above 1 mg/l may contribute significantly to the blocking of the filter media (Spencer et al 1991). This is approximately the equivalent of a dose of 4.8 mg/l of ferro chloride (Cleasby 1990). Table 4.3 presents a summary of criteria which according to the literature apply for a prefered water source for direct treatment for slow sand filtration.

Table 4.3 Some water quality limitations for direct treatment by slow sand filtration

Criteria	Spencer et al, (1991)	Cleasby, 1991	Di Bernardo (1991)
Turbidity	5 - 10 NTU (1)	5 NTU	10 NTU
Alga (Units/l)	200000 (2)	5 mg/m^3 (3)	250000
True Color	15 - 25 UPC		5 UPC
Absorbance UV, 254 nm	0.08 UA		
Dissolved Oxygen	> 6 mg/l		
Phosphorus	30 μ g/l		
Ammonia	3 mg/l		
Total Iron	1 mg/l	0.3 mg/l	2.0 mg/l
Manganese		0.05 mg/l	0.2 mg/l
Faecal Coliforms (MPN)			200 org/100 ml

(1) If possible, the nature of the turbidity and particle size distribution needs to be taken into account.
(2) This not only depends on the number but also on the type of algae present in the water. The recommendation applies for covered filters.
(3) This limit corresponds to chlorophyll-a, as an indirect measurement of the algae level.

5. Pre-treatment Systems

Pre-treatment systems which need to be applied together with slow sand filtration have to have similar characteristics. In this section a number of pre-treatment alternatives are being explained which are relatively easy to operate and maintain and do not require chemical dosing (Smet et al 1989).

5.1 Infiltration wells

One of the oldest techniques of pre-treatment consist of infiltration wells along the side of rivers. Depending on the water quality in the river and the soil conditions, water drawn from such wells can be used, put into supply after disinfection or brought to a slow sand filter plant. Problems with the resuspension of iron and manganese oxides have been reported by Engels et al 1989 when levels of oxygen in the ground and the 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 that cannot be remedied by maintenance activities.

5.2 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. In figure 5.1 two possibilities to install the filter material are being indicated. 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 5.1 (b) have been reported as 98% for turbidity removal from rivers with turbidity levels ranging from 48 to 200 NTU (Salazar 1980). An evaluation made in Colombia has indicated that the real efficiency of the systems are rather low and may reach only some 20% (CINARA, IRCWD, 1988).

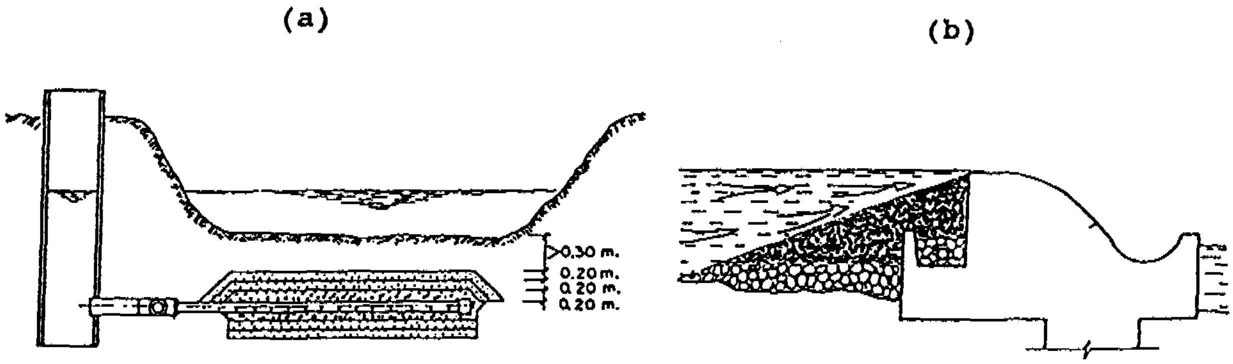


Figure 5.1 Infiltration Gallery. (a) Without interfering in the flow gradient (Smet et al 1989) and (b) an abstraction system with a weir (Salazar 1980)

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 the water. The recognition of this type of limitations has motivated CINARA to develop a modified system is now being known as dynamic roughing filters, as is further explained in section 5.4.

5.3 Plain sedimentation

Plain sedimentation can very much contribute to reducing the level of suspended solids in the water source. It will depend however on the nature and the level of contamination if a sufficiently good effluent can be obtained for slow sand filter treatment.

Some rivers in the south east of the U.S.A. are transporting up to 10,000 mg/l of suspended solids. Plain sedimentation with very short retention periods of 3 to 8 hours can reduce this level of suspended solids significantly to a value below 1,000 mg/l (Steel 1953; cited by Cleasby 1991). This is a very good reduction but still the effluent is not suitable for slow sand filter treatment.

Plain sedimentation has a limited impact for water sources with a turbidity of colloidal nature. 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 (Turneaure et al 1940; cited by Cleasby 1991).

Two different applications can be identified: a system using a short retention time, less than one day, and a system with a very long retention time in the order of several days or weeks. The level of suspended solids in the water source can be reduced considerably by plain sedimentation but still the resulting quality may not be sufficiently good for treating by slow sand filtration.

For systems with a short retention time tests with sedimentation columns are recommended to explore the expected quality improvement. 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 (Cleasby 1991).

Long term storage is very common in England. In London turbidity reduction in large storage basins have been reported to reduce turbidity levels from 30 NTU to values below 4 NTU (WRC 1977). Long term storage may also have a significant effect on the bacteriological quality. 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 (Windle Taylor 1974). However, the periodic blooms of algae made it necessary to introduce micro-screeners or rapid filters before the slow sand filters, which are treating the stored water (Ridley 1967). To control the algae growth in the storage tanks, methods have been developed in England to avoid stratification primarily by using pumps which ensure a continuous water flow in the basins. The potential of long term storage for tropical countries has to be evaluated carefully before large scale application can be promoted.

Tilted plate settlers or tube settlers can be applied as sedimentation units with short retention times and may reduce the required surface area with 60 to 70%. 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. In principle, the same possibilities and limitations apply for this technology as for plain sedimentation with a short retention time. Yet more frequent cleaning will be required and needs special attention (Castilla et al 1989).

5.4 Roughing filters

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 sections different types of roughing filters are being discussed and are classified according to their characteristics and direction of flow.

Dynamic Roughing Filters (DyRF)

The DyRF consist of a layer of fine gravel (3 to 6 mm) of some 0.2 to 0.3 m height 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 being placed as drainage system (Figure 5.2).

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 clogging may go much quicker and depending on the 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. 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 (Galvis et al 1992).

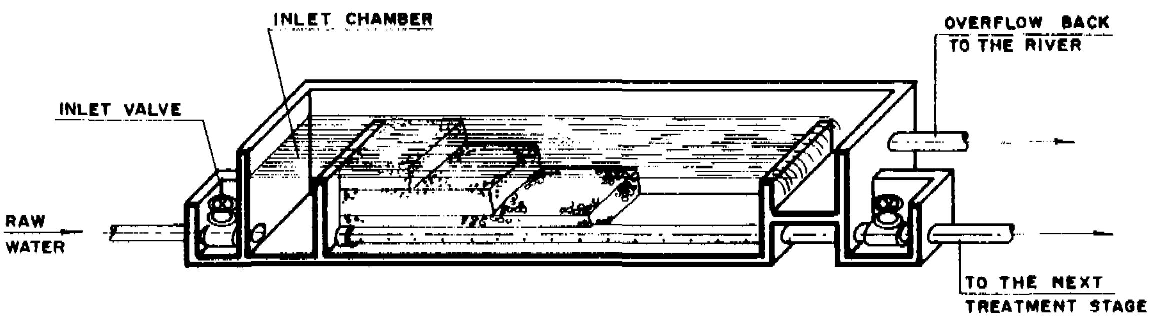


Figure 5.2 Dynamic Roughing Filter

Downflow coarse sand filters

As of the beginning of this century downflow rapid sand filters have been applied prior to slow sand filters in Europe and the U.S.A. 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 (Cleasby 1991). Another system comprising pebbles and sand called pebbled matrix filtration has been developed in Rusia and is currently being investigated as pre-treatment to be combined with slow sand filtration (Rajapakse et al 1989). 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 only a pre-treatment method for surface water sources with a suspended solid content below 2000 mg/l, whilst applying a filtration rate of 0.7 m/h. These alternatives seems to have good potential for pre-treatment of surface water with high suspended solid content. However, its operation and maintenance requirements which include backwashing under considerable pressure may restrict its application. Operating at a flow velocity of 0.72 m/h and treating water with 500 mg/l of suspended solids the effluent concentration in suspended solids increases after a filter run of 16 hours to values above 25 mg/l. Although some 70% of the retained suspended solids can be removed by just draining the unit, it is still necessary to backwash hydraulically the filter ensuring that the filter bed is fluidiced to fully clean the sand layer. The true potential of this technology cannot be assessed at present as no full scale experience exists.

Downflow roughing filters in series (DRFS)

The DRFS system is based on the system used by Pueb-Chabal in Paris and other european cities in the beginning of this century (Ellms 1919; cited by Cleasby 1991). In this system water passed through three or more filter units. The first unit comprised gravel of 25 mm diameter and the following units comprise smaller gravel. Subsequently, the water was treated by a slow sand filter.

Studies realized in Peru (Perez et al 1985; Pardon 1989) and in Colombia (Galvis et al 1987; Quiroga 1988) have stimulated new interest in this pre-treatment option. Figure 5.3 indicates a schematic design of a DRFS 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 help of a fast drainage valve connected to the drainage system.

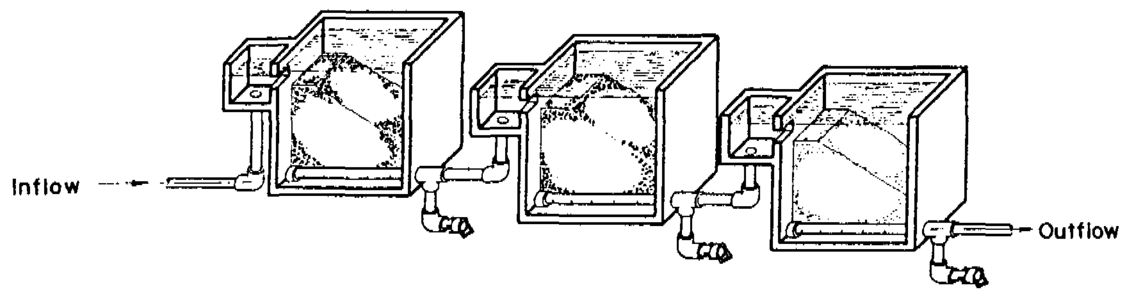


Figure 5.3 Schematic diagram of a downflow roughing filter in series

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 increased 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 effluent 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.

The results of this study were used to build a full scale plant with a capacity of 0.4 l/second operating at 0.3 m/h. For raw water turbidities ranging from 20 to 100 NTU an average removal of 63% was identified. For the range between 100 and 300 NTU the efficiency reached 79%. The average removal efficiency of faecal coliforms was in the order of 66% (Pardon 1987). Filter runs in the range of 3 to 7 weeks were obtained but the study does not report the increase in headloss which was being caused. The filter system was drained through the application of hydraulic shocks when the efficiency in removal dropped below 60% for turbidity and also when the effluent turbidity was above 20 NTU (Lloyd et al 1986).

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. Filtration rates of 0.7 m/h were used. 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, Galvis et al 1989). The first results of the studies with the 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% (Quiroga et al 1988; Galvis et al

1989). In subsequent investigations with technical scale units of 2 m diameter the potential of this pre-treatment alternative was confirmed. However, very little experience on full scale plants is available and therefore only very preliminary design guidelines can be established (See chapter 10). Further studies are needed to establish the potential and limitations and develop more concrete guidelines. In Asia these systems were also used but instead of applying gravel other filter material was used such as coconut fiber (Frankel 1974; Wolters et al 1989). 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.

Upflow roughing filters

In this alternative the water flows upwards through a series of different gravel layers which are decreasing in size. Two alternatives can be distinguished upflow gravel filtration in layers (URFL) when the gravel layers are installed in the same unit (Figure 5.4) and upflow roughing filtration in series (URFS) when the gravel layers are installed in two or three different units (Figure 5.5). 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, 1987).

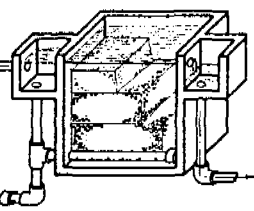


Figure 5.4 Overview of an Upflow Roughing Filter in Layers

Both alternatives of upflow roughing filtration have been evaluated in Cali, Colombia, using water from the River Cauca. For the first trials with 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. 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% (Galvis et al, 1989; Wolters et al 1989). The alternative of URFL evaluated for the same water quality showed lower removal efficiencies. The URFL system was also evaluated for another surface water source of better quality with an average turbidity level of approximately 12 NTU and higher values over 100 NTU for very short periods (in the order of several hours). Average values for true color were 34 TCU and for faecal coliforms 1680 MPN per 100 ml. Average removal efficiencies identified were 69% for turbidity, 45% for true color and 89% for faecal coliforms. After some two weeks of operation the hydraulic resistance of the system was in the order of 25 cm and after hydraulic cleaning the initial headloss over the unit was restored, which was in the order of 7 cm. The hydraulic cleaning was completed with a cleaning of the surface layer of the gravel to control the growth of algae (Galvis et al 1989).

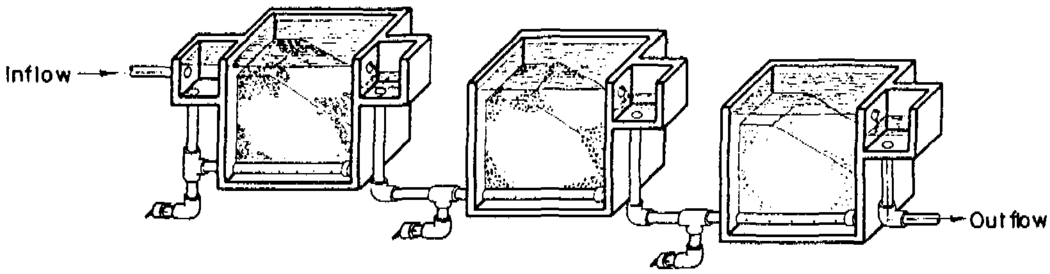


Figure 5.5 Overview of an Upflow Roughing Filter in series

Between 1990 and 1993 CINARA has evaluated both systems again using water from Cauca river but now 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 mm/l and faecal coliforms counts between 30,185 and 148,575 MPN per 100 ml. Removal efficiencies for the URFL were 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 (CINARA-IRC 1993). Additional information can be found in the final report on the Pre-treatment Project.

Horizontal roughing filters

Horizontal roughing filters (HRF) (Figure 5.5) are normally divided in three compartments of reducing length each filled with gravel with the first compartment comprising the coarsest gravel and the last compartment the finest. 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 (Wegelin 1989). Studies about this method have been established in Thailand (Than et al 1977) and Tanzania (Wegelin et al 1981; Mbwette, 1983). More recently the International Reference Center for Waste Disposal (IRCWD), based in Switzerland, has realized laboratory studies with experimental pilot units using different kaoline suspensions (Wegelin, 1986). This study continued with a monitoring of full scale units constructed in different countries including Tanzania, Peru, Sudan and Colombia. In Colombia one experimental unit and three full scale plants have been constructed and monitored in the context of the first phase of the pre-treatment project. 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 2 mm and the filtration rate ranged from 0.3 to 0.6 m/h. Studies were continued by comparing two units, one with a length of 7.14 m and without drainage system and another of 4.35 m length but with a drainage system. Those units were filled with gravel between 25.4 and 1.6 mm operating at flow velocities between 0.3 and 0.75 m/h. Additional information on the findings is being presented in chapter 11 and in the final report of the pre-treatment project.

An important characteristic of this system is its large sludge storage capacity. In the first experiments the units were used until they were fully obstructed by accumulated solids in the filter bed. Thereafter the gravel was taken out of the box, washed and re-installed. More recently the option of hydraulic cleaning was introduced and regular cleanings were made.

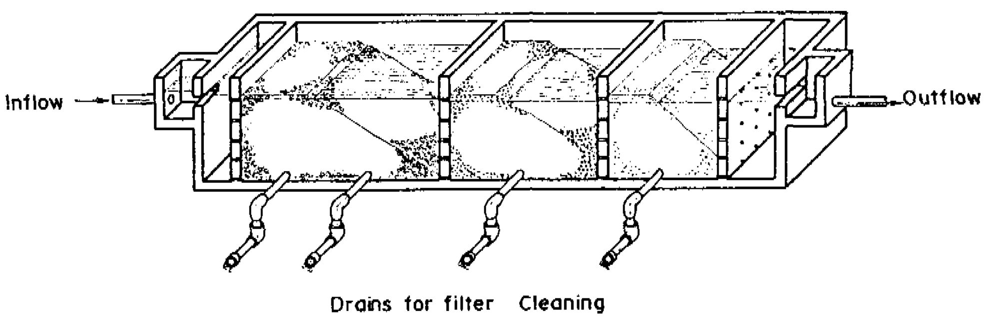


Figure 5.6 Schematic overview of an horizontal roughing filter

6. *Roughing Filtration*

6.1 Basic concepts

The application of slow sand filtration requires water low in turbidity often making pre-treatment necessary. Roughing filtration using coarse and fine gravel is a simple and efficient method for the removal of suspended solids due the large surface area available for sedimentation, absorbtion, and biological and bio-chemical activities in the filters (Wegelin 1991).

Roughing filters usually comprise filter material which gradually decreases in size from coarse in the first part of the system to relatively fine in the 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 indicated in the following comparison (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. The applied rate will depend very much on the type of filter, the sanitary risk of the water source and the required treatment efficiency (Shulz and Okun 1984, Galvis 1992). The treatment mechanisms which are taking place in roughing filters can be classified as mechanisms of transport, attraction and purification.

6.2 Transport mechanisms

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

- *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 also be retained. This mechanism however, has only limited importance in roughing filtration due to the large diameter of the gravel (approximately 2 mm), which according to Huisman 1984, corresponds with a pore size of 500 µ. 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 (Wolters 1988).

- *Sedimentation*

This process removes suspended material in a similar way as in a sedimentation tank. The difference is that in the 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 flocks which then also can precipitate on the gravel surface. Colloidal material however, is not being removed by sedimentation. Wegelin (1986) and Siripatrachai (1987) are reporting that sedimentation is the principal mechanism for particle removal in horizontal roughing filters. Because of the similarities it may be assumed that also in upflow and downflow roughing filtration sedimentation plays an important role.

- *Diffusion*

The brown movement or molecular diffusion is caused by the collision of particles with water molecules. This movement is important for the removal of colloidal material and does not affect particles above 2 μ (Huisman 1982). 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.

- *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. Particle removal through this mechanism is limited in roughing filtration because low filtration velocities are being applied and the gravel grains are relatively large.

- *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.

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

6.3 Attachment mechanisms

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

- *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 µ. 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 electrical 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.

- *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 origin deposited on the surface of the gravel will quickly become the breading ground of bacteria and other micro-organisms. This will produce a sticky 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.

6.4 Purification mechanisms

The purification processes whereby the trapped impurities on the filter grains are broken down are interdependent 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.

- *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.

- *Bacteriological activity*

Increasing evidence is available of the importance of biological processes in roughing filters. (Trueb, 1982; Pardon, 1988, Wolters, 1988, Smet et al. 1989, Wegelin, 1989). 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 the 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. This explains the considerable reduction of total and faecal coliforms (indicator mechanism for faecal contamination) in roughing filters.

7. *Selection of Roughing Filtration Alternatives in Combination with Slow Sand Filtration*

On the basis of the selection criteria indicated in chapter 2 and studies carried out by CINARA and IRC (1993) a preliminary guideline has been established for the selection of pre-treatment alternatives which are operating in combination with slow sand filters.

7.1 Selection on the basis of hygiene risk and removal efficiency

The study by CINARA-IRC (1993) indicated that the alternatives of URFS, DRFS, and HRF preceded by a DyRF show similar removal efficiencies for turbidity and faecal coliforms, while URFL presents similar efficiencies only for less polluted water. With these results preliminary guidelines can be established to ensure sufficiently good water is flowing to the slow sand filters. On the basis of the results of the pre-treatment project and ample revision of the literature, the following maximum acceptable levels for different parameters have been identified for water to be treated by slow sand filtration (Table 7.1). The limits have been established for slow sand filters which are being operated at 0.15 m/h. They are based on project findings with statistical reliability levels over 90%, except for faecal coliforms which have a reliability of 72%.

Table 7.1. Water quality limitations for slow sand filtration

PARAMETER	AFFLUENT		EFFLUENT	
	Max.	Average	Max.	Average
Turbidity (NTU)	25	10	5	2
Real Color (UPC)	40	20	15	7
Faecal Coliforms (UFC/100ml)	1900	550	10	2
Total Iron (mg/l)	1.0	0.5	0.3	0.2
Manganese (mg/l)	0.2	0.1	0.1	0.05

On the basis of the limits presented in table 7.1 and the results of the project, indicative water quality limits have been identified for treatment systems including slow sand filters and two pre-treatment stages (DyRF and Roughing Filtration). The DyRF systems were used with a filtration velocity between 1 and 3 m/h and filter medium as indicated in table 7.2. The flow velocity in the roughing filters as indicated in table 7.3 ranges between 0.2 m and 0.6 m/h. Because of the similarity between results, limits indicated for URFS in table 7.3 also apply for DRFS and HRF.

Table 7.2 Gravel media in the pre-treatment systems (CINARA-IRC 1993)

PRE-TREATMENT UNIT	LENGTH (m)	DIAMETER (mm)
DyRF	0.60	3 - 19
URFS	4.65	1.6 - 25
DRFS	4.65	1.6 - 25
HRF	7.15	1.6 - 25
URFL	1.55	1.6 - 25

Table 7.3 Indicative raw water quality limits for water treatment systems including two stages of pre-treatment and slow sand filtration (*) (**)

Filtration velocity (m/h)	Pre-treatment	Turbidity (NTU)		True Color		Faecal Coliforms (1000 /100ml)		Total Iron (mg/l)		Manganese (mg/l)	
		Max	Average	Max	Average	Max	Average	Max	Average	Max	Average
0.30	URFS	650	84	230	60	300	89	5.5	4.5	1.3	0.9
	URFL	500	70	100	48	200	84	5.5	4.5	1.3	0.9
0.45	URFS	440	53	115	48	300	89	5.5	4.5	1.3	0.9
	URFL	240	44	61	38	200	84	5.5	4.5	1.3	0.9
0.60	URFS	330	44	72	35	300	89	5.5	4.5	1.3	0.9
	URFL	150	39	48	32	200	84	5.5	4.5	1.3	0.9

* The limitations for URFS also apply for DRFS and HRF

** The maximum values indicated correspond with changes in raw water quality of a duration of less than three hours

As indicated in Table 7.3 and according to the results of the project the removal efficiencies for faecal coliforms, total iron and manganese are not very much affected by the flow velocity which has been studied ranging from 0.3 to 0.6 m/h. Higher velocities are now being studied in the next project phase.

7.2 Preliminary criteria on basis of cost estimates

The comparison of systems, operated under the same velocity, clearly shows the advantage of URFL as it presents only one third of the construction volume of URFS and DRFS and only 1/5 of the volume of HRF. This suggests that wherever possible the URFL should be selected when raw water quality permits. However, in practice the situation is more complicated. In terms of treatment efficiency, an URFL operated at 0.3 meters per hour and an URFS operated at 0.6 meters per hour show similar efficiencies (Table 7.3), so these systems can be used for the same raw water quality and therefore a comparison between the two is of interest. In Table 7.4 the construction volume of these two systems operated at 0.3 and 0.6 m/h respectively, is being compared. It shows that the actual construction volume of the URFL although being operated at a lower flow velocity is still smaller and therefore this option will be more economic.

Table 7.4 Comparison of construction volumes of URFS and URFL operated at 0.3 and 0.6 m/h

PARAMETER	PRE-TREATMENT SYSTEM	
	URFS	URFL
Filtration velocity (m/h)	0.3	0.6
Number of phases	1	3
Total filtration area m^2(*)	3.4	5.0
Perimeter m (*)	7.4	18.4
Volume of gravel m^3 (*)	5.0	7.5
Draining System	1	3

* Calculated for a flow of 1 m^3/h

Clearly an URFL is a very interesting alternative when compared to the other systems URFS, DRFS and HRF both for economic reasons and for treatment efficiency. Based on these considerations and the experiences of the project, a chart for selecting water treatment systems including slow sand filtration and pre-treatment systems is being presented in Figure 7.1. It should be noted however, that it only concerns a preliminary guide because the investigations so far have not been evaluating higher filtration velocities and therefore future changes may still be considered. Furthermore, it is necessary to review the possibility using finer gravel material as planned in the new project phase. Finally, also longer term behaviour of the units under continuous field operation needs to be explored better, as this may also provide additional information to refine the selection chart.

The chart presented in figure 7.1 includes the utilization of a DyRF as first pre-treatment step. If however raw-water pumping is required utilization of a DyRF may be not convenient and it may be necessary to divert from the table and construct an URFL or an URFS instead of the DyRF.

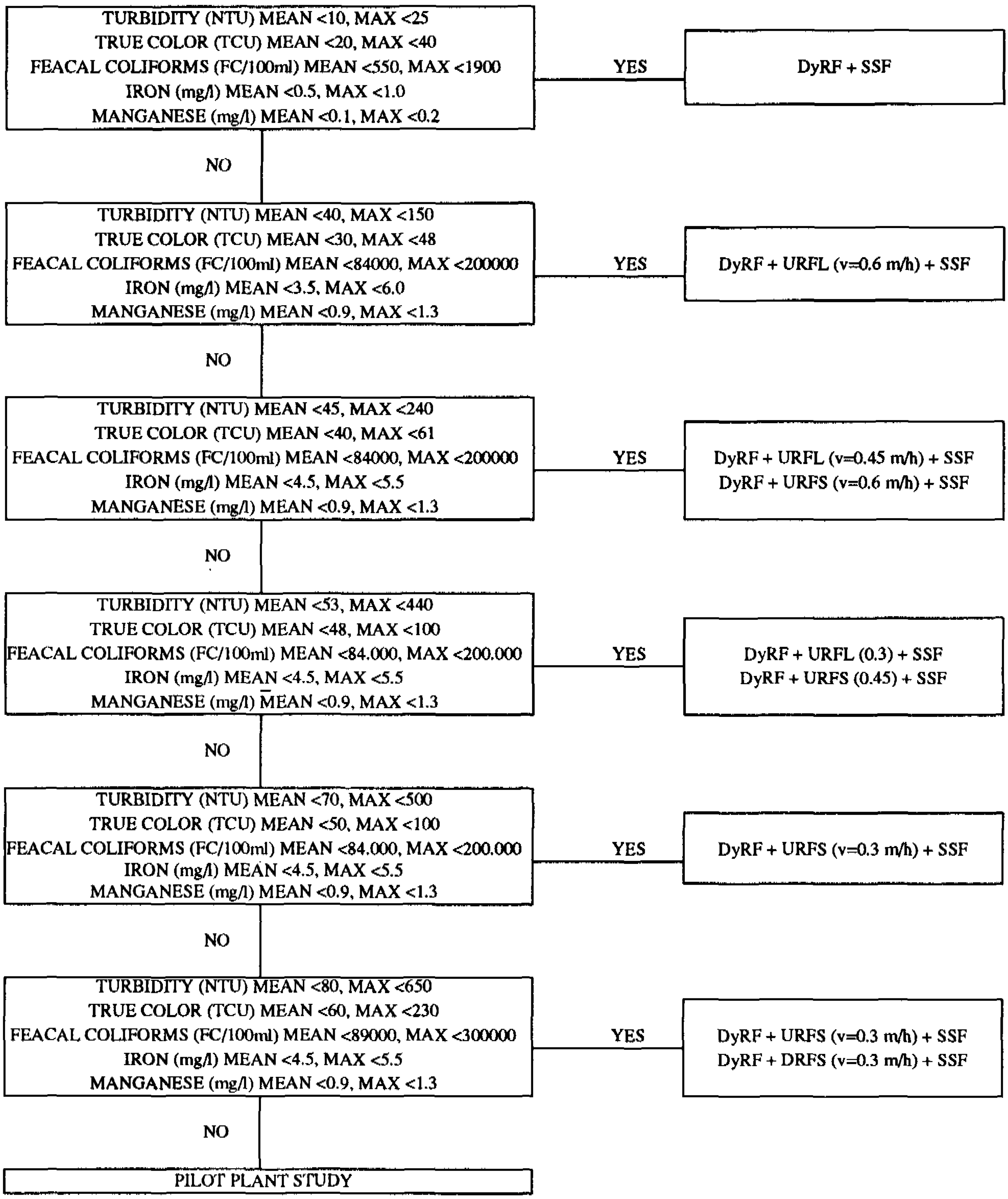


Figure 7.1 Preliminar selection chart for pre-treatment and slow sand filter systems, based on cost and removal efficiency considerations

8. *Dynamic Roughing Filters*

8.1 General description

In Figure 8.1 a general overview of a dynamic roughing filter (DyRF) is presented. The water flow Q_c entering the filter is distributed into two flows. The first flow Q_c passes through the filter medium and on to the subsequent treatment units; the other flow Q_e passes over the gravel bed, removing material which is being deposited. The latter flow 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 Q_f 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. The potential to react to increased loads in suspended solids is the reason for the name dynamic roughing filtration.

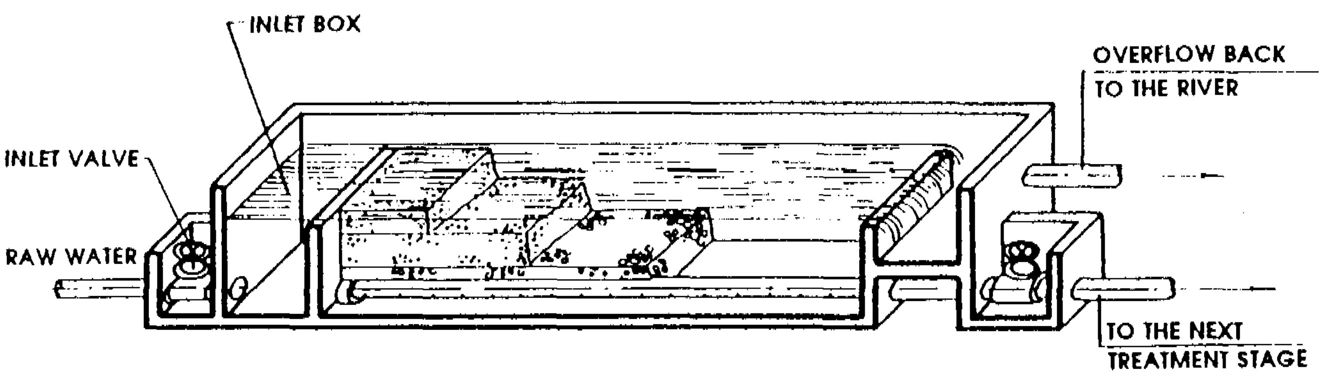


Figure 8.1 General overview of a dynamic roughing filter

8.2 Design criteria

Two types of dynamic roughing filters 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. Important reductions in suspended solids can be obtained (in the order of 70%) and also other parameters can be reduced such as turbidity and faecal coliforms in the order of 40 to 60%. This system is a clear first sanitary barrier to improve the water quality.

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 considered 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 filter bed is the most important element and requires special attention as it is crucial to the functioning of the system. It is composed of different gravel layers, a layer with fine grains on the top and a layer with coarser grains at the bottom. The grading of the gravel differs from other types of roughing filters where grain size reduces and not increases in the direction of flow. With the fine gravel at the top, most of the suspended solids will accumulate at this point in the system which very much facilitates the cleaning of the unit. 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. In Table 8.1 preliminary design criteria are being indicated differentiating the two design alternatives of DyRF. In Table 8.2 preliminary specifications of the filter medium are been indicated as recommended by CINARA.

The filtration rate which can be applied may be rather high and will depend on local conditions. At present, filtration rates between 1 and 3 m/h are applied in Colombia.

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, a surface flow between 0.05 and 0.15 m/s is being recommended 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 quickly when peaks arrive at the unit, whereas 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.

Table 8.1 Preliminary Design Criteria for two DyRF design alternatives

PARAMETER	PRINCIPAL ROLE	
	First barrier to improve quality of water	Protection against peak loads
Filtration Velocity (m/h)	0.5 - 3	3-5
Range size of gravel in the upper layer (mm)	3 - 5	< 3
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 8.2 Specification of Filter Media for DyRF

Position of layer	First Barrier to improve quality		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
Middel	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

During a filter run the flow through the filter bed will gradually reduce with time. This reduction maybe between 20 to 40% during an operation period of one week provided that no peaks in suspended solids are being received (Figure 8.2). This implies that the design capacity needs to be at least 1.4 times the required capacity of the system. The DyRF is then operated as a declining rate filter. The operation of this type filter will be different when it is included in a pumped system. A constant design flow is then much more recommended as the cost of water pumping will otherwise be much higher. This implies that the outlet valve of the system will have to be adjusted daily to ensure that the total output of the system is kept constant by compensating for the gradual increase of the resistance in the gravel layer.

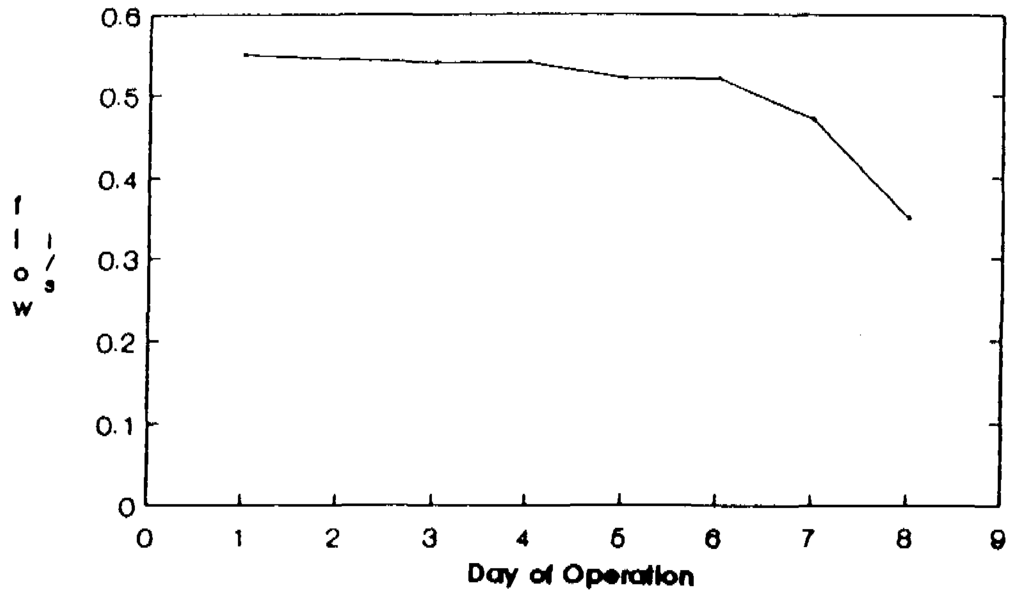


Figure 8.2 Reduction of the capacity of a DyRF, with a filtration rate of 1.5 m/h CINARA-IRC 1993

8.3 Design

Location

The location of the dynamic filter depends on the accessibility of the intake site. Easy access for the operator needs to be guaranteed particularly in the rainy season when maintenance work will be more frequent. Good accessibility will encourage the operator to carry out the maintenance tasks and not to neglect this important issue. Preferably, the dynamic filter is located close to the intake because it then protects the total system including the transmission main. If however, the intake site is not easily accessible then it is important to locate the dynamic filters closer to the other treatment units.

Dimensions

The dimensions of the DyRF and particularly those corresponding to the surface area (length and width) are conditioned by the water flow available for washing the surface. During washing, the flow velocity has to guarantee that the resuspended solids will be carried away from the filter. The selection of the width of the unit therefore has to guarantee that the horizontal flow is sufficient to facilitate maintenance and not exceed the recommended values because it may enhance the risk of carrying away filter material.

A practical relation between the width of the structure, the design capacity of the washing flow and the flow velocity is:

$$b = c^2 \frac{Q_L}{Vs^3}$$

b = width of the structure
V_S = flow velocity of wash water
Q_L = design capacity of wash flow (m^3/s)
c = discharge coefficient for a horizontal weir which on the basis of the formula of Francis can be taken as 1.84 $m^{½}/s$

Figure 8.3 presents a schematic design of a dynamic roughing filter and includes the procedures to calculate the dimensions of the unit.

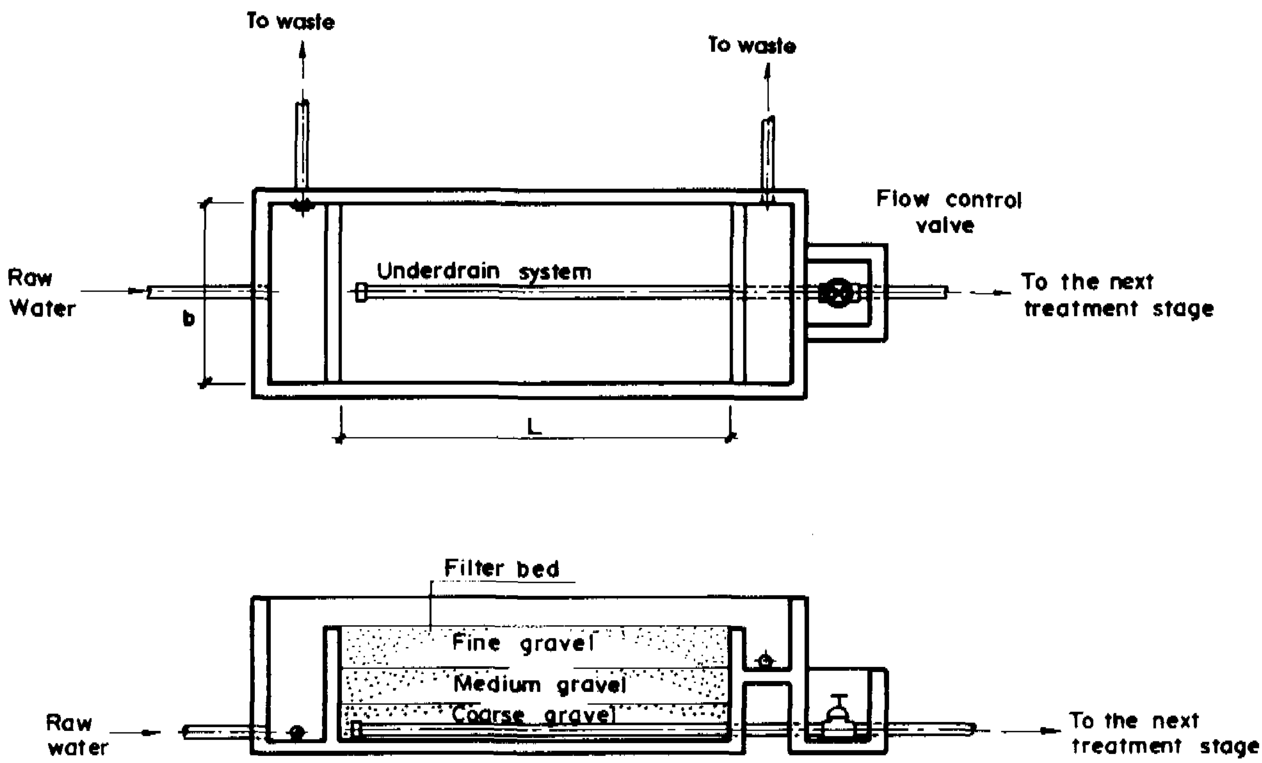


Figure 8.3 Schematic design of a dynamic roughing filter

The formulas used for the design calculation of the dynamic roughing filter are presented here.

I) Surface area of a DyRF

$$A_S = \frac{Q_F}{V_F}$$

with:

Q_F = filtration capacity (m^3/h)
V_F = filtration rate (m/h)
A_S = surface area (m^2)

II) Width of a DyRf

$$b = 3.4 \frac{Q_L}{Vs^3}$$

with:

b = width (m)
Q_L = wash flow (m^3/s)
V_S = velocity of wash flow (m/s)
c = 1.84 ($m^{½}/s$)

III) Length of a DyRF

$$L = \frac{A_S}{b}$$

L = length (m)

9. *Upflow Roughing Filtration*

9.1 General description

Upflow roughing filtration systems consist of one or several units which comprise gravel of different sizes which reduce in size from coarse to fine in the direction of flow. In systems with one unit (Figure 9.1) this is filled with layers of gravel of different sizes ranging from coarse at the bottom to fine at the top (upflow roughing filtration in layers: URFL). In systems with more units (upflow roughing filtration in series: URFS), the first unit comprises coarser gravel than the last unit. Upflow roughing filtration has the advantage that the first filtration occurs at the bottom of the filter where the drainage system is situated, thus facilitating drainage of the suspended solids which have been deposited. (See chapter 14). Furthermore, the flow direction reduces interferences, due to temperature or density differences. This improves the hydraulic behaviour, helps to avoid dead zones and results in a more homogeneous retention time and thus a better process of treatment.

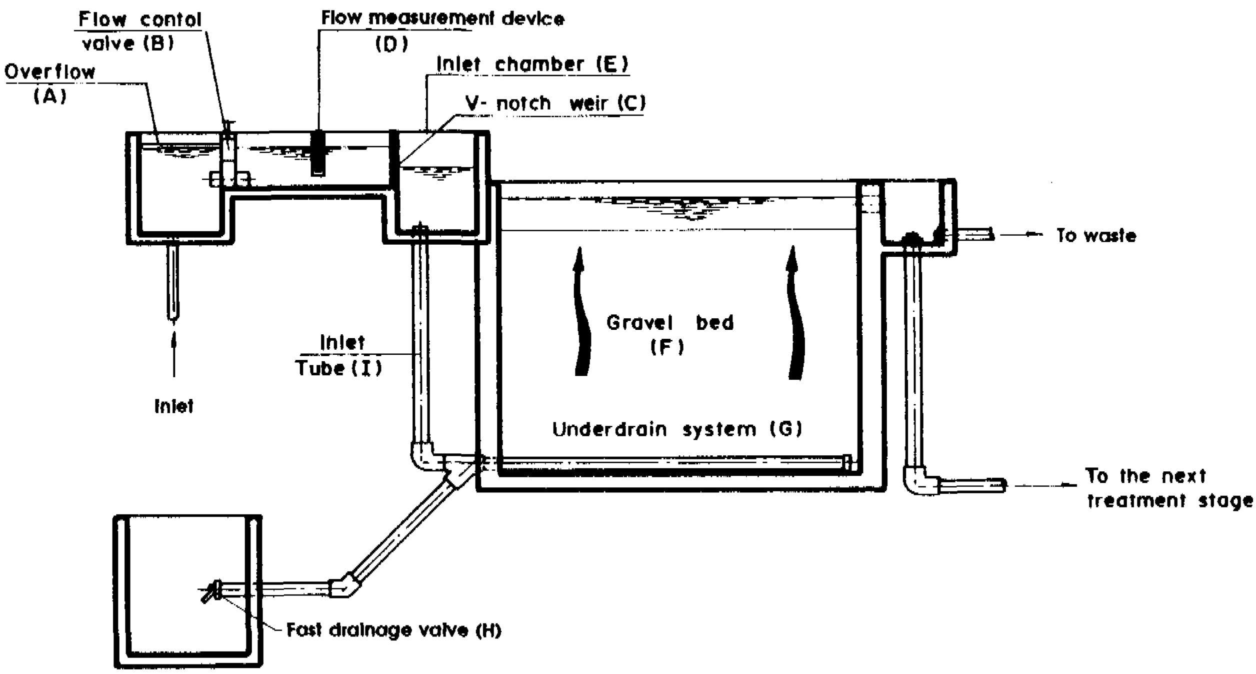


Figure 9.1. Schematic design of an upflow roughing filter in layers

Both systems have been evaluated in Colombia (CINARA, IRC 1993) on pilot scale and in full scale plants. For the URFS system removal efficiencies ranged for suspended solids between 92 and 97%; for turbidity between 69 and 83%; for true color between 29 an 68% and for faecal coliforms between 97.7 and 99.7%. For URFL removal efficiencies ranged for suspended solids between 49 and 94%; for turbidity between 46 and 71%; for true color between 10 and 46% and for faecal coliforms between 73.3 and 98.4%. These data are all based on units operating at filtration velocities between 0.3 and 0.75 m/h. Both systems also show removal of iron and manganese above 65% and COD above 50%.

The main components of an upflow roughing filter are:

a. Filter box: 1, 2 or 3 compartments
b. Gravel bed
c. Inlet and Outlet box
d. Drainage system
e. Regulation and control valves

Filter box

The height of the filter box depends on the height of the filter layer, the height of the drainage system, including support material, the level of supernatant water, the additional water volume required for adequate washing and free board. The height of the systems constructed so far range between 1.0 and 1.7 meters. The walls of the filter can be vertical or partially inclined. Construction may be in ferrocement, reinforced concrete or brickwork.

Filtration media

The filtration media usually consists of 5 different sizes of gravel distributed over one, two or three compartments. Selection and grading of the gravel is important to ensure that the different layers are uniform. This will create the best storage capacity and will facilitate cleaning. Table 9.1 presents the preliminary recommendations for gravel size and height of gravel layers for different alternatives with one, two or three units as recommended by CINARA-IRC 1993.

Table 9.1 Filter media recommended for upflow roughing filtration

Filter Media (mm)	Height (m)			
	URFL	URFS		
		1	2	3
19 - 25	0.20 - 0.30	0.2 - 0.3	0.15	
13 - 19	0.20 - 0.30	0.9 - 1.25	0.15	0.15
6 - 13	0.20 - 0.30		0.8 - 1.25	0.15
3 - 6	0.20 - 0.30			0.4 - 0.6
1.6 - 3	0.25 - 0.35			0.4 - 0.65

Inlet and outlet structure

The inlet structure needs to cater for the stabilization and measurement of the inflowing water and should allow for an overflow of excess water. In general, the structure comprises a small channel and a small shallow box. Both structures are separated by a wire which includes a calibrated indicator to measure the flow capacity (Annex 1). This structure also permits the control of the filters as the increase of the water level in the box indicates an increase of the hydraulic resistance in the system. This will provide a good indication for the operator when he needs to clean the filter. In chapter 13, the routine checks which need to be realized are indicated.

The outlet structure permits the collection of the filtered water and in case an URFS is being applied also serves as the inlet box for the next unit.

Drainage system

The drainage system has three basic objectives. It provides the uniform distribution of the water in the filter medium. It supports the filter medium and it facilitates a uniform distribution of the wash water during the washing of the filter. The design of the drainage system needs to cater for the difference in capacity and flow velocity during normal operation and during washing. Usually the drainage system consists of a perforated pipe (PVC of a diameter of 3 to 4 inches covered with a small layer of gravel with a grain size between 25 to 19 mm and a height of 20 to 30 cm. To facilitate cleaning a special drainage valve is being installed which can be opened quickly (Annex 1). The design of the number of orifices in the pipe needs to be based on the criteria used for the design of multiples as specified in Annex 2.

Regulation and control valves

The equipment used in the roughing filter units include flow regulating valves, fast drainage valves, weirs and flow indicators as indicated in Annex 1.

9.2 Design criteria

The design criteria which are indicated in Table 9.2 are based on experience with pilot plants and full scale installations evaluated in the context of the pre-treatment project in Colombia (CINARA-IRC, 1993). The recommended values for the rate of filtration and the depth of the filter medium show a considerable variation. The value which needs to be selected will depend as discussed in chapter 7 on the hygienic risk which is available in the raw water source. The higher the risk, the more gravel needs to be installed.

Table 9.2 Preliminary design guidelines for upflow roughing filters

ITEM	RECOMMENDED VALUE
- Design Period (years)	10 - 15
- Operation Period	24
- Rate of filtration	0.3 - 0.75
- Number of units (in series)	1 - 3
- Filter medium	
Depth (m)	0.85 - 1.25
Size (mm)	1.6 - 19
- Support Layer	
Depth (m)	0.20 - 0.30
Size (mm)	19 - 25
- Height of supernatant water	0.20
- Static head of wash water (m)	2.0 - 2.5
- Filtration area per unit (m^2)	15 - 25

9.3 Operation and maintenance

The operation of the upflow roughing filters includes the control of the filtration rate and the quality of the effluent, particularly in conditions of peak loads in suspended solids. The maintenance consist of weekly or monthly cleaning of the units by just opening the fast drainage valve at the bottom and cleaning the surface layer of the gravel (see chapter 13).

The control of the rate of filtration consists of measuring the water inflow and ensuring that the quantity does not exceed the specifications. Continuous operation of the filter is very important to ensure that the physical, chemical and biological processes taking place in the unit are sustained. Intermittent operation will result in reduction of the efficiency of these processes and may result in a penetration of impurities in the interior of the filter and subsequently in the effluent.

9.4 Design examples

This section comprises two design examples, treating water from different rivers under different technical, socio-economical and environmental conditions.

Treatment plant La Doctrina, Córdova

This plant is constructed in the neighbourhood la Doctrina of the municipality Lorica in the department Córdova in Colombia. It provides water to a population of 3745 persons with a design flow of 4.4 l/s. It consists of a dynamic roughing filter followed by two treatment lines, each comprising an upflow roughing filter in series (3 units) and a slow sand filter (Figure 9.2). The raw water has an average turbidity level between 30 and 80 NTU with peak values upto 380 NTU. Faecal Coliform Counts may reach values of 122000 FCU/100 ml.

Treatment plant of the College Colombo-Británico, Cali

This plant is built in the southern outskirt of Cali and provides water to a school population of 1200 students. The design flow of the system is 1.0 l/s. This plant used to be a chemical treatment plant with a very poor performance and high operation cost. In 1987 therefore the plant was modified into a combination of dynamic filtration, upflow roughing filtration and slow sand filtration. The plant is rectangular in shape and constructed in reinforced concrete (Figure 9.3). The raw water has an average turbidity level of 12 NTU with peak values upto 100 NTU. Faecal Coliform Counts average 27000 FCU/100 ml with maximum values of 228000 FCU/100 ml.

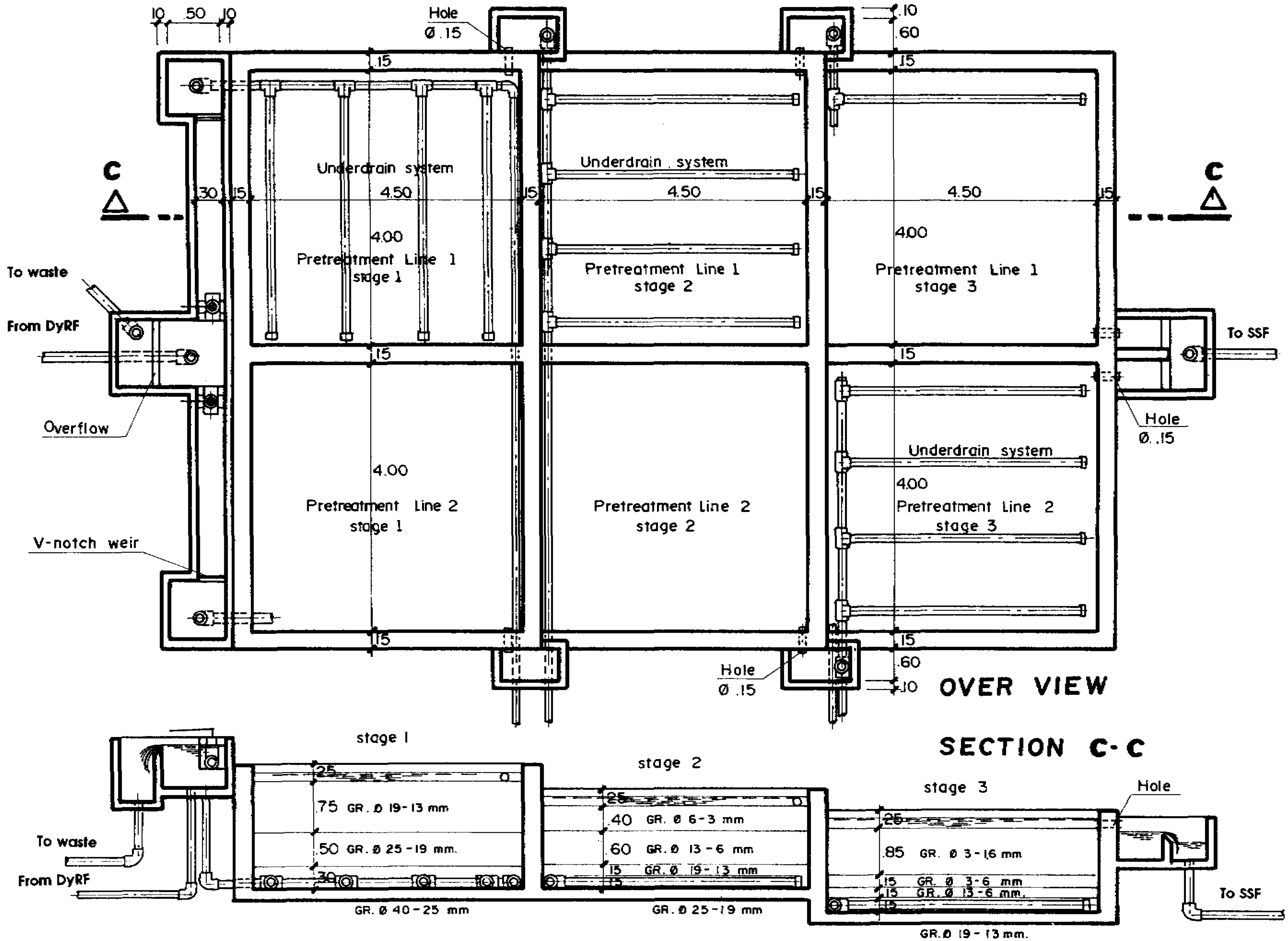


Figure 9.2 Upflow roughing filters in series, Treatment Plant in La Doctrina, Córdova

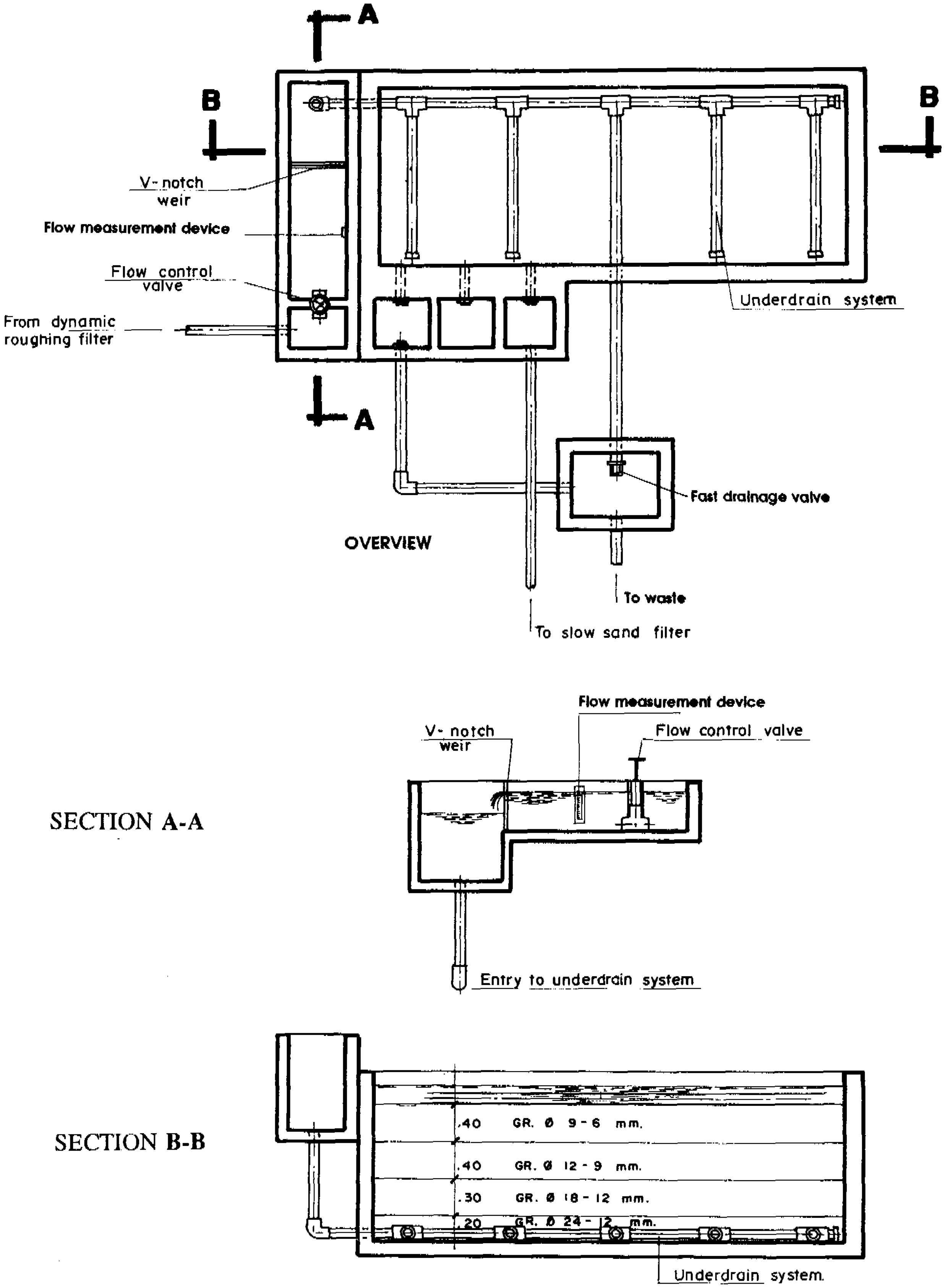


Figure 9.3 Upflow roughing filter in layers in the Treatment Plant of the College Colombo-Británico in Cali.

10. *Downflow Roughing Filtration*

10.1 General description

In downflow roughing filtration (DRF) (Figure 10.1) water flows through different units filled with gravel ranging from coarse in the first unit to fine in the last unit. On a pilot scale downflow roughing filtration has been tried in several places, but only in Peru have a few full scale plants been constructed. The experience available with the systems, including the research carried out by CINARA, is not sufficient to fully establish the possibilities and limitations. The units in the research station of CINARA treating water from Cauca River operating at flow velocities between 0.3 and 0.75 m/h, show removal efficiencies for suspended solids in the order of 98%, turbidity around 85%, true color between 54 and 62%, and faecal coliforms between 99.6 and 99.9%. In general, the removal efficiencies are very similar to those reported for upflow roughing filtration in series.

The basic components of a downflow roughing filter are:

a. Filter box: 2 to 3 compartments
b. Filter media
c. Inlet and outlet box
d. Drainage system
e. Regulation and control valves

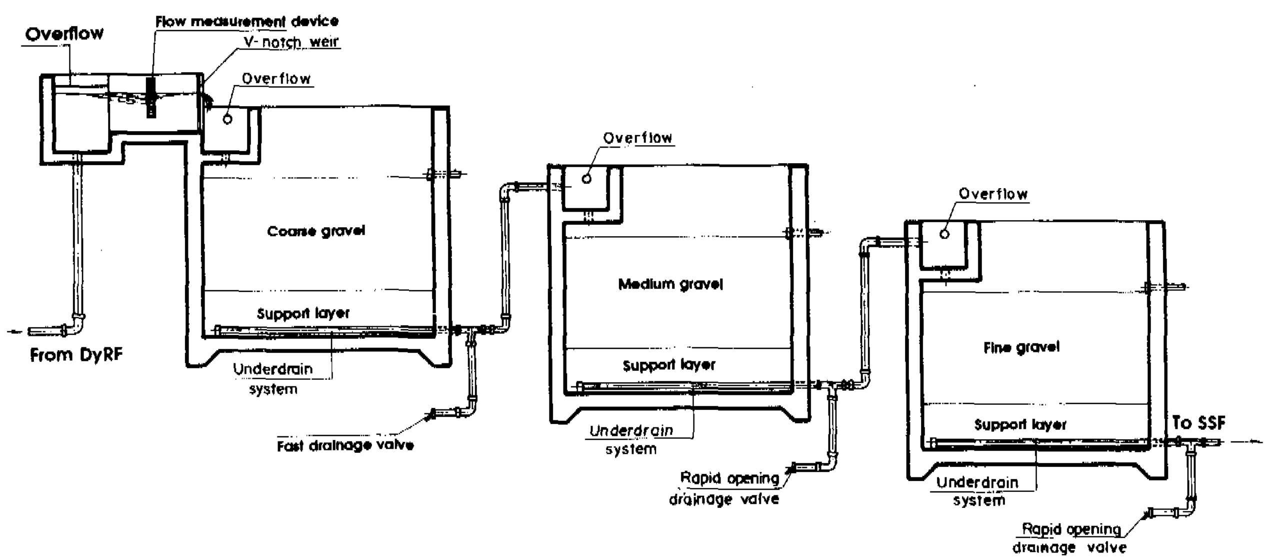


Figure 10.1 Schematic overview of a downflow roughing filter in series

Filter layout

A DRFS is somewhat higher than an URFS, as the height has to compensate for the headloss. The systems constructed so far range from 1.4 to 1.9 m in height. The structure can be built in reinforced concrete, ferrocement, or brickwork.

On pilot scale different filter materials have been used. For example the Asian Institute of Technology in Bangkok has experimented with coconut fiber, burned rice husk and carbon. Coconut fiber was found to be the filter medium with the most promising results (Frankel 1974). Removal efficiencies of 98% in turbidity were found for water turbidities between 24 and 130 NTU using a flow velocity of 0.25 m/h (Sevilla 1971).

Other researches in Peru and Colombia have focused on the use of gravel as filter medium. The results of these investigations show lower removal efficiencies for turbidity between 50 and 90% (Perez et al 1985, Quiroga, 1988; CINARA-IRC 1993). The use of gravel however has the advantage that the material does not have to be removed, but can be cleaned by fast drainage. Furthermore, gravel is an inert material and therefore will not have any effect on the quality of the water whereas the coconut fiber may be partly dissolved in the water.

Filter media

The DRFS systems constructed up till now compose of three compartments, filled with gravel ranging from coarse in the first unit to fine in the last. The filter media cannot be placed in layers in one unit because drainage of the system would carry all the deposited material to the fine gravel at the bottom. This would lead to clogging of the filter bottom and would require a removal of all the gravel from the units. Thus the alternative of a downflow roughing filter in layers is not feasible.

Inlet and outlet box

The inlet structure needs to cater for the stabilization and measurement of the inflowing water and to enable overflow of excess water. In general, the structure comprises of a small channel and a small shallow box. Both structures are separated by a weir which includes a calibrated indicator to measure the flow. (Annex 1). This structure also permits the control of the operation of the filters as the increase of the water level in the box indicates an increase of the hydraulic resistance in the system. This will provide a good indication for the operator when he needs to clean the filter. In chapter 13 the routine checks which needs to be realized are indicated.

Drainage system

The drainage system has three basic objectives: it provides the uniform distribution of the water in the filter medium, it supports the filter medium and it facilitates a uniform distribution of the wash water during the washing of the filter. The design of the drainage system needs to cater for the difference in capacity and flow velocity during normal operation and during washing. Usually the drainage system consists of a perforated pipe (PVC of a diameter of 3 to 4 inch covered with a shallow layer of gravel with a grain size between 25 to 19 mm and a height of 20 to 30 cm. To facilitate cleaning a special drainage valve is being installed which can be opened quickly (Annex 1). The design of the number of orifices in the pipe needs to be based on the criteria used for the design of multiples as specified in Annex 2.

10.2 Design criteria

As the experience with downflow roughing filters on full scale is limited only preliminary design criteria can be provided, based on the experience of Perez, 1986; Pardon, 1987, Quiroga, 1988 and CINARA-IRC 1993 (Table 10.1).

Table 10.1. Preliminary design criteria for downflow roughing filtration

PARAMETER	PEREZ, 1986 QUIROGA, 1988	PARDON, 1987
- Operation Period (h/d)	24	
- Filtration Rate (m/h)	0.3 - 1.2	0.30
- Number of Units	min. 2	1
- Height of filter media (m)	0.5 - 0.80	1.20
- Gravel size (mm)		
First compartment	50 - 25	40 - 25
Second compartment	25 - 12	25 - 12
Third compartment	12 - 6	12 - 6
- Height of drainage system including support layer (m)	0.15 - 0.30	

10.3 Operation and maintenance

Operation and maintenance of a DRF system is similar to URF systems and includes flow control, verification of filtration rate and weekly or monthly cleaning (See Chapter 13). It is not yet fully clear if the hydraulic cleaning of the units by just draining them is sufficient to remove all the suspended material which is deposited on the gravel in the filter. It may well be that some material remains in the box which may gradually result in a more rapid clogging of the units. If this is the case then the full gravel pack has to be removed, washed and put back into the unit again. Only further research and experience will clarify this point. The experience to date however is rather promising as full scale systems have been operating for over two years at flow velocities between 0.3 and 0.6 m/h with raw water with an average suspended solids content between 61 and 187 milligrams per liter. Monthly hydraulic cleaning which later on was intensified to weekly cleaning, was sufficient to recuperate almost the full headloss. Only after two years a full cleaning of the system was carried out taking out the gravel and washing it outside the system before putting it back into the box.

11. Horizontal Roughing Filtration

11.1 General description

In a horizontal roughing filter (Figure 11.1), water flows in a horizontal direction through a gravel medium which decreases in size. The filter medium is divided in three or four sections which reduce in length and comprise gravel reducing in size with coarse gravel in the first section to fine in the last section. The advantage of this system is that the gravel layers can be extended without the need of a higher structure or the need of more hydraulic energy. According to Wegelin, 1986, this type of systems are already being used for over 30 years for artificial recharge of aquifers in Germany, Switzerland and Australia. More recently the IRCWD supported research on this technology in projects in different countries including Peru, Colombia, Ghana, Tanzania and China.

The investigations of different alternatives realized in Colombia (CINARA-IRC 1993) with pilot scale and full scale units indicated removal efficiencies for suspended solids in the order of 90% for turbidity between 69 an 88% and for faecal coliforms over 95%. These systems were all operated at flow velocities between 0.3 m/h and 0.6 m/h.

The main components of a horizontal roughing filter are:

a. Filter box with three or four compartments
b. Filter media
c. Inlet and outlet box
d. Drainage system
e. Separation walls
f. Regulation and control valves

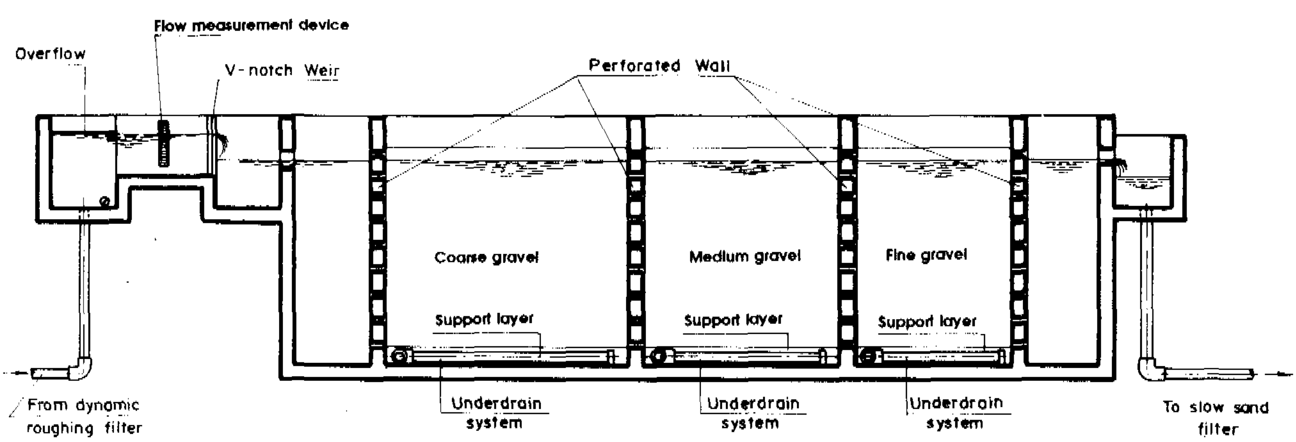


Figure 11.1 Schematic design of a Horizontal Roughing Filter

Filter box

The filter box is divided in three or four sections; each filled with gravel of a specific size and separated by vertical walls or wire mesh. The height and the width are less than 1.5 m and 5.0 m respectively (Wegelin, 1986) and the length is still under investigation and may range from 8 to 16 meters.

Filter material

The three or four sections of the filter are filled with gravel ranging from coarse in the first unit to fine in the last. Table 11.1 indicates the dimensions of the different gravel layers and the gravel size as given by Wegelin, 1986.

Inlet and outlet structure

The inlet structure needs to guarantee a uniform distribution of the water over the full cross section of the filter. Furthermore, it needs to prevent the inflow of coarse and floating material. The width of the inlet structure therefore is rather large and is recommended to be about 0.8 meters (Wegelin, 1986). The inlet box is also important as it directly visualizes the headloss in the unit which helps to control operation and maintenance.

The outlet box has to permit the uniform abstraction of the water and includes a fixed weir to ensure a minimum water level at the end of the filter. Wegelin, 1986 recommends that the width of this box is 0.6 meters.

Drainage system

Although initially this technology did not include a drainage system and was to be cleaned by taking out all the gravel, the application of draining systems in the other roughing filter alternatives have shown their importance for good operation and maintenance. Therefore, the horizontal roughing filter is now also designed with a drainage system. (Annex 2).

Separation walls

The function of these separation walls is to separate the different gravel sections to avoid gravel from one section mixing with the next section. The walls permit taking out the gravel in one section for cleaning without disturbing the gravel in the next section. The walls also have to ensure a more uniform distribution of the water over the filter. The walls can be made out of brick-work or concrete blocks which have perforations of 3 centimeters in diameters or open sections of 5 x 5 centimeters. The total area of the openings should be between 16 to 20% of the surface area of the cross section (Wegelin 1986).

11.2 Design criteria

In Table 11.1 the preliminary design criteria for this technology as recommended by Wegelin are indicated. These criteria are based on pilot scale and full scale experiences in different countries including Colombia.

11.3 Operation and maintenance

As with the other roughing filtration alternatives, the simplicity of operation and maintenance procedures permits that persons from the community with little formal education can easily realize the tasks involved. In chapter 13 the tasks which the operator has to carry out in relation to operation and maintenance are described in detail.

Table 11.1 Preliminary design criteria for horizontal roughing filters (Wegelin, 1989)

PARAMETER	MAXIMUM CONCENTRATION OF SUSPENDED SOLIDS (mg/l)		
	< 100	100 - 300	> 300
Filtration Rate (m/h)	1.00 - 1.50	0.75 - 1.00	0.50
Length of gravel layers(m)			
20 mm	3(*)	3(*)	3 - 5
15 mm	2 - 3	2 - 4	2 - 5
10 mm	2	2 - 3	2 - 4
5 mm	1	1 - 2	1 - 2
Total Length of Filter Media (m)	8 - 9	8 - 9	8 - 16

* This section can be left out.

11.4 Design example

In Figure 11.2. some general information is provided on the design of the horizontal roughing filters of the treatment plant of the University Javeriana in Cali. The plant is designed by CINARA with a projected population of 3000 students and 200 permanent staff, including teachers, technical staff and security staff. The design flow is 3.0 l/s. The raw water has an average turbidity level of 18 NTU with peak values of 110 NTU. Faecal coliform counts average 6800 FCU/100 ml with maximum values of 100000 FCU/100 ml.

-Design capacity: 3.0 l/s

-Rate of filtration: 1.2 $m^3/m^2/h$

-Number of systems: 2

-Number of units per system: 2 per line

-Geometric form: Rectangular prisma

-Construction material: Concrete blocks

-Filter dimensions:
Width 3.8 m
Length 4.0 m

-Height of drainage system: 0.2 m

-Height of filter media: 1.2 m

-Location fast drainage valve: 1.0 m

-Number of fast drainage valves: 4

-Quantities of filter media:

Gravel:	19 - 12 mm	8.8 m^3
	12 - 6 mm	9.0 m^3
	6 - 3 mm	13.0 m^3
	3 - 2 mm	5.5 m^3

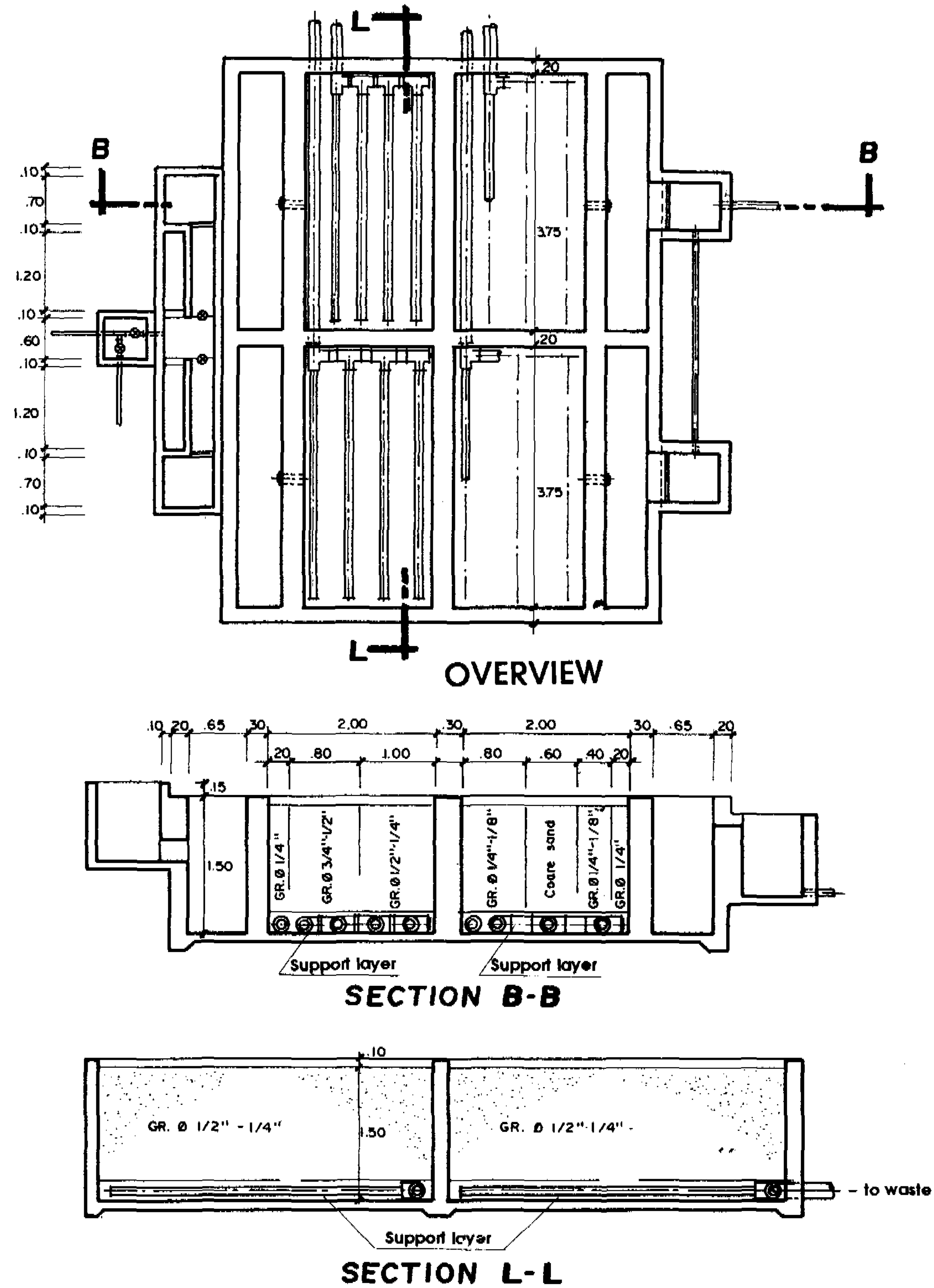


Figure 11.2 Horizontal roughing filters in the Treatment Plant of the University Javeriana, Cali, Colombia

12. *Construction of Pre-treatment Systems*

Good design does not necessarily mean the construction of a good plant because much depends on the quality of materials used, available skills, and quality of supervision provided during construction. It is necessary to discuss the technical aspects with the contractor who usually has not much experience with the construction of this type of works. Therefore, it is important to monitor progress with a high frequency particularly if the contractor is not familiar with the use of new materials. A supervisor should particularly pay close attention to the mixing, compaction and curing of concrete as this largely determines the strength and the lifetime of the construction. A number of key issues need to be evaluated during the planning and design stage, such as type of materials available in the region, access to the site, topographical and geotechnical characteristics of the site and quality of labor available. These aspects need to be known before the project can be fully designed. This will enable the development of a good design and identification of suitable locations for the different units and hydraulic elements such as weirs, overflows, valves and flow indicators.

The filter box can be constructed in reinforced concrete, ferrocement, brickwork, plastics or stabilized earth, or even wood. Water tightness of this structure needs to be guaranteed which implies special attention during construction particularly with reference to the structural stability and to details such as wall floor connection and sections where pipes pass through walls. This is very important to avoid future cracks in the construction and leakages. It is better to have a relatively coarse wall finish to ensure a high resistance against the flow, making it pass through the filter media in order to avoid short circuits. Another point of special attention is the location of the weirs, overflows and flow channels which have to be located in such a way that it facilitates operation and maintenance and ensures continuous flow at the designed level.

The filter media needs to be gravel free from organic material and clay particles and needs to match the specifications for the grain size. Preferably, different layers of gravel with different grain sizes are being separated by plastic wire mesh. This very much facilitates maintenance during the washing of the filter media as it avoids the mixing of gravel of different size. (See chapter 13). Grading of the gravel will often be necessary to ensure that the right size is being obtained. This can be done manually or with a machine using sieves of different openings corresponding to the required diameter of the grains. After sieving a grain size distribution diagram needs to be made of each fraction which has been separated to verify that the grain size distribution is within the set limits. Before it is placed in the filter box the gravel needs to be carefully washed. Filling of the units can best be done in layers which are leveled before the next layer is put in place.

13. *Operation and Maintenance*

Adequate operation and maintenance is essential to ensure the good functioning of the gravel filters. The procedures which have been developed are easy in their execution and do not require special equipment or highly qualified staff. Staff does require however, adequate training and a working environment which fully supports and respects them and stimulates them to carry out their important task which ensures the provision of good quality water to the community. This section presents the different operation and maintenance procedures involved in the maintenance of dynamic filters and upflow roughing filters. Procedures for the other systems are not presented because the URF alternative is being considered as the most promising and maintenance procedures of the other systems are rather similar.

13.1 The operator's role

To enable the operator of the system to carry out his maintenance tasks, adequate support from the organization responsible for the system and from the community is required. This implies that good communication needs to be established between the different parties involved. Furthermore, it is particularly important that the community is well informed about possible interruptions of the water distribution resulting from repairs or maintenance activities.

The work of the operator needs to be facilitated by the establishment of a detailed schedule of key activities and their frequency. Although the general activities involved in the operation and maintenance of the systems will not differ very much, their frequency can be affected by the quality of the water and the local conditions. This makes it necessary to develop individual schedules for each plant. These schedules need to be developed together with the operator and the responsible organization, and needs to be reviewed regularly on the basis of plant performance. The performance of the pilot units and the full scale plans in Colombia show that better results can be obtained if the maintenance activities are carried out on a regular basis. Often the tasks of the operator includes other activities outside the treatment system for example, in the inlet structure, the distribution system and may also include the collection of water tariffs and the support to the administrative body. Almost always such other activities can be carried out by the operator as they do not interfere much with maintenance tasks proper to the functioning of the system. These tasks also bring the operator in close contact with the community and will permit to establish good communication and facilitate the exchange of information between the operator and the community about interruption in the systems, repairs, problems in the distribution network, and satisfaction of the consumers about the water quality. To specify the activities of the operator, these have been broken down in activities which have to be carried out daily, weekly, monthly and occasionally.

13.2 Daily tasks

The daily tasks of the operator are very much focused on ensuring adequate operation of the system keeping it clean and checking different aspects to ensure that the small changes in headloss and flow velocities which may occur during normal operation are being detected so that an adequate response can be given. Floating material in the supernatant water of the roughing filter has to be removed whenever this is present as this will avoid possible blockages of the system and makes the visual outlook of the plant more pleasant. The level of clogging of the DyRF needs to be checked by measuring the outlet flow from the unit. This flow needs to be larger or equal to the minimum flow which still guarantees adequate water supply for the community. A smaller value implies that the unit needs to be cleaned immediately. Larger values than 1.4 times the minimum design flow are also to be avoided as this will mean that the unit may be operating at a lower efficiency level. Dynamic filters in pumped schemes however may be operated differently to ensure that they continuously produce the design flow. This requires gradually opening of the outlet valve to compensate for the increase in the resistance in the filter bed.

In each of the roughing filter lines the rate of filtration has to be checked by measuring the output of the units to ensure that it is not over the design rate. The installation of an overflow weir and a flow indicator D, are adequate systems to control the flow and facilitate the work of the operator (figure 13.1). In Annex 1 these instruments are shown in detail. When the rate of filtration is above the design rate, valve B needs to be closed somewhat to ensure that part of the inflowing water is wasted over the overflow weir A.

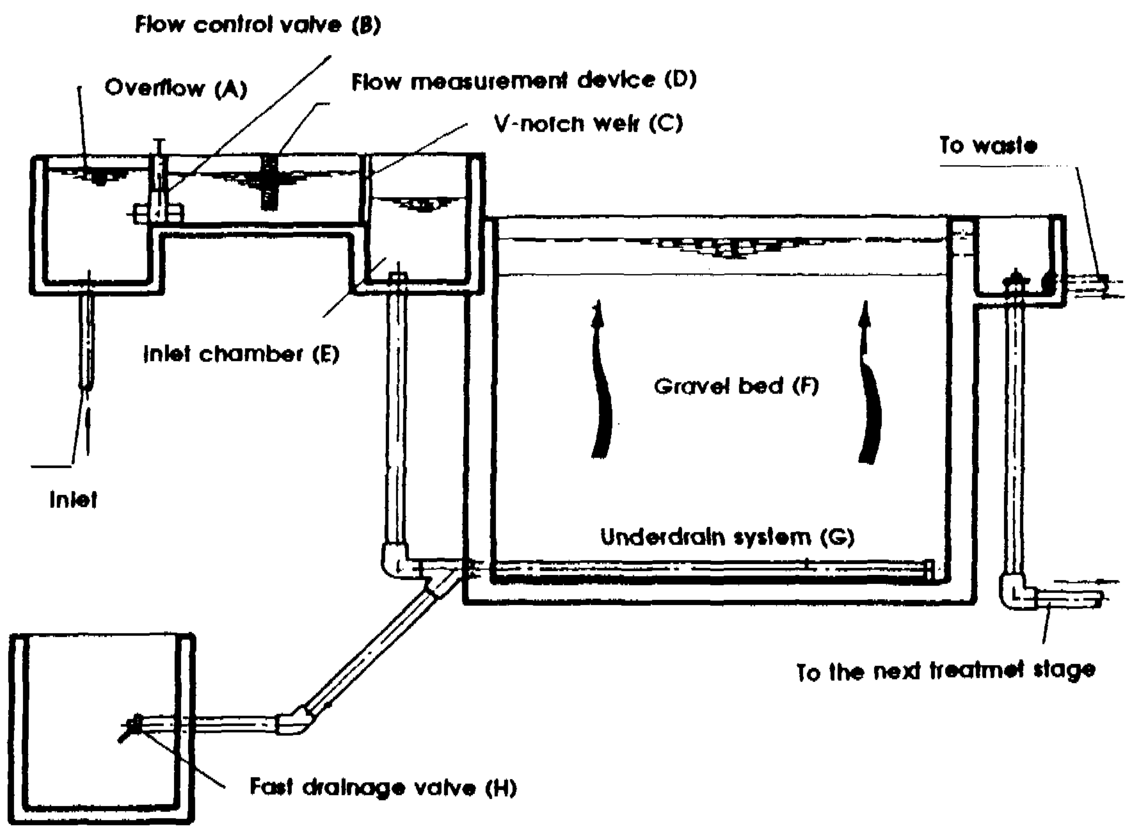


Figure 13.1. Upflow roughing filter in layers

Checking the headloss in the roughing filter will permit a decision to be taken about the cleaning frequency. The headloss can be observed by checking the water level in the inlet structure E. This level will increase day by day until it reaches the level of the overflow. Then cleaning will be required to bring the headloss back to the initial level. Cleaning of the unit needs to be realized weekly unless the headloss development is quicker. This then needs to be done immediately. If headloss is not being restored after washing then this may be due to one of the following reasons:

a. Entering of air in the drainage system
b. Inefficient washing
c. Full obstruction of the filter media

The cases a and b can be restored by draining the filter again. If however, the filter is fully blocked then new drainage will not solve the problem and only the full washing by taking out the gravel can then restore the initial headloss. This work needs to be discussed with the community as it involves a great deal of labor.

The operator needs also to carry out activities related to quality control of the water. This can be done by simple equipment for measuring turbidity, PH and residual chlorine. Daily checking of the water turbidity and registering it in a log book is essential. This information will permit the detection of operational problems in the units and will help in the decision making about modifications in the system.

13.3 Weekly operation

In most schemes the weekly routine will include the cleaning of the roughing filters and the dynamic filters. Cleaning of the DyRF comprises basically of the cleaning of the surface layer of the gravel. This cleaning is an easy task which usually only takes some 30 minutes. In this work it is important not to mix the fine gravel layer on top with the coarser gravel deeper in the bed. Therefore, the installation of a plastic wire mesh between the coarse and the fine gravel is very useful to help the operator to do a good job. Under normal conditions the DyRF needs cleaning once a week. During the rainy season or for sources which carry a high load of suspended solids the cleaning frequency may have to be increased to two or three times per week.

The cleaning of the URFL consists in draining the units after having opened and closed the fast drainage valve several times. Experience on pilot scale and in full scale plants has shown that opening the valve some 8 times within 10 seconds produces the best results. This was amongst others identified in the project "Hydraulic Cleaning of Roughing Filters" implemented with support of IDRC. The inlet to the roughing filter (I, Figure 13.1) needs to be closed before the fast drainage valve is being manipulated to ensure the highest impact of the opening and closing of the valve and avoid the entrance of air in the gravel bed.

In the weekly tasks, cleaning of the plant site needs to be included and particularly of those components where materials may settle such as inlet and outlet boxes. In the inlet box sludge is usually accumulated and in the outlet structure algae may grow. Keeping the plant site and the structures as clean as possible is important to establish a pleasant and professional environment which will raise the confidence of consumers.

13.4 Monthly operation

For the DyRF there are no real additional tasks to be fulfilled on a monthly basis. The monthly tasks in the roughing filters comprise: cleaning of the surface layer of the gravel bed, manipulation of the fast drainage valve 10 times, and subsequent drainage of the unit. These tasks may involve the operator for approximately half a day. The cleaning of the surface layer is carried out while the filter is operating under the design flow. The water however is not passed on to the subsequent unit, but wasted, as it comprises a lot of suspended solids and other impurities. The gravel in the surface layer is moved about from one side to the other to ensure adequate cleaning. This activity is ended when a visual inspection shows that the outflowing water and inflowing water are approximately of the same quality. Then the inlet and outlet box are cleaned. Subsequently, the fast drainage valve is being manipulated and the filter is drained. The draining is stopped when the quality of the drainage water shows a marked improvement. Then the inflow is re-opened until the filter is filled up completely and the process of hydraulic cleaning by opening the fast drainage valve 10 times can be repeated.

13.5 Occasional operation

Once the blockage of the filter media in the roughing filters or the DyRF is such that normal cleaning does not recuperate the initial headloss and allows the unit to operate on the designed rate of filtration, then it will be necessary to remove the gravel from the box and clean it outside the unit. Whether this activity is required depends very much on the efficiency of the weekly and monthly cleanings and should be avoided as much as possible because it is a costly and laborous process. For the DyRF, this task is less costly and laborous because the volume of gravel is much smaller than in the roughing filters. When this type of cleaning is required it is very important to handle the gravel with care in order to avoid the mixing of layers with different grain size. Such mixing will otherwise very much reduce the sludge volume which can be retained in the gravel layer. If mixing occurs then the gravel needs to be graded again before replacing it in the unit.

When the raw water quality deteriorates particularly due to an increase in suspended solids, the DyRF filter will very much protect the subsequent unit from excessive loads in suspended solids. Nevertheless, under these conditions the operator has to be alert and more frequently check the quality of the water flowing in and out of the different treatments units. If quality deteriorates it may be necessary to ensure the continuation of the flow through the units at a lower rate to avoid the interruption of the biological processes. Reduction of the flow velocity may be particularly necessary when the DyRF is fully blocked and peak turbidities are still present in the raw water. Another reason for reducing the flow is the quality of the outflowing water. The DyRF may not clog fully but still the water flowing to the next unit may show an increase in the level of turbidity. If this level exceeds 25 NTU, at the inlet of the slow sand filter then the operator has to reduce the filtration rate in the slow sand filter and the pre-treatment units with 50%. If even after this reduction turbidities are being presented over 25 NTU, a further reduction to 25% or in extreme cases 10% of the designed filtration rate is required. In the extreme the full treatment plant needs to be shut down. It needs however to be realized that the biological processes in the system will be affected and as a result re-establishing good quality effluent will take one to two days. It is therefore necessary to avoid wherever possible the complete shut down of the plant.

In case the water flow in the plant is being reduced it needs to be explored if the users need to be advised. This will be the case if as a result of the lower production of water, consumption has to be reduced. Experience will have to show the operator under which conditions it will be important to advise the community. It may however be obvious that if the full plant is being shut down then the consumers need to be informed preferably before hand so they can store some water in their homes and furthermore, they have to be advised not to use the water for drinking without boiling it a few days after the system is put in operation again. Advising the community can for example be done with help of the water committee.

13.6 Work plan

In each treatment system a work plan has to be established for the different activities together with the operator. Tables 13.1 and 13.2 give examples of the tasks of the operator related to the DyRF and URFL.

Table 13.1 Operators tasks in Dynamic Roughing Filtration

Frequency	Activity
Dynamic Filter Daily Tasks	- Check the level of clogging - Measure the quality of water at the intake - Register the water quality
Periodic Tasks	- Realize the cleaning of the filter media * Close the outlet valve * Open the inlet valve further to obtain the flow velocity for cleaning * Rake the surface of the filter media in the direction opposite to the flow to re-suspend the settled material * Move about the first 20 cm of the gravel layer with a spade * Stop the inflow * Quickly open and close the fast drainage valve 10 times; thereafter leave it open until the quality of the drainage water shows a marked improvement * Close the drainage valve * Re-establish the inflow * Open de outlet valve NOTE: In the dry season this labor needs to be done weekly and in the wet season frequency may have to be increased to two or three times per week depending on the local conditions
Occasional Tasks	- Cleaning of the filter by removing the gravel. If possible carry out a normal cleaning procedure; thereafter close all valves and open the drainage valve to fully empty the filter * Remove the gravel layer avoiding mixing of the different gravel sizes. * Wash the gravel layers separately * Reinstall the gravel in the original position * Restart the flow in the units

Table 13.2 Operator Tasks in Upflow Roughing Filtration

Frequency	Activity
Daily Tasks	- Check the water flow in each unit - Remove floating material - Measure water quality in the inlet and the outlet of each unit - Register the water quality data and other possible observations
Weekly Tasks	- Clean the inlet and outlet box ensuring that the wash water is being wasted through the overflow - Implement the weekly cleaning of the roughing filter * Close the inlet flow to the unit * Insert the stop in the inlet pipe * Open and close the fast drainage valve 8 to 10 times within some 10 seconds - Drain the water until a marked improvement in the quality of the drainage water is being observed - Close the valve, re-establish the flow in the filter by removing the stop and opening the inlet valve
Monthly Tasks	- Carry out the monthly cleaning of the filter * Block the flow to the subsequent units * Move the first 20 cm of the gravel layer about with a spade until the visual quality of the inflowing and overflowing water is similar * Clean the inlet and outlet box ensuring that the water is wasted through the overflow * Close the inlet valve and insert the stop in the inlet pipe * Quickly open and close the fast drainage valve 10 times and drain the unit until the quality of the drainage water shows a marked improvement * Close the valve and re-fill the unit * Repeat the process of opening and closing the fast drainage valve and draining of the unit * Re-establish normal operation
Occasional Activity	- When the quality of the inflowing water deteriorates as a result of a temporary peak load, the flow to the filter needs to be reduced or stopped - Cleaning of the inlet pipe to the filter using a brush - Full cleaning of the gravel layer by removing the gravel and cleaning it outside the filter box. In this operation care has to be taken not to mix the different gravel layers.

Bibliography

Acodal (1992). *Revista Acodal,* No. 150.

Bellamy, W.D.; Silverman, G.P. and Hendricks, D.W. (1985). *Filtration of Giardia cysts and other substances.* (Slow Sand Filtration; Vol. 2). Cincinnati, OH, USA. Water Engineering Research Laboratory.

Camp, T.R. (1964). "Theory of water filtration". In: *Journal ASCE.* vol 90, no. SA4.

Canepa, L. (1982). *Filtros de arena en acueductos rurales :* informe final. Lima, Peru, CEPIS.

Castilla, A. and Smet J.E.M. (1989). "Tilted plate settling". In: Smet, J.E.M. and Visscher, J.T. (ed.). *Pre-treatment methods for community water supply: an overview of techniques ad present experience.* The Hague, The Netherlands, IRC International Water and Sanitation Centre. p. 75-91.

Cinara and IRC (1993). *Proyecto integrado de investigación y demostración de método de pretratamiento para sistemas de abastecimiento de agua: informe final.* Cali, Colombia, CINARA.

Cinara and IRCWD (1988). *Proyecto de promoción de la tecnología HRF en Colombia: informe final.* Cali, Colombia, CINARA.

Cleasby, J.L.; Hilmoe, D.J. and Dimitracopoulus, C.J. (1984). "Slow sand and direct in-line of a filtration surface water". In: *Journal of the American Water Works Association,* vol. 76 no. 12, p. 44-55.

Cleasby, J.L. (1991). "Source water quality and pre-treatment options for slow sand filters". In: *Task Committee on Slow Sand Filtration.* New York, N.Y., USA.

Collins, M.R. et al (1989). *Modifications to the slow sand filtration process for improved removals of Trehalomethane Precursors.* AWWARF Final Report.

Collins, M.R.; Eighmy T.T. and Malley, J.P. (1991). "Evaluation modifications to slow sand filters". In: *Journal of the American Water Works Association,* vol. 83, no. 9, p. 62-70.

Craun, G.F. (1988). "Surface water supplies and health". In: Journal of the American Water Works Association, vol. 99 no.2, p. 40-52.

DNP (1991). Agua potable y saneamiento básico : plan de ajuste sectorial : segunda fase 1991-1994. Santa Fé de Bogotá, Colombia, DNP.

Ellis, K.V. (1985). "Slow sand filtration". In: *C.R.C. Critical Reviews in Environmental Control.* vol. 15, p. 315 - 328.

Ellms, J.W. (1917). *Water purification.* New York, N.Y., USA, McGraw-Hill.

Engels, C. and Poggenburg, W. (1989). "Riverbank Filtration". In: *Pre-treatment methods for community water supply*: The Hague, The Netherlands, IRC International Water and Sanitation Centre. p. 5-14.

Fox, K.R.; et al (1984). "Pilot Plant Studies of Slow Rate Filtration". In: *Journal of the American Water Works Association*, vol. 76, no. 12, p. 62-68.

Frankel, R.J. (1974), "Series filtration using local filter media", *Journal of the American Water Works Association*, vol. 66, no. 2, p. 124-127.

Galvis, G. (1992). "Innovative technology for reduction in biological contaminants and THM precursors. In: Craun, Gunther F. (ed.) *The Safety of water disinfection: balancing chemical and microbial risk.* Washington, D.C., USA, ILSI Press. p. 415-428.

Galvis, G.; Fernández, J. and Visscher J.T. (1992). *Comparative study of different pre-treatment alternatives:* paper presented at the Workshop in Zurich on roughing filters for water treatment 25-27 June 1992.

Galvis G. and Visscher J.T. (1989). *Proyecto Integrado de Investigación y Demostración en Filtración Lenta en Arena: informe final.* Versión Resumida. Cali, Colombia, CINARA and The Hague, The Netherlands, IRC International Water and Sanitation Centre.

Galvis, G.; Visscher J.T. and Lloyd, B. 1991. *Overcoming Water Quality Limitations with the Multi-Barrier Concept, A Case Study from Colombia.* Paper presented in the AWWA Conference. New Hampshire, USA. November 1991.

Haarhoff, J. and Cleasby J.L. (1991). "Biological and physical mechanisms in slow sand filtration". In: *Task Committee on Slow Sand Filtration.* New York, N.Y., USA. Chapter 2.

Haberer, K.; Drews, M. and Waltenberg, (1984). Entfernung organischien der Wasserinhaltstoffe bei der Langsamsandfiltration. In: *Vom Wasser*, vol. 63, p. 121-140.

Hazen, A. (1913). *The filtration of public water supplies.* 3rd ed. New York, N.Y., USA.

Hendricks, D.W. and Bellamy, W.D. (1991). *Microorganism removals by slow sand filtration.* New York, N.Y., USA.

Hespanhol J. (1969). *Investigacao sobre o Comportamento e Aplicabilidades de Filtros Lentos no Brazil.* Sao Paulo, Brazil. Universidade de Sao Paulo, Facultadad e Higiene e Saúde Pública.

Hudson, H.E.; Uhler, R.B. and Bailey, R.W. (1979). "Driving flow manifolds with square-edged laterals". In: *Journal of the environmental engineering division.*

Hudson, H.E. (1988). *Water clarification processes : practical design and evaluation.* New York, N.Y. USA, Nostrand Reinhold.

Huisman, L. (1982). *Slow sand filtration.* Delft, The Netherlands, Delft University of Technology. Department of Sanitary Engineering.

Huisman, L. and Wood, W.E. (1974). *Slow Sand Filtration.* Geneva, Switzerland, World Health Organization.

Ives, K.J. (1957). *Algae and Water Supplies, Physical Removal of Algae, Water Engineer.* 61.432.

IRC and CINARA (1992). *Filtración lenta en arena, tratamiento de agua para comunidades : planeación, diseño, construcción, operación y mantenimiento.* Cali, Colombia, CINARA.

IRCWD (1992). *Surface water treatment by roughing filters, with special emphasis in horizontal flow roughing filtration.* Duebendorf, Switzerland, IRCWD.

Joshi, N.S. et al. (1982). "Water Quality Changes During Slow Sand Filtration". In: *Indian Journal of Environmental Health,* vol. 24, no. 4, p. 261-266.

Letterman, R.D. and Cullen T.R. (1985). *Slow sand filter maintenance : cost and effects on water quality.* Cincinnati, O.H. USA. U.S. Environmental Protection Agency.

Lloyd, B. (1974). *Functional microbial ecology of slow sand filters.* University of Surrey, Guildford, U.K., Ph. D. Thesis Report.

Lloyd, B.; Pardon, M. and Wheeler, D. (1986). *Final report on the development evaluation and field trials of a small scale multi-stage modular filtration system for the treatment of rural water supplies.* Lima, Perú, DelAgua.

Lloyd, B.; Bartram, J. and Pardon, M. (1987). *Report on drinking water surveillance programme : phase 3c: evaluation of the pilot region diagnostic.* Lima, Perú, DelAgua.

Lloyd, B. and Helmer, R. (1991). *Surveillance of drinking water quality in rural areas,* Harlow, U.K., Longman Scientific and Technical.

Logsdon, G. and Fox, K. (1988). "Slow sand filtration in the United States". In : Graham N.J.D. (ed.) *Slow sand filtration : recent developments in water treatment technology.* Recent Developments in Water Treatment Technology. U.K., Chichester, Ellis Horwood. p.29-45.

Mbwette, T.S.A. (1983). *Horizontal flow roughing filters for rural water treatment in Tanzania.* Dar es Salaam, Tanzania, University of Dar es Salaam. M.Sc. Thesis.

Okun, D.A. (1991). "Best available source". In: *Journal of the American Water Works Association.*

O'Melia, C.R. (1985). "Particles, pretreatment and performance in water filtration". In: *Journal of environmental Engineering*, vol. III no. 6.

O'Melia, C.R. and Statumm (1967), "Theory of filtration". *Journal of the American Water Works Association*, vol. 59.

PAHO (1990). *Las condiciones de salud en las Américas : problemas ambientales que afectan a la salud.* Washington, D.C., USA, Pan American Health Organization.

Pardon, M. (1987). *Consideraciones, desarrollo y evaluación de un sistema de tratamiento que implementa la filtración gruesa de flujo vertical en gravas.* Lima, Perú, DelAgua.

Pardon, M. (1988). Final report evaluation of the water treatment plants of Cocharcas, La Cuesta and Compin : *Components of a rehabilitation Programme Introducing Horizontal Roughing Filtration Technology to Rural Water Supply Schemes.* Switzerland. Lima, Peru, Del Agua.

Pérez, J.; Pardon, M; Lavado, I. and Vargas, C. (1985). *Informe preliminar de la investigación sobre prefiltros de grava.* Lima, Perú, CEPIS.

Quiroga, E. (1988). *Estudio preliminar sobre pretratamientos : reporte final.* Cali, Colombia, Universidad del Valle.

Rachwal, A.J.; Bauer, M.J. and West, J.T. (1988). "Advanced techniques for upgrading large scale slow sand filters". In: Graham, N.J.D. (ed) *Slow sand filtration* : recent developments in water treatment technology. Chichester, U.K., Ellis Horwood, p. 331-347.

Reiff, F. (1988). "Desinfección del agua potable con una mezcla de gases oxidantes producidos in situ". (Moggod). In: *Boletín oficina Sanitaria Panamericana*, vol. 105, no. 4, p. 371-389.

Rook, (1974). "Formation of haloforms during chlorination of natural water". In: *Journal Water Treatment and Examination.* vol. 23, no. 2, p. 234.

Salazar, L. Guillermo (1980). "Agua potable para la zona rural Colombiana". In: *ACODAL* no. 96, p. 9-24.

Schulz, C.R. and Okun, D.A. (1984). *Surface water treatment for communities in developing countries.* New York, N.Y., USA, Wiley.

Sevilla, A. (1971). *A study of filtration methods for providing inexpensive potable water to small communities in Asia.* Bangkok, Thailand, Asian Institute of Technology.

Sims, R.C. and Lloyd, A.S. (1991). "Slow sand filtration : present practice in the United States. In: *Task Committee on Slow Sand Filtration.* Slow Sand Filtration. New York, N.Y., USA.

Siripatrachai, T. (1987). *Physical and mathematical analysis of the performance of horizontal roughing filtration.* Delft, The Netherlands, Institute of Hydraulic Engineering, M.Sc. Thesis Report.

Sleazak, L.A. and Sims, R.C. (1984). "The application and effectiveness of slow sand filtration in the United States". *Journal of the American Water Works Association*, vol. 76, no. 12, p. 38-43.

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

Steel, E.W. (1953). Water supply and sewerage. 3rd ed., New York, N.Y., USA, MacGraw-Hill.

Than, N.C. and Ovano, E.A.R. (1977). *Horizontal flow coarse material prefiltration.* (AIT Research report; No. 70). Bangkok, Thailand, Asian Institute of Technology.

Toms, I.P. and Bayley, R.G. (1988). "Slow sand filtration: an approach to practical issues". In: Graham, N.J.D. (ed.). *Slow sand filtration: recent developments in water treatment technology*. Chichester, U.K. Ellis Horwood. p. 11-28.

Turneaure, F.E. and Russel, H.L. (1940). Public water supplies, 4th ed., London, U.K., John Willey.

Visscher, J.T. and Galvis, G. 1992. *Slow Sand Filtration an Essential Component in Many Multi-Barrier Water Treatment Systems.* Paper presented at a workshop in Zurich, Switzerland, June 1992. IRCWD workshop. Zurich, Switzerland. June 1992.

Visscher, J.T. and Galvis, G.C. (1987). "Participación y perspectivas de la Universidad del Valle en el desarrollo del proyecto de filtración lenta en arena (FLA)". In: *ACODAL.* Seminario internacional sobre tecnología simplificada para potabilización del agua, agosto 17 al 21 de 1987: memorias. Cali, Colombia, ACODAL.

Visscher, J.T.; Galvis, G. and Smet, J. (1991) Slow Sand Filter Design and Construction Learning from Experience. *Paper presented in the AWWA Conference*, New Hampshire, USA. November 1991.

Visscher, J.T.; Paramasivam, R.; Raman, A. and Heijnen, H.A. (1987). *Slow sand filtration for community water supply: planning, design, construction, operation and maintenance.* (Technical Paper; No. 24). The Hague, The Netherlands, International Reference Centre for Community Water Supply and Sanitation.

Wegelin, M. (1986). *Horizontal flow roughing filtration (HRF): a design, construction and operation manual.* (Report; No. 06/86). Duebendorf, Switzerland, IRCWD.

Wegelin, M. and Mbwette, T.S.A. (1989). "Horizontal flow roughing filtration". In: Smet, J. and Visscher, J.T. (eds). *Pre-treatment methods for community water supply: an overview of techniques and present experiences.* The Hague, The Netherlands, International Reference Centre for Community Water Supply and Sanitation, p. 127-162.

Wegelin, M.; Schertenleib, R. and Boller M. (1991). "The decade of roughing filters: development of rural water treatment process for developing countries". In: *Aqua*, vol. 40, no. 5, P. 304-316.

WHO (1987). *Technology for water supply and sanitation in developing countries: report of a WHO study group.* (Technical Report Series; No. 742). Geneva, Switzerland, World Health Organization.

Williams, P.G. (1984). "First report on slow sand filter investigation". In: *Water Sewage and Effluent*, 20, 33-39.

Windle, T.E. (1974). *45th Report on results on bacteriological, chemical, and biological examination of London water for the years 1971-73.* London, U.K., Metropolitan Water Board.

Wolters, H.; Smet, J.E. and Galvis G. (1989). "Up-flow roughing filtration". In: *Pre-treatment methods for community water supply : an overview of techniques and present experience.* The Hague, The Netherlands, International Reference Centre for Community Water Supply and Sanitation.

Wolters, H. (1988). *Roughing filtration : a literature study.* Delft, The Netherlands, Delft Technical University. MSc thesis report.

WRC (1977). *Water purification in the EEC.* Water Research Centre State of the Art Review. New York, N.Y. USA, Pergamon Press.

Yao, Kuan M. (1975). "Extended plain sedimentation". In: *Journal of the Environmental Engineering Division ASCE*, vol. 101, no. EE3. Proc. Paper 11392. June. p. 413-423.

Yao, K.M.; Habibian, M.T. and O'Melia, C.R. (1971). "Water and wastewater filtration : concepts and applications". In: *Environmental science and Technology.* vol. 5, no. 11.

Annexes

I. Control devices for operation and maintenance

I.1. Flow regulation devices

Gate valves

The first gate valves CINARA has used comprised of a metal plate protected with an anticorrosive paint and operated with a wheel as shown in figure A1-1,a. This simple system however required regular repainting and does not allow small modifications in the flow rate. Therefore, a different type of gate valve has been established (figure A1-1,b) which is more sensitive to the variation in flow. Still some quality problems exist with this valve and although it can be produced quite cheaply, its cost is not always low as it is not yet available in the commercial market.

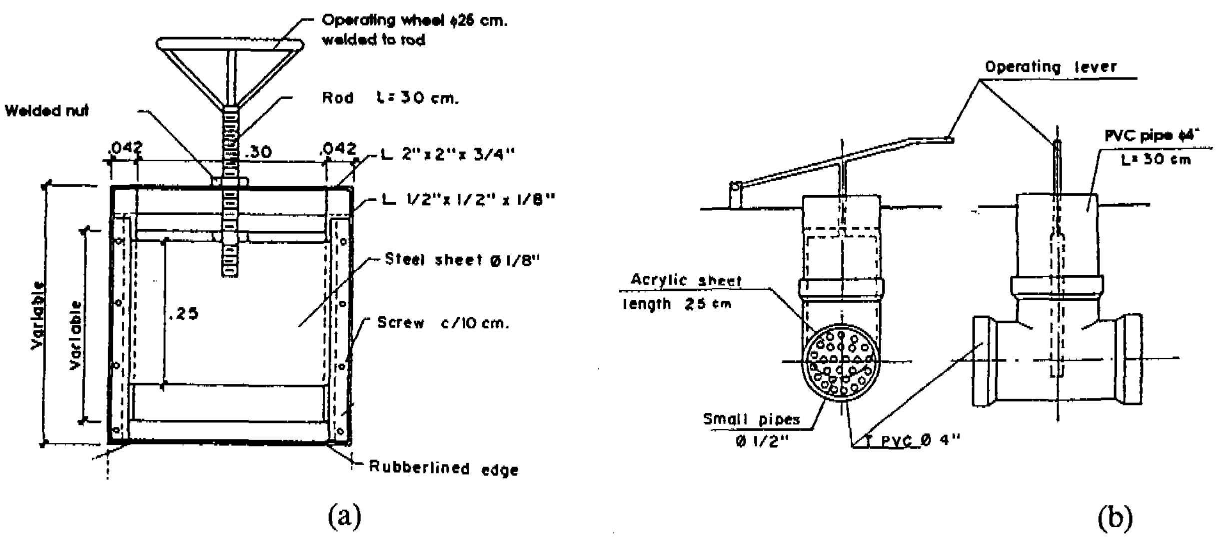


Figure A1-1 Gate Valves (a) in open channel (b) in pipeline

Weirs

The flow over a weir in an open channel is related to the depth of the water above the crest of the weir. This makes it possible to use a weir as a flow measurement device. Two types of weirs are most commonly used, those with a rectangular opening and those with a triangular opening. See figure A1-2.

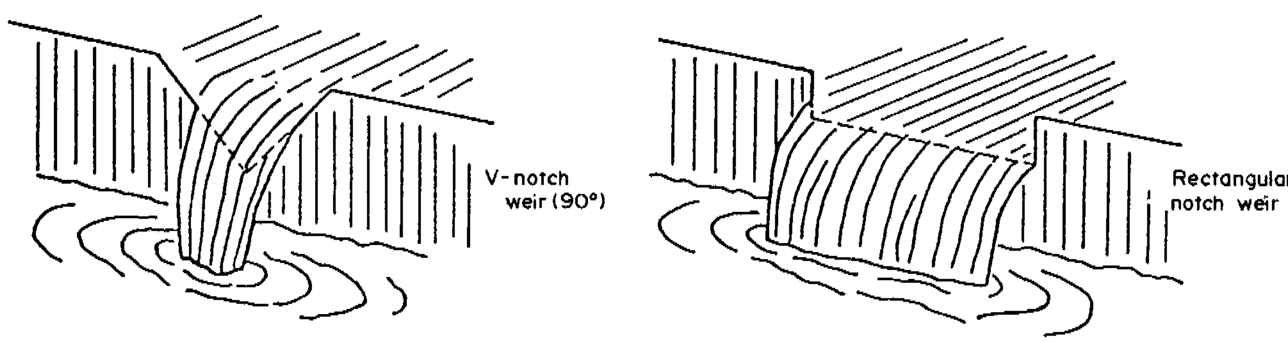


Figure A1-2 Different types of weirs

The triangular weir is mostly used for smaller flow discharges. The weir usually consists of a small metal or acrylic plate which is being installed and calibrated at the end of the construction process. It has to be installed in such a way that the water surface is smooth and not disturbed by turbulence. Furthermore, in measuring the height of the overflow to calculate the volume of water which is passing through the weir, the water depth needs to be measured somewhat upstream of the weir.

Flow measurement

Flow measurement can be facilitated by installing a flow indicator upstream of the weir. The flow indicator developed by CINARA is based on the principle of communicating vessels. A tube with a U bend is installed next to the flow channel and is directly connected to this channel. In this tube a float is installed to which a flow indicator is attached. The pointer of the indicator will now show the height of the water over the weir on a calibrated scale indicating the rate of filtration.

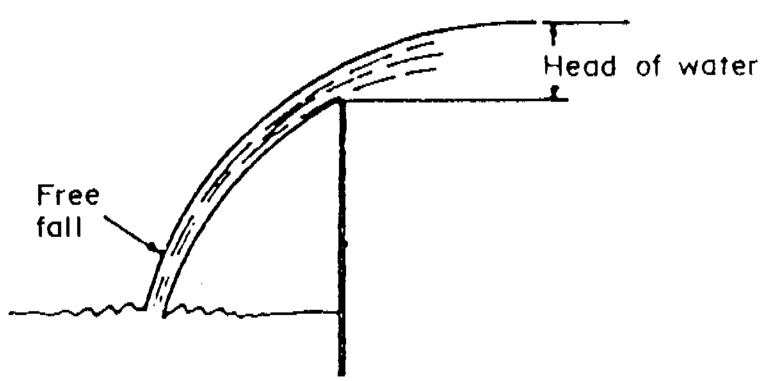


Figure A1-3 Measuring the height over the weir

To facilitate the use of this system CINARA has adjusted the calibrated scale by using different colors (green, orange and red) which give a very clear visual indication of the flow ranges which can be permitted (Figure A1-4). The green zone indicates the conditions under which the plant is operating at the design rate. The orange marks the zones in which the filter is operating at a higher or lower but still acceptable rate, usually 20% above and 50% below the design flow. The latter serves as the zone where the filter can be operated when peak loads in suspended solids are apparent in the raw water. The red color is used for the zones which indicate that the filter is either operated at an extremely high or low velocity.

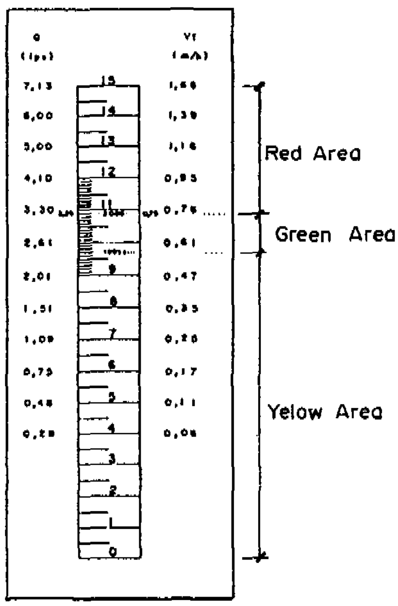


Figure A1-4 Measuring scale

I.2. Maintenance facilitating devices

Fast drainage valve

The fast drainage valves are connected to the drainage system. As a result of the rapid opening and closing of the valves, shock waves are produced which assist in removing the material accumulated on the gravel which would otherwise only be partly removed by draining the unit (Wolters, 1988).

In Figure A1-5 two designs for this type of valves are being presented, one based on experience in Peru (a) and the other (b) developed in Colombia using a model which is also applied for milk cans.

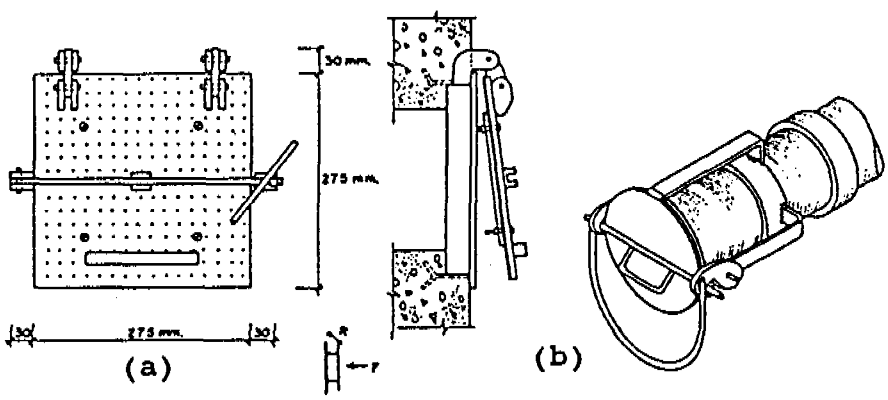


Figure A1-5 Fast drainage valves (a) peruvian model (b) colombian model

Plastic wire mash

To facilitate maintenance it is important to install plastic wire mash between different layers of gravel. This facilitates the work of the operator as it indicates the level to which the material can be moved about with a spade and avoids the mixing of gravel of different sizes.

II. Design of manifolds

II.1 Design criteria

Manifolds consist of a main pipe and lateral pipes which usually are placed on regular intervals. The layout should ensure an equal distribution of flow to establish the best possible hydraulic behaviour of the tank or reactor in which the manifold is being installed. Manifolds are hydraulic structures of considerable importance which are often used in water and waste water treatment plants, but also in many industrial processes.

Two principal systems exist: **dividing-flow manifolds** which are used to distribute a liquid in a filter medium or tank and **combining-flow manifolds** to abstract a liquid from a filter medium or tank, which ensures a uniform abstraction. Some manifolds are being designed to comply with both functions as is the case of drainage systems in rapid sand filters. Because of the lack of a straightforward calculation method, manifold design is often neglected. As a result many treatment plants show poor hydraulic performance and are having low efficiency levels.

A number of preliminary design criteria have been established to facilitate the design of manifolds in roughing filters (Table A2.1). These criteria are based on recommendations of Fair, Geyer and Okun (Limusa-Wiley, 1971), studies by Hudson and collegues (Hudson et all 1979), work by Mr. Galvis and Mr. Castilla and experience in Colombia. They are supported by a theoretical calculation which is presented in section 2.2 and assume a reasonable uniform flow with less then 20% in flow between the first and the last opening. Although further verification is still needed and planned in the pre-treatment project, systems designed on the basis of the preliminary design criteria are already in operation for several years and show good results.

Table A2-1 Summary of design criteria for manifolds in roughing filters

Item	Combining-flow	Dividing-flow
$R_o = \frac{\text{surface area openings}}{\text{surface area gravel}}$	0.001 - 0.005	0.001 - 0.005
$R_l = \frac{\text{surface area openings}}{\text{surface area lateral}}$	0.3 - 0.5	0.4 - 1.0
$R_p = \frac{\text{surface area laterals}}{\text{surface area main}}$	0.3 - 0.5	0.4 - 1.0
Diameter of openings	6 - 19 mm	6 - 19 mm
Space between openings	0.1 - 0.3 m	0.1 - 0.3 m
Space between laterals	0.5 - 1.0 m	0.5 - 1.0 m
Flow in openings	4 - 5 m/s	4 - 5 m/s

Adapted from Fair, Geyer and Okun, 1971

In an upflow roughing filter, a manifold system is used which combines both functions. The system acts as a dividing-flow manifold under normal operation conditions and as a combining-flow manifold when being drained for cleaning. Under normal operation conditions flow velocities are low (0.6 m/h) and thus manifold design is not critical as also the gravel bed very much ensures uniform distribution of flow. When the filter is being drained flow velocities are considerably higher (20 m/h) and therefore this situation presents the main conditions for designing the manifold.

II.2 Theoretical calculation of manifold hydraulics

In the "Proceedings of the American Society of Civil Engineers" (Hudson et al 1979) a study was mentioned of two engineers Mr. H. Uhler and R. Bailey who on the basis of experiments of other researchers (Messrs. Mcnown, Hartigan, Lansford, Miller, Thomas and Vennard) developed equations which permit to calculate the headloss coefficients in the lateral pipes. These coefficients can be calculated for both types of manifolds. In this section the equations obtained by Hudson are being used to establish a relation between the surface area of the openings and the surface area of the lateral, and the surface area of the laterals and the surface area of the manifold main.

Dividing-flow manifolds

The following symbol definitions are being used (Figure A2-1)

Q	=	Total flow at the entrance of the manifold
V_i	=	Flow velocity in cross section i of the manifold main, immediately after lateral i.
Q_i	=	Flow through section i with flow velocity V_i.
v_i	=	Flow velocity in lateral i.
q_i	=	Flow in lateral i with flow velocity v_i.
A_i	=	Cross section area of manifold main in section i.
a_i	=	Cross section area of lateral i.
$h'f$	=	Lateral entry loss over the port entrance
H_i	=	Total headloss over a lateral i, including the headloss over the entrance and outlet.
g	=	Acceleration of gravity
α	=	Flow velocity coefficient to express the headloss per opening and function of the flow velocity in the lateral, as expressed in equation (2).

ß = Term used to include all headloss components

Considering the experimental results the following assumptions are being made:

- The surface area in the manifold main remains constant before and after a branch section.
- The laterals are connected under a 90 degree angle with the manifold main.
- The lateral pipes are circular and are smoothly connected to the manifold main without extending into its interior.

On the basis of the preceding information and Figure A2-1 the following equations can be derived:

$$H_i = h'f_i + 1.0 \frac{(v_i)^2}{2g} \tag{1}$$

$$h'f_i = \frac{\alpha (v_i)^2}{2g} \tag{2}$$

Hudson et al, using the experimental results, established a relation between α and $(V_i/v_i)^2$

$$\alpha = \phi\left(\frac{V_i}{v_i}\right)^2 + \phi_i \tag{3}$$

The values of ϕ and ϕ_i are different for short and long laterals (Figure A2-2). A short lateral is defined as a lateral which length is smaller than three times its diameter. The values found by Hudson for ϕ and ϕ_i are indicated in table A2-2.

Table A2-2 Values of ϕ and ϕ_i

LATERAL	ϕ_i	ϕ
Long	0.4	0.90
Short	0.7	1.67

The coefficients for the long lateral are smaller than those for the short ones probably because the long lateral permits a partial recovery of the velocity head in the contracted lateral jet.

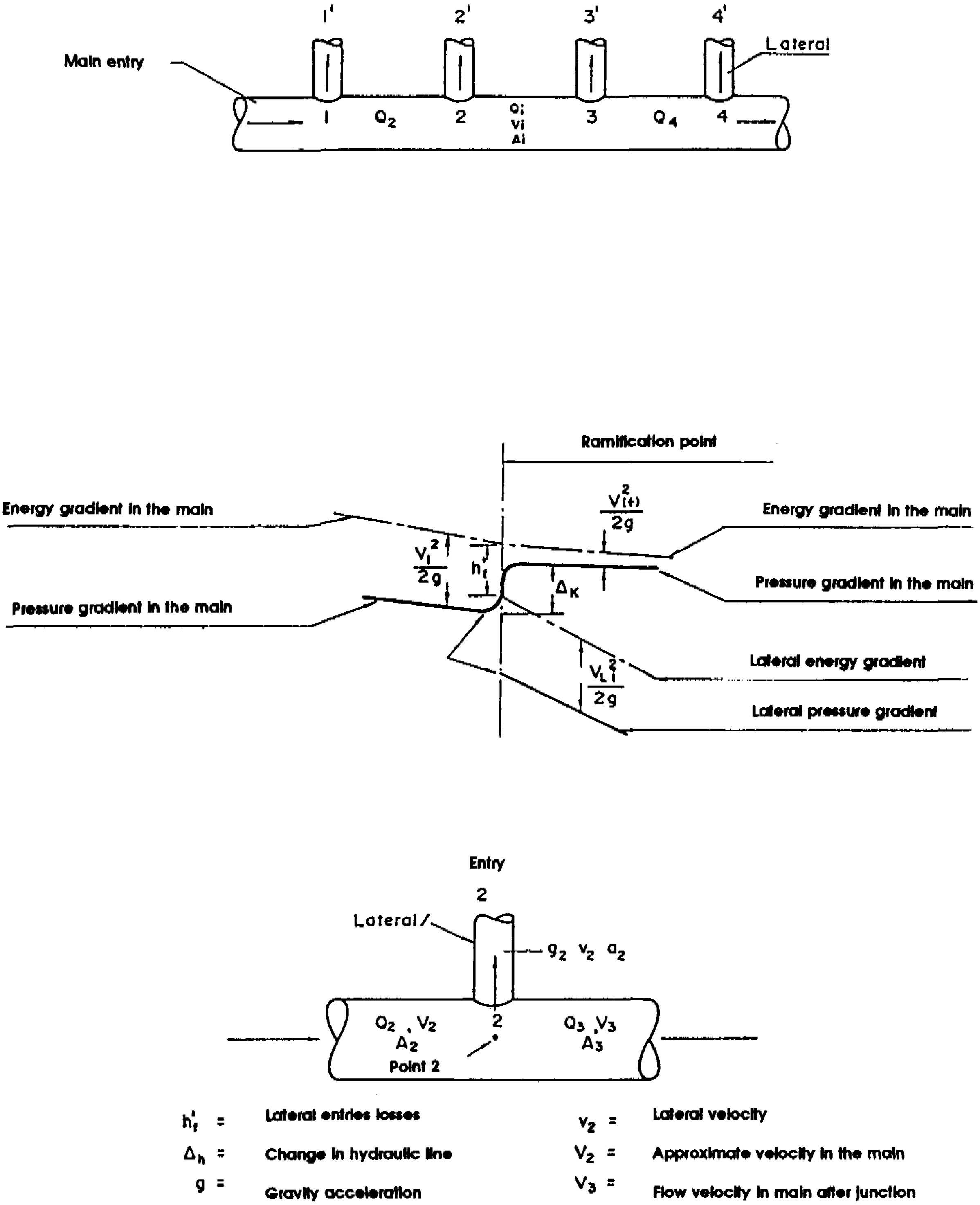


Figure A2-1 Conditions in a diffusor manifold

According to Hudson the loss at the outlet can be indicated as follows:

$$ß = \phi \left(\frac{V_i}{v_i}\right)^2 + \phi_i + 1.0 \qquad (4)$$

Using R as the relation a/A (surface area of one orifice divided by surface area of the lateral or surface area of one lateral divided by surface area of the manifold main) and assuming that the laterals are short, then:

$$ß_i = 1.67 \left(\frac{R\, Q_i}{q_i}\right)^2 + 1.7$$

It is interesting to analyze the coefficient ß and the pressure along the length of the manifold main as explained in the next sections.

- *Coefficient ß*

The ideal flow through an orifice which discharges in a lateral, arises when the water above the opening is not moving. In a dividing-flow manifold the ultimate lateral is in this condition. At the entrance of the manifold main the full flow is being transported which results in a maximum flow velocity in this zone, if the diameter is uniform over the full length. At the end of the manifold, most of the flow will already have been distributed through the laterals and therefore the flow velocity is almost zero. The lower the flow velocity in the manifold the less energy is needed to change the flow direction towards the lateral. The last lateral will have the most advantage situation in this respect. For this reason the coefficient ß will gradually reduce towards the last laterals and these therefore, will have the tendency to be able to take a larger part of the flow. It is obvious that this coefficient is very important for the distribution of the flow over the manifold.

- *Pressure over the length of the manifold main*

If the headloss through friction and reduction in flow velocity would be less than the recovery of the head along the length of the manifold main due to the deceleration of the contracted flow, the pressure would gradually increase towards the end of the manifold. Under this conditions the discharge of the last laterals would be higher than the initial laterals. Generally, however, the headloss due to friction is larger than the headloss recovery over the length of the pipe and therefore the discharge tends to reduce towards the last laterals. Thus the increasing headloss over the length of the manifold main has a compensating effect on the effect of the coefficient ß, in a dividing-flow manifold. In summary, the discharge to the laterals is the result of the combined effect of headloss recovery, the coefficient ß and the headloss through friction and changes in direction of flow over the length of the manifold main. The importance of each of these factors depends on the conditions of the individual case.

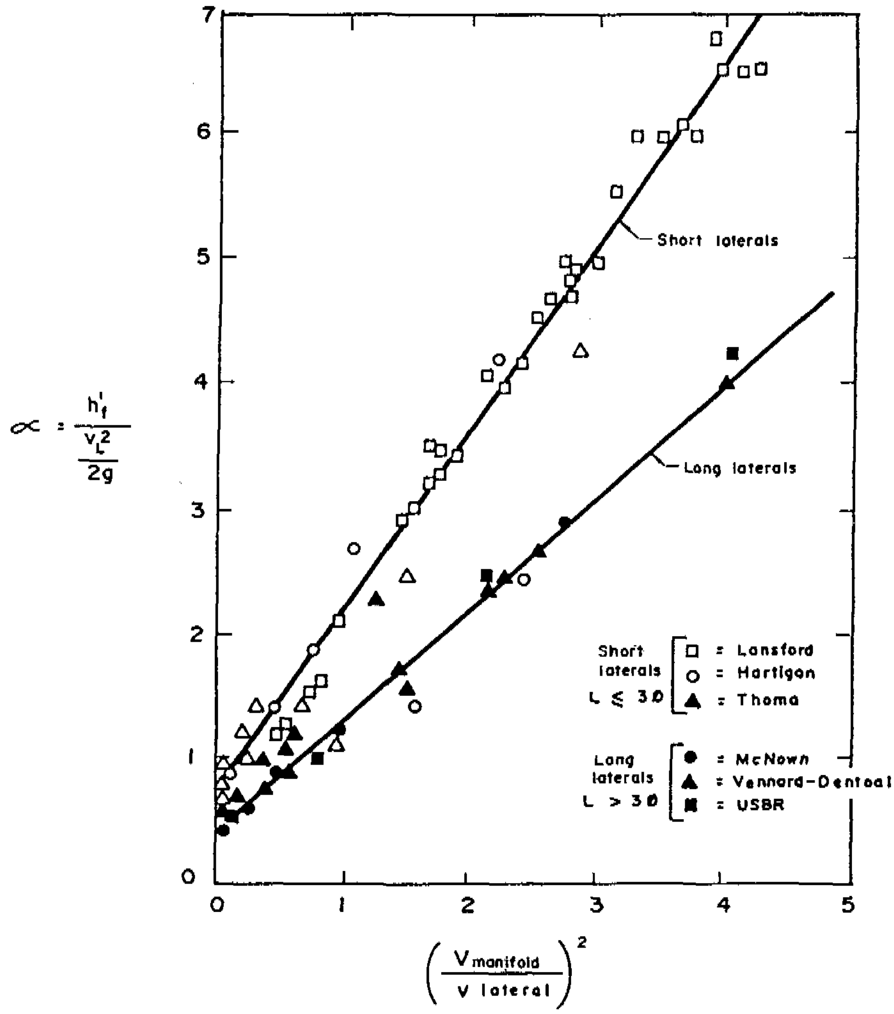


Figure A2-2 Port entry headloss coefficient as a function of the ratio of manifold main velocity to lateral velocity

Often the headloss in the manifold main can be neglected particularly in a new system. Under this conditions it is possible to estimate directly the relation between the surface area of the lateral and the manifold main to produce a uniform distribution with a given variation.

If a 10% variation is selected the following can be established:

- The last lateral ß can be taken as 1.7 as V/v = 0 (Short lateral).
- The headloss in the last lateral n equals:

$$H_n = \beta_n \frac{\left(\frac{q_n}{a}\right)^2}{2g} = 1.7 \frac{\left(\frac{q_n}{a}\right)^2}{2g}$$

- Under the given condition the headloss in all laterals are equal and therefore:

$$H_n = \beta_1 \frac{\left(\frac{q_n}{1.20 * a}\right)^2}{2g} = 1.7 \frac{\left(\frac{q_n}{a}\right)^2}{2g}$$

$$\beta_1 = 1.7 * 1.20^2 = 2.448$$

- For β_1 with R as the relation between the surface area of the lateral and the manifold main:

$$\beta_1 = 1.67 \quad \left(R \frac{Q}{q_1}\right)^2 + 1.7 \tag{5}$$

- Furthermore:

$$\frac{q_1 + q_n}{2} = \frac{Q}{n}$$

and as q_n has to be 1.1 q_1 because of the acceptance of 10% variation:

$$q_1 = \frac{0.909\ Q}{n} \tag{6}$$

- Inserting in (6) in (5) the following is obtained:

$$\beta_1 = 1.67 \left(R \frac{Q_n}{0.909\ Q}\right)^2 + 1.7 = 2.448 \tag{7}$$

which gives: $R = \frac{0.61}{n}$

However, in dividing-flow manifolds the headloss in the manifold main has an important compensating effect. It is reasonable to apply $R = 0.8/n$ as confirmed by experience.

Collecting-flow manifold

The collecting-flow manifold is the opposite of the dividing flow manifold, with the flow in the beginning of the manifold main being smaller than at the end. The headloss over the entrance equals 0.4 to 0.5 times the headloss through the flow velocity in the lateral and is not included in the headloss h'f in the equation (8) as will be shown further on. The headloss h'f is the headloss between the entry point of the lateral in the manifold main and a point somewhat further down the manifold main directly after the lateral. It is a result of the change in velocity between those points and the change in flow direction. (Figure A2-3). In Figure A2-4 the relation is presented between α and V/v.

$$\alpha = \frac{h'fi}{\frac{(v_i)^2}{2g}} = 1 - 0.7 \left(\frac{V_i}{v_i}\right)^{0.5} \qquad (8)$$

Relation between the areas with uniform distribution and friction

The following equation is being derived from Hudson:

$$\beta_1 = 1.5 - 0.7 \left(\frac{V_i}{v_i}\right)^{0.5} \qquad (9)$$

For the lateral farest away from the outlet, the coefficient Beta is as follows:

$$\beta_1 = 1.5$$

$$H_n = \frac{1.5 \left(\frac{q_n}{a}\right)^2}{2g} \qquad (10)$$

$$H_1 = \frac{\beta_1 \left(\frac{q_1}{a}\right)^2}{2g} \qquad (11)$$

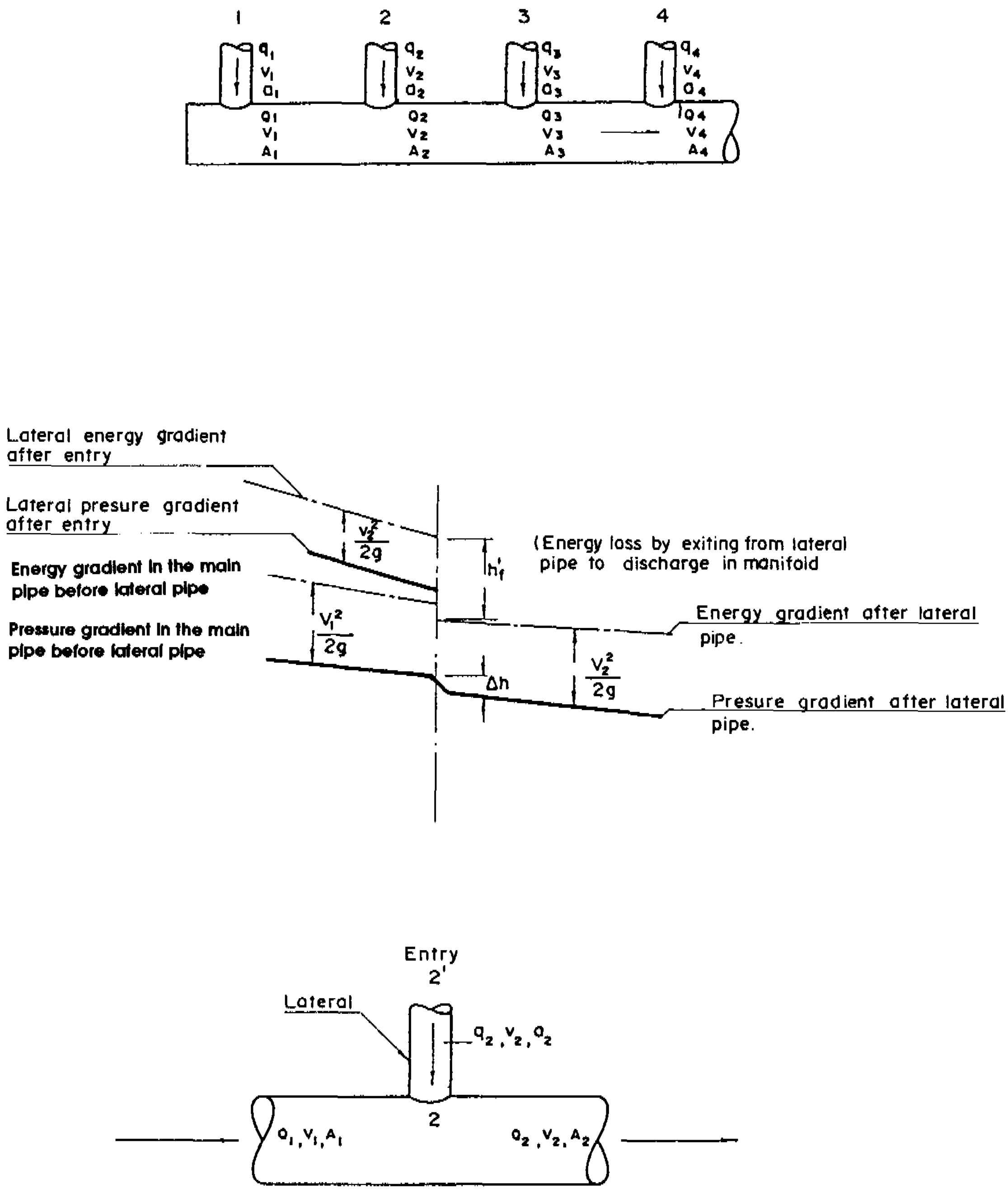


Figure A2-3 Conditions in a combining-flow manifold

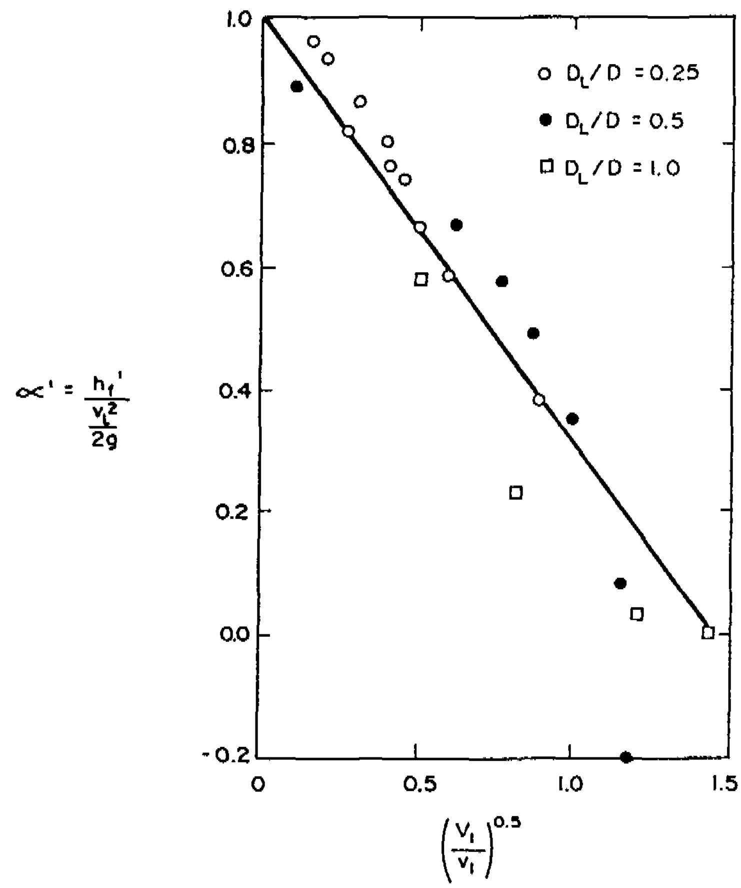


Figure A2-4 Coefficient of the headloss at the outlet of the laterals discharging in manifold main

Accepting a 20% difference in flow distribution between the first and the last orifice implies that:

$q_1 = 1.2\ q_n$, and neglecting friction loss ($H_1 = H_n$)

the following is obtained

$$1.5\ q_n^2 = \beta_1\ q_1^2 = \beta_1\ (1.2\ q_n)^2$$

$$\beta_1 = \frac{1.5}{(1.2)^2} = 1.042$$

Furthermore we can state:

$$\frac{q_1 + q_n}{2} = \frac{Q}{n} = \frac{1.2\, q_n + q_n}{2} = \frac{2.2\, q_n}{2}$$

$$q_n = 0.909 \frac{Q}{n} \tag{12}$$

$$q_1 = 1.2 * 0.909 \frac{Q}{n} = 1.091 \frac{Q}{n} \tag{13}$$

Inserting these values in the expression (9) we find:

$$\beta_1 = 1.042 = 1.5 - 0.7 \left(\frac{R\, Q_1\, n}{1.091\, Q_1}\right)^{0.5} \tag{14}$$

and so:

$$R = \frac{(1.5 - 1.042)^2\ (1.091)}{(0.7)^2\, n}$$

$$R = \frac{0.47}{n}$$

II.3 Example

In this example the manifold system is designed for an upflow roughing filter of 2.0 x 2.7 m. The critical situation will arise when the filter is being drained. Flow velocity then may be estimated at 20 m/h. The total output of the system thus will be:

$$Q = V \times A = \frac{20 \times 2.0 \times 2.7}{3.6} = 30.0 \text{ l/s}$$

It is assumed that 4 laterales are being installed at a distance of 0.7 m. The flow per lateral then becomes:

$$q_L = \frac{Q}{4} = 7.5 \text{ l/s}$$

For an opening size of 0.95 cm and accepting a flow velocity of 5 m/s the discharge per opening is:

$$q_o = v_o \times A_o = 5 \times 0.785\ (0.95 \times 10^{-2})^2 = 3.55 * 10^{-4} \text{ m}^3\text{/s}$$

the number of openings thus becomes:

$$n = \frac{q_l}{q_o} = \frac{7.5 \times 10^{-3}}{3.55 \times 10^{-4}} = 21$$

For a length of 2.0 m and considering two lines of openings, the spacing between openings becomes 18 cm. The ratio between the combined surface of the openings and the surface area of the filter becomes 0.0011 which is acceptable according to table A2-3. The diameter of the lateral can be calculated with:

$$R = \frac{0.5}{n} = \frac{A_o}{A_l}$$

$$A_l = 2n\ A_o$$

$$\tfrac{1}{4}\pi\, d_l^2 = \tfrac{1}{4}\pi\ .2n\ .\ d_o^2$$

$$d_l = \sqrt{2n}\ d_o = \sqrt{42}\ .\ 0.95 = 6.1 \text{ cm} \approx 2.5''$$

The diameter of the lateral thus can be taken as 2.5".

If 2.5" is not readily available a pipe of 3" can be taken as well. The diameter of the manifold main can now be calculated assuming:

$$R = \frac{0.5}{n}$$

this gives

$$d_m = \sqrt{2n}\ d_l$$

With two laterals discharging in each section of the manifold main the diameter of the manifold main thus becomes:

$$d_m = \sqrt{2.2} \times 2.5"$$

$$= 2 \times 2.5"$$

$$= 5"$$

If 5" is not readily available in the market a pipe diameter of 6" can be taken as well.

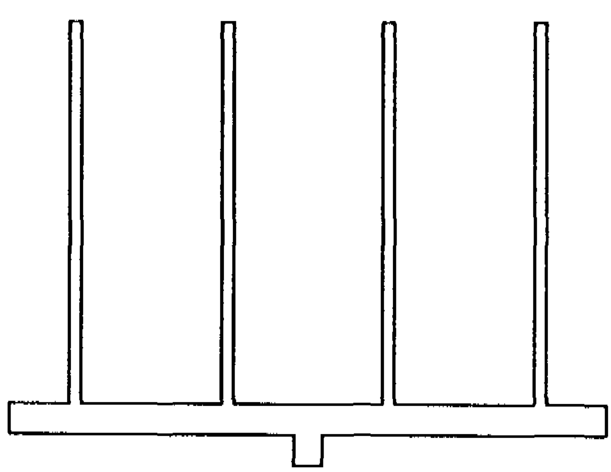


Figure A2-5 Schematic design of manifold

II.4 References

French, J.A. (1980). "Discussion to dividing-flow manifolds with square-edged laterals", *Journal of the Environmental Engineering Division, ASCE.*

Hudson, H.E. (1981). "Water Clarification Processes. Practical design and evaluation". *Van Nostrand Reinhold environmental engineering series.*

Hudson, H.E.; Umler, R. and Bailey, (1945). "Dividing flow manifold with square edged laterals". *Journal of the Environmental Engineering Division, ASCE,* August.

McNown. J.S. (1954). "Mechanics of manifold flow". Trans. *ASCE,* 119 (Paper 2714): 1103.

Rawn, A.M.; Boweman, F.S. and Broores, N.H. (1961). "Difussers for disposal of sewage in sea water". Trans. of the *American Society of Civil Engineers,* Vol. 125. part III, No. 344 m.

Vennard, J.K. and Dentoni, D.K. (1954). "Discussion to Mechanics on Manifold Flow". Trans. *ASCE,* 119 (paper 2714): 1136.

III. Integrated research and demonstration programme on pre-treatment methods for drinking water supply

III.1. Summary and conclusions

This annex presents a summary of the results of the first phase of a comparative study of different multi-stage water treatment systems. Each of these systems consisted of a combination of two stage roughing filtration and slow sand filtration. This research has been carried out in Colombia in the period 1989 - 1992, in the context of the development and demonstration project on pre-treatment technology. The project was implemented by CINARA, Centro Inter-Regional de Abastecimiento y Remoción de Agua and IRC, International Water and Sanitation Centre, with financial support from the Research and Development Department of the Netherlands Ministry of Foreign Affairs. Inputs were also provided by Colombian institutions including: the National Planning Department, the Ministry of Health, Universidad del Valle, Empresas Municipal de Cali EMCALI, Comité Cafeteros and ACODAL and by international organizations including: WHO, PAHO, CEPIS, IDRC, IRCWD, Robens Institute, University of Surrey, the School of Engineering of Sao Carlos and the University of Sao Paolo.

After a gradual start, in which a research station was established in Puerto Mallarino, Cali, Colombia, the project has developed very well and has been able to demonstrate the vast potential of the combination of two-stage roughing filtration and slow sand filtration. It has proven the feasibility of applying a multi-barrier concept towards water treatment without using chemical coagulants leaving low dose disinfection as a possible final safety barrier. Even if this last barrier fails, still a water low in sanitary risk is being produced. This is very important as chemical water treatment which was sofar the only alternative for highly polluted surface waters, has a poor performance record outside the larger cities and a limited potential particularly for smaller townships. In Colombia, for example, over 40% of the population lives in communities with a population below 12,000. Many of these have to rely on polluted surface water sources. The present coverage with water supply systems of these communities is estimated at 24%. Out of these 30% include some and only 4% include full treatment, which unfortunately seems not very reliable in many cases.

The project results show that this situation can be changed. Although further technical and economical optimization is required, the combination of two-stage roughing filtration and slow sand filtration is a very reliable and effective water treatment which can be managed even by small communities. These findings bring reliable water treatment within reach of many people now having to rely on deteriorating surface water sources. The results also show the importance of the concept of integrated water treatment, which promotes the selection of treatment processes which are complementary (Lloyd et al., 1991). The combination of pretreatment and slow sand filtration, concerns treatment processes with different possibilities and limitations (Figure A3-1). These processes are complementary and together sufficiently remove the most important contaminants from the surface water. Applying the combination of integrated water treatment and the multi-barrier concept is very promising. It not only increases the reliability of water treatment, but also may considerably reduce cost.

With the potential of pre-treatment technology now fully established indepth research is needed to optimize the technology, improve its design criteria and its operation and maintenance procedures, in order to fully benefit from the potential of this technology at lowest cost.

Another important feature of the project is that it helped to build-up research capacity in Colombia and establish a very good infrastructure for future investigation and technology transfer.

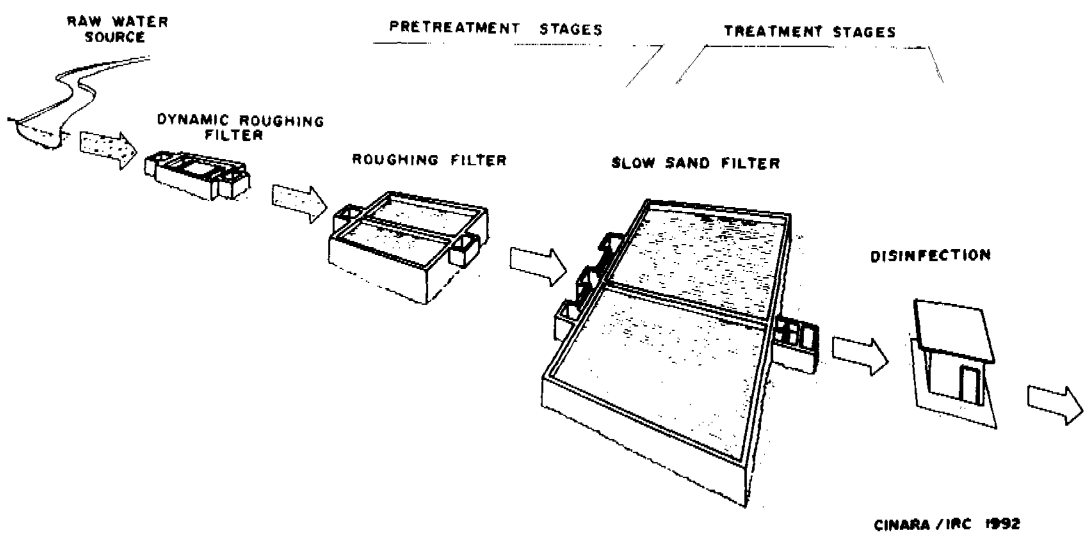


Figure A3-1 Multi-stage filtration water treatment plant

Slow Sand Filtration

The research concerns systems which combine slow sand filtration (SSF) with different two-stage roughing filtration alternatives. Experience in several countries has shown that slow sand filtration is a very appropriate water treatment method, which combines simple and low cost operation and maintenance with high removal efficiencies for a wide range of substances (Visscher et al., 1987). In this process water slowly percolates through an emerged sandbed and during its passage impurities and harmful substances are being removed by a combination of filtration, sedimentation and biological and bio-chemical processes. It has a very good performance efficiency, but this not always prove sufficient to cope with the high load in suspended solids and other harmful substances in the water including faecal coliforms and colour, resulting in an effluent not able to meet prevailing water quality criteria. Also its performance me be reduced by low temperature, low nutrient levels and low oxygen levels. Finally a high load of suspended solids, algae proliferation and excess concentrations of iron and manganese may result in very short filterruns.

Several authors have established guidelines describing the raw water quality which can be treated by slow sand filters. Although the maximum levels for these parameters still show a considerable range (eg. turbidity levels of 5 - 10 NTU, colour of 5 - 25 TCU, iron and manganese content of 0.3 - 2.0 mg/l and 0.05 - 0.2 mg/l respectively), they are quite low. A reason for this is that they are developed for countries such as the USA were drinking water criteria are very stringent. In less developed countries water quality guidelines are more relaxed, and therefore also water with somewhat higher pollution levels can still be accepted for SSF treatment. Nevertheless, the drawback that SSF alone can cope only with relative clean water has limited the application of this technology.

Pre-treatment

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 for the conditions in most less developed countries. This limitation has revitalized research in other pre-treatment alternatives not requiring the addition of chemicals and 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. The experiments reported in literature focus particularly on the removal of suspended solids, some using kaoline suspensions, and the performance of the systems is mostly being explained on the basis of sedimentation theory.

Experiments with small roughing filters in Colombia treating different types of surface water showed that next to suspended solid removal also good reduction in coliform counts and in true colour was obtained (Galvis and Visscher, 1987). These first results and the findings presented in literature triggered the construction of a small number of full scale plants in Valle del Cauca, Colombia, comprising both slow sand filters and pre-treatment units mostly build to very conservative design criteria and treating water from rivers with low or moderate pollution levels.
The positive results obtained in the pilot plants, the need to cope with water quality deterioration due to soil erosion and expansion of human activities and the decision of national and international organizations to promote water supply pressure in the region Valle del Cauca to provide better water quality to the population resulted in the initiation of a comparative research project. This research comprised: i) a comparison at technical scale of five different treatment systems in the research station in Puerto Mallarino each comprising two-stages of roughing filtration, followed by slow sand filtration and ii) performance monitoring of full scale water treatment plants operated and maintained by local caretakers under supervision of community-based organizations.

The first research period

The project went through a first period of seven months, in which the selected systems were put to the test. These systems were build to design criteria resulting from a broad literature review and field experience in Cali. This phase proved to be very important to get a good idea about the performance of the systems and adjust the research programme. It also allowed to gradually establish the research team and create a format for the computerized data base. The results in this period were already quite good and demonstrated the importance of the biological activity in the roughing filters. Nevertheless with average faecal coliform counts between 56 and 213 per 100 ml, the quality of the treated water from the systems under test, still was rather far outside the water quality standards in Colombia. Furthermore, the filterruns in the slow sand filters were rather short, ranging from 8 - 30 days.

Establishment of the subsequent research periods

To overcome the problems indicated above a number of changes were made in the research plant. Dynamic roughing filtration was adopted as the first pre-treatment step as this gave much better results to remove suspended solids than plain sedimentation and tilted plate settling. Essentially, this first step is meant to remove coarse suspended material at low cost, therefore a low retention time of less than one hour was applied.

Furthermore finer filter media was allocated in the subsequent roughing filters and one treatment line was added including a modified horizontal roughing filter with a shorter retention time. Finally the filtration rate was reduced in the second period from 0.6m/h to 0.3 m/h and gradually increased again in period III and IV to the initial rate of 0.6m/h. At present the rate has been even further increased to 0.75m/h still providing good results, showing the potential for further optimization.

Description of the test plant

The systems tested in the subsequent periods thus comprised a dynamic roughing filter, an upflow, a downflow or horizontal flow roughing filter and a slow sand filter as outlined in Figure A3-2.

The dynamic roughing filter (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 passes over the gravel bed and part of it is drained through the bed to the next treatment unit, the other part is returned to the river. The filter works primarily as a surface roughing filter. Under normal conditions the unit will gradually clog and will need a simple cleaning every week. When peak loads of suspended solids are received, clogging will go much quicker and will reduce the water volume flowing to the subsequent treatment units, thus protecting the total treatment plant. This is the more important as most of the rivers in the Andean region have considerable peaks in turbidity but usually of short duration.

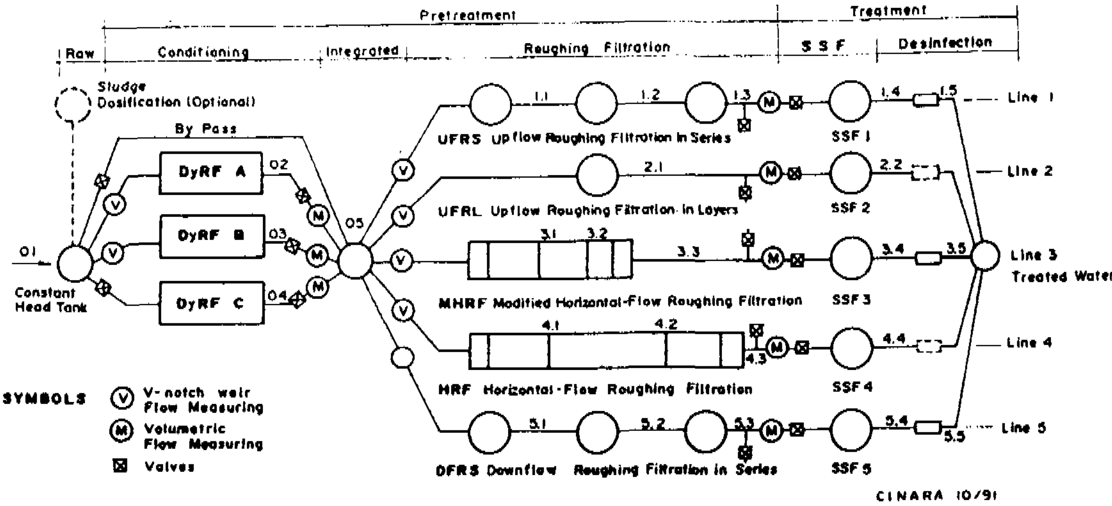


Figure A3-2 Schematic flow diagram of the test plants

Two types of upflow roughing filters are tested: i) Upflow Roughing Filtration in Series (URFS), a system of three units operated in series with the first unit filled with coarse gravel, the second with medium size gravel and the third with fine gravel, and ii) Upflow Roughing Filtration in Layers (URFL), a system comprising only one unit filled with layers of gravel of different size ranging from coarse at the bottom to fine at the top. Upflow roughing filtration, which sofar has had limited application in the world, has the advantage that the first filtration occurs at the bottom of the filter. This facilitates the removal of deposits from the gravel by draining the units through a system of underdrains.

One type of downflow roughing filtration is tested, Downflow Roughing Filtration in Series (DRFS) which consists of three units operated in series, each comprising gravel of different size similar to the upflow filters. This type of system has been researched somewhat more widely in different parts of the world and particularly in Perú.

Two types of horizontal-flow roughing filters were tested, one Standard Horizontal Roughing Filter (HRF) comprising three compartments filled with the same gravel sizes as the upflow and downflow units and build to the design criteria proposed by Wegelin (1986) and one Modified Horizontal Roughing Filter (HRFM) filter which includes a drainage system to facilitate cleaning and comprises shorter compartments to bring the theoretical hydraulic retention time at the same level as of the upflow and downflow systems in series, thus making the systems competitive.

Each line includes at the end a slow sand filter, filled with a sand layer of 1.0 meters depth. The sand has an effective diameter of 0.2 mm and a uniformity coefficient of 1.57. During the research period reported here, this layer was gradually reduced due to subsequent scrapings, but never fell below 0.6 m even in the filter which needed most scraping. To date the research is being continued and with sand levels now even as low as 0.45 m still excellent effluent is being obtained from the slow sand filters, which holds a promise for further cost savings.

All units are made of ferro-cement and are circular in shape with a diameter of 2.0 m and a height of 2.0 m. The horizontal roughing filter is built in brickwork and is 1.2 m high, 1.2 m wide and 8 m long including the inlet and outlet structure. The modified horizontal filter has the same dimensions but a length of 5 m. Summary data on all systems are presented in Table A3-1.

Three six month test periods have been carried out with flow velocities of respectively 0.3, 0.45 and 0.6 m/h in the roughing filters and after the period reported here, this rate has been increased to 0.75 m/h. A wide range of parameters were measured in the research including: turbidity, true colour, suspended solids, settlable solids, pH, dissolved oxygen, alkalinity, total hardness, total iron, manganese, COD, faecal coliforms and faecal streptococci.

Table A3-1 Technical characteristics of the treatment systems in the research station

	DYRF	URFL	URF	and	DRF	HRFM			HRF		
			1	2	3	1	2	3	1	2	3
Surface Area (m2)	0.75/2	3.14	3.14	3.14	3.14	1.57	1.57	1.57	1.57	1.57	1.57
Gravel depth											
19 - 25 mm		0.30	0.30	0.15							
13 - 19 mm	0.20	0.30	1.25	0.15	0.10	2.06			3.27		
6 - 13 mm	0.20	0.30		1.25	0.10		1.14	0.10		2.32	
3 - 6 mm	0.20	0.30			0.67			0.60			1.00
1.6 - 3 mm		0.35			0.68			0.55			0.55
Effective depth	0.60	1.55	1.55	1.55	1.55	2.06	1.14	1.25	3.27	2.32	1.55

Description of full scale plants

In parallel with this research, seven community managed full scale treatment plants comprising roughing filters and slow sand filters have been monitored (Table A3-2). Most of these plants were build using very conservative design criteria because of the lack of data available in the literature, scarce information on raw water quality and limited experience of the CINARA team. Two plants very near to the CINARA office however were built to less conservative design criteria, as they could be monitored and controlled more easily.

Table A3-2 Some key characteristics of the full scale plants all treating water from highland rivers

TREATMENT PLANT	ROUGHING FILTERS					SLOW SAND FILTERS
	FLOW (l/s)	TYPE	LENGTH (m)	SIZE RANGE (mm)	VELOCITY (m/h)	FILTRATION RATE (m/h)
CEYLAN	9.4	URFS	2.08	25-3	0.70	0.14
EL RETIRO	8.8	DyRF	0.3	4-25	1.5	
		URFL	0.7	25-3	0.70	0.15
COLOMBO	0.7	DyRF	0.6	6-25	1.5	
		URFL	1.2	25-4	0.60	0.11
RESTREPO	0.7	HRF	7.0	16-5	0.80	0.15
JAVERIANA	1.5	DyRF	0.6	4-25	0.75	
		HRF	4.0	16-3	1.00	0.08
LA MARINA	7.9	URFS	2.0	25-3	0.90	0.16
C.GORDAS	8.6	DyRF	0.6	7-25	10.20	
	2	URFS	1.8	25-3	0.70	0.16

Quality of the raw water sources

The full scale plants and the research station are located in different places and together cover a range of raw water qualities (Figure A3-3). The full scale plants draw water from highland rivers with low or moderate pollution levels and the research station draws water from a highly polluted lowland river, which receives both water from the highland rivers and untreated sewage from small and large settlements.

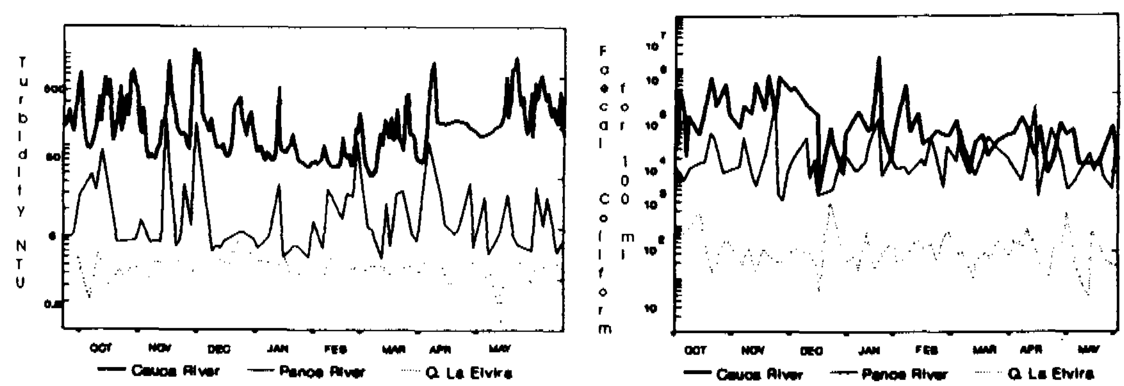


Figure A3-3 Indication of water quality of different rivers included in the research programme

The highland rivers have average turbidity levels ranging from 2.5 to 20 NTU with peaks ranging from 20 to 180 NTU, faecal coliform counts range from 368 to 26,000 with peaks between 2,600 and 228,000. The lowland river has periods with average turbidity levels over 150 NTU, faecal coliform counts over 100,000 per 100 ml and colour over 60 TCU. After rains the water quality deteriorates further, developing peak turbidity lasting several hours with values sometimes over 3000 NTU. When applying the classification of microbiological quality proposed to WHO and UNEP (Lloyd et. al. 1991) all but one of the raw water sources included in the research fall in the worst category (E), which implies a very high sanitary risk.

The raw water quality is influenced by the climate, in which rainy periods and dry periods exist (Figure A3-4). Furthermore steep changes in water quality are being observed, but usually of short duration (Figure A3-5). The temperature of the water is rather constant with averages ranging from 18 to 23 °C with standard deviations around 3 °C.

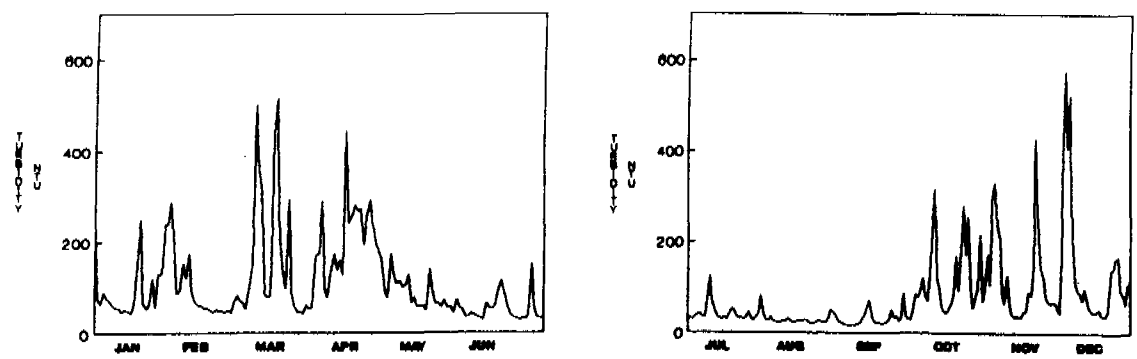


Figure A3-4 Raw water quality of Cauca river, period (January-December 1990)

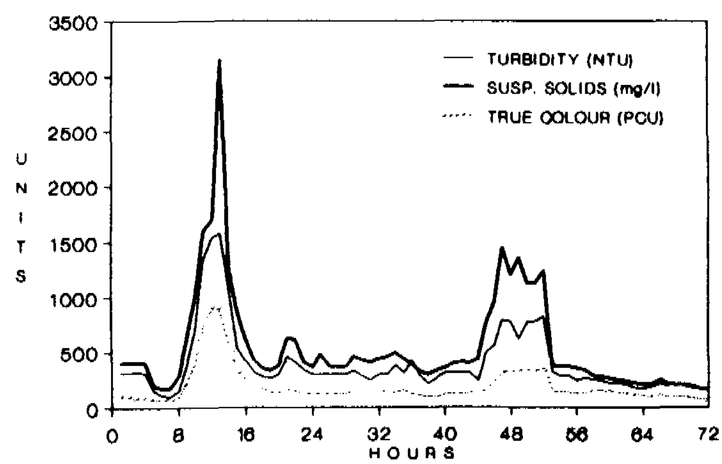


Figure A3-5 Abrupt water quality changes in Cauca river

Overall performance of the treatment combinations

The overall performance of the treatment lines which are being compared in the research station in Puerto Mallerino has been even better than expected. This not only concerns the removal of suspended solids, but particularly the bacteriological quality improvement and the removal of other impurities (Figures A3-6 and A3-7).

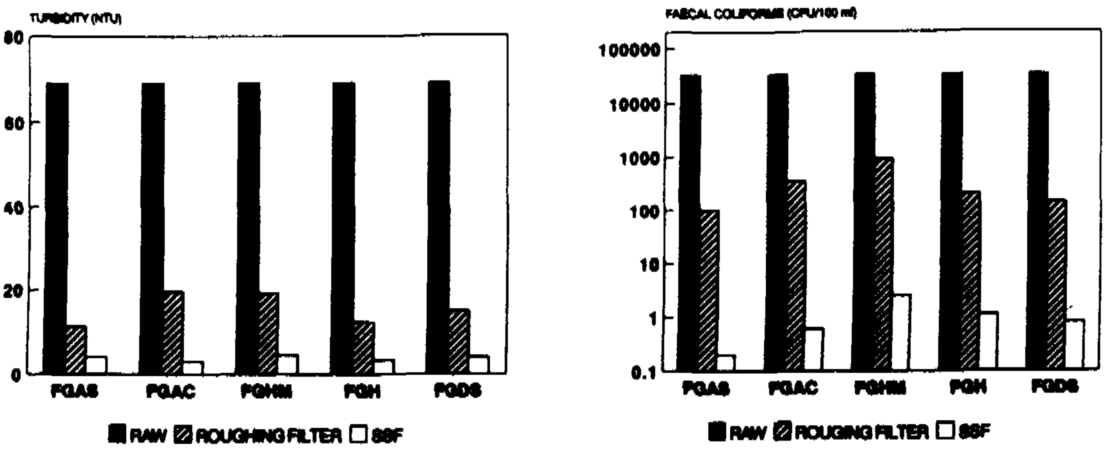


Figure A3-6 Average effluent turbidities and faecal coliform counts of the different treatment units (Jan.-July 1991)

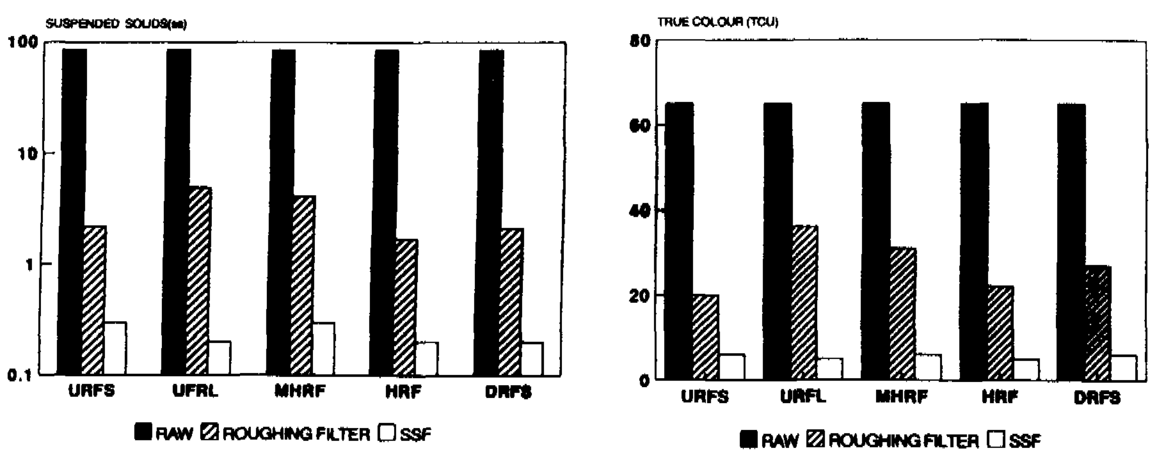


Figure A3-7 Suspended solids and average colour levels in the effluent of the different treatment units (Jan. - July 1991)

The combination of two-stage roughing filtration and slow sand filtration for example proved capable of consistently bringing about a reduction in faecal coliforms between 4.0 and 5.6 logaritmic units (99.9911 - 99,9980%) , which is well above the figures indicated in the literature. But also good removal was obtained for other parameters including suspended solids (99.7 - 99.9%), turbidity (94.4 - 97.5%) and true colour (77.1 - 93.0%).

The systems also proved to be able to cope with short steep changes in water quality, using turbidity as an indicator. Peaks of longer duration were also very much reduced, but did result in a gradual deterioration of the effluent. This point needs further research to explore what adjustments can be made to the units in case longer periods of high turbidity or other impurities have to be coped with.

Monitoring of full-scale plants

Monitoring of seven community-managed full-scale plants confirmed the research findings obtained in Puerto Mallarino. With average inflow turbidities ranging from 3 to 20 NTU average effluent turbidities varied from 0.4 to 1.0 NTU. With inflow peaks over 180 NTU, the outflow remained consistently below 2 NTU. This is better than the data reported by Slezak et al. (1984), on 27 SSF plants in the USA, treating water with average turbidities of 0.4 - 10 NTU producing effluent values between 0.08 and 2.5 NTU.

Average turbidity removal efficiencies in the seven plants ranged from 80 to 96%. Similar findings were obtained for average removal efficiencies for suspended solids (93 to 99% with average value in effluent below 0.3 mg/l, true colour (60 to 87%, with average in the effluent below 6 TCU) and faecal coliforms (99.788 to 99.9985% with average effluent value below 1 FC/100ml). Figure A3-8 shows that the treatment processes adapted to the concentration of impurities in the raw water. Better removal efficiencies were obtained for higher contamination levels. In Ceylan the average removal of faecal coliform bacteria is 2.8 log units. In the Colombo plant, which receives more contaminated water, this is 4.8 log units. The data obtained indicate that all these plants which are being operated by caretakers from the community, consistently produce water within the water quality standards prevailing in Colombia, except for the occasional occurrence of a very low number of faecal coliforms in the effluent.

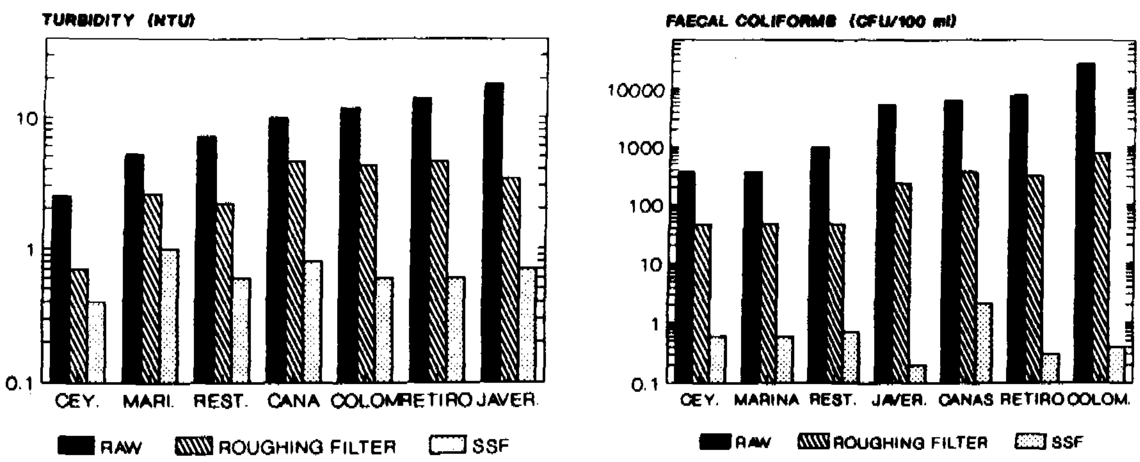
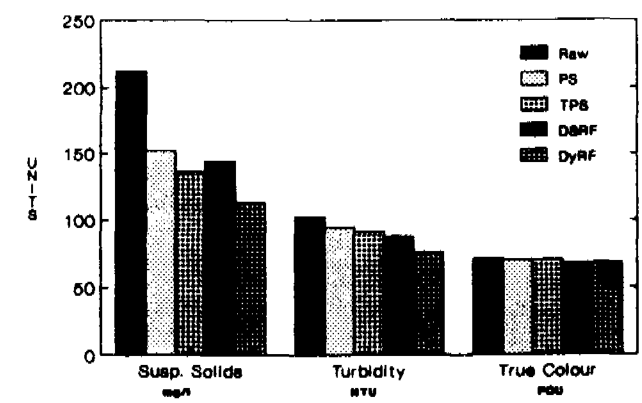
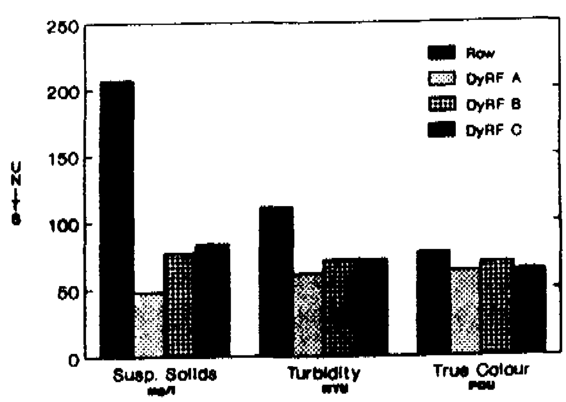


Figure A3-8 Performance results of several full scale plants

Performance of the dynamic roughing filters

The dynamic roughing filter being the first treatment step plays a very important role and holds a lot of promises for further cost savings. Figure A3-9 provides data from the first research period showing that the dynamic roughing filter gave better results in suspended solid removal than the plain sedimentation units and the tilted plate settlers. Removal efficiencies in the subsequent three periods are consistently good, between 57 and 80% in for average suspended solid loads in the raw water of 60 - 190 mg/l. These units also significantly reduce faecal coliform counts (33-78%), iron (46-75%) and manganese (52-60%) but have less impact on turbidity (36 - 45%) and much less on colour figures (11 - 17%). The fact that suspended solid loads are reduced to a higher extent than turbidity clearly shows that particularly coarser suspended material is being removed in the dynamic roughing filters. The results in Puerto Mallarino, are consistent with the findings resulting from the monitoring of the full scale systems treating water with low suspended solids concentrations (average values ranging between 10 and 16 mg/l) with removal efficiencies of 58 - 77% at flow velocities of 1.5 m/h and turbidity removal efficiencies between 44 and 49%. In one system operating at 10.2 m/h suspended solid removal reached only a level of 31%.





Period I

Period II

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

Figure A3-9 Average levels of different contaminants in raw water and effluent of dynamic roughing filters and other conditioning processes

Performance of the second stage roughing filters

A comparison of the different roughing filtration alternatives gives very interesting results. For turbidity removal, efficiencies between 54 and 83% were obtained in Puerto Mallarino, with best results for URFS and HRF, closely followed by DFRS. For the below 50 NTU turbidity range however all systems produce effluent values below 10 NTU except for the

URFL, the alternative with lowest cost, which in period III reached upto 12 NTU. Whereas 10 NTU is recommended by different authors as an acceptable influent for a slow sand filter, based on the very stringent water quality standards in the USA. In less developed countries drinking water quality guidelines however are less stringent, so slightly higher inflow turbidities can also be accepted, making the URFL also a viable alternative.

Removal of suspended solids was ranging from 84 - 98% again with the higher efficiencies for URFS, HRF and DRFS. but with the effluent remaining below 5mg/l for influent values below 187 mg/l. For removal of faecal coliform bacteria with influent values over 89,000, efficiencies ranged between 93.7 and 99.7%. For true colour removal they were lower but still ranged between 28 and 69% which are findings which sofar have not been reported in literature. The efficiencies were hardly affected by the increase in filtration rate, nor did water quality changes in the influent have a major impact. The best results were obtained with URFS, HRF and DRFS, the systems with longer retention times, but results with URFL and HRFM were still quite good. The somewhat lower efficiency of the URFL is easily explained by the much shorter length of the gravel of 1.55 m. The less good results of the HRFM, although having a length of 4.35 m is very likely due to poor hydraulic performance of the unit. A similar tendency as observed for coliform removal and colour presents itself for faecal streptococci and Iron and Manganese removal.

Although the performance of the process in the roughing filters is very good, the combination of mechanisms is not yet fully understood. For example, headloss development was measured in the different units resulting in different profiles which need further study and clarification. This be of importance to better understand and predict the performance of the different processes, which is essential to make design improvements and to develop the most suitable cleaning procedures.

The roughing filters in the full scale plants, all treating water with lower turbidity levels than the systems in Puerto Mallarino, showed lower removal efficiencies ranging from 30 - 72%, but all producing an effluent below 5 NTU. Also coliform counts in the raw water of the full scale plant are less, and removal efficiencies somewhat lower, ranging from 78 to 95%. True colour levels in the raw water were also lower and removal efficiencies ranged between 17 and 45%. The effluent of all pre-treatment units was sufficiently good that it could very well be treated with SSF.

Performance of the slow sand filters

The removal efficiencies of the slow sand filters have been similar to values reported in literature, but in the first research period, the pre-treated water was of rather poor quality leading to high levels of particularly suspended solids and in relative short filterruns between 8 and 30 days. Modifications in the pre-treatment systems in the subsequent periods resulted in lower levels of suspended solids and much better influent quality for the slow sand filters, resulting in longer filterruns between 30 and 55 days in period II (filtration rate of 0.1 m/h) to 30 to 35 in period IV (filtration rate 0.15 m/h) (Figure A3-10).

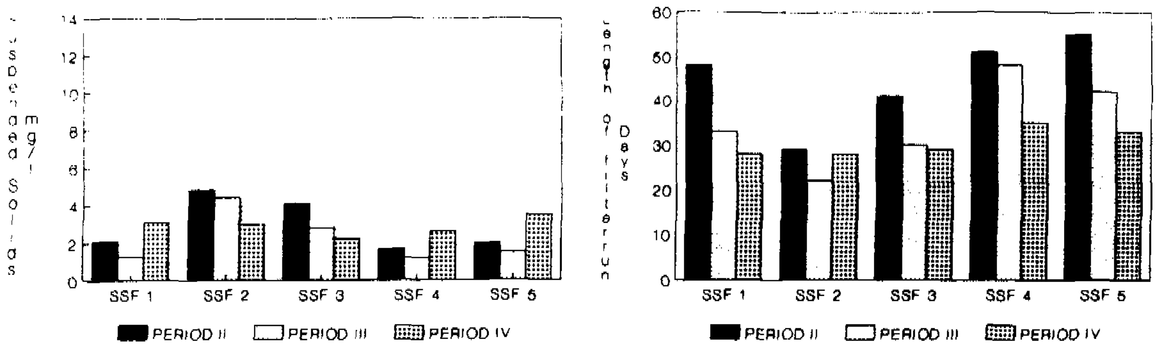


Figure A3-10 Graphic presentations of average loads of suspended solids in SSF influent and length of filterruns

The filterruns in the slow sand filters included in the full scale plants, varied between 2.5 and 7 month, with filtration rates around 0.15 m/h. These units received water with turbidity values ranging from 0.7 to 4.6 NTU and produced effluents ranging from 0.4 to 1.0 NTU. Only one plant had a shorter filterrun of 1 month, probably due to algae problems.

Conclusions and recommendations

The research results clearly show the potential of combining two-stage roughing filtration with slow sand filtration to treat water from different Andean rivers and are also promising for other regions in the world. Although all rivers contain water which falls in category E, the highest pollution category established by WHO and UNEP, water of very low sanitary risk was produced consistently by the treatment systems falling in category A (0 faecal coliforms/100 ml) or B(1-10 faecal coliforms/100 ml). The monitoring of seven full-scale community water supply plants which are operated by local caretakers, shows that a satisfactory effluent can be produced consistently without the addition of chemicals.

The microbiological and physico-chemical improvements brought about by the processes are considerably better then thought possible on the basis of the review of literature. More importantly the results clearly show the possibilities to introduce a multi-barrier concept to integrated water treatment without adding chemical coagulation. This is particularly so because the systems show higher removal efficiencies when the raw water quality is lower and thus presents a higher sanitary risk.

Two-stage roughing filtration vastly enhances the potential application of slow sand filtration and the combination fits very well in the concept of integrated water treatment as the processes are complementary in the removal of different contaminants. Roughing filtration 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. Having overcome the combination of high and fluctuating levels of faecal contamination, suspended solid loads, turbidity, true colour, iron and manganese, a very low dose disinfection stage could be introduced wherever it could be accepted and sustained at local level (Galvis 1992).

The dynamic roughing filter provides a very good first step in the treatment process at very low cost, on average less than 5% of the capital investment in the treatment plant. It plays a very important role as first treatment step were it contributes to the removal of different substances including suspended solids, micro-organisms, iron and manganese. Further improvement of this step is advisable as this may strongly contribute to the overall optimization of the subsequent treatment processes.

The different roughing filtration alternatives compared in Puerto Mallarino, all performed very well. It was clearly shown that they not only have a great capacity to remove suspended solids and turbidity, which has been the main reason for their development, but equally or maybe even more important is their potential to reduce faecal coliform counts, iron and manganese levels and colour figures. The removal efficiencies for all systems were not very much affected by the gradual increase in filtration rate from 0.3 to 0.6 m/h, which seems to imply that further optimization is possible. The treatment results in the full scale plants confirm the findings in the research station, although removal efficiencies were somewhat lower, which are in line with the lower contamination in the raw water sources from which these plants draw their water. Further optimization of both the units in the research station and the full scale can be obtained as all systems have been over designed due to the very limited availability or complete absence of design criteria in literature.

On the basis of a statistical analysis, URFS, DRFS and HRF provide comparable results, which are slightly better than the other two alternatives URFL and HRFM. However when taking into account the depth/length of the gravel bed, the URFL presents the cheapest alternative for the less and the URFS and DRFS for the more polluted water sources. When also following the present experience that upflow systems are more easy to clean the prime emphasis goes to the URFL and URFS.

Preliminary guidelines can be established on the basis of this first comparative research in roughing filtration alternatives. These concern roughing filters with design criteria as indicated in Table 1 and operating at a flow velocity of 3m/h for DyRF and 0.6 m/h for the other roughing filter systems and 0.15 m/h for SSF.

- For relative clean water sources, below 10 NTU in turbidity and 500 Faecal coliforms/100ml, a combination of DyRF and SSF would be appropriate.

- For highland rivers having limited changes in water quality and of short duration DyRF, URFL and SSF is a good combination upto 100 NTU and 10,000 FC/100ml.

- For lowland rivers with stronger water quality variations of longer duration, DyRF, URFL and SSF is a good combination upto 50 NTU and 50,000 FC/100ml and the combination DyRF, URFS and SSF is good upto 100 NTU and 120,000 FC/100ml

- For more contaminated water sources the systems under study still provide a good potential, but the present research findings do not permit firm recommendations for these situations implying that for the time being on-site pilot plant testing will be needed to explore which performance may be expected. This will depend amongst others on the type of suspended solids and their particle size distribution which needs more study then sofar has been possible in the research carried out sofar.

It is presently being looked into if similar criteria can also be established for some of the other parameters measured in the research including true colour, iron and manganese, by carrying out statistical analyses of the data incorporated in the data base.

The research results provide a very good starting point for designing suitable multi barrier water treatment systems for the majority of raw water conditions prevailing in Colombia and other countries without the need to use chemicals. The research findings show the need to take into account both the multi-barrier concept and the concept of integrated water treatment. This implies that not only water quality but more important the sanitary risk involved in a certain water source and the response of the treatment system towards this risk is being assessed when planning and designing a water treatment system.

However as the true limits of pre-treatment technology are not yet known, designs would have to be rather conservative and would leave considerable room for optimization. Furthermore operational problems may show up due to the fact that research in pre-treatment technology is very recent and therefore has not lead to a full understanding of the processes involved. Further research and development is thus needed to obtain better insight in the processes to enable their technical and economical optimization, establish adequate operation and maintenance procedures and analyze their effect on possible disinfection. Late 1992 another area for additional research was identified when first bench tests proved that roughing filtration has potential application in combination with conventional treatment processes which are based on dosing of chemicals. Not only does pre-treatment strongly reduce the required chemical dose, but they can also improve the operational conditions, thus having a very good potential even for optimizing existing conventional systems.