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DOI
10.4233/uuid:61328f49-8ab8-4da1-8e84-4650bddf9a1d

Publication date
2016

Document Version
Final published version

Citation (APA)
Sanchez Torres, L. (2016). *Upflow gravel filtration for multiple uses*. [Dissertation (TU Delft), Delft University of Technology]. https://doi.org/10.4233/uuid:61328f49-8ab8-4da1-8e84-4650bddf9a1d

UPFLOW GRAVEL FILTRATION

FOR MULTIPLE USES

Luis Darío Sánchez Torres

Upflow gravel filtration for multiple uses

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Proefschrift

Ter verkrijging van de graad van doctor
aan de Technische Universiteit Delft,
op gezag van de Rector Magnificus Prof. Ir. K. Ch. A. M. Luyben,
voorzitter van het College voor Promoties,
in het openbaar te verdedigen op
28 april 2016 om 12:30 uur

door

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The dissertation was supported by:

Universidad del Valle Cinara

Keywords: Upflow gravel filtration, multi stage filtration, groundwater, micro-irrigation

Printed by: Gildeprint
Cover by: L.D. Sánchez Torres
ISBN: 978-94-6186-640-0

Dedication

This thesis is dedicated to my mother, Maria Elvia Torres, for all your love and dedication by our education.

"Si realmente quieres entender algo, trata de cambiarlo"

(Kurt Lewin, 1890-1947)

Contents

Nomenclature

Abbreviations

Abbreviation	Parameter
APHA	American Public Health Association
AWWA	American Water Works Association
CEPIS	Centro Panamericano de Ingeniería Sanitaria y Ambiental
CF-UGF	Coagulation- flocculation in up-flow gravel filter
CGR	Contraloría General de la República
CMRS	Completely Mixed Reactors in Series
CNR	Comisión Nacional de Riego
DRF	Dynamic roughing filtration
EIDENAR	Escuela de Ingeniería de los Recursos Naturales y del Ambiente
FR	Filter run
GF	Gravel filtration
HPC	heterotrophic bacteria plate count
IDRC	International Development Research Centre
INTA	Instituto Nacional de Tecnología Agropecuaria
IRC	International Water and Sanitation Centre
LHFI	Localized High Frequency Irrigation
MDGs	Millennium Development Goals
MHL	Maximum head loss
MSF	Multi-Stage Filtration
O&M	maintenance and operations
R&TT	Research and Technology Transfer
RF	Rapid filters
RMC	Rapid Mixing Chamber
SDG	Sustainable development goals
SSF	Slow sand filtration
TSS	Total Suspended Solids
UGF	Upflow Gravel Filtration
UN	United Nations
UNESCO	United Nations Office for Science and Culture
UNICEF	United Nations Children's Fund
Univalle	Universidad del Valle
PCU	Platinum Cobalt Units
WHO	World Health Organization
WPCF	Water Pollution Control Federation

Variables and Constants

Symbol	Parameter	Unit
A	Area	m^2
As	average geometric surface area	m^2
C_s	Coefficient of sphericity	(-)
D_{mg}	average grain size	(-)
DO	dissolved oxygen	mgL^{-1}
$Dulq$	distribution uniformity of the lower quarter	%
E	Efficiency	%
E_p	loss of energy in the channel unit coagulation	m
Fr	Filtration Run	h
hf	head loss	m
h_v	table of water over the weir	m
Δh	declining water level	m
J	loss of unit load	mm^{-1}
k	permeability	cm^{-1}
L	Length	m
Lr	hydraulic jump length	m
R_e	Reynolds number	(-)
Red	Reduction	%
RT	Residence time	min
t	time	h
°t	Temperature	°C
T_m	average time of mixing	%
U_c	Uniformity coefficient	(-)
vf	filtration rate	mh^{-1}
ε	Porosity	%
γ	specific weight of water	Nm^{-1}
μ	absolute viscosity	Nms^{-2}
v	kinematic viscosity	m^2s^{-1}
g	gravity constant	ms^{-2}
ρ	density of water	kgm^{-3}

CHAPTER 1

General introduction.

1.1. Relevance of improving water quality in small communities and small towns.

Safe drinking water supply and basic sanitation, together with hygiene education, are considered fundamental components to improve the quality of life and productivity in human settlements. Water access problems and poor water quality affect human health and wellbeing of communities. To improve access to these services and contribute to poverty reduction the international community through the United Nations (UN) agreed to the Millennium Development Goals (MDGs) until 2015. According to UN (2013) more than 2,100 million people have obtained access to improved water sources in the last 21 years. In 2010, the proportion of population with access to such sources was 89% (76% in 1990). This would mean that the target on drinking water was achieved five years before the scheduled date, despite significant population growth. Today still 800 million people are without access to an improved water source and many more remain without safe and sustainable sanitation (SDSN, 2014). Whereas the overall picture seems positive, the statistics hide a number of problems. In Latin America there are considerable differences between urban and rural coverage, and, according to the Inter-American Development Bank, (BID, 2013), 12 countries had not yet reached its implicit goal of rural coverage in access to safe water. Colombia, Haiti, Nicaragua and Venezuela are among the lowest performers (differences of over 8% with the MDG goals). Table 1.1 presents the variation in use of water supply and sanitation facilities between 1990 and 2015 in Colombia and in Latin America and the Caribbean compared to the proportion of the 2015 population that gained access since 1990.

Table 1.1. Use of water supply sources and sanitation facilities at Colombia and Latin American and the Caribbean (adapted from UNICEF and WHO, 2015).

Area	Use of facilities (% population) 1990		Use of facilities (% population) 2015		Proportion of population with access since 1990 2015 MDGs (%)	
	Water	Sanitation	Water	Sanitation	Water	Sanitation
Colombia						
Urban	97	82	97	85		
Rural	69	41	74	68		
Total	88	69	91	81	32	35
Latin American and the Caribbean						
Urban	94	80	97	88		
Rural	63	36	84	64		
Total	85	67	95	83	35	36

The values in Table 1.1 are in a way masking the underlying problem that many water supply systems do not provide a sustainable service of good quality water. This is even more of a problem in rural areas. A related problem is that the limited attention for sustainability aspects entails a considerable risk for a fall back in the gains made in extending water supply services (WHO/UNICEF, 2012).

The UN are therefore in the process of developing action oriented sustainable development goals (SDG). These are global in nature and universally applicable, taking into account different national realities, capacities and levels of development, respecting national policies and priorities. They build on the foundation laid by the MDGs, seek to complete the unfinished business of the MDGs, and respond to new challenges. These goals constitute an integrated, indivisible set of global priorities for sustainable development (SDSN, 2014). Goal 6 of the proposed SDGs 2030 (UNDP, 2016) includes: “To ensure availability and sustainable management of water and sanitation for all”. This goal is much more ambitious than the related MDG, as it aims at universal and equitable access to safe and affordable drinking water. This is an enormous challenge taking into account that sustained functioning of water systems and adequate water treatment are important limitations particularly in community water supply. For example, in rural areas of Colombia it is estimated that 79% of the 11,608 rural water systems do not provide satisfactory water quality (INS, 2013). Reasons include: use of inadequate technologies (which in several cases has led to abandoning the treatment systems), design problems, absence of trained operators, lack of resources for buying necessary materials and equipment, deteriorating water quality due to insufficient catchment protection. This situation clearly calls for action and innovation and for technologies that can cope with water quality deterioration and can be operated by local operators and sustained by rural communities at reasonable costs.

The challenge is important for Colombia taking into account that over 81% of the water supply systems use surface water sources (Figure 1.1) which are being affected by the continuous process of deforestation and erosion, and the effects of climate change. Furthermore the treatment of domestic wastewater is poor as only 20% of total wastewater that is produced is subject to a form of treatment (CGR 2009).

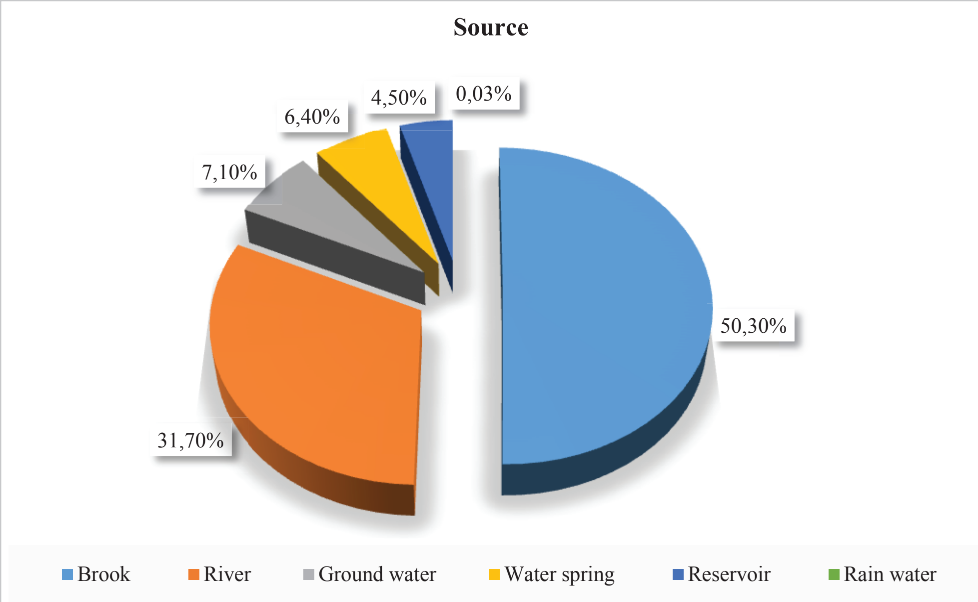

Figure 1.1. Sources in rural water supply systems in Colombia (based from INS 2013).

The consequence of this situation is that existing and future water treatment systems will need to cope with higher loads of suspended solids and bacteriological contamination that will require additional water treatment barriers to be able to provide good quality drinking water in a sustainable way to its users.

1.2. Upflow gravel filtration in water treatment

One particular technology that seems to offer a potential to help facing the challenges of deteriorating raw water sources and increased demands for improved water qualities is upflow gravel filtration (UGF). UGF technology has mainly been applied as pretreatment step in multi stage filtration (MSF). This system is a combination of different types of gravel filtration (dynamic gravel filtration (DyGF), upflow gravel filtration (UGF), down flow gravel filtration (DGF), and/or horizontal gravel filtration (HGF)) and slow sand filtration (SSF) (Galvis, 1999; Galvis et al., 1998; Di Bernardo and Sabogal, 2008) to overcome problems in small water supply systems using surface water sources with high levels of turbidity, total suspended solids (TSS) and faecal coliforms which exceed the treatment capacity of SSF alone. UGF was considered in this research because it is the main pretreatment used in full scale MSF systems in Colombia also by its ability to maintain treatment simplicity comparable to that of SSF at accessible investment costs, facilitated by the use filter material from local sources. UGF and DGF in series have good removal efficiencies of suspended solids and fecal coliforms with a better hydraulic performance than HGF (Galvis, 1999). In an UGF the accumulated solids can be drained through bottom discharge by a drainage system that can be constructed of perforated pipes of low costs (Galvis, 1999).

In an UGF the water passes through the gravel bed from the bottom to the top. During this passage impurities are retained in the filter, with sedimentation being the dominant particle transport and removal mechanism (Boller, 1993). UGF has a high potential for TSS and turbidity removal, because it allows for storing large amounts of solids with limited increase in head loss (Boller, 1993), facilitating long filter runs.

In an UGF good turbidity reductions are obtained at filtration velocities < 2 mh^{-1}, preferably <1 mh^{-1}, because high solids removal efficiency is only achieved under laminar flow conditions (*Re* 2-10, efficiency between 40-80%) (Wegelin and Mbwette, 1989). For influent turbidity less than 70 NTU an UGF is expected to produce an effluent with turbidity <10-20 NTU or TSS < 2-5 mgL^{-1} (Galvis, 1999).

Biological activity has been reported in UGF when water with organic matter and nutrients is treated (Galvis, 1999; Di Bernardo and Sabogal, 2008; Arakawa *et al*., 2014). Bacteria and other microorganisms may form sticky layers (biofilms) or produce exocellular polymers that contribute to particle destabilisation and attachment. However, macro-biological organisms, inhabiting the gravel filters, contribute to the sloughing off of stored material or biofilm (CEHE, 1999; Galvis 1999). In addition, the low filtration velocity and the upflow current promote the gradual removal of impurities from the bottom to the top, resulting in clear water on top, allowing for the penetration of sunlight and algae growth.

1.3. Performance of UGF

Several UGF in MSF systems, are already in operation for more than 20 years at a cost that never exceeded 4% of total family income (Sánchez et al, 2007). UGF is considered to be a suitable pretreatment technique for rural water supply systems because of its ability to maintain treatment simplicity comparable to that of SSF with accessible investment costs due to the use of filter material from local sources that can be sieved and cleaned by community labor. In addition, the area required for construction is easily obtained in rural areas. These systems can also be administrated, maintained and operated by local operators.

During the 1800s some sort of upflow filters were already built in England, France, Scotland and the USA (Baker, 1981). UGF was introduced in Brazil during the 1960s, and in Colombia during the 1970s (Sánchez *et al*, 2007). In Colombia, pilot and full scale studies with MSF were developed during the 1980s (Visscher and Galvis, 1987). Other pilot studies with UGF were developed later in Brazil (Di Bernardo et al, 1988; Di Bernardo, 1993). During the 1990s studies of UGF and MSF continued at Cinara (Galvis et al, 1993; Wegelin et al, 1997; Galvis et al, 1998). Based on the experience with UGF in Colombia, during the 2000s a growing number of full scale MSF systems were constructed in Latin America. Today UGF is the main pretreatment technology used for rural water supply systems in Valle del Cauca, Colombia possibly because of the technology selection guide that was developed by Cinara and the MSF technology transfer project TRANSCOL (Galvis et al. 1998). A recent survey identified 62 MSF treatment plants from a total of 115 (Veldt and Burger, 2015).

The performance and operation and maintenance (O&M) of MSF can be compromised by sudden changes in raw water quality, which may interfere with the efficiency of the treatment process. The most serious problem concerns peaks in turbidity level and *E-coli* concentrations. To cope with these problems costly interventions have been made in some

MSF systems. In water supply systems like El Retiro (see chapter 2) in the periphery of Cali a 4,000 m^3 settling basin was constructed in 2003 to reduce the turbidity peaks in the water from the Pance River before the MSF system. A similar settling basin of 2,000 m^3 is planned in the regional water system in the north of the department Valle del Cauca, which has an MSF system that treats water from the Palomino River. This river has increasing problems with turbidity peaks, some lasting for 24 hours, and water pollution, usually water leaching from coffee pulp and mucilage fermentation from coffee plantations (Fields, 1987). These changes in water quality particularly affect the efficiency of the UGF.

In view of the deterioration in surface water quality and the costly solutions that are already being adopted in some MSF systems, a better understanding of the performance of the UGF in these systems is needed.

Because of the decreasing availability of good water quality in surface sources, the use of groundwater has been increasing in Colombia in the last years. However, this source has limitations due to the high iron and manganese content, e.g. in Valle del Cauca region 85% of the wells have problems with iron and manganese. High concentrations of iron and manganese may lead to rejection of the water by consumers (WHO 2011) and may also affect water distribution systems, valves, meters and other accessories. Removal of iron and manganese is therefore key to ensuring sustainable water services, particularly in rural water systems.

1.4. Objectives

1.4.1. General objective of the thesis

This thesis focuses on learning more about the performance of the UGF treatment process and the relation with O&M in MSF systems. In addition, the improvement of the performance of UGF and its potential application for other uses are explored to contribute to a better response to the water quality problems that water supply entities face as a consequence of deterioration of surface water sources, aggravated by climate change and environmental deterioration.

1.4.2. Specific objectives

- To evaluate the robustness of design, O&M procedures and performance of four full-scale UGFs that are part of full scale MSF systems, comparing practice with the criteria and procedures recommended in literature.

- To evaluate the performance and design aspects of coagulation and flocculation in UGF in a MSF plant, defining influent turbidity levels to allow operation with and without coagulant during peak turbidity loads.

- To analyze the *E-coli* and TSS removal in UGF with a filter fabric on top and the influence of algae growth in the fabric cover.

- To explore the removal of iron and manganese from groundwater by UGF under high and low oxygen concentrations at different pH levels.

- To analyze the potential application of UGF for micro-irrigation, examining the effect of six water treatment combinations for the treatment of turbid surface water on four types of emitters looking at clogging potential and distribution uniformity of irrigation.

An overview of the approach for this research is shown in Figure 1.2. The research combined studies in full-scale plants, pilot filters and at bench scale. Part of the pilot studies were carried out at the Cinara institute's Research and Technology Transfer (R&TT) Station based at EMCALI, Puerto Mallarino, using water from Cauca River. All MSF plants are fed with water from small rivers or streams draining relatively small watershed areas. Measurements comprise water quality (influent and effluent), conditions of O&M, and efficiencies, including the review of plans and design criteria.

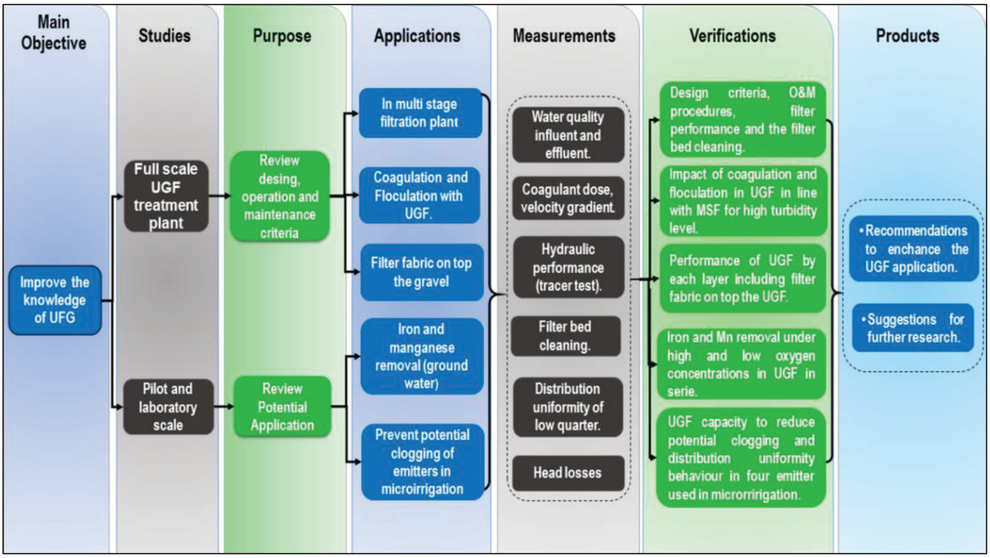

Figure 1.2. Overview of the approach for this research

1.5. Thesis outline

Chapter 2 presents the results of a study of four full scale UGFs that are part of full scale MSF systems located close to Cali, Colombia. The study analyzed the design criteria that were applied, the way O&M is carried out, and the performance of the systems including filter bed cleaning. The analysis was further based on multiple sources of evidence e.g., observation, interviews, O&M procedures review, water analysis and the literature. Also, feedback from system operators was used to check whether performance and conditions during the research period deviated from the normal situation.

Chapter 3 provides an analysis of low-cost MSF enhanced by coagulation-flocculation in UGF (CF-UGF). The performance and the design and operational aspects of the CF-UGF units followed by UGF and SSF was reviewed, defining influent turbidity levels to allow operation with and without coagulant, taking advantage of the SSF as the final stage of filtration for the removal of micro-organisms. The study was carried out in a full scale system with seven years of operation. During the study, water quality and system operation was monitored including coagulant dosage and measurements of the washing velocity and head loss (h_f) over the filter bed. Also the hydraulic behavior of CF-UGF units and the investment and O&M costs were considered.

Chapter 4 focuses on the (improved) performance of an UGF covered with a filter fabric looking at the removal efficiency for TSS, *E-coli*, particle size distribution in each gravel layer and also the head loss development and algal growth on top of the filter fabric. The study was conducted in a pilot filter to determine the effect of head loss development in relation to algal biofilm growth on filter fabric, the role of each of the gravel layers on TSS, *E-coli* and particles removal, and the effect of algal growth on the filter fabric in relation to *E-coli* removal.

In Chapter 5 the iron and manganese removal in UGF under high and low oxygen conditions is explored. The analysis started with a bench scale test to establish the adsorption capacity for iron and manganese of coated gravel at different pH. The second step was to compare the two oxygen conditions for iron and manganese removal in pilot filters, taking into account the effect of different filtration velocities.

Chapter 6 evaluates the effect of six water treatment combinations for the treatment of turbid surface water on four types of emitters looking at clogging potential and distribution uniformity of irrigation. A pilot plant was used to analyze the removal of physical chemical parameters that affect emitters clogging. For each treatment line, the efficiency and clogging potential was determined. The effect of treatment on the performance of the four emitters were estimated by distribution uniformity of the lower quarter and was measured by the performance of discharged flow at each emitter over time.

In Chapter 7 the main conclusions of this research are provided and suggestions are made for further research and recommendation to enhance the UGF application.

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CHAPTER 2

Performance of upflow gravel filtration in multi stage filtration plants.*

This chapter presents the results of a study of four full scale upflow gravel filters that are part of full scale multi-stage filtration. The study explored the design criteria, the operation and maintenance (O&M) practices, and the performance of the systems. Findings showed that most design criteria and O&M procedures are following the recommendations as presented in the literature but several diversions were also identified. Performance data showed that removal efficiencies were on the low side when compared to the literature, possibly because of the good influent quality water that was treated. Cleaning efficiency was analyzed and the overall conclusion is that an adjustment of the design criteria and O&M procedures is needed to enhance system performance. This includes drainage system design, surface cleaning by weir, and filter bed cleaning to allow a reduction in cleaning cycles and an improvement in operation control.

*This chapter is based on:

Sánchez, L.D., Visscher J.T. & Rietveld L.C. 2015. Performance of upflow gravel filtration in multi-stage filtration plants. Water science & technology, 71 (4), 605-614.

2.1. Introduction

Upflow gravel filtration (UGF) is an important component in multi-stage filtration (MSF) systems, particularly because it protects slow sand filters (SSF) from receiving high loads of suspended solids and other pollutants including microbiological contamination. The main development of UGF technology emerged in Colombia in the 80's, where it was introduced first at technical and thereafter at full scale (Galvis *et al*. 1999). In 2005, more than 140 MSF systems existed in Colombia (Visscher 2006), and to date the number surpasses 200. In the Valle del Cauca region, about 54% of existing rural water treatment plants use MSF.

A UGF consists of a box, or a series of boxes, filled with gravel where the water enters from below and flows out from the top (Figure 2.1). During this passage, impurities are retained in the filter. When filters are cleaned, accumulated solids are removed through gravity flow by opening the drainage (butterfly) valve. The gravel has a large surface area where particles can be retained by sedimentation (main removal mechanism) and attachment (Boller, 1993; Galvis, 1999), and where biomass can play a role in biodegradation, as was mentioned in chapter 1 section 1.2, thus facilitating longer filter runs. Operation of a UGF involves the control of the filtration velocity, the head loss over the filter, and effluent water quality. O&M mainly comprises control of the filtration velocity, head loss, influent and effluent water quality as well as different types of cleaning procedures such as gravel surface cleaning and filter bed cleaning, which may be undertaken daily, weekly, monthly or even less frequenlty (Galvis *et al*. 1999).

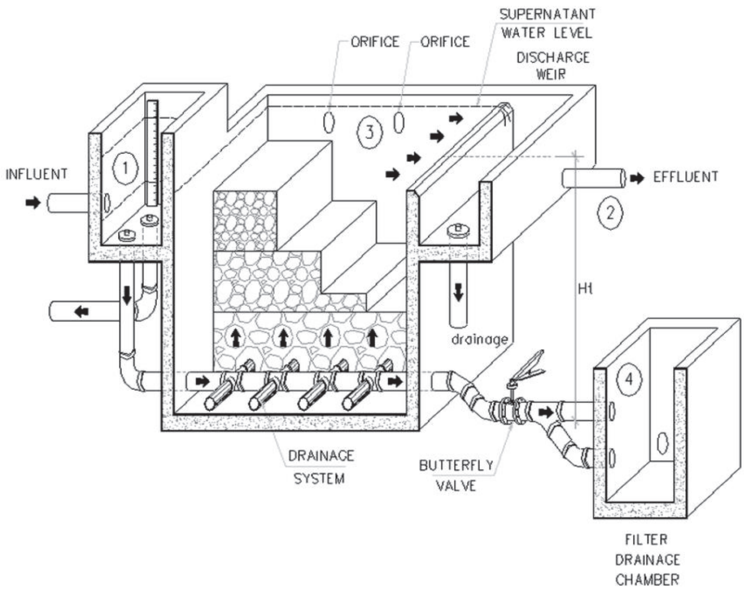

Figure 2.1. Schematic overview of a UGF system with different gravel layers.①, ② monitoring water quality parameter: turbidity, E. coli, total coliforms, pH; ① head loss measure; ③ surface cleaning; ④ drainage during filter bed cleaning.

Several systems already operate for a long time making it relevant to evaluate the robustness of design, operation and maintenance (O&M) procedures and performance of such systems, particularly because most systems are managed by local water committees. Therefore, these issues are addressed in this chapter, as well as comparing practice with the criteria and procedures recommended in the literature.

2.2. Materials and methods

2.2.1. Approach

Four treatment systems were selected near to Cali, Colombia. These systems were selected because they represent different situations that together make up a large part of the UGF systems currently available in Colombia. Differences include social and community conditions (middle and low-income communities), gravity versus pumped systems (three gravity systems and one pumped system), surface sources with and without storage reservoir, and differences in O&M practices. The analysis explored the design criteria applied in the systems, the O&M procedures that are used and the UGF performance, including the treatment efficiency for turbidity, *E-coli*, total coliforms and also total suspended solid (TSS) removal, hydraulic behaviour and cleaning efficiency. The analysis was further based on multiple sources of evidence e.g., observation, interviews, water analysis and the literature (Yin 1989), this enhance the validity of the findings by triangulation (Stake 1995) which consists of using a combination of methodologies to study the same phenomenon, thus making it possible to compare, enrich the interpretations, and contrast data from different sources. In this case, feedback from system operators was used to check whether performance and conditions during the short research period deviated from the normal situation. Although the research of a reduced number of systems has limitations for the generalization of findings, the four selected systems are still fairly representative for a much larger number of UGF systems that treat water from surface water sources in Colombia.

Design criteria were established by reviewing drawings and physical inspection. O&M procedures were reviewed by looking at operational instructions (if available), observation and interviews with operators. Water quality parameters were measured in the UGF (see Figure 2.1, points (1) and (2)); surface cleaning was observed (point 3) and filter bed cleaning was monitored (point 4). The flow was measured by a calibrated triangular weir installed in the inlet channel of the UGF units. Samples of the filter material were sieved to verify gravel size and porosity was determined following the procedure described by Ives (1990). Head loss (h_f) over the UGF was measured daily over 7 days, covering a full cycle of operation between two cleanings.

Turbidity and TSS in the influent and effluent of UFG were measured daily during a 2-week site visit and were used to estimate TSS accumulation during the filter run. This was compared with the TSS measured in the cleaning process. Discharge during cleaning was measured at point (4) (Figure 2.1). To observe possible differences in cleaning efficiency, the standard cleaning procedure with shock loading by opening and quickly closing the drainage valve (some 10 times), was compared with an uninterrupted drainage process (which is easier for the operator).

2.2.2. Water quality analyses

Influent and effluent quality was monitored in the UGF units at each treatment plant, looking at the following parameters: TSS, Turbidity, *E-coli*, total coliforms and pH (see Figure 2.1, points (1)-(2)), using standard methods (APHA, AWWA, WPCF, 2005). During filter bed cleaning, turbidity and TSS were measured and the correlation was verified.

2.2.3. Filter bed cleaning

To obtain more insight in to filter bed cleaning, the following procedure was followed: (1) the surface area of each filter was measured (A); (2) the declining water level in the filter (Δh) was measured over time (t) and (3) the washing velocity was set by the expression $Q = \Delta h * At^{-1}$ (m^3s^{-1}). The drop in the water level in the filter was measured in the inlet pipe until the filter was empty.

2.2.4. Hydraulic behavior of UGF Units

The hydraulic behavior of the UGF units was established by applying tracer tests (see chapter 3) with sodium chloride, which makes it possible to determine the presence of dead zones resulting from the hydraulic design and possible permanent clogging. The concentration curve of the tracer was analyzed using the mathematical simulation models of Wolf-Resnick, the Morril index, and the model of completely mixed reactors in series CMRS such as are described in chapter 3.

2.2.5. Description of the UGF systems

The four full-scale MSF plants are described in box 2.1. All systems include dynamic gravel filtration, except for El Retiro, which was selected because this system is preceded by a reservoir (4,000 m^3) to prevent peak loads of suspended solids reaching the UGF. All plants have UGF in layers as a secondary filtration stage. The system in La Sirena was selected because it has two stages of UGF, both in layers. This diverts from what is described in the literature as a two-stage UGF with two filtration stages of different gravel size with crushed gravel, which is different to the other systems that use river cobble. The plant in Arroyohondo was selected because it has a special feature in that it makes it possible to dose a coagulant prior to the UGF when turbidity is high, to stimulate coagulation and flocculation in the UGF (see chapter 3). All plants have SSF as the final filtration stage. Golondrinas is a typical system located in a mountainous rural area with deforestation problems in the watershed. A summary of the treatment plant components is shown in Table 2.1.

All UGF units are made from reinforced concrete. Drainage systems, consisting of perforated PVC pipes, are placed at the bottom of the structure, serving both to distribute the flow during filtration and to discharge the water during periods of cleaning.

Table 2.1. Treatment plant components.

		Pretreatment system					SSF		
Treatment plant	**Flow (Ls^{-1})**	**Type**	**#**	***A* (m^2)**	**Filter length**	**ε (%)**	**#**	***A* (m^2)**	***vf* (mh^{-1})**
El Retiro	20	Reservoi	1	2,000	-	-	4	480	0.15
		UGFL	4	28	1.6	44-46			
La Sirena	10	DyGF	2	9.0	0.6		2	240	0.20
		$UGFS_2$	3	17.7	2.2	54-56			
Arroyohondo	6	DyGF	2	5.4	0.60		4	72	0.15-0.30
		UGFL	2	10.6	1.05	38-42			
Golondrinas	9	DyGF	2	8.1	0.60		4	216	0.15
		UGFL	2	23.1	1.1	42-43			

#: Units number; *A*: Area; ε: Porosity; $UGFS_2$: upflow gravel filtration in series with two stages; DyGF: dynamic gravel filtration; UGFL: upflow gravel filtration in layers.

Box 2.1. The four MSF systems included in this study

The MSF system in El Retiro, replaced a conventional water treatment plant with rapid filtration in 1987; the system provides water to a better-off neighborhood with 500 inhabitants and a number of private schools. The system was financed through the tariff and is managed by a team of operators supervised by an users committee.

The MSF system in Arroyohondo, replaced a compact conventional water treatment plant with rapid filtration. In this MSF it proved possible to use coagulation and flocculation in combination with UGF which enhances the flexibility to respond to variations in turbidity as discussed in chapter 3. The system was built in 2005 with financial resources raised by local organizations and communities. Today it supplies water to 840 inhabitants.

The MSF system in La Sirena, was built in 1988 in response to several cases of cholera that occurred in the community. Initially it only comprised SSF, but subsequently this was transformed into a MSF system to cope with the deterioration of water quality in the watershed. It provides water to 4500 inhabitants of a low-income settlement. It is managed by a water committee and was built with support from central and local governments and a small grant from the Dutch Embassy.

The MSF system in Golondrinas, is located in a remote rural low-income community. It provides water to 2500 inhabitants, is managed by a water committee and was built in 2005 with financial resources from central and local governments.

2.3. Results and discussion

2.3.1. Design characteristics of the UGF systems

The design characteristics of the systems are shown in Table 2.2. , which also includes the guideline values given by Galvis *et al*. (1999).

Some design characteristics (filter length, number of stages, and period of operation) are in line with Galvis *et al*. (1999). Gravel sizes, however, are different and the observed filtration velocities were all above the recommended levels (0.6 mh^{-1}). In El Retiro and Golondrinas, the minimum filtration area per UGFL unit is over 20 m^2, which may influence the washing efficiency. Important differences also exist for the minimum static head (difference between supernatant water level and the outlet pipe in the drainage chamber (*Ht* Figure 2.1)) only matches the criteria in La Sirena, which is part of the hydraulic design of drainage systems, to ensure sufficient initial washing velocity. This velocity was low in two systems showing deficiencies in the design of these two systems.

Rulers to measure flow rate and head loss were missing in all systems. The absence of these tools suggests that the operators and their supervisors did not grasp the importance of either flow control to avoid overloading or head loss measurement to follow the clogging process.

Table 2.2. Design criteria applied and design criteria recommended for each upflow gravel filtration

Criterion	Guide	Treatment plant			
		El Retiro	La Sirena	Arroyo-hondo	Golon-drinas
Design period (years)	8-12	15	15	15	15
Period of operation (hd^{-1})	24	24	24	24	24
Filtration velocity (mh^{-1})	0.3-0.6	0.64	0.67	0.45-0.9	0.7
Number of stages					
UGFL	1	1		1	1
UGFS	2-3		2		
Filter bed					
Length of gravel bed (m)					
UGFL	0.6-0.9	1.0		0.75	0.80
UGFS	1.15- 2.35		1.6 [(a)]		
Size (mm)	1.6 - 25	4.0-28	4.0-28	3.2-25	2.2-25
Support bed					
Length (m)	0.3	0.5	0.3	0.30	0.30
Supernatant water height (m)	0.1-0.2	0.05	0.10	0.05	0.10
Minimum static load of washing flow (m)	3.0	2.2	4.0	1.62	1.55
Area per filtration unit (m^2)	<20	28	17.7	10.6	23.1
Initial washing velocity (mh^{-1})	>10	5.4	10.2	10.4	5.4

[(a)]two stages of 0.8 m

2.3.2. Operation and maintenance as practiced in the systems

O&M procedures were compared with the procedures proposed in the literature (Table 2.3). All systems were operated based on visual inspection of the water, closing the inlet if the operator observes that the turbidity is too high. Flow velocity, head loss and turbidity were not measured.

Weekly cleaning was applied in all systems but operators added the envisaged monthly surface cleaning and carried this out before filter bed cleaning. All operators followed the procedure as suggested in the literature, which entails interrupting the outlet and inlet flows to the unit whilst maintaining a layer of supernatant water on top of the gravel bed. Surface cleaning was then done manually with a shovel, stirring the surface layer of the filter to remove solid material adhering to the gravel. The supernatant water was discharged with the released solids. In the two systems with orifices, the water discharge is low and much lower than the two systems with overflow weirs, which may result in the removal of fewer solids.

Table 2.3. Qualitative comparison of applied and recommended operation and maintenance activities

Activity recommended	Treatment plant			
	El Retiro	La Sirena	Arroyo-Hondo	Golon-drinas
Daily operation				
Flow measurement and adjustment [(a)].	No	No	No	No
Turbidity measurement.	Yes	No	Yes	No
Head loss measurement[(b)].	No	No	No	No
Remove any floating material.	Yes	Yes	Yes	Yes
Record of turbidity.	Yes	No	Yes	No
Weekly maintenance				
Cleaning walls of the inlet and outlet chamber.	Yes	Yes	Yes	Yes
Hydraulic filter cleaning (filter draining) [(c)].	Yes	Yes	Yes	Yes
Restarting the UGF.	Yes	Yes	Yes	Yes
Checking of filter cleaning efficiency.	Yes	Yes	Yes	Yes
Monthly maintenance				
Gravel surface cleaning[(d)].	Weekly	Weekly	Weekly	Weekly
Implement normal cleaning	Weekly	Weekly	Weekly	Weekly
Less frequent				
Gravel bed removing, cleaning and put back into the unit	No	No	No	No

[(a)]Visual adjustments, but there are no records; [(b)]Visual inspection of the water level is done in the inlet chamber to verify the maximum level, but no record is made; [(c)]Hydraulic filter cleaning was performed with successive closures of the fast drainage valve. This is a butterfly valve which facilitate operation; [(d)]Done as part of weekly maintenance.

The cleaning procedure by filter drainage also matches the procedures indicated in the literature. Filter units were filled to 20 cm above the gravel bed by opening the inlet valve, thus increasing the static head at the start of the cleaning. During drainage, the butterfly valve on the drain pipe was quickly opened and closed (approximately 10 times). The filter was then filled again from the top and drained, and thereafter put back into operation.

The envisaged occasional extraction and washing of the gravel has never been done in any of the systems according to the operators, and one system has been operating for over 15 years with only weekly cleanings. Table 2.4 presents additional O&M data for each system. Differences exist in the maximum turbidity levels that operators accept before closing the inlet, to avoid turbidity peaks reaching the system. Operator judgement is based on visual inspection (no measurement); and interestingly, when water samples were taken it turned out that their visual assessment was quite in line with the indicated levels (Table 2.4). Frequency and duration of interruptions are low, thus not affecting the continuity of the overall system.

The total time for all maintenance activities was observed and divided by the surface area of the unit (operator hm^{-2}), to be able to compare systems. Maintenance time is highest in Golondrinas and La Sirena, mainly as a result of low drainage velocity during surface cleaning.

Table 2.4. Summary of operating, monitoring and maintenance conditions

	Treatment plant			
Variable	**Arroyo-hondo**	**El Retiro**	**La Sirena**	**Golon-drinas**
Operational parameters				
Maximum turbidity (NTU) at inlet (before closing)	30	20	50	60
Filter run (d)	7	7	7	7
Operation velocity(mh^{-1})	0.5-1.0	0.64	1.0	0.6
Years of operation	8	1	15	9
Monitoring parameters				
Number of interruptions per year	11	No	11	15
Maximum duration interruption (h)	6	No	3	4
Maximum head loss in UGF (m)	0.15	0.10	0.20	0.25
Head loss before weekly cleaning (m)	< 0.05	<0.05	<0.05	<0.05
Maintenance activities				
Required time (min)	59.8	137.2	116	168.9
Operator-hm^{-2}	0.083	0.082	0.109	0.122
Discharge method for surface cleaning	Weir and channel	Weir and channel	Orifice	Orifice

2.3.3. Water quality

Water quality monitoring is very limited and only concerns the end product (outflow SSF). In El Retiro *E-coli* is monitored daily. Monthly monitoring of the effluent of the SSF is done in El Retiro and Arroyohondo (measurements: turbidity, color, pH and *E-coli*). No monitoring is applied in the other two systems.

During the site visits, the water quality at the inlet and outlet of the UGF and the outlet of the SSF was additionally monitored for a period of two weeks (Table 2.5). The mean turbidity level of the effluent of the UGF was less than 5 NTU. In all cases, the turbidity of the effluent of the UGF was below 10 NTU, which is the guideline value of inflow water to the SSF units (Galvis *et al.* (1999), Di Bernardo & Sabogal (2008). The best turbidity removal was obtained in El Retiro. La Sirena showed the worst performance, possibly due to the high filtration velocity and the type of filter material (crushed gravel with a higher porosity, a larger shape factor (8.7) and lower sphericity (0.69), Di Bernardo & Sabogal (2008).

The best removal efficiency for *E. coli* was found in the UGF units in El Retiro and Golondrinas with 66 and 72%, respectively. These plants were operated with a relatively constant flow, following the guidelines. The other systems had larger flow variations and lower removal efficiencies.

Table 2.5. Water quality

Treatment plant	Stage	Parameters (statistics)								
		Turbidity (NTU)			*E-coli* (Log CFU(100 ml)$^{-1}$			Total coliform (Log CFU(100 ml)$^{-1}$		
		Av.	SD.	E %	Av.	SD.	Red.	Av.	SD.	Red.
La sirena	Influent	1.70	0.58		1.92	0.30		3.65	0.27	
	Eff. UGF	1.40	0.41	16	1.60	0.27	0.30	3.44	0.27	0.21
	Eff. SSF	0.26	0.06	80	0.0	0.0	1.60	1.16	0.78	2.28
El Retiro	Influent	4.01	3.16		2.70	0.40		3.89	0.19	
	Eff.UGF	1.70	0.99	55	2.23	0.50	0.47	3.61	0.30	0.28
	Eff. SSF	0.40	0.20	71	0.0	0.0	2.20	1.50	1.50	2.10
Arroyo-hondo	Influent	2.50	1.30		3.35	0.22		4.32	0.45	
	Eff. UGF	1.70	0.23	36	3.12	0.23	0.23	4.10	0.45	0.26
	Eff. SSF	0.18	0.06	89	0.0	0.0	2.60	0.85	0.37	3.21
Golondri-nas	Influent	5.70	2.60		1.92	0.16		2.80	0.51	
	Eff. UGF	3.70	1.30	40	1.37	0.21	0.55	2.49	0.54	0.31
	Eff. SSF	0.60	0.17	78	0.0	0.0	1.37	0.89	0.22	1.60

Eff: Effluent; Av: Average; SD: Standard deviation; E: Efficiency; Red: Reduction

2.3.4. Hydraulic behavior of UGF units

Table 2.6 summarizes the results of the tracer tests for each treatment plant. Results show that the UGFs corresponded to a "dual system" with plug flow and mixed flow while also presenting dead zones, which is consistent with the UGF behavior as reported by Galvis (1999).

The UGF in El Retiro had the best performance with the largest portion of plug flow and the lowest fraction of dead zones. The highest fraction of dead zones was found in the systems that do not have a weir (La Sirena and Golondrinas), which suggests that the limitations in surface cleaning had a negative effect on the hydraulic behavior. The dead zones suggest that some permanent accumulation of solids occurred in the gravel bed. This accumulation was more severe in the systems with limitations in surface cleaning.

Table 2.6. Results based on the analysis for the Wolf and Resnick and CMRS models

UGF Unit	Plug flow (%)	Mixed flow (%)	Dead Zone (%)	CMRS	Morril Index	RT (min)	Washing velocity (mh^{-1})	Weir
El Retiro (vf=0.6 mh^{-1})	48	50	2	10	2.5	94	5.4	Yes
La Sirena (vf=0.6 mh^{-1})	40	62	8	6-7	2.8	120	10.2	No
Arroyohondo (vf=0.6 mh^{-1})	37	60	3	7	2.8	59	10.4	Yes
Golondrinas (vf=0.6 mh^{-1})	20	65	15	3-4	4.5	65	5.4	No

RT: Residence Time

2.3.5. Cleaning behavior in UGF Units

The TSS concentration during drainage (Figure 2.2) showed four zones: 1) a first peak of TSS during a high washing velocity; 2) a low concentration of TSS during declining washing velocity; 3) a peak in TSS during low washing velocity; 4) a low concentration of TSS and a low flow. The first peak results from the high initial flow, which quickly dragged particles to the drainage system. Thereafter, the velocity reduced and fewer particles were dragged. The second peak is most likely the result of air being pulled into the gravel bed, which helped to disturb the particles that remained on top of the grains. Earlier reports on filter cleaning (Wolters, 1988; Cinara& IDRC, 1993) only reported the first peak. In Arroyohondo, two identical UGF units with the same operation time (7 days) and equal influent water quality were cleaned at the same time: one with shocks and the other only draining the filter. The behavior in terms of TSS removal was very similar in the two units, which suggests that shock loading, by quickly closing the drainage valve, did not have an effect on the TSS removal pattern (Figure 2.2d). This confirms the suggestion of Mataix (2004) and Collins et al (1994) that stirring of the deposits does not happen because the energy is dissipated by deformation of the pipe and by the viscosity of the water.

The highest washing velocities during UGF cleaning were obtained in Arroyohondo and La Sirena and these were in line with those reported by Wolters (1988) and Galvis (1999), but low in comparison to the range of 60 to 90 mh^{-1} found by Pardón (1989). For the other two systems, values were much lower, probably due to hydraulic limitations in the drainage system.

The effectiveness of filter bed cleaning was checked by analyzing the quantity of TSS removed during cleaning. Results were compared with turbidity data and showed to have a good linear correlation (TSS= 0.16 (turbidity) + 0.138; R^2 =0.93; n= 16). Based on this correlation, the TSS concentration in the drainage water for both drainage cycles was calculated using the data from Figures 2.2 and 2.3. Results of the accumulated removal are shown in Figure 2.4. Furthermore, the mass balance of TSS was established based on TSS values in influent and effluent for the same 7 days to calculate the total amount of solids retained in the filter over the filter run (dotted horizontal line in Figure 2.4).

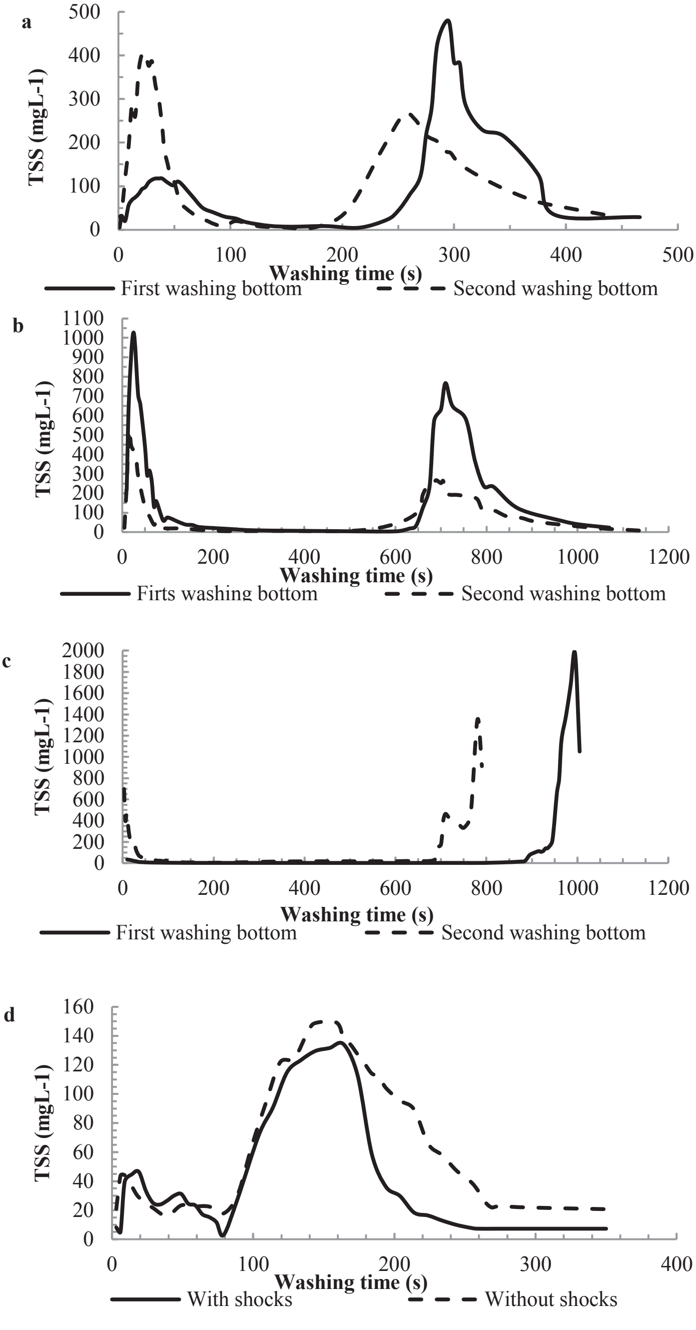

Figure 2.2. TSS behaviors on time during filter bed cleaning in UGF units. (a) UGF La Sirena, (b) UGF Golondrinas, (c) UGF El Retiro, (d) UGF Arroyohondo.

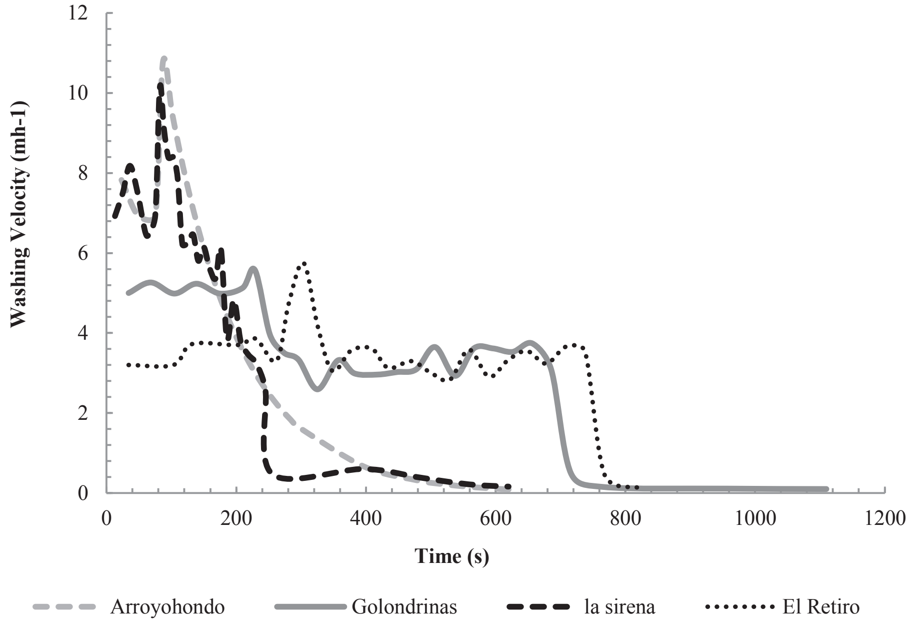

Figure 2.3. Washing velocities in UGF units

Figure 2.4 shows that on average, in all UGFs about 90% of the retained solids were removed by two drainage cycles. The other 10% is expected to be removed by surface cleaning as it has not been necessary to remove the gravel for washing; in three UGFs, the second filter drainage removed a larger quantity of solids than the first. A possible reason may be the relatively low TSS concentration in the influent (0.8 – 2 mgL^{-1}), resulting in a low volume of deposits in the filter.

In a way, the lower initial washing velocity may have facilitated the solids removal. Whereas the peak velocity was lower a higher velocity was sustained for a longer period (Figure 2.3), which contributed to solids removal over a longer period of time (Figures 2.4b and 2.4c).

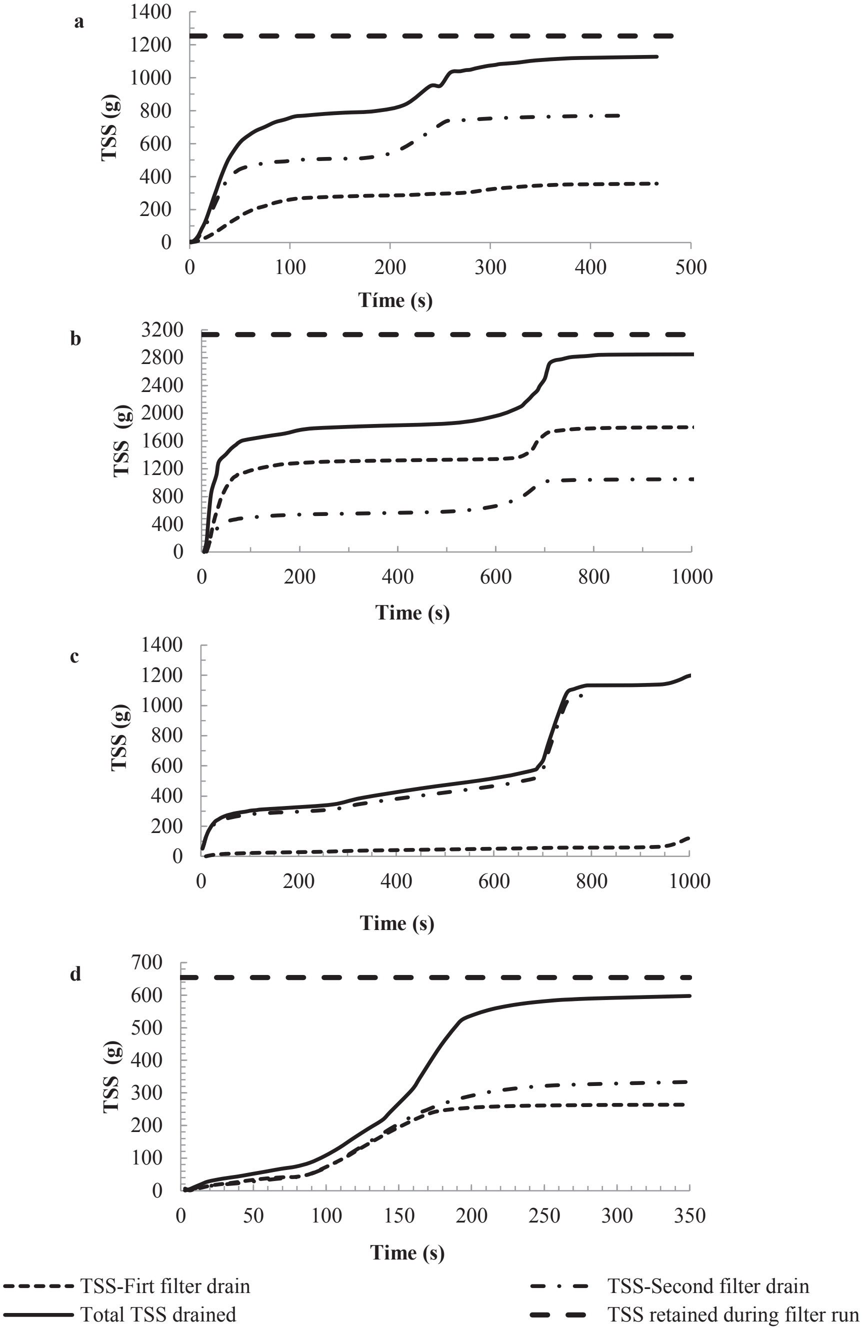

Figure 2.4. TSS evacuated during filter bed cleaning on time for each UGF. (a) UGF La Sirena (mean TSS 1.8 mgL^{-1}), (b) UGF Golondrinas (mean TSS 2.0 mgL^{-1}), (c) UGF El Retiro (mean TSS 0.8 mgL^{-1}), (d) UGF Arroyohondo (mean TSS 1.3 mgL^{-1}).

Whereas Pardón (1989) indicates the need for frequent cleaning to avoid permanent clogging of the filters, our findings related to weekly cleaning suggest that cleaning frequency can be even lower. The systems only developed a small head loss after one week (< 0.05 m). Furthermore, gravel was not removed for cleaning, in any of the UGFs because of advanced clogging, and one system had been in operation for 15 years. Hence, it is relevant to explore the cleaning cycles in more detail since reduced frequency reduces the work load of the operator, and reduces water loss, which may be particularly relevant in pumped systems. Longer periods between cleaning may also have a positive effect on treatment efficiency by allowing more biomass development in the filters as was described in chapter 1

2.4. Conclusions

This chapter presents the results of a study of four full-scale UGFs that are part of full-scale MSF systems. The study explored the design criteria that were applied, the way O&M procedures are carried out, and the performance of the systems, including filter bed cleaning. This study shows that in general, the design characteristics of the systems follow the literature with the exception of the drainage system and flow velocities; in two cases this resulted in lower washing velocities than recommended in the literature. Performance data showed that removal efficiencies were on the low side when compared to the literature, possibly because of the good quality influent water that was treated. Head loss and flow measurement are not possible in the systems due to the lack of measurement tools in the UGFs. A weir should be included in the design criteria of UGFs to facilitate water drainage during surface cleaning. Operators follow, to a fair extent, the recommended O&M procedures but they do not: take samples to monitor water quality, measure head loss, or control the flow velocity. Based on the first observations shock loads did not influence cleaning efficiency of the lowly loaded filters, implying that this practice can be replaced by just twice draining the UGFs, thus facilitating the work of the operator. Head loss build up in one week was low, suggesting that fewer cleaning cycles may be needed, but more controlled studies are necessary to improve the understanding of this cleaning method. Results show that the procedures applied for filter bed cleaning are effective despite some limitations found in the drainage systems and low washing velocity. About 90% of the retained solids were removed in two drainage cycles; the remaining 10% is probably removed during surface cleaning of the gravel bed. Adjustment of the design criteria and O&M procedures is needed to enhance system performance. This includes drainage system design, surface cleaning by weir, and filter bed cleaning to allow a reduction in cleaning cycles and to improve operation control.

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CHAPTER 3

Low-cost multi-stage filtration enhanced by coagulation-flocculation in upflow gravel filtration*

This chapter assesses the operational and design aspects of coagulation and flocculation in upflow gravel filters (CF-UGF) in a multi-stage filtration (MSF) plant. This study shows that CF-UGF units improve the performance of MSF considerably, when the system operates with turbidity above 30 NTU. It strongly reduces the load of particulate material before the water enters in the slow sand filters (SSF) and therewith avoids short filter runs and prevents early interruption in SSF operations. The removal efficiency of turbidity in the CF-UGF with coagulant was between 85 and 96%, whereas the average efficiency without coagulant dosing was 46% (range: 21-76%). Operating with coagulant also improves the removal efficiency for total coliforms, *E-coli* and HPC. No reduction was observed in the microbial activity of the SSF, no obstruction of the SSF bed was demonstrated and SSF runs were maintained between 50 and 70 days for a maximum head loss of 0.70 m. The most important advantage is the flexibility of the system to operate with and without coagulant according to the influent turbidity. It was only necessary for 20% of the time to operate with the coagulant. The CF-UGF unit represented 7% of total construction costs and the O&M cost for the use of coagulant represented only 0.3%.

*This chapter is based on:

Sánchez, L. D., Marin, L. M., Visscher J. T. & Rietveld, L. C. 2012. Low-cost multi-stage filtration enhanced by coagulation-flocculation in upflow gravel filtration. Drink Water Eng. Sci., 5, 73–85.

3.1. Introduction

Water quality and quantity from surface sources are changing due to the deterioration of watersheds caused by deforestation, erosion, and the discharge of untreated wastewater (e.g., in Colombia only 3.1% of the total volume of wastewater produced is treated (CGR, 2009)). These changes are intensified by the global climate change causing longer dry periods on the one hand and more intense rainfall on the other (Bates et al., 2008). The two most serious problems concern the peaks in turbidity level and high *E-coli* concentrations of long duration. These changes are affecting the existing water treatment plants, causing higher operation and maintenance (O&M) requirements and even interruptions in their operation (Bates et al., 2008). These growing water quality problems are not unique for Colombia but imply a significant challenge in the Andean region, because surface water is the main source in the water supply systems. In Colombia about 80% of the water supply systems are based on surface water supply (Ministerio de Desarrollo, 1998). The problems particularly affect water supply systems in rural areas and small towns, many of which even lack adequate water treatment.

Multi-Stage Filtration (MSF) is one of the more promising and reliable water treatment options for small communities. This technology uses a combination of gravel filtration (GF) in combination with slow sand filtration (SSF). Upflow gravel filtration (UGF) is the most common pre-treatment system used for MSF in Colombia (Sanchez et al., 2006a). In UGF the water passes through the gravel bed from bottom to top. During this passage impurities are retained in the filter. Upflow filtration has the advantage that the heavier particles are removed first at the bottom of the filter. Burganos et al. (1994) have reported that upflow units have an increased collection efficiency at small and medium pore inclination angles. This concept is interesting for the theoretical analysis of particle motion and deposition, but complex to manage in practice. When the time comes to clean the filters, the accumulated particles can be removed by opening the drainage valve, allowing gravity flow to drain and clean the filter.

Research carried out by Cinara over more than 15 years showed that different MSF alternatives that were tested, including UGF (filtration rate, $vf = 0.6$ mh^{-1}) and SSF (operating at 0.15 mh^{-1}), were able to produce effluents with a low microbial risk (Galvis, 1999). Research also explored the use of coagulation and flocculation with UGF, called CF-UGF, focusing, in pilot plants, on the combination with rapid filters (RF). The results showed a reduction in the consumption of coagulants by up to 30% compared to the conventional system of coagulation, flocculation and high rate sedimentation (Cinara and IRC, 1996).

Other research into CF-UGF has emphasized the laboratory variables and the removal efficiencies. Richter and Moreira (1981) reported that a flocculation time of 3-5 min in a UGF is equivalent to a time of 15 min in the jar test under laboratory conditions and 25 min in non-compartmentalized flocculation units in full-scale plants. Santamaría (1999) showed that, using UGF, the flocculation time can be reduced by up to 60% compared to mechanical flocculators; Salazar and Ocampo (1999) found that in CF-UGF, producing the same water quality would require between 10 and 20 times less retention time compared to a sludge blanket clarifier; Kawamura (1985), working on pilot units with UGF and RF, reported a turbidity removal of 50%; Ahsan (1995) found that horizontal gravel filtration (HGF) with coagulation removes more particles compared to the HGF without chemicals.

In addition, studies with different packed gravel beds found that a stratified bed is more efficient than a uniform bed (Attakoya et al., 1991).

Di Bernardo and Sabogal (2009) have further refined some of the parameters for design and operation and maintenance, but have only applied these at pilot scale. The use of CF-UGF with MSF has had few full-scale applications. Full-scale experiences using UGF based on conventional technology with rapid filtration as a final stage were reported by Kardile (1981), working with a *vf* = 4-10 mh^{-1} and turbidity levels between 300-500 NTU, achieving construction costs between 30% to 50% less than conventional systems of equal capacity. Bohle (1981) reported a velocity gradient in a truncated pyramid filter of $G = 1230\ s^{-1}$ at the bottom layer and 35 s^{-1} at the upper layer with a *vf* = 11.3 mh^{-1}. More recent studies indicated that aluminium residual has not shown any effects on biological activity in SSF, when coagulation with aluminium sulphate has been used, Dorea and Clarke (2006). This indicates that the addition of coagulant with UGF has potential to improve the performance of MSF during variations in influent water quality, conserving biological processes in the following stages. Consequently full-scale evaluations are necessary to better understand the design variables and operation and maintenance conditions.

The community of Colinas de Arroyo Hondo, located in a rural area of Yumbo municipality, Colombia, had a treatment plant functioning with the processes of coagulation, flocculation, sedimentation, rapid filtration and disinfection with chlorine and ultraviolet light. However, in field studies on water quality in the distribution network, biofilms were found in the pipes, which generated problems of re-growth of micro-organisms in the water supplied to the users (average values of 2183 CFU $(100\ ml)^{-1}$ for heterotrophic bacteria, 7 CFU $(100\ ml)^{-1}$ for *E-coli,* and 39 CFU $(100\ ml)^{-1}$ for total coliforms were also found at four points in the distribution network), (Sánchez et al., 2006b). In the treatment plant, which had been operating for 4 years, failures were identified in its functioning, allowing solids and micro-organisms to pass into the distribution system. After an investigation, the treatment system was redesigned and rebuilt to CF-UGF with MSF. This new system has been in operation for 7 years at the time that this evaluation study was carried out.

This chapter assesses the performance and the design aspects of the CF-UGF units followed by UGF and SSF, defining influent turbidity levels to allow operation with and without coagulant, taking advantage of the SSF as the final stage of filtration for the removal of microorganisms.

3.2. Materials and methods

3.2.1. Set-up of the treatment system

In Figure. 3.1 the set-up of the CF-UGF MSF plant is shown. The system operation is performed by pumping raw water to the plant and then, after treatment, pumping it into the distribution network. The system uses two pumping flow rates: 6 Ls^{-1}, which is the maximum capacity for 12 hours during the day and 3 Ls^{-1} for another 12 hours at night. The treatment plant consists of 5 components. Raw water passes through the flow control unit in the dynamic gravel filter (DyGF), whose main function is to protect the next steps from excessive loads of suspended solids and turbidity. Filtered water flows into a rapid mixing chamber (RMC), where the coagulants are dosed. Afterwards, the water enters the CF-UGF stage which consists of 2 units in parallel, where the processes of coagulation, flocculation, sedimentation and filtration of destabilized particles occur. Water is collected in a front weir to later enter the UGF where remaining flocculated particles are removed in the different layers of gravel. From there, the water enters the SSF stage for final removal of suspended particles and microorganisms.

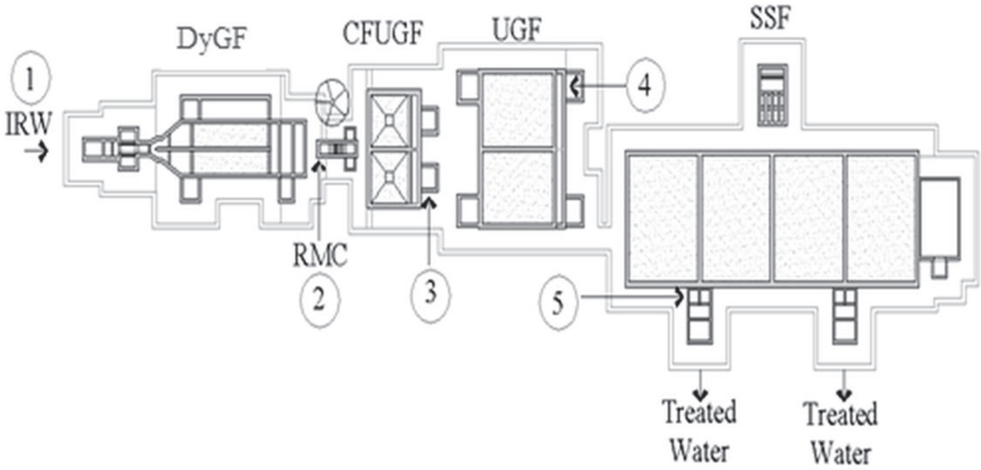


Figure 3.1. Layout of the treatment system (1-5: monitoring points of water quality)

The RMC unit allows a proper rapid mixing time and a velocity gradient, while the CF-UGF unit facilitates flocculation and deposition of particles. The hydraulic RMC operates through a rectangular weir that allows free flow of water to form a hydraulic jump for mixing the coagulant with the raw water. Above the RMC a channel provided with holes is installed to distribute the coagulant and initiate the destabilization of the particles, which will be removed in the CF-UGF and the UGF. The RMC is divided into 3 compartments, one for arrival and energy dissipation, one for mixing and one for the outlet of the coagulated water to the CF-UGF.

Table 3.1 lists the parameters considered for the design of the unit, the formulae and the values obtained during the operation, while Figure. 3.2 shows the layout of the CF-UGF unit.

Table 3.1. Parameters and formulae

Parameter	Formulae	Value
RMC		
Length to the mixing point	$L_m = 1 + 0.5h_v(P_v/h_v)^{0.54}$ (3.1)	0.17 m
Velocity gradient for rapid mixing	$G = (\gamma E_p/\mu T_m)^{0.5}$ (3.2)	1282 s^{-1}
Average mixing time (Tm)	$T_m = L_r/V_{ap}$ (3.3)	0.31 s
CF-UGF		
Average velocity gradient CF-UGF (Di Bernardo and Sabogal, 2009).	$G = \sqrt{\frac{(\gamma)(V_{ap})(J_{mg})}{(\mu)(\epsilon_o)}}$ (3.4)	28.5–3.1 s^{-1}
Unit head loss in the porous medium (Ergun, 1952).	$J = \frac{150v(1-\epsilon_o)^2(V_{ap})}{g(\epsilon_o)^3(C_s)(D_{mg})^2} + \frac{1.75(1-\epsilon_o)(V_{ap})^2}{g(\epsilon_o)^3(C_s)(D_{mg})}$ (3.5)	0.0016- 0.0009 mm^{-1}
Reynolds number in the porous medium, (Dinoy, 1971).	$R = \frac{\rho x V_f x k^{1/2}}{\mu}$ (3.6)	732-41

Where: L_r= hydraulic jump length; L_m = length between the base of the weir and the mixing point (m); h_v= table of water over the weir (m); P_v= height of water from the base of the weir and fill up sheet of water (m); E_p= loss of energy in the channel unit coagulation, (m); T_m = average time of mixing (s); γ= specific weight of water (N m^{-1}); μ= absolute viscosity (N m s^{-2}); V_{ap} = approach velocity (m s^{-1}); ε_o = porosity of clean filter; J = loss of unit load (m m^{-1}), v = kinematic viscosity (m^2 s^{-1}), g = gravity constant (m s^{-2}), C_s= coefficient of sphericity; D_{mg} = average grain size, ρ = density of water (kg m^{-3}), R_e = Reynolds number, k = permeability (cm^{-1}).

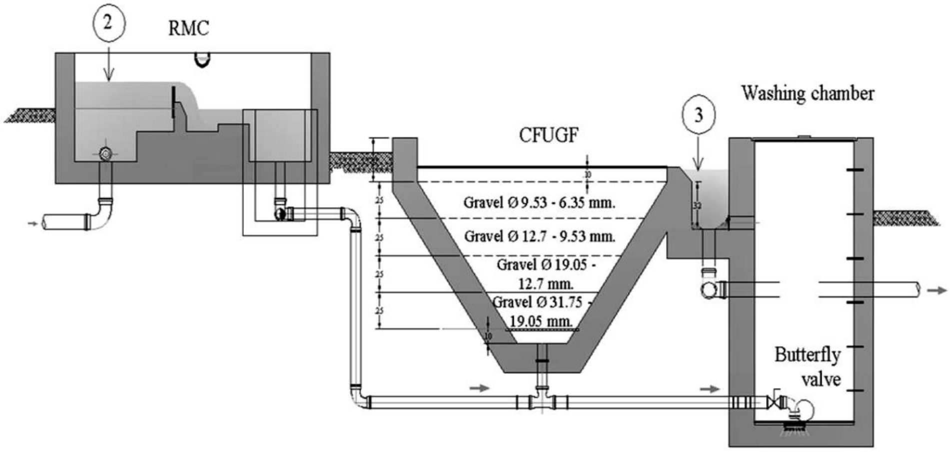


Figure 3.2. CF-UGF Unit Scheme (2-3: monitoring points of water quality)

The CF-UGF step consists of 2 units in parallel and forms the third component in the treatment scheme. The system has 4 layers of gravel bed and was designed as a truncated pyramid to facilitate variation in the velocity gradient, producing a variable gradient from the highest to the lowest value from the bottom to the surface of the unit. In Table 3.2 the values of the velocity gradient for two flow operations are listed, depending on the properties of the filter bed and the average fluid velocity in each gravel layer, the cross-sectional area of the filter and the head loss in the bed. The calculation of the velocity gradient was done by Eq. 3.4 (see Table 3.1) and the head loss by Eq. 3.5 proposed by Ergun (1952), which is valid for any flow regime as long as the bed is not fluidized (Di Bernardo and Sabogal, 2009).

Table 3.2. Velocity gradients in the CF-UGF for the 2 flows of operation

Filter media size mm	ε_o	D_{mg} (m)	Flow 3 Ls^{-1}				Flow 6 Ls^{-1}			
			V_{ap} (ms^{-1})	J (mm^{-1})	G s^{-1}	R_e	V_{ap} (ms^{-1})	J (mm^{-1})	G s^{-1}	R_e
31.7-19.0	0.31	0.0317	0.00375	0.0016	14.2	366	0.0075	0.0032	28.5	732
19.0-12.7	0.38	0.0191	0.0014	0.0007	5.3	124	0.0029	0.0015	11	257
12.7 - 9.5	0.40	0.0111	0.00075	0.0009	4.2	43	0.0015	0.0019	8.5	86
9.5 - 6.3	0.42	0.0079	0.00045	0.0009	3.1	20	0.0009	0.0018	6.3	41

T = 22.8 °C, γ = 9,737 Nm^{-1}, μ = 9.44 E-04 N ms^{-2};ν= 9.47 E-07 m^2s^{-1}, C_s = 0.81

The flocculation gradient is greater at the bottom of the bed, decreasing towards the top of the filter, basically to promote the formation of flocs (Fair et al., 1984). It should be noted that the velocity gradient values were lower than those reported by Ahsan (1995) (200-300 s^{-1}) because the filtration rate in CF- UGF was lower. The Reynolds numbers indicate that the units work in the hydraulic transition regime; values obtained above 10 indicate a stable inertial regime for flocculation in the porous medium, as reported by Wright (1968)**.** For the calculation of the head loss in the filter the Ergun equation was used.

The calculations for each barrier of the treatment system were done on the basis of the design parameters presented in Table 3.3 and the dimensions of each barrier in Table 3.4.

Table 3.3. Treatment barriers and design parameters

Criteria	Treatment Stage				
	DyGF	RMC	CF-UGF	UGF	SSF
Design period (years)	15	15	15	15	15
Operation time (h)	24	24	24	24	24
Number of units in parallel	2	1	2	2	4
Flow per unit (Ls^{-1})	3	6	3	3	1.5
Filtration rate (mh^{-1})	3		3.2-27	1	0.30
Initial washing velocity (mh^{-1})	20		20	20	
Area by unit (m^2)	3.6		4	10.8	18
Gravel					
Length (m)	0.6		1	1.1	0.2
Size (mm)	25.4-3.2		19 -6.3	25.4-3.2	12.5
Sand					
Length (m)	----	----	----	----	0.85
d_{10} (mm)					0.15-0.35
C_u.	----	----	----	----	2-3.5

Table 3.4. Dimensions of each barrier

Treatment barrier	Number of units	Dimensions			Flow (Ls^{-1})	Material structure
		L (m)	*A* (m)	*H* (m)		
Input chamber	1	5.15	0.6	0.7	6.0	Concrete
DyGF	2	4.0	0.9	0.8	3.0	Concrete
RMC	1	2.2	0.4	0.5-0.7	6.0	Concrete
CF-UGF *	2	2.0	2.0	1.4	3.0	Concrete
UGF	2	3.8	2.84	1.3	3.0	Concrete
SSF	4	3.7	4.7	1.75	1.5	Concrete

L: length; *A*: width; *H*: depth; * Bottom Area: 0.16 m^2 and Surface Area: 4.0 m^2

3.2.2. Monitoring water quality and system operation

During the study, the raw water was monitored for the following parameters: turbidity, true colour, *E- coli*, total coliforms, heterotrophic bacteria plate count (HPC) and pH. The water quality parameters and related methods are listed in Table 3.5. Measurements of the head loss (h_f) over the filter bed were done for short periods of operation (4-6 hours), when coagulants were applied. However, when operating without coagulant, daily measurements were done during periods of 8 days, according to the schedule defined by the plant operators for cleaning the CF-UGF and UGF.

This is because with time large quantities of solids will accumulate within the gravel bed and then filter resistance will increase gradually and the water level within the inlet chamber will rise to a maximum h_f. To facilitate the measurements of h_f, steel rules were installed in the inlet chamber, taking care that the zero of the rule coincides with the water level in the chamber for the no-flow condition.

Table 3.5. Water quality parameters and analysis method

Parameters	Method	Limit of detection
Total coliform	9222B, filtration x membrane	0
E coli	9222B, filtration x membrane	0
HPC	9215A, discharge in plate	0
Turbidity	2130B	0.1
Aluminium	3500-Al B	0.03
pH	4500 H^+	2
True colour	2120C	1
TOC	5310 B	0.18

(APHA, AWWA, WPCF, 2005)

The dose of coagulant for operation of the CF-UGF system was previously defined according to studies developed by Cinara (2004). In these studies two types of jar tests were carried out: a) to define basic parameter such as rapid mixing time, gradient of rapid mixing, slow gradient mixing, slow mixing time, and sedimentation time; the rapid mixing intensity and the slow mixing during the jar test were expressed in the velocity gradient *G* (s^{-1}) following the method described in CEPIS (2004) and were used for a first approximation of the velocity gradient in the CF-UGF and b) one second set of jar tests were done with raw water of the source to define the optimal dose of coagulant (turbidity up to 100 NTU), following the method presented by Di Bernardo and Sabogal (2009), where rapid mixing is done using jar test equipment with 2-liter glass jars. The coagulant used was 50% liquid aluminium sulphate diluted with water to 2%, and was added for a period of 60 seconds at a velocity gradient greater than 300 rpm. After rapid mixing, water was extracted and filtered in a funnel containing filter paper Whatman 40 (pore size 8 μm), to obtain a volume sufficient to realize analyses.

Tracer tests were performed according to the methodology described by Pérez and Galvis (1990), in order to understand the hydraulic performance of the CF-UGF and UGF units. Trials were conducted following the experimental stimulus-response method, in which a tracer, easily detectable and not involved in any of the physical and chemical processes that may alter the actual fluid hydrodynamics and with a known concentration, is injected into the influent (Rocha et al., 2000). The concentration curve of the tracer was analysed to determine the portion of plug flow, dead zones, and the fraction that works as a completely mixed flow. These tests were conducted using sodium chloride. The substance was dosed continuously through a constant hydraulic head dispenser with a sodium chloride concentration between 50-100 mgL^{-1}. The dosing period was three times the theoretical retention time of each unit and the response was measured at the output of each unit by means of electrical conductivity. Measurements were taken after the CF-UGF unit every 2 min.

Results were analysed with the mathematical model Wolf-Resnick, the Morril Index (relationship between the time between the 90% and the 10% passage of the tracer), and the model of completely mixed reactors in series (CMRS), (Pérez and Galvis (1990). Wolf-Resnick model Eq. 3.7 indicates that by plotting the fraction of tracer remaining in the filter ($1 - F(t)$) versus tt_o^{-1} (relationship between measured time and the theoretical retention time), it is possible to estimate the values θ and tanα (the slope of the straight line), and to identify the characteristics of the reactor using Eqs. 3.8 to 3.11. CMRS model was analyzed using Eq. 3.12, where n is the number of reactors in series and CC_o^{-1}, is the relationship between the concentration of tracer which remains in the reactor at a time and the concentration of tracer applied.

$$Log\,1 - F(t) = -\,tan\alpha\,[\,(t/to) - p\,(1-m)] \qquad (3.7)$$
$$Tan\alpha = (0.434 * p)\,/\,(\theta * (1-p)) \qquad (3.8)$$
$$\text{Plug flow } (\theta)\text{: } \theta = p\,(1-m) \qquad (3.9)$$
$$\text{Dead zones } (m)\text{: } m = 1 - (\emptyset/p) \qquad (3.10)$$
$$\text{Mixed flow } (Mf)\text{: } Mf = (1-p)\,(1-m) \qquad (3.11)$$
$$\text{CMRS model: } \frac{C}{Co} = n\left[\frac{(n*\frac{t}{to})^{n-1}}{(n-1)!}\right]\left(e^{-n*\frac{t}{to}}\right) \qquad (3.12)$$

The porosity (ε_0) of the filter material was determined following the procedure defined by Ives (1990): first, the mass (M) occupied by the sample of gravel in a container of known volume and the apparent volume (V) occupied by the gravel in the container were both measured; then the density (ρ_s) of the gravel was determined by the ratio between the mass of the sample and the volume occupied by the sample. The porosity was calculated by Eq. 3.13.

$$\varepsilon_0 = 1 - M/\rho_s V \qquad (3.13)$$

The filter cleaning procedure was the following: the water inlet to the unit was interrupted; without removing the supernatant water the surface of the gravel bed was cleaned manually with a shovel, stirring the surface layer of the filter to remove solid material adhering to the gravel; the supernatant water with the removed deposits from the top of the filter bed was discharged through a front weir; then the filter was prepared for removal of the deposits in the filter by adding water to the unit by opening the flow control valve in the inlet chamber to restore the supernatant water layer and increasing its height to a level of water between 20-25 cm above the gravel in the main compartment; this increased height adds some pressure for the cleaning procedure, in which fast drainage was carried out by quickly opening and closing (some 10 times) the butterfly valve on the underdrains; the unit was thereafter refilled with water and the cleaning and drainage procedure was repeated for adequate cleaning of gravel; then the filter was put back into operation.

The flow rate (Q) of the draining procedure in the UGF units was determined by the following procedure: a) the unit was filled to the maximum level of the top; b) the water flow into the filter was interrupted; c) the surface area was measured (A); d) the butterfly valve was opened until the water level lowered by 5 cm; e) the declining water level (Δh) was measured over time *(t)* and f) the initial flow rate was set by the expression $Q = \Delta h * A/t$ (m^3s^{-1}).

3.3. Results and discussion

3.3.1. Water quality of the source water

The water source is a small mountain river (the Arroyo Hondo River) which drains an area that has problems of deforestation and erosion, strong activity in the basin of rocky material exploitation for construction and discharge of untreated wastewater. The behaviour of microbiological parameters over a 3-month period indicated that the source has, according to Lloyd and Helmer (1991) and WHO (2011), a high microbiological risk, because faecal coliform values at all times exceeded 1000 CFU $(100\ ml)^{-1}$. *E-coli* bacteria were below 4.2 logs CFU $(100\ ml)^{-1}$ for 95% of the time, but were never less than 3.2 logs CFU $(100\ ml)^{-1}$. The average values of HPC and total coliforms were 5.3 logs CFU $(100\ ml)^{-1}$ and 5.1 logs CFU $(100\ ml)^{-1}$, respectively.

The behaviour of turbidity in the source was measured for a period of one year. This parameter, which is easy to measure, is a good indicator to control the system and facilitates decision-making by the operator. The results indicated that the turbidity in the source did not exceed 100 NTU for 97% of the time, while the turbidity was lower than 25 NTU for 75% of the time. Minimum values of 3 NTU were recorded during the summer period and maximum values of 350 NTU during the rainy season. Turbidity peaks were of short duration (4-6 hours) but sometimes lasted up to 24 hours. For 95% of the time the true colour level in the river was below 25 UPC and at no time the level dropped below 5 UPC.

3.3.2. Coagulant dosage

The basic parameters defined for the operation with coagulant were as follows (Cinara, 2004): rapid mixing time 60 s, gradient of rapid mixing 300 rpm (G=280 s^{-1}), slow gradient mixing of 60 rpm (G=28 s^{-1}), slow mixing time 25 min, and sedimentation time of 20 min.

Figure 3.3 presents the coagulant dose of aluminium sulphate for different turbidity levels, applied to the operation with coagulant based on previous studies developed by Cinara (2004). The dosage behaves as a logarithmic function and a little variation in the dose of coagulant is presented for affluent turbidities between 60-100 NTU. The optimum pH was in the range of 6.6-7.6 for an alkalinity between 59-133 mgL^{-1} $CaCO_3$.

The pH range of the water source has facilitated an efficient and low cost operation because only very small changes occurred during the coagulation- flocculation process, thus avoiding the need for pH adjustment. The dosing conditions as shown in Figure 3.3 are clearly within the range suggested by CEPIS (2004), of 3-30 mgL^{-1} of aluminum sulfate and a pH close to 7, which suggest that prevailing coagulation mechanism is due to charge neutralization of the aluminium hydroxide. Dosing of aluminum sulphate is carried out by a dispenser at the point of greatest turbulence in the RMC; the concentration of solution of aluminum sulphate was 2%, which is in line with the recommendation of CEPIS (2004) which suggests a coagulant concentration between 1-2% for water treatment plants. This level of concentration in combination with sufficient turbulence, allows for a good coagulant dispersion which facilitates its coming into quick contact with a large number of particles (Di Bernardo and Sabogal, 2009).

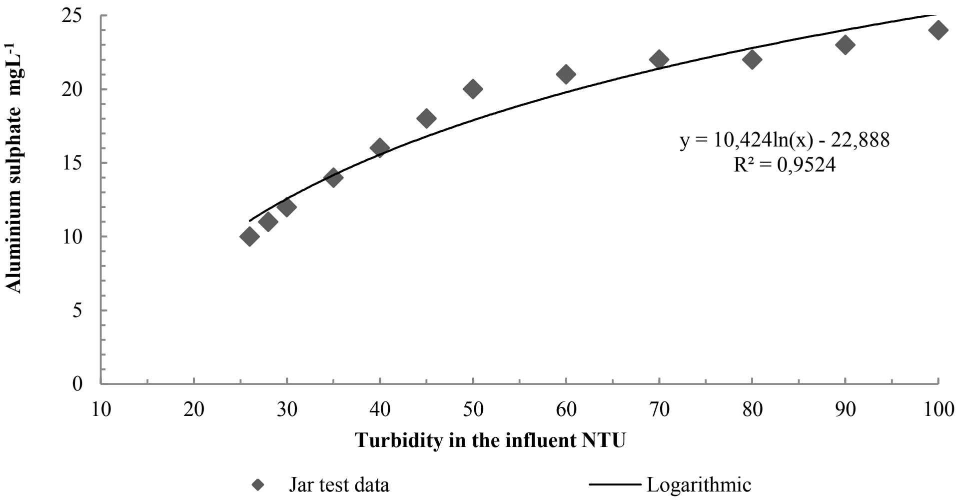
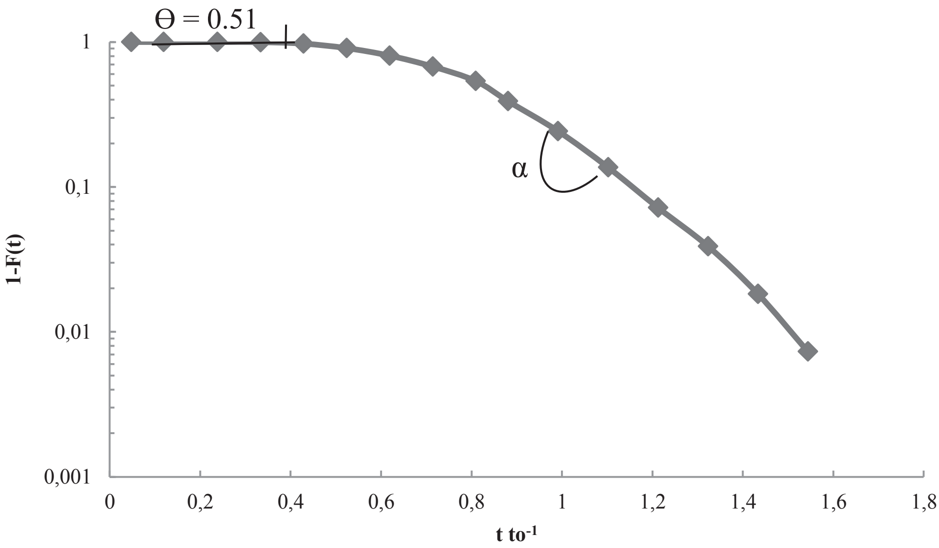


Figure 3.3. Aluminium sulphate dose as a function of influent turbidity in CF-UGF Unit.

3.3.3. Hydraulic behaviour of CF-UGF units

Figure 3.4 shows the results of the mathematical model Wolf-Resnick. The model results for CF-UGF indicate that the system worked with a plug flow fraction of 51%, a mixed fraction of 46% and a dead zone fraction of 3% (r^2 = 0.90). Taking into account the dead zones in the CF-UGF, the velocity gradient in the unit varied between 3.2 and 29.4 s^{-1}, which is close to the value calculated in Table 3.2 and obtained in the jar test by Cinara (2004) to define the coagulant dose. The Morrill Index (MI) was 1.82, which suggests, according to experiments by Perez and Galvis (1990), the presence of plug and mixed flow in the CF-UGF unit. Figure 3.5 presents the results of the CMRS model. The continuous lines show the hydraulic behaviour with *n* reactors in series, while the dotted line represents the measurements at the CF-UGF unit. When comparing the results of the theoretical model with experimental data, the hydraulic behaviour of the reactor CF-UGF tends to *n* =6 reactors in series (see continuous black curve, Figure 3.5), confirming the presence of a relative plug flow.



Log 1- F(t) = -0.95 (t/to) + 1.24 (r^2 = 0,902)
Tanα= 0.95; ϴ = 0.51; p= 0.53; m = 0.03; Mf = 0.46

Figure 3.4. Hydraulic Wolf-Resnick model results for the CF-UGF

The mean residence time (tm) was estimated from the experimental curve, determining the centroid under the curve, the mean residence time for the CF-UGF was 19.7 min for the flow of 3 l s^{-1} while theoretical retention time was 19 min. These differences may be explained by the presence of dead zones and implies that the curve of distribution residence time has a tail and therefore the time will be displaced in the time axis (Figure. 3.5), the fluid elements that were trapped in the dead zones is conducted very slowly and will have a much larger residence time.

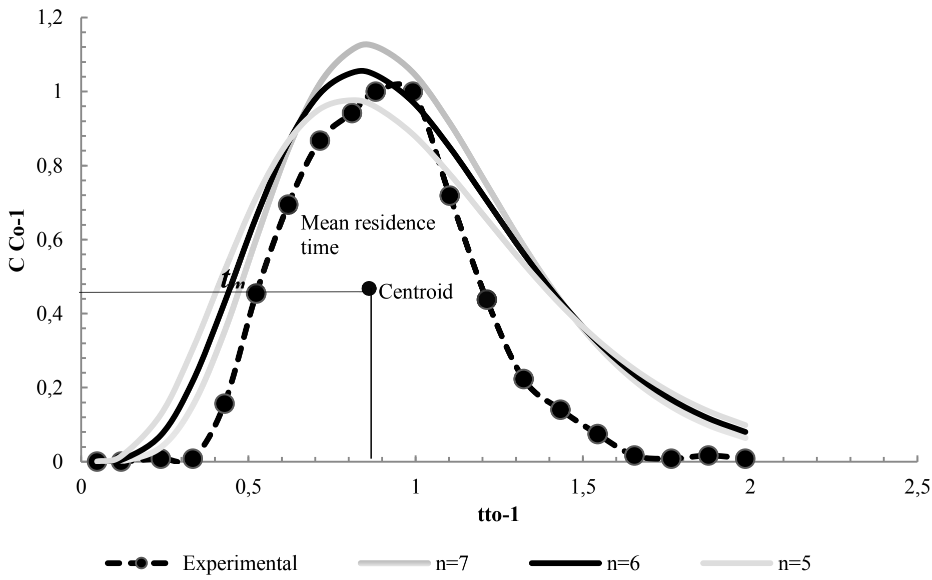


Figure 3.5. CMRS model for CF-UGF

3.3.4. Removal of turbidity in the operation without coagulant

The operation of the CF-UGF without coagulant dosing was used in dry periods. Figure 3. 6 shows the frequency of turbidity in raw water and after different treatment barriers, including DyGF, CF-UGF, UGF and SSF. The CF-UGF units produced an effluent between 2.5-7.5 NTU, with mean removal efficiency of 46%, while for 78% of the time the UGF showed turbidity levels lower than 1 NTU and another 22% were between 1.5- 2 NTU. SSF units processed water with 0.3 NTU for 98% of the time. CF-UGF and UGF always produced water with turbidity levels below 10 NTU, which is the guideline value of inflow water to the SSF units, according to Di Bernardo (1993) and to Galvis et al. (1999) who add the requirement that filtration rates should be lower than 0.20 mh^{-1} in SSF units. In this case it was a little higher because the system operated at filtration velocities of between 0.15 and 0.30 mh^{-1} (3-6 Ls^{-1}).

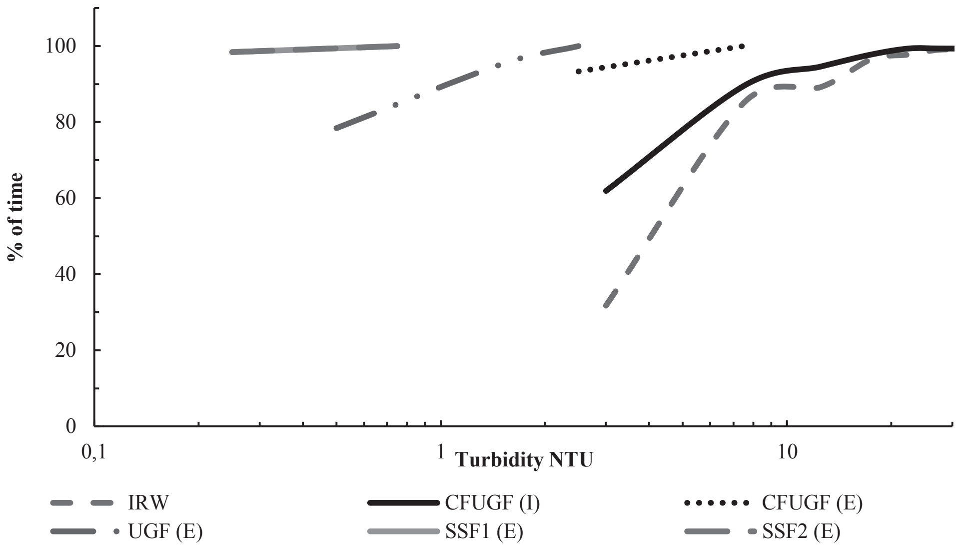


Figure 3.6. Frequency of turbidity levels (operation without coagulant)

3.3.5. Removal of turbidity in the operation with coagulant

Figure 3.7 shows the frequency of turbidity in raw water and different treatment barriers, when the CF-UGF is operated with coagulants. The application of coagulant in the RMC varied as indicated in Figure. 3.3. In the effluent CF-UGF units, for 97.2% of the time turbidity levels lower than 10 NTU were achieved, while the maximum value of turbidity in the effluent in this step ranged between 15-20 NTU for 1.4% of the time. The UGF showed turbidity levels lower than 6 NTU for 100% of the time, thereby facilitating the operation of the SSF, which produced water with turbidity below 1 NTU for 98% of the time. The addition of coagulant in the CF-UGF enabled water with turbidity levels below 10 NTU after UGF to be obtained. Increments in turbidity levels, which occur basically in the rainy season, could be managed in the treatment plant by the CF-UGF unit, which contributed to an effective operating system, preventing reductions in the SSF filtration runs. Additionally, stops in the operation of the treatment plant were prevented.

The removal efficiency of turbidity in the CF-UGF with coagulant was between 85 and 96%, which is higher compared to operating without coagulant, and average efficiency of turbidity in the CF-UGF was 46%, ranging between 21-76%. The removal efficiencies in CF-UGF with velocity gradients between 28.5- 3.1 s^{-1} (for 3 and 6 Ls^{-1}) was consistent with the findings of Di Bernardo and Sabogal (2009), who established that the gradient must be less than 100 s^{-1}. The efficiency in the removal of turbidity can be explained by the good opportunities for contact with particles in the gravel bed, which is consistent with Richter (1987), Attakoya et al. (1991), and Di Bernardo and Cruz (1994).

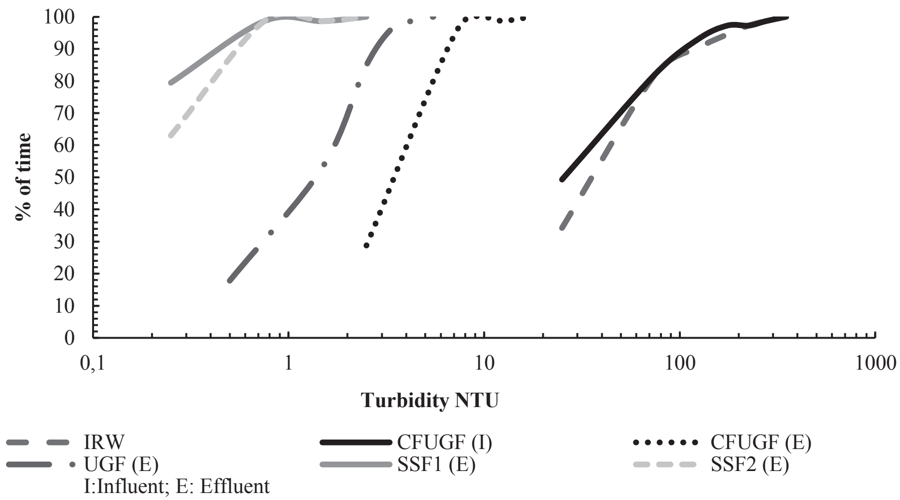


Figure 3.7. Frequency of turbidity levels (operation with coagulant)

3.3.6. Removal of microorganisms in the operation without coagulant

When operating without aluminium sulphate, the duration of the filter run was 8 days in the CF-UGF and UGF, regardless whether it achieved the maximum head loss or not (normally head loss is less than 5 cm in 8 days, to a maximum value of 15 cm). The typical removal of microorganisms for a filter run is presented in Table 3.6 for samples taken in the influent of CF-UGF unit and the effluent of the UGF.

The removal efficiency in the CF-UGF and UGF steps was 0.16 logs for total coliforms, 0.16 logs for *E-coli* and 0.17 logs for HPC, lower than that reported by Galvis et al. (1999), which was probably due the effect of a higher filtration rate. SSF achieved a total reduction of *E-coli* of 3.2 log units, facilitating the work of chlorination as a security barrier. These results are consistent with the WHO (2011) which indicated that the range of log removal of bacteria for SSF must be between 2-6 under presence of schmutzdecke and appropriate: grain size, flow rate, operating conditions (mainly temperature, pH), cleaning and refilling and in the absence of short circuiting.

The reduction of the filtration velocity in the UGF from 0.9 mh^{-1} to 0.5 m h^{-1} contributed to improved efficiency in the removal of microorganisms. The average removal for total coliforms and *E-coli* was 0.57 and 0.5 logs respectively in CF-UGF and UGF steps, while HPC reached a removal of 0.64 logs. These reductions are close to those reported by Galvis et al. (1999). The average removal in SSF was 3.0 logs. The efficiency of the SSF was not influenced by changes in the filtration rate, probably due to the high level of maturity of the filters.

Table 3.6. Microbiological behaviour without the use of coagulant (UGF: vf = 1.0 mh^{-1}, SSF: vf = 0.30 mh^{-1})

Descriptive statistics	Raw water			CF-UGF and UGF effluent			SSF effluent		
	TC	EC	HPC	TC	EC	HPC	TC	EC	HPC
No data	9	9	7	9	9	7	10	10	9
Average	15,131	2,262	202,629	12,227	2,161	139,625	12	0	744
Maximum	25,000	3,600	403,500	19,400	6,700	272,000	68	0	1900
Minimum	8,000	1,250	120,000	6,100	800	73,000	2	0	100
STD deviation	6,733	914	123,549	5,893	1,859	80,237	19.7	0	651.6
Average log CFU 100^{-1} ml^{-1} removal units				0.16	0.16	0.17	3.3	3.2	2.3

TC: Total coliforms (CFU (100 ml)$^{-1}$) EC: *E-coli* (CFU (100 ml)$^{-1}$) HPC: heterotrophic play count bacteria (CFU (100 ml)$^{-1}$), operational flow 6 l s^{-1}.

3.3.7. Removal of micro-organisms, operation with coagulant

Table 3.7 presents the results of operating with coagulant, the data correspond to samples taken in the influent of CF-UGF unit and the effluent of the UGF for a period of 6 hours, because when turbidity was less than 30 NTU the coagulant dosing was stopped. The dose of aluminium sulphate corresponded to 10 mgL^{-1}, with an operation flow of 6 Ls^{-1}. Overall, the pre-treatment with CF-UGF and UGF contributed to the reduction of microbiological load: average 0.44 log removal for total coliforms, 0.40 log removal of *E-coli,* and 0.44 log removal for HPC. Only the CF-UGF unit contributed with average efficiencies for total coliforms equivalent to 0.19 log, 0.12 logs for *E-coli* and 0.15 log for HPC. The last stage of treatment, SSF, allowed a total reduction of 3.4 log of total coliforms, 3.1 log for *E-coli*, and 3.9 log of HPC. This suggests that the dosage of aluminium sulphate did not affect the biological activity in pre-treatment and SSF, which is consistent with that reported by Dorea and Clarke (2006).

When comparing operation with and without coagulant an increase in the average efficiency of removal of microorganisms, between 0.16-0.17 log to 0.40 to 0.44 log was observed, i.e. in the operation with coagulant the removal efficiency for total coliforms, *E-coli* and HPC, was 2.75, 2.5 and 2.6 higher respectively compared to the operation without coagulant in the CF-UGF and UGF units.

Table 3.7. Microbiological behaviour with the use of coagulant (UGF: *vf* 1.0 mh^{-1}, SSF: *vf* 0.3 mh^{-1})

Descriptive statistics	Raw water			CF-UGF and UGF effluent			SSF effluent		
	TC	EC	HPC	TC	EC	HPC	TC	EC	HPC
No data	5	5	5	5	5	5	5	5	5
Average	13,240	2,780	110,740	6,070	1,125	41,100	2.8	0	540
Maximum	16,050	5,600	160,000	10,100	2,150	47,100	4	0	800
Minimum	8,900	1,200	79,300	3,900	1,000	35,000	1	0	200
STD deviation	2,746	1,684	31,494	2,402	465	5,290	1.3	0	219
Average log CFU $100^{-1}ml^{-1}$ removal units				0.44	0.40	0.44	3.4	3.1	3.9

TC: Total coliforms (CFU (100 ml)$^{-1}$) EC: *Eschirichia coli* (CFU (100 ml)$^{-1}$); HPC: heterotrophic play count bacteria (CFU (100 ml)$^{-1}$), operational flow 6 Ls^{-1}

3.3.8. Aluminium, pH, colour and organic matter

Residual aluminium in the system was low in the effluent of CF-UGF and 53% was removed in the UGF, so that the concentration in the influent of the SSF varied between 0.07 and 0.09 mgL^{-1}, and the average effluent concentration was 0.04 mgL^{-1} (STD 0.005 mgL^{-1}). This value was lower than the WHO (2011) guidelines, which recommended less than 0.2 mgL^{-1} for aluminium in drinking water. In a pilot study that examined the impacts of chemical pre-treatment by gravel filters on SSF Dorea and Clarke (2006), reported an average aluminium concentration of 0.041 mgL^{-1} in the effluent of the SSF and indicated that the chemical improved the overall treatment efficiency. However as indicated by these authors careful control of the coagulation step is needed to avoid carry-over of aluminium to the SSF as this might contribute to possible filter clogging even though turbidities of less than 10 NTU are achieved. In this thesis however premature clogging did not occur at all and filter runs of SSF were maintained between 50 and 70 days with a maximum head loss of 0.70 m, which is in line with the range of 20-60 days reported by Schulz and Okun (1984), the minimum of 45 days recommended by Cleasby (1991) and the range of 46-178 days recommended by Galvis et al. (1999).

The pH in the influent varied between 8.2 and 8.5 and between 8 and 8.2 in the effluent of CF-UGF, which is expected not to affect the biological development of the Smutzdecke in the SSF, respect Galvis et al. (1999) reported pH in the range of 7.1-8.0 for operation of SSF and indicate that the adsorption of virus the sand improves with increasing ion concentration and valence of the cations in solution.

The true colour reduction recorded an average efficiency of 54% in the CF-UGF stage and 57% in the UGF stage, and the net efficiency of the true colour reduction of the CF-UGF and the UGF stage together was 76%. The organic matter content measured as total organic carbon (TOC) was low, the influent had an average value of 1.1 mgL^{-1} (± 0.075), and the removal efficiency in the CF-UGF and UGF step together was 9%, with an efficiency at the end of the treatment of 28%.

3.3.9. Operation and maintenance

The treatment plant operated without coagulant for turbidity levels below 30 NTU. When influent turbidity was greater than 100 NTU, operators interrupted the operation to reduce the turbidity load on the plant and, depending on water needs, operated with coagulant, reducing the filtration rate by half. When rain events occurred and the influent turbidity was greater than 30 NTU, a dosage of coagulant was applied. Figure 3.8 shows the behaviour of turbidity in CF-UGF and UGF and head loss in the CF-UGF units for an event of short duration when the turbidity increased above 30 NTU up to a maximum value of 58 NTU, and operation with coagulant was necessary.

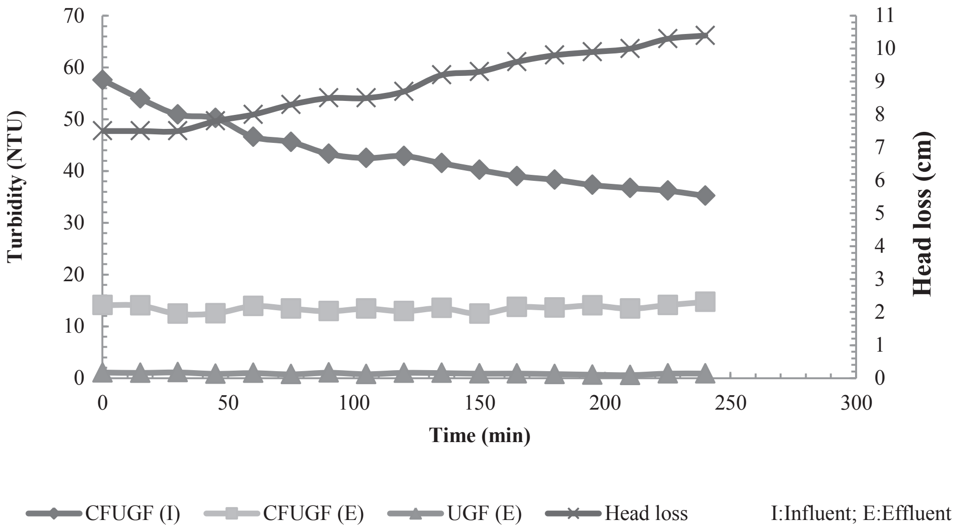


Figure 3.8. Turbidity in CF-UGF and UGF units and head loss on CF-UGF

The monitoring of the head loss was only done in the CF-UGF units. In the UGF unit, there was no change in head loss detected in the relatively short period of the event. The total loss over a period of 4 hours of operation was 3.5 cm and did not achieve the maximum value of 20 cm. Table 3.8 summarizes the operating and monitoring conditions. Based on the information registered by the operators of the water treatment plant, cleaning frequency for the CF-UGF and the UGF units was every 8 days (without a coagulant dosage). During the rainy period the CF-UGF units, operating with aluminium sulphate, registered cleaning frequencies in the range of 6 -100 hours. The cleaning frequency of 6 hours was obtained when influent turbidity reached values of above 100 NTU, while the 100-hour operation mode was observed when influent turbidity levels were between 30-60 NTU.

The behaviour of the cleaning of the CF-UGF in terms of turbidity is presented in Figure 3.9. In the CF-UGF, cleaning was done by operating the butterfly valve in such a way that 10 shock waves were created. It can be observed that, for the CF-UGF, in the first 15 s of the discharge a turbidity peak occurred, for 35 s the water was clear, and then after 85 s a second peak was observed. For the UGF, the solids discharge started immediately to reach a second peak after 150 seconds, about 5 times higher than the first peak. The behaviour of the water quality in the discharge of the wash water was different from that reported by Wolters (1988) and Cinara and IDRC (1993). When the valve was opened suddenly, the particles which stayed on the filter media experience a change in velocity, whose effect may be to drag the particles to transport to the drainage system.

Table 3.8. Summary of operating and monitoring conditions

Variable	Value
Operational parameters	
Influent turbidity for coagulant dosing	> 30 NTU
Maximum turbidity of operation	Normally 100 NTU, sometimes can operate with peaks above 100 NTU for short periods of time (4-6 hours).
Cleaning period CF-UGF and UGF, operation without coagulant.	Each 8 days
Time cleaning of CF-UGF	15 minutes
Time cleaning of UGF	45 minutes
Monitoring parameters	
Coagulant used	Liquid aluminium sulphate, type A, 50% concentration
Filtration run of the CFUGF with application of coagulant	6-100 hours
Percentage of time with coagulant dosage	20% year
Percentage of operating time without coagulant	80% year
Maximum period of time recorded dosing coagulant	100 hours
Maximum duration of registered plant shutdown	24 hours
Percentage of stops in the year	3%
Maximum head loss in CF-UGF units	20 cm
Maximum head loss in UGF units	15 cm
Maximum washing velocity CF-UGF	9.1 to 7.5 m h^{-1}
Maximum washing velocity UGF	9.4 to 8.4 m h^{-1}

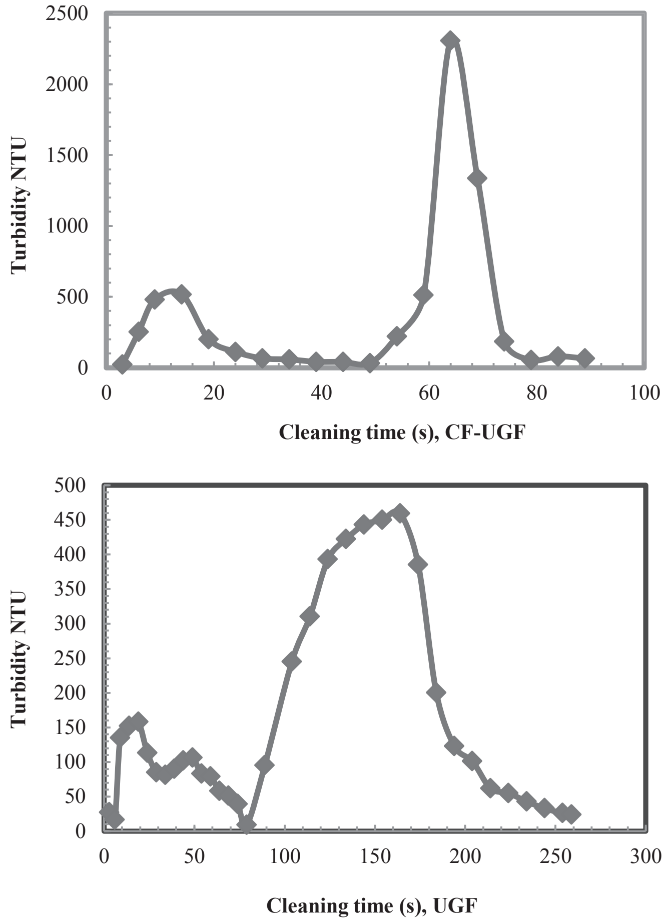


Figure 3.9. Behaviour of turbidity on a deep clean-up of CF-UGF and UGF

The flow rate of the drain water for cleaning the filter was variable over time; measurements were done for different heights of water level relative to the position of the butterfly valve. The maximum backwashing rate for the UGF was 9.4 mh^{-1} while for the CF-UGF it was 9.1 mh^{-1}

3.3.10. Investment, operation and maintenance costs

Tables 3.9 and 3.10 list the construction costs (year 2011) and operation and maintenance costs (year 2010). The CF-UGF stage and the pre-treatment by UFG represent respectively 7% and 28% of the total construction costs of the water treatment plant (see Table 3.10). The cost per m^3 of produced water was US$0.05 (discount rate of 12% for Colombia and project horizon of 15 years). Per capita investment costs are US$18, for an average consumption of 150 $Lc^{-1}d^{-1}$. The costs of O&M for the use of coagulant are low, representing only 0.3% of total O&M costs, because the operator only doses in periods of high turbidity, making the technology attractive. The highest costs of O&M represent pumping energy and staff. The O&M costs of US$ 0.264 m^{-3} is low (depreciation was included), when compared to the costs of US$ 1.04 m^{-3} (includes the average investment costs) for the utility of the city of Cali, close to the community. The O&M costs in a gravity system would reduce to US$ 0.14 per m^3.

Table 3.9. Initial investment costs

Stage of treatment	* Cost (US $)	%
DyGF	9,821	16
CFUGF	4,554	7
UGF	17,002	28
SSF	29,783	49
Total cost (US $) *	61,160	
Cost L/s (US $)	10,193	
Per capita cost (US $)	18	

* Costs up to February 2011

Table 3.10. Operation and maintenance costs

Item	Monthly cost (US $)*	%
Cost of coagulant (aluminium sulphate)	13	0.3
Cost of the chlorine	365	9.5
Staff costs (including benefits and social security)	1,399	36.4
Electrical energy costs	1,661	43.3
Costs of materials and equipment	249	6.5
Costs for water quality analysis	155	4
Total monthly cost O & M (US $)	3,842	
O&M costs per m^3 produced (US $ m^{-3})	0.25	

* Costs to December 2010

3.4. Conclusions

CF-UGF is a relatively new technology that has been applied in a few cases with rapid sand filtration. In this study CF-UGF has been used in combination with MSF technology comprising DyGF, UGF and SSF. This chapter shows that combining CF-UGF with MSF greatly contributed to the removal efficiency of the system without negatively affecting the biological activity of the treatment system in terms of the efficiency of microorganism removal in the UGF and SSF when coagulant was dosed. This strongly contributes to the operational flexibility of the system as it allows to dose coagulant only when high influent turbidity peaks occur.

CF-UGF improved the operation of MSF compared to only UGF, when the system operated with turbidity levels above 30 NTU, facilitating the performance of the SSF by reducing the load of particulate material to avoid short filter runs and possible interruptions in treatment plant operation. The removal efficiency of turbidity in the CF-UGF with coagulant dosing was between 85 and 96%, which is higher compared to operation without coagulant dosing; average efficiency of turbidity in the CF-UGF was 46%, ranging between 21-76%. The addition of coagulant in the CF-UGF allowed for obtaining water with turbidity levels below 10 NTU after UGF, which contribute to the effective operation of the SSF. The overall system produced water with turbidity below 1 NTU for 98% of the samples that were taken in the research period. In the operation with coagulant the removal efficiency for total coliforms, *E-coli* and HPC, was 2.75, 2.5 and 2.6 higher respectively compared to the operation without coagulant in the CF-UGF and UGF units. No reduction was observed in the microbial removal efficiency of the SSF, no obstruction of the SSF beds were demonstrated and SSF runs were maintained between 50 and 70 days for a maximum head loss of 0.70 m.

The hydraulic behaviour of CF-UGF indicated that the system worked with a plug flow fraction of 51%, a mixed fraction of 46% and a dead zone fraction of 3%. The comparison between the theoretical model and experimental data indicated that hydraulic behaviour of the reactor CF-UGF tends to n = 6 reactors in series, confirming the presence of a relative plug flow. The mean residence time was 19.7 min for the operating flow of for the flow of 3 Ls^{-1}; theoretical retention time was 19 min.

It was only necessary to operate the system with coagulant for 20% of the time. The CF-UGF run time was 6-100 hours depending on raw water turbidity. In the operation without coagulant, the run time for the CF-UGF and UGF was 8 days. Whereas the designed drainage flow was established at 20 mh^{-1} in practice this level was not reached and the real flow according to the measurements in the units was about 9 mh^{-1} indicating that more research is needed on the effect of the drainage system during the cleaning operation.

The CF-UGF unit represented only 7% of the total construction costs, and the pre-treatment CF-UGF and UGF represented 35% of total costs, while the cost of m^3 produced by the MSF with the CF-UGF system was US$0.05, for a per capita investment of US$18. The O&M costs for the use of coagulant represented only 0.3% of the total O&M costs. The production cost was US$0.264 m^{-3} for the operation with pumping. In a gravity system the cost is reduced to US$0.14 per m^3.

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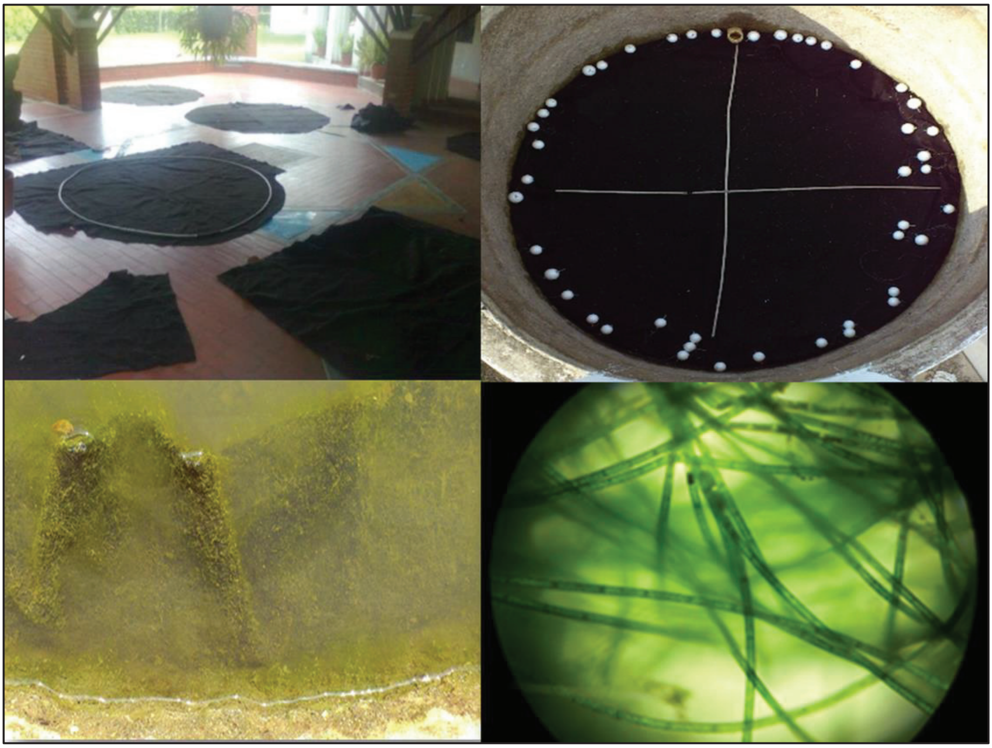


CHAPTER 4

Effect of upflow gravel filter with fabric cover on suspended solids and E-coli removal and algal growth

This chapter examines the performance of an upflow gravel filter covered with a filter fabric in relation to removal of total suspended solids, particles and E-coli, head loss development and algal growth. The results of the study indicate that in upflow gravel filters the operation is characterized by a period of ripening, effective filtration and breakthrough. Best suspended solids removal efficiency was obtained for an effective filtration period between 30 and 39 days, with all filter layers contributing to the performance. The larger accumulation of solids per volume of filter layer occurred in the filter fabric (thickness of 0.0056 m), but this layer did not contribute to E-coli removal. Removal efficiency of E-coli was highest in the bottom layer (gravel size of 19.1 mm) and reduced to the top of the filter. A direct relationship was found between the increase in head loss in the filter fabric and algal biofilm development. In addition, suspended solids with particles sizes larger than 40 μm were completely removed, whereas particles with sizes less than 2 μm were hardly removed. The study also suggests that applying a filter fabric allows for reducing the filter height without losing suspended solids removal efficiency and that the once-a-week cleaning procedure, common for upflow gravel filter operation, can be adapted to local circumstances as cleaning at longer intervals can improve efficiency, reduce water losses and decrease maintenance costs.

*This chapter is based on:

Sánchez L.D., Dominguez, E.A., Visscher, J. T., Rietveld L. C. (in preparation). Effect of upflow gravel filter with fabric cover on suspended solids and E-coli removal and algal growth.

4.1. Introduction

Upflow gravel filtration (UGF) is a pretreatment technology used in multi-stage filtration (MSF), to reduce the load of suspended solids to facilitate the performance of the final stage of slow sand filtration (SSF). This technology is e.g. being applied in rural areas in Latin America to cope with considerable variations in the level of turbidity and total suspended solids (TSS) in surface water sources. UGF units consist of gravel beds of different sizes with coarse gravel at the bottom, to fine gravel at the top. The treatment process is a combination of physical-chemical and biological processes in the filter layers (Galvis, 1999; Arakawa *et al*, 2014). This filtration technology is attractive because it requires little energy and is relatively simple to operate and maintain in rural areas. However, better understanding of the physical-chemical and microbiological processes that take place within these systems is necessary to analyze its multiple benefits, as well as, to optimize operation and maintenance and to review its potential to enhance subsequent treatment processes.

Following the experience with SSF reported by Di Bernardo and Sabogal (2008) where placing a filter fabric on top of the filter contributed to a reduction in the necessary filter bed height, it is anticipated that the same will be the case for UGF. Filter fabrics are light and relatively easy to remove which allows for efficient surface cleaning.

Whereas the biological activity in SSF has been studied in more depth it may be expected that a similar type of biofilm develops on top of the surface layer in the UGF, due to the retention of some solids in the filter fabric. In addition, low filtration velocity (< 1 mh^{-1}) and the upflow current facilitate the gradual removal of solids from the bottom to the top of the UGF. The clarity and the height of the supernatant water on top of the UGF allows then for penetration of sunlight which reaches the filter fabric on top of the granular medium. This possibly facilitates further biofilm development including a variety of microorganisms, such as bacteria, protozoa, and algae, that can further penetrate in the inter-granular voids of the gravel layer (Di Bernardo and Sabogal, 2008).

In this chapter, the *E-coli* and TSS were analyzed over a pilot UGF with a filter fabric on top, to explore the efficiency in disinfection and suspended solids removal of this combination. Also algae growth in the filter fabric was studied measuring chlorophyll-a concentration (Di Bernardo and Sabogal, 2008).

4.2. Materials and methods

The study was conducted in a pilot plant consisting of an uncovered circular filter with four gravel layers and a filter fabric on top, operating at a filtration velocity of 0.5 mh^{-1} (Table 4.1). The height of the supernatant was kept constant (0.09 m above the filter fabric). Gravel sizes and layer thickness were based on Galvis (1999) with the coarse gravel at the bottom and the smallest gravel on top (Figure 4.1).

The UGF was fed with natural water from the Cauca River, Cali-Colombia. To reduce the entrance of peaks in turbidity and TSS a dynamic gravel filter (DyGF) unit was installed before the UGF. Gravel layers were washed with clean water and grain size analyses were performed to check the grain size of each layer. Furthermore the uniformity coefficient was determined, U_c ($d_{60}d_{10}^{-1}$) and the porosity (ε_o) of the filter material was estimated following the procedure defined by Ives (1990). Piezometers and sampling points were installed in each gravel layer to allow taking measurements of head loss (*hf*) and water quality samples during the filter run.

Table 4.1. Pilot filter characteristics

Characteristics	Value				
Construction material	Ferrocement				
Form	Circular				
Diameter	1.93 m				
Filtration media					
	Thickness (m)	**Media size (mm)**	**Porosity**	**Specific surface area (m^2m^{-3})**	**Uc[1]**
Gravel layer 1 (bottom)	0.25	25.4 – 19	0.47	428	1.30
Gravel layer 2	0.25	19 – 12.7	0.41	525	1.28
Gravel layer 3	0.25	12.7 – 6.3	0.42	1,180	1.46
Gravel layer 4 (surface)	0.16	6.3- 3.2	0.46	3,100	1.50
Fabric cover 1	0.0016	0.000033[2]	0.88	15,563	na.
Fabric cover 2	0.0018	0.000033[2]	0.88	15,860	na.
Fabric cover 3	0.0022	0.000033[2]	0.83	26,061	na.

[1] $U_{c=} d_{60}d_{10}^{-1}$*;* n.a: not applicable, [2] fiber size (33µm).

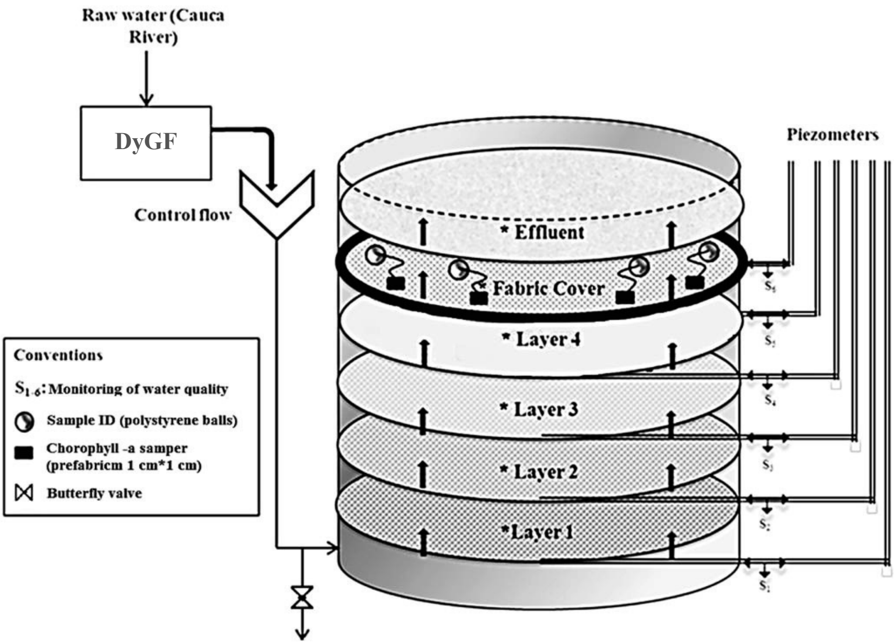


Figure 4.1. Pilot filter scheme

4.2.1. Filter operation

Two filter runs were analyzed. The first filter run with filter material that was washed outside the filter. The second filter run was initiated after a normal filter cleaning procedure by drainage. Initially two criteria were explored related to ending the filter run: Drop in the removal efficiency for turbidity or TSS (filter breakthrough) and the maximum permissible head loss (*hf* maximum) of 15 cm (Galvis, 1999). When breakthrough occurred in the first run the experiment was continued for another 24 days to explore any further effect from continued accumulation of solids in the filter. Head loss development increased in this period but never reached the envisaged maximum. The second run was ended when breakthrough occurred. Once, after the first filter run, the filter was taken out of operation, the surface of the unit was washed and the filter cleaning was carried out by quickly opening and closing the butterfly valve in the under drains. When the initial *hf* was recovered the second filtration run was started.

4.2.2. Measurements

Flow control was carried out in a channel with a triangular weir and the *hf* was measured daily with piezometers located in each filter layer (Figure 4.1). Turbidity, TSS, *E-coli*, chlorophyll- a, were daily measured for each of the layers (at 0.25, 0.50, 0.75, 0.91 and 0.916 m from the bottom of the filter). Turbidity measurements were performed using a Hach 2100P portable turbidimeter. TSS was measured according to Standard Methods 2540 B, using Whatman paper filter of 1.2 µm, *E-coli* was measured by membrane filtration technique 9222 D; the filtration instruments were properly sterilized each time a sample was processed to avoid contamination. Samples were taken in sterile vials and filtration was done in a UV sterilized cabinet. The filtered volume was between 1.0 ml or 0.1 ml to facilitate the colony counting (20-60 colonies per membrane). The other parameters were measured according to standard methods (APHA, AWWA, WPCF, 2005). Particle size distributions were measured during the first filtration run for each filter layer using laser diffraction, detecting particle sizes in the range of 0.02µm to 2000µm (Mastersizer 2000, vers. 5.6), following the procedures described in the operating manual of Malvern Instruments (2005). The suspended solids of the influent and effluent for each filter layer were partitioned into different size classes; weighted according to the particle size distributions, following the procedure of Lin *et al*. (2008). Removal efficiency was determined, making the respective mass balances (between influent and effluent) for TSS fractions with the respective particle sizes for each filter layer.

Sampling of chlorophyll-a on top of the filter fabric was conducted by placing pieces of fabric cover of 1 cm * 1 cm (Figure 1).

Residence time distribution was verified for the clean bed with tracer tests according to the methodology described in chapter 3. Results were analyzed with the mathematical model Wolf-Resnick and the model of completely mixed reactors in series (CMRS).

4.3. Results and discussions

4.3.1. Filter operation

Residence time distribution in the UGF was verified for clean bed before starting the first filtration run. The test revealed a plug flow fraction of 80%, a mixed flow of 20% and the results of the CMRS model indicated n = 10 reactors in series, which provided good hydraulic conditions for the experiment.

Turbidity removal efficiency in the UGF during the two filtration runs showed three distinct periods (Figure 4.2). During the first period, (the ripening period) removal efficiency was low but increased quickly towards the end. This period is characterized by a conditioning process of the filter media, where clean media captures particles and becomes more efficient at collecting additional particles (Crittenden et al, 2012). The duration of this ripening period depended on filter conditions. In the first filter run the filter was filled with clean gravel which resulted in a ripening period of 10 days (Figure 4.2). In the second test period the filter was only washed facilitating the capturing of new particles which resulted in a ripening period of only one day. The second period of the filtration run is denominated as effective filtration (Crittenden et al, 2012), which is characterized by a good removal efficiency. This period lasted 30 days in the first run and 39 days in the second run before breakthrough occurred (Figure 4.2). The second filtration run was thus characterized by a shorter ripening period, longer effective filtration and best net turbidity removal efficiency during the effective filtration period (first filter run: 42.9%; second filter run: 52.8%) with less variation (first filter run: STD 13.3%; second filter run: STD 5%). In the third period, the filter reached breakthrough, i.e. a sharp reduction in removal efficiency, which implies that filter bed cleaning is needed (Crittenden et al, 2012).

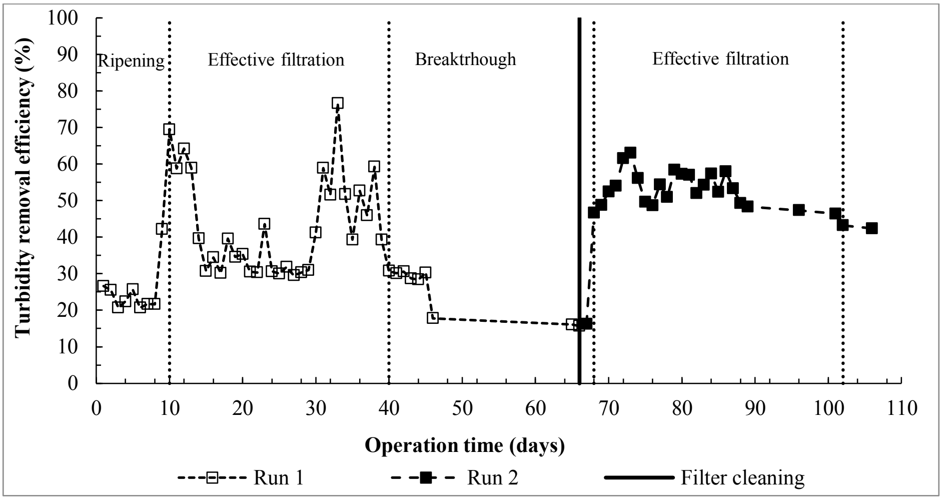


Figure 4.2. Turbidity removal efficiency versus operation time (vf 0.5 mh^{-1})

4.3.2. Effect of filter layers on particles, TSS, and E-coli removal

Figure 4.3a shows the particle size distribution in each filter layer, while Figure 4.3b shows the results of total TSS removal efficiency in the filter layer for different particle sizes. Figure 4.3a illustrates that all filter layers contribute to particle removal, but particles in the larger size range were better removed in the gravel with diameter of 19.1 and 12.7 mm. These results are in line with Collins *et al.* (1994) who observed that large particles were preferentially removed in the first 0.30 m of a UGF.

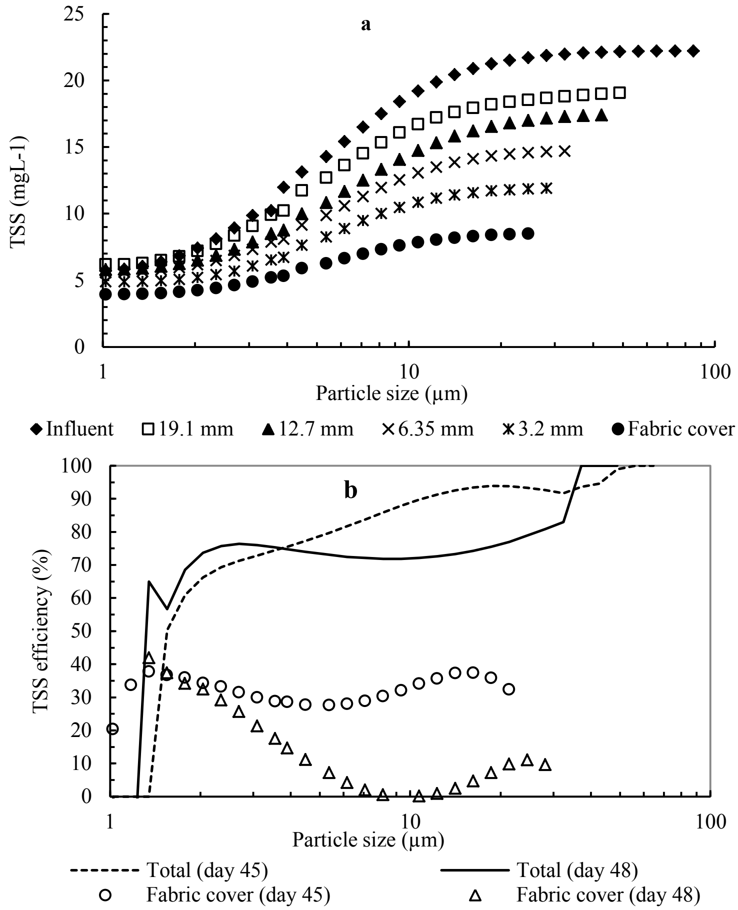


Figure 4.3. TSS distributed by particle size: a) in each filter layer (influent TSS 22.2 mgL^{-1}) and b) Total TSS efficiency in UGF and fabric cover for different operation time: Total TSS removal efficiency: 67% (Influent TSS 24.4 mgL^{-1}) and 38% (influent TSS 29.1 mgL^{-1}).

Figure 4.3b shows a high TSS removal efficiency in the UGF for particles sizes above 40 µm, while smaller particles sizes, in part, passed the different filter layers. From a total TSS removal efficiency of 67%, particles in the range of 10-40 µm reached removal efficiencies of 90%, whereas particles in the range of 2-10 µm reached a removal efficiency of 70% and particles less than 2 µm were hardly removed. This last result was consistent with Lin et al. (2008).

The filter fabric contributed particularly in removing small particles, as larger particles were retained in the different gravel layers. Removal efficiencies of particles smaller than 25 µm were between 30-40%, but a considerable reduction of small particles was observed when total TSS removal efficiency in the UGF was low.

Results for average TSS and *E-coli* removal efficiency in each layer are shown in Figure 4.4 TSS removal efficiency was better in the top layer (gravel size of 3.2 mm) followed by gravel layer size of 6.35 mm and the filter fabric. Little difference in efficiency was observed for gravel sizes between 19.1 to 12.7 mm. Net TSS removal efficiencies in the UGF after ripening was 62%. The median net removal efficiency for *E-coli* in the UGF was 90.2 %. Removal efficiency of *E-coli* was highest in the bottom layer (gravel size of 19.1 mm), and reduced towards the top. The high removal in the bottom layers may be explained by the adherence of bacteria to the particles, the formation of clusters of bacteria (WHO, 2011), natural die-off and predation by organisms inhabiting in the bottom layer. In Figure 4.5 the removal efficiency of *E-coli* was plotted against the quantity of TSS retained in each gravel layer that contributed to the *E-coli* removal. A relation seems to exist between increased *E-coli* removal and the quantity of TSS retained in these filter layers. When the accumulated solids in each filter layer were near to 2 gL^{-1} a maximum efficiency was observed, after this, removal efficiency of *E-coli* was independent of solids concentration.

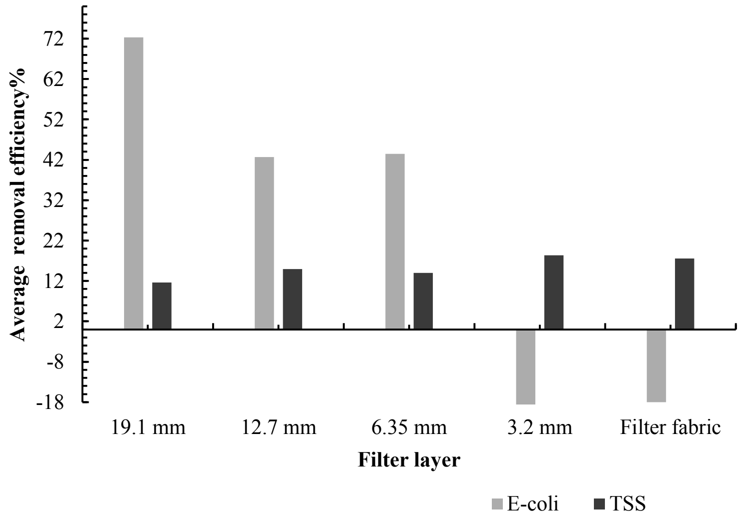


Figure 4.4. Removal of E-coli and TSS in the filter layers (*vf* 0.5 mh^{-1}, E-coli influent average 3.9 log CFU(100 ml)$^{-1}$, SD= 0.32 log CFU(100 ml)$^{-1}$

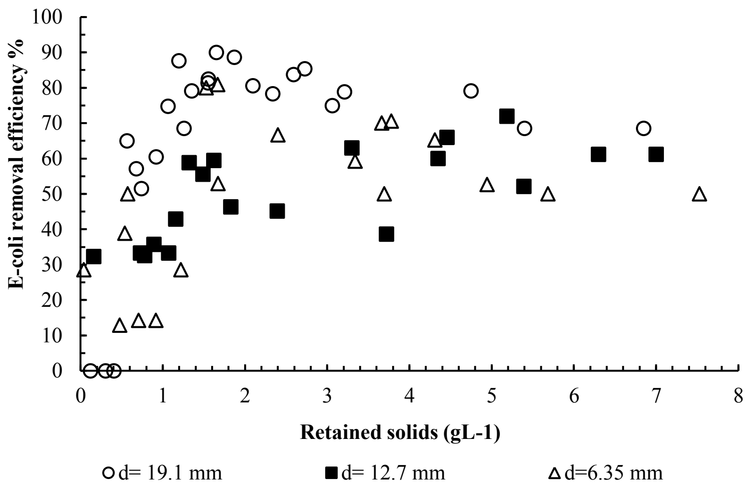


Figure 4.5. Relation between E-coli removal efficiency and TSS retained in the gravel layer (vf = 0.5 mh^{-1})

4.3.3. TSS retention in the filter

The results of daily absolute TSS accumulation per volume in each filter layer are shown in Figure 4.7. The data show that the deposited TSS in each layer increased progressively over operation time. The total TSS retained per filter volume at the end of the effective filtration period reached 200 gL^{-1} at 40 days of operation. The filter fabric (top of the filter) had the largest TSS retention per volume of filter layer, contributing with 171.6 gL^{-1}due to small layer thickness (0.0056 m). In chapter 2 was reported that in UGF surface cleaning is a maintenance activity to be improved as this represents the highest time input in maintenance and is complicated by the low surface water drainage velocity which does not sufficiently allow for the removal of solids with the result that the filter stays dirty. Using filter fabrics can contribute to this because these fabrics can be easily removed and washed to clean the retained solids and the algal biomass, before putting them back in the filter. The gravel below the fabric has very little or no algal biomass and therefore the retained solids can then easily be removed. All gravel filter layers contributed to TSS retention. However, in the gravel bed the greater accumulation of solids per volume of gravel layer occurred in the small gravel size (d= 3.2 mm) of 0.16 m and small differences were observed in the other three layers towards the bottom (gravel size between 6.3 to 19.1 mm). Greater TSS retention in filter fabric and small gravel sized filter layer may be associated with larger surface area available for deposition. Findings in chapter 2 showed that UGF units in full scale plants are cleaned every seven days. This pilot study, however, revealed that at seven days of operation (Figure 4.7 red dotted line) the quantity of retained TSS was only 1/5th of the total TSS retention capacity of the filter. Here weekly cleaning of UGF units was not necessary, reducing water losses during cleaning.

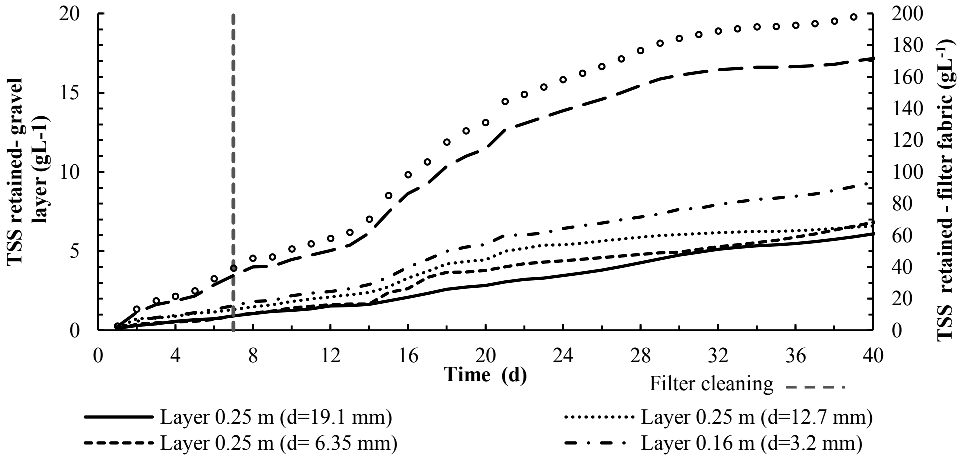


Figure 4.6. TSS retention behavior over the depth of the filter for run 2 (the first 0.25 m correspond to the filter bottom and increases to the top; TSS influent average: 30.6 mgL^{-1}, SD= 18.9 mgL^{-1}).

Chlorophyll-a concentration in filter fabric

Average water temperature in the UGF was 23.6 ºC, STD 1.2 ºC, and dissolved oxygen in the influent was 5.2 mg/L, STD 1.2 mg/L, and in the effluent 5.8 mg/L, STD 1.0 mg/L, indicating aerobic conditions throughout the unit. Chlorophyll-a was monitored in the set of filter fabrics (Table 4.1) to identify algal activity (Figure. 4.7). A progressive increase of algal growth on the surface of the filter fabric was evident over time for both filtration runs. Chlorophyll-a development in the filter fabric showed three phases: 1) the adaptation phase (ripening phase) which took 20 days in the first filter run (new filter) and only 4 days in the second run (washed filter); 2) the growth phase between day 20 - 60 and 3) the stabilization or equilibrium phase. During the first filtration run the growth rate of the algal biofilm was lower, probably because starting the process needed more time for colonization of organisms in the filter fabric.

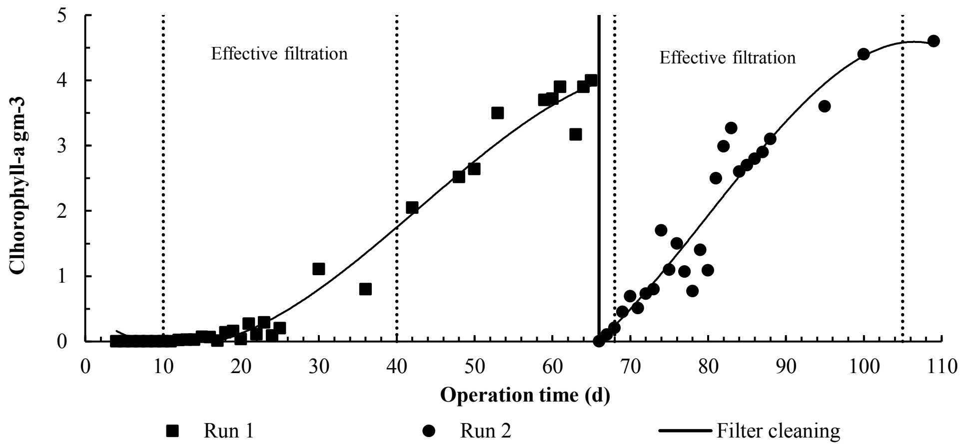
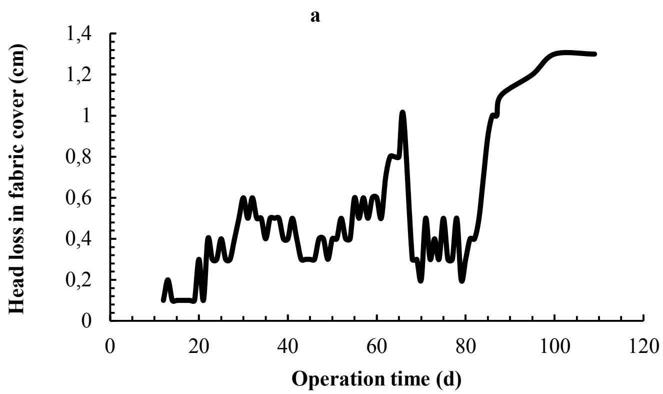


Figure 4.7. Chlorophyll-a growth in the fabric cover

Algae growth in the first run reached a chlorophyll-a concentration of 1.8 gm^{-3} at the end of the effective filtration stage (40 days) and growth tended to stabilize at 3.8 gm^{-3} after 65 days. During filtration run 2 growth started immediately, increased faster and stabilization was observed after 34 days with a concentration of 4.4 gm^{-3} (Figure 4.7). The relative stabilization can be explained in terms of space availability and competition for nutrients, as reported by Di Bernardo (1995).

4.3.4. Head loss development in relation to algal biofilm growth

Biofilm development contributes to the solids retention and can cause an increase in *hf* in the filter fabric (Mälzer and Gimbel, 2006). For this reason *hf* development in the filter fabric was analyzed versus chlorophyll-a concentration. Figure 4.8a shows the *hf* in the filter fabric. The *hf* in the filter fabric and its relation with progressive chlorophyll-a concentration is shown in Figure 8b.The filter operation with a filtration velocity of 0.5 mh^{-1} during the first and second filtration run, reached a total *hf* of 4.5 and 5.0 cm for an operation time of 64 and 44 days respectively. In the filter fabric *hf* varied between 1.0 and 1.3 cm and represented 26% of the total *hf*. A direct relationship was found between the *hf* in the fabric cover and Chlorophyll-a concentration that represents algal biofilm growth. The progressive *hf* increase thus coincided with the algal biofilm increase but this could also be due to TSS accumulation in time, as was discussed previously.



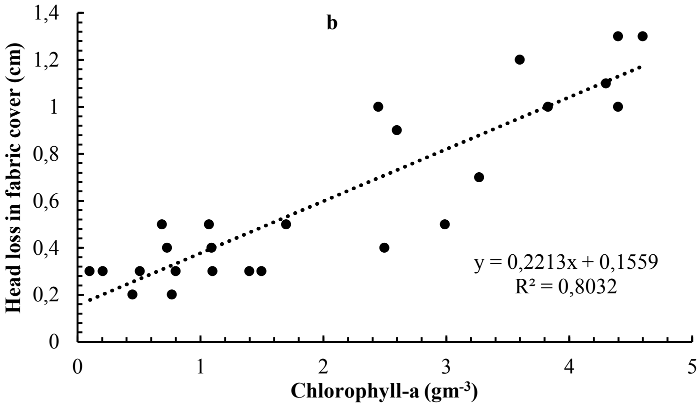


Figure 4.8. Relation between: a) head loss in fabric cover over time; b) head loss and chlorophyll-a.

4.3.5. Effect of filter fabric in relation to TSS removal efficiency

Figure 4.9 shows the TSS removal efficiency over time and in relation to chlorophyll-a in the filter fabric for filter run 2. TSS removal efficiency in the filter fabric increased progressively over time until day 5, after the efficiency remained stable during the effective filtration period (until day 40). TSS removal efficiency increased progressively with the increment of the chlorophyll-a presence in the filter fabric, until a chlorophyll-a concentration of 1.0 gm^{-3}. Afterwards TSS removal efficiency remained relatively stable until a chlorophyll-a concentration of 4.0 gm^{-3}, while removal efficiency decreased subsequently. Apparently the solids removal in the filter fabric was during the first period enhanced by the algal growth as reported by Hirschi and Sims (1991), Mälzer and Gimbel (2006), and Langenbach et al. (2010), but not in the later stages.

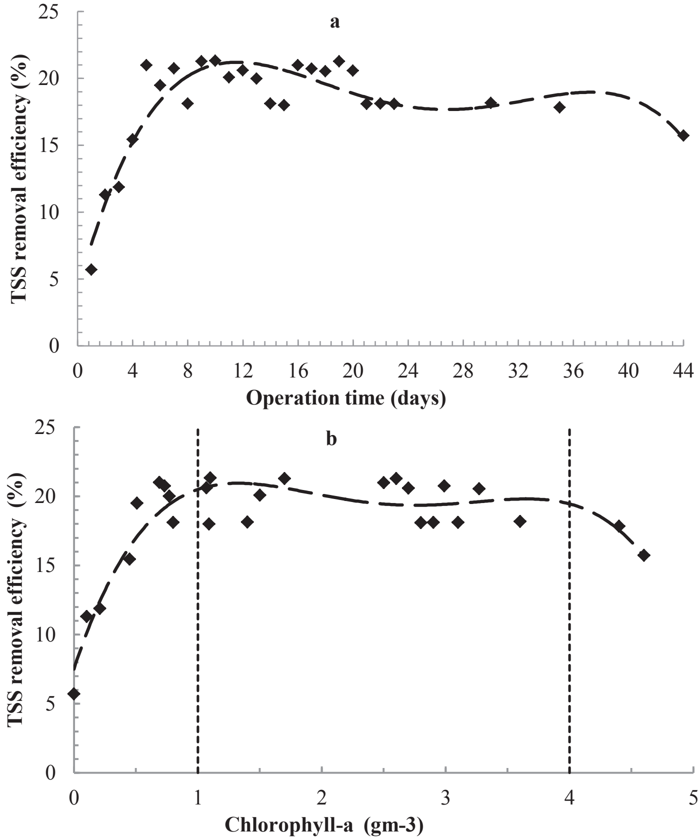


Figure 4.9. TSS removal efficiency over operation time (a) and chlophyll-a (b) in the filer fabric (*vf* 0.5 mh^{-1}).

4.3.6. Effect of algal grows on filter fabric in relation to *E-coli* removal

The relationship between algal growth measured as chlorophyll-a concentration and the *E-coli* removal efficiency over time in the filter fabric is presented in Figure 4.10. At the beginning of the filtration run, when chlorophyll-a concentration was minimal, *E-coli* was removed, but this removal reduced after 4 days whereas chlorophyll-a concentration increased.

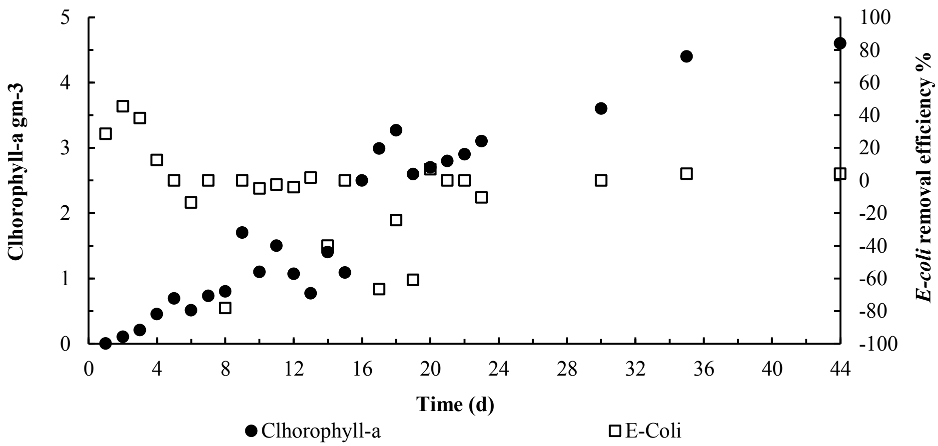


Figure 4.10. Chlorophyll-a and E-coli removal efficiency over time for run 2 (vf= 0.5 mh^{-1})

Even negative removal efficiencies were observed (42% of the data) which seems a confirmation of the suggestions of Campos *et al.* (2006) and Nakamoto (2014) that at high temperatures biological activity in the top layers may favor regrowth of bacteria and protozoa within the pores of the biofilm developed in the filter fabric and the gravel bed. Studies conducted in tropical waters by Carrillo *et al*. (1985) shown that *E-coli* can survive, remain physiologically active, and regrow at rates that were dependent on nutrient levels of the ambient waters. Also *E-coli* survived better in filtered than in untreated water as was reported by Korhonen and Martikainen (1991) and suggests that predation and/or competition for nutrients affect the survival in an aquatic environment.

4.4. Conclusions

The removal efficiency of TSS, particles and *E-coli*, head loss increase and development of algal biofilms in UGF with a filter fabric on top was studied. The results of the study indicate that in upflow gravel filter the operation is characterized by a period of ripening, effective filtration and breakthrough. Best TSS removal efficiency was obtained for an effective filtration period between 30-39 days.

Median removal efficiencies in the UGF after ripening were: for TSS 57% and for *E-coli* 90%. All filter layers contributed to TSS removal but the greater accumulation of solids per volume of filter layer occurred in the filter fabric (thickness of 0.0056 m).

Removal efficiency of *E-coli* was highest in the bottom layer (gravel size of 19.1 mm) and was reduced to the top of the filter. In the gravel bed the greater accumulation of solids per volume of gravel layer occurred in the small gravel size (d =3.2mm) (thickness of 0.16 m).

The removal efficiency in the filter fabric was 17% for TSS but the fabric did not contribute to *E-coli* removal.

The UGF developed a maximum head loss of 4.5 cm before breakthrough in the first test period after 66 days and of 5.0 cm in the second test period after 44 days. The head loss in the filter fabric was 1.0 and 1.3 cm for the first and second filtration run respectively. A direct relationship was found between the increase in head loss and the algal biofilm growth ($r^2 = 0.8$) on this layer media, but this is also due to TSS accumulation.

All filter layers contributed to particle removal, with a larger number of larger particles being removed in the gravel with diameter of 19.1 and 12.7 mm. TSS with particles size larger than 40 µm were completely removed, but small particles < 2 µm were hardly removed. Filter fabric contributed to removal of particles less than 25 µm.

The study suggests that TSS removal efficiency by a layer of fabric cover of 0.56 cm is equivalent to a gravel layer of 0.16- 0.25 m with a gravel size between 6.3-3.2 mm at the top of the filter bed. Thus using a filter fabric may allow a reduction in filter bed height without losing removal efficiency, which, in turn, has an impact on the reduction in investment costs.

Finally it was found that the practice of weekly cleaning of UGF units as finding in chapter 2 needs to be adapted to local circumstances, as cleaning at longer intervals can improve efficiency and reduces water losses.

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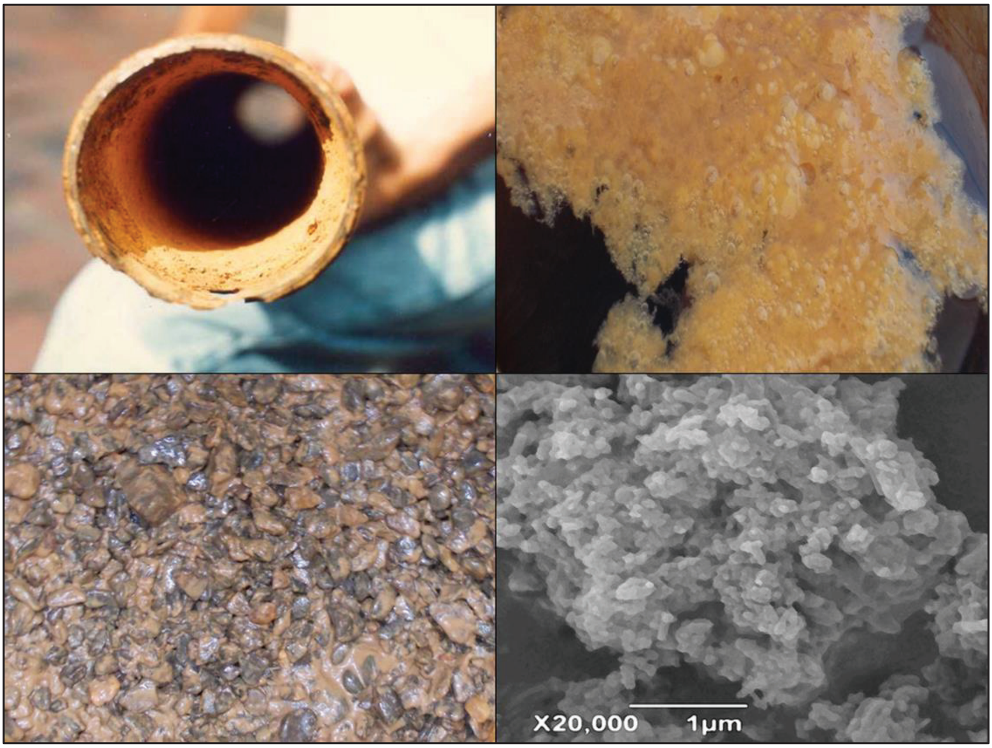
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CHAPTER 5

Iron and manganese removal from groundwater by upflow gravel filtration*

This chapter explores the removal of iron (Fe) and manganese (Mn) in Upflow Gravel Filtration (UGF) both at laboratory and pilot scale at high and low oxygen concentrations and different pH levels. In batch experiments with coated gravel results showed only small differences in Fe and Mn removal between high and low oxygen concentrations (pH 5-8). Removal efficiencies were influenced by pH with best Fe removal (64%) being obtained at pH 7 at high oxygen concentrations with the ultimate concentration being reached after five hours. For Mn best removal (72%) was obtained at pH 8 reaching the ultimate concentration after four hours. Removal efficiencies were higher in a UGF pilot plant with median removal efficiencies for Fe between 75% - 95% and for Mn between 60-95%. In the pilot plant the effect of different oxygen concentrations were also small. Filtration velocity had an impact with the best efficiency being obtained with a filtration velocity between 1-3 mh^{-1}.

*This chapter is based on:

Sánchez L.D., Visscher, J. T., Rietveld L. C. (in preparation). Iron and manganese removal from groundwater by upflow gravel filtration.

5.1. Introduction

Use of groundwater is increasing in Colombia as surface water becomes more polluted by the discharge of untreated domestic wastewater, and more turbid due to erosion as a result of deforestation and poor management of water catchment areas (CGR, 2009). Unfortunately many groundwater sources contain high concentrations of iron (Fe) and manganese (Mn) which may lead to the rejection of the water by consumers and may affect water distribution systems, including valves, meters and other accessories. WHO (2011) indicates a guideline value for total Fe of 0.3 mgL^{-1} based on aesthetic and other not health related considerations. For Mn the health-based guideline value is 0.4 mgL^{-1} but for piped water supply a maximum acceptable threshold of 0.1 mgL^{-1} needs to be adopted to prevent black deposits.

If Fe and Mn concentrations in groundwater exceed these levels treatment is necessary to avoid the formation of deposits and incrustation in the pipes and to reduce the formation of bio-films which may cause changes in water quality. Most Fe and Mn removal systems combine aeration (Ae) and rapid sand filtration (RSF), but the involved treatment mechanisms are still not fully understood (De Vet, 2011, Crittenden et al. 2012, Bruins et al. 2014).

Upflow Gravel Filtration (UGF) is another, low cost, treatment process that has a potential for Fe and Mn removal from groundwater. Ingallinella et al. (2002) presented a pilot study, with an Ae system followed by UGF and RSF where 90% reduction in Fe and Mn was obtained at influent concentrations of 0.3 to 5.0 mgL^{-1}. Pacini et al. (2005), reported Fe and Mn removal efficiencies of 85% to 95% for a combination of UGF and Slow Sand Filtration (SSF) and UGF in combination with RSF. Sánchez and Burbano (2006) reported efficiencies of 50 – 90% for Fe and Mn removal by UGF combined with SSF.

Different mechanisms may contribute to Fe and Mn removal, depending particularly on the oxygen level and pH (Hatva, 1988; Mouchet, 1992; Sharma, 2001). The two main mechanisms are: oxidation-floc-formation and adsorption-oxidation (Sharma, 2001). Several researchers (Mouchet, 1992; Mann et al. 1998; Zhang et al. 2002; De Vet, 2011) mention biological oxidation as a third process which is facilitated by microbiological activity on the filter grains.

Under oxic conditions, oxidation of Fe^{2+} and Mn^{2+} to insoluble Fe^{3+} and Mn^{4+} takes place which results in the formation of Fe flocs and Mn oxides that can be retained by filtration (O'Connor 1971). This retention will gradually increase the head loss in the filter which needs to be restored by filter cleaning. The pH has an important influence on the Fe^{2+} oxidation rate, being relatively quick at pH >7 (Mouchet 1992) and a pH between 7.1 and 8.0 has proven to be sufficient for effective Mn removal in full-scale treatment plants with Ae and RSF (Bruins et al., 2014).

Under anoxic conditions, Fe^{2+} and Mn^{2+} are mainly removed by adsorption onto the surface of the filter media. This process will stop after some time unless the adsorbed Fe^{2+} and Mn^{2+} is oxidized on the filter media surface (Buamah, 2009). The latter can be obtained by interrupting the flow when the adsorption capacity is exhausted and backwashing the filter with oxygen-rich water or with a chemical oxidant, after which the filter can be put back in operation. An alternative option is to adopt a continuous regeneration by allowing a low concentration of oxygen in the water and adopting a short pre-oxidation time to keep floc formation to the minimum (Sharma 2001).

UGF has the advantage that it is a low cost solution and it allows for storing large amounts of solids with a limited increase in head loss (Boller, 1993), facilitating long filter runs. This is basically associated with the high pore size of the gravel material. The study of the mechanisms involved in Fe and Mn removal in UGF has been limited but is relevant to enhance the knowledge about UGF and to contribute to improving the design and maintenance practice. This study contributes to enhancing the knowledge base by comparing the removal of Fe and Mn in UGF operating at high and low oxygen concentrations. The study comprises a laboratory scale test to establish the removal capacity for Fe and Mn on coated gravel from an existing UGF at different pH levels and an experiment with pilot UGFs operated at different filtration velocities.

5.2. Materials and methods

5.2.1. Batch experiment

Batch experiments were carried out to define the Fe and Mn adsorption capacity on the gravel, at high and low oxygen concentrations, with water prepared in the laboratory with fixed Fe and Mn concentrations and different pH levels. For the experiments 2000 ml jars were filled with 100 g gravel (equivalent grain size of 4.35 mm). The gravel was obtained from a full scale UGF that has been treating groundwater with Fe and Mn for over 10 years, enhancing the adsorption (Sharma, 2001). The gravel was washed with demineralized water and dried at room temperature. The jars were filled with a mixture of demineralized water and a stock solution to obtain water with a concentration of 6 mgL^{-1} Fe^{2+} and 2 mgL^{-1} Mn^{2+}. The stock solution comprised ferrous sulphate ($FeSO_4.7H_2O$) (1490 mgL^{-1}, pH<2) and analytical-grade manganese sulphate ($MnSO_4H_2O$), (956 mgL^{-1} Mn^{2+}, pH<2). Each jar was adjusted to the desired level of pH and oxygen concentrations. The experiments were carried out under an ambient temperature of, on average, 25 ºC, and at pH levels of 5, 6, 7 and 8. The pH levels were continuously monitored and maintained at these levels (within a range of 0.05 units) during the experiment by adding 6N HCL or 1N NaHCO3 respectively, whenever required.

For the experiment at low oxygen concentrations nitrogen gas was infused into the solution in the jar to attain and maintain a dissolved oxygen (DO) concentration of less than 0.1 mgL^{-1}. At high oxygen concentrations air bubbles were permanently infused in the jar maintaining a DO concentration of approximately 6.0 mgL^{-1}. For both experiments blank tests were conducted under the same conditions but without the addition of gravel media to be able to confirm the adsorption on the gravel grains. Jars were placed on a shaker operated at 80 rpm to ensure mixing and to prevent possible flocs settling on the gravel, as also used by Buamah (2009).

Sampling was done at time intervals of 0.5 to 1.0 hours to determine the adsorption capacity at equilibrium concentration which was considered to be reached when the difference in Fe and Mn concentrations in two consecutive samples was less than 0.05 mgL^{-1} for Fe and 0.02 mgL^{-1} for Mn (Sharma 2001, Buamah 2009).

5.2.2. Pilot study

In the second phase two pilot filters were used to compare Fe and Mn removal by UGF operating at high and low oxygen concentrations, using natural groundwater from a well localized in Terranova's water supply treatment plant (Jamundí-Colombia). The system consisted of four filter columns (254 mm internal diameter and 1m height) with two lines operating in parallel with sampling taps and piezometer connections (Figure 5.1).

Each line consisted of two filter stages in series, the first filter stage (F1) basically to remove Fe and the second filter stage (F2) for Mn removal and to polish the Fe removal (Table 5.1). The gravel composition of the two stages differed; the first stage with a predominant gravel size between 12.7-6.35 mm to facilitate large pore sizes for greater accumulation of flocs and the second with a predominant gravel size of 6.35-3.2 mm with a larger surface area and smaller pore sizes to facilitate adsorption and to remove small flocs passing the first filtration stage.

Water from the pumping main, coming from the groundwater well, was stored in a tank of 120 liters. From this tank the pilot systems were fed using a hydraulic dispenser of constant head to ensure constant flow even when the pumps of the water system were turned off, which happened occasionally for a maximum period of 2.5 hour. Water from the tank with a DO level < 2 mgL^{-1} was directly supplied to the low oxygen line. In this condition it was expected that Fe and Mn were present as Fe^{2+} and Mn^{2+} and adsorb on the filter grains, while the limited concentration of oxygen allowed for continuous regeneration of the Fe. Given that the Mn in the influent was low (less 0.09 mgL^{-1}) it was decided to dose Mn in the influent of the two filtration lines. The Mn solution was prepared using analytical-grade manganese sulfate ($MnSO_4H_2O$), (956 mgL^{-1} Mn^{2+}, pH<2). The solution was dosed using 500 ml bottles with a tap to allow for drip feeding whilst ensuring that the Mn concentration did not exceed 2 mgL^{-1}. The influent to the filters operating at high oxygen concentrations was aerated. The water flow to the UGF was kept constant and any increase in resistance due to the deposits of Fe flocs and Mn oxides in the filter media was measured as a water level rise in the inlet box. Removal efficiencies were tested for four filtration velocities (*vf*) of respectively 1, 2, 3 and 10 mh^{-1} for a duration of the experiments of 34, 21, 17 and 11 days respectively.

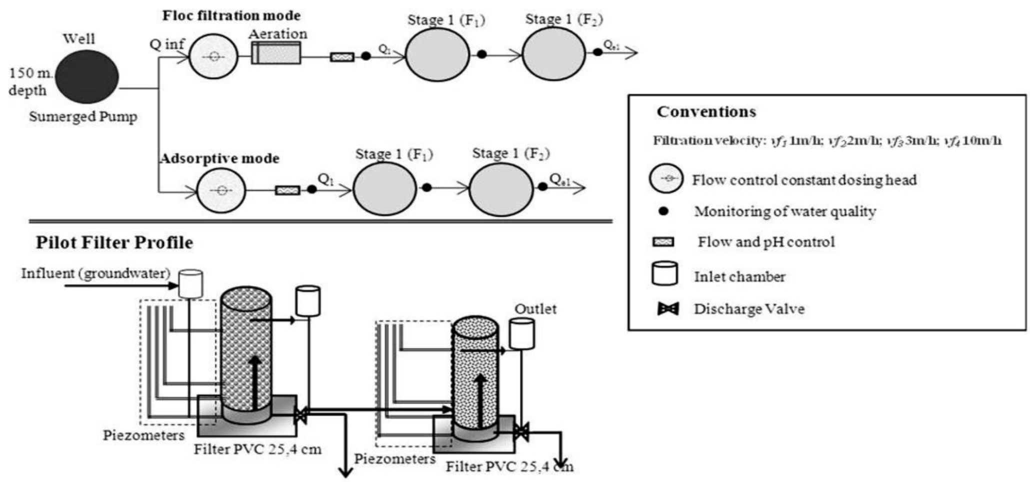


Figure 5.1. Experimental set-up of the pilot plant

Table 5.1. Experimental conditions and specifications of each UGF stage

Experimental conditions	Line 1	Line 2
Expected dominant process	Oxidation-floc formation	Adsorptive-oxidation
Filtration velocity (mh^{-1})	1,2,3 and 10	1,2,3 and 10
Feed water	Aerated well water	Well water with <2 mgL^{-1} O_2
Specification for each filter	**UGF Stage 1 (F1)**	**UGF Stage 2 (F2)**
Filter material	PVC corrugated pipe	PVC corrugated pipe
Diameter (cm)	25.4	25.4
Total height (cm)	100	100
Maximum head loss (cm)	15	15
free border (cm)	5	5
Supernatant water level (cm)	5	5
Filtration area per unit (m^2)	0.051	0.051
Grain size (mm)	**Thickness of each layer of filter bed (m)**	
6.35 - 3.17	-	0.50
12.7-6.35	0.50	0.20
19.05-12.7	0.20	0.20
25.4 - 19.05	0.20	-
Total height of the filter media (m)	0.90	0.90

5.2.3. Procedures and measurements

The head loss (hf) was measured daily with piezometer tubes located at different filter heights (Figure 5.1). When the total head loss surpassed 0.15 m (as suggested by Galvis et al. 1999) the filter was taken out of operation and the filter bed was cleaned. The water intake was closed and the surface of the gravel was washed by raking. Thereafter the filter was drained by quickly opening and closing the drain valves at the bottom (Figure 5.1). The drainage velocity ranged between 40-50 mh^{-1}, determined by measuring the fall in water level in the filter (Δh) over time (t). Cleaning was repeated once and for the second cleaning the filter was refilled up to a water level of 0.10 m above the surface of the gravel. Immediately after cleaning the filter was put back into operation. Before a new filter run, with a different filtration velocity, was initiated the gravel was taken out of the filter and washed with clean water.

Volumetric flow measurements were daily carried out at the inlet. In the batch and pilot experiments the pH, oxygen and temperature were measured using a multi-parametric device (Orion 4star thermo scientific). Turbidity measurements in the pilot plant were performed using a Hach 2100P portable turbidimeter. Fe and Mn in the batch and pilot experiments were measured using standard methods (APHA, 2005). In the case of the batch tests at high oxygen concentrations at pH 7 and 8, however, a different method was used. Tamura et al. (1974) indicate that using the standard method does not give adequate results when Fe^{2+} is in the presence of a concentration of Fe^{3+}, above 10 mgL^{-1}. As this was the case in these batch tests the modified method for Fe^{2+} measurement developed by Tamura was used. The detection level was 0.02 mgL^{-1} for Mn and 0.05 $mgL^{-}1$ for Fe. Samples for Fe and Mn were put directly into acid containing bottles to set the pH below 2. Hydrochloric and nitric acid were used to stabilize the samples with Fe and Mn respectively. All samples were kept cool and analyzed within 24 hours after sampling.

5.2.4. Data analysis

During the batch experiments the adsorption efficiency (%) was calculated by using Equation (5.1) and the amount of adsorbed Fe and Mn per m^2 of gravel (*qs*) with Equation (5.2). The pilot study was conducted under continuous flow and the same equations were used.

$$Adsorption\ efficiency\ (\%) = (Co - Cs)\ /\ Co\ * 100 \qquad (5.1)$$

$$q_S = \frac{V*(C_O - C_S)}{A_S} \qquad (5.2)$$

Where: V is the volume of the solution (L), *As* is the geometric surface area of used gravel mass (m^2), *qs* (gm^{-2}) is the amount of removed Fe and Mn per m^2 of gravel surface. In batch experiments: *Co* is the initial concentration in the liquid phase for Fe or Mn (gm^{-3}); *Cs* is the final concentration of Fe and Mn in the liquid phase (gm^{-3}). In pilot filter: *Co* is influent Fe or Mn concentration (gm^{-3}); *Cs* is effluent Fe or Mn concentration (gm^{-3}); Fe^{3+} concentration was determined by the difference between the concentration of total and dissolved Fe.

A sieve analysis was carried out to obtain the geometric surface (*As*) with the method established by Huisman (1986). The average weight of the fractions retained on each sieve was measured and the shape factor for each fraction was established based on a comparison of the grain form of each fraction with the forms and related shape factor reported by Di Bernardo and Sabogal (2008).

5.3. Results and discussions

5.3.1. Iron and manganese removal during batch experiments

Batch experiments were done to determine the best pH for the operation of the pilot filter experiments, and to verify if the coated gravel from an existing UGF, envisaged to be used in the pilot filter, had the ability to adsorb Fe and Mn. The main findings related to Fe and Mn removal at different pH levels in the batch experiments are shown in Figure 5.2 and Table 5.2. Best removal was obtained for Fe at pH 7 at high oxygen concentrations, reaching an ultimate concentration of 2.15 mgL^{-1} after 5 hours. At low oxygen concentrations this was 2.48 mgL^{-1} after 5.5 hours. The time to reach the ultimate concentration was quite similar for pH 5-7, while for pH 8 the time was shorter. Differences in the ultimate concentrations for Fe were small or negligible between high and low oxygen concentrations for pH 8 (Figure 5.2a; Table 5.2), while for Mn small differences were observed for varying pH. For all pH levels the Fe^{2+} concentration dropped sharply during the first hour which suggests a rapid adsorption on the surface area of the gravel due to an effective external mass transfer (Buamah 2009) and/or a rapid oxidation during high oxygen concentrations. The ultimate concentrations of Mn were also almost similar at low and high oxygen concentrations and were best at pH 8. The result at pH 8 was also reached faster than for lower pH levels (Figure 5.2b; Table 5.2), while the effect of pH being larger than for Fe. The blank test performed at pH 7 at low and high oxygen concentrations in the jar without gravel (Figure 5.2a and 5.2b) resulted in negligible reduction of Fe and Mn. This result suggests that removal of Fe and Mn was mainly by adsorption on the gravel or heterogeneous oxidation and also that catalytic oxidation at higher pH and high oxygen concentrations by the formed Fe^{3+} flocs was negligible.

Findings at high oxygen concentrations are in line with Stumm and Lee (1961) who have reported that oxidation of ferrous Fe should be expected to occur rapidly in oxygenated waters at pH values exceeding 7.0. Best removal results for Fe at pH 7 at low oxygen concentrations confirms the findings of Mouchet (1995) and Sharma (2001), who reported an optimum pH of 6.5-7.0 using coated sand for adsorption.

However the small difference between the removal at high and low oxygen concentrations, suggests that Fe removal was basically at the surface of the gravel and probably the rate limiting step is the transport of the iron to the gravel surface. At low oxygen concentration Fe^{2+} is absorbed on the gravel, while for high oxygen concentration it can be adsorbed on the gravel and Fe^{3+} flocs can be formed by oxidation (catalytic effect). The small difference between the Fe^{2+} removal at high and low oxygen concentration might indicate low Fe^{2+} adsorption on Fe^{3+} flocs, which is in line with Tamura et al. (1976) who found that only 17% of Fe^{2+} was adsorbed in flocs dosing an excess of 100 mgL^{-1} of Fe^{3+}, thus suggesting that adsorption and heterogeneous oxidation on the gravel are the most important mechanisms in these tests.

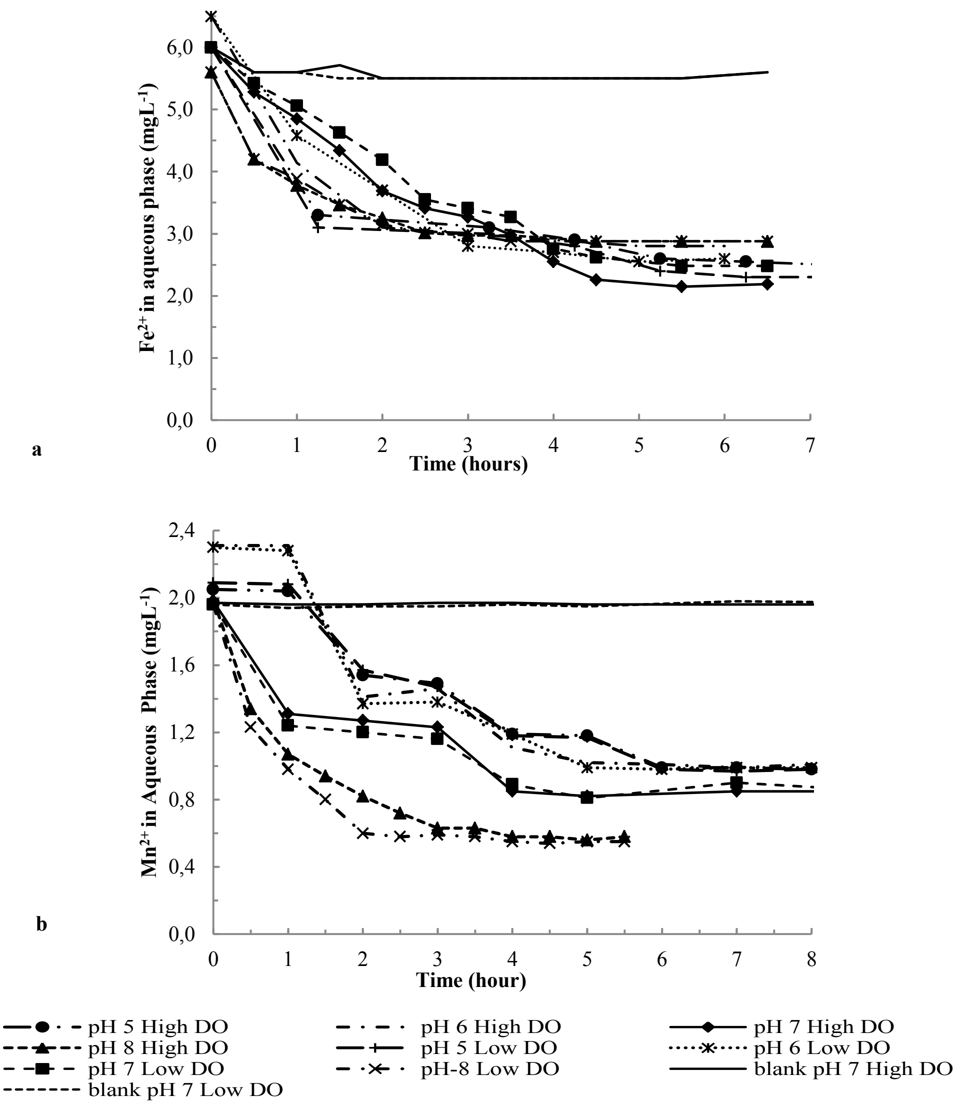


Figure 5.2. Fe and Mn adsorption in batch reactors at high and low oxygen concentrations: a) Fe^{+2}; b) Mn^{+2}; (pH 5-8).

Results for Mn removal are in line with Buamah (2009) who reported an optimum pH of 8.0 for Mn adsorption on different filtering materials including sand. The Mn adsorption may be associated with the negative charge of Mn oxides (Liu *et al.* 2004) and the auto-catalytic oxidation of adsorbed Mn (Buamah 2009).

Table 5.2. Fe^{2+} and Mn^{2+} adsorption in batch reactors for different pH (batch experiments 100 g coated gravel).

pH		5		6		7		8	
Experiment conditions		HOC	LOC	HOC	LOC	HOC	LOC	HOC	LOC
Time to reach ultimate concentration , (hrs)	Fe	5.25	5.25	5.0	5.0	5.0	5.5	4.5	4.5
	Mn	6.0	6.0	6.0	6.0	5.0	5.0	4.0	4.0
Ultimate concentration, (mgL^{-1})	Fe	2.4	2.6	2.8	2.55	2.15	2.48	2.88	2.88
	Mn	1.0	0.99	0.98	0.97	0.8	0.81	0.58	0.55
Fe^{2+} adsorbed (%)		58	62	57	60	64	59	49	49
Adsorbed Fe^{2+} (gm^{-2} of gravel)		0.058	0.063	0.063	0.064	0.065	0.060	0.046	0.046
Mn^{2+} adsorbed (%)		52	53	56	57	59	57	72	72
Adsorbed Mn (gm^{-2})		0.019	0.019	0.022	0.022	0.020	0.019	0.024	0.025

HOC: High oxygen concentration; LOC: Low oxygen concentration; values reported for ultimate concentration.

The time to reach the ultimate concentration of Fe and Mn was in line with experiments done by Sharma (2001) for the adsorption of Fe on coated sand with sizes less than 1.5 mm (4-6 h) and Buamah (2009) for Mn adsorption in pulverized Fe oxide coated sand at high and low oxygen concentrations (5 h). The use of previously coated gravel with Fe oxide enhanced the capacity to adsorb and oxidize new Fe and Mn, contributing to the creation of new adsorption and oxidation sites. When comparing with the results of Sharma (2001) for coated sand (sand grains of 0.18 gm^{-2}, for a test with 50 g of sand media) specific Fe adsorption and oxidation was three times lower than can be explained by the smaller total surface area of the grains of the gravel.

5.3.2. Pilot plant experiments

The results of the removal of Fe and Mn for the two-stage pilot plants, operating at four different filtration velocities and at high and low oxygen concentrations, are presented in Table 3. Natural groundwater was used with a pH that was slightly above 7. In Table 5.3 also information on the average levels of oxygen, temperature, filtration velocities and Fe and Mn concentrations for the different experimental periods is given. The pH and the temperature were similar for all filter runs. Oxygen concentrations showed some variations between the filter runs and in most cases it was lower in the second stage of the filtration set-up (F2).

Fe levels in the raw water varied which was probably caused by changes in the pumping regime, as the wells have multiple well screens tapping from different groundwater layers. For Mn this was different, since dosing of $MnSO_4H_2O$ was constant.

Table 5.3. Overview of water quality parameters in the four test runs pilot UGFs.

Experiment		Influent high oxygen concentration						Influent low oxygen concentration					
		F1			**F2**			**F1**			**F2**		
		pH	DO (mgL^{-1})	t (∘c)	pH	DO (mgL^{-1})	t (∘c)	pH	DO (mgL^{-1})	t (∘c)	pH	DO (mgL^{-1})	t (∘c)
T1 *vf* 1 mh^{-1} Fe 2.6 mgL^{-1} (STD 0.49)	Av.	7.2	5.6	26.5	7.5	4.4	26.6	7.1	1.4	26.6	7.1	1.0	26.8
Mn 1.34 mgL^{-1} (STD 0.45)	STD	0.2	1.6	0.6	0.3	1.7	0.3	0.3	0.5	0.5	0.3	0.5	0.7
T2 *vf* 2 mh^{-1} Fe 3.6 mgL^{-1} (STD 1.4)	Av.	7.0	6.2	26.4	7.2	2.9	26.3	6.9	0.8	26.4	7.0	1.4	26.5
Mn 1.57 mgL^{-1} (STD 0.45)	STD	0.2	1.0	0.4	0.3	0.9	0.4	0.2	0.4	0.5	0.2	0.4	0.5
T3 *vf* 3 mh^{-1} Fe 3.1 mgL^{-1} (STD 0.7)	Av.	7.0	6.4	26.2	7.3	4.1	26.1	7.1	1.5	26.3	7.3	1.8	26.4
Mn 1.32 mgL^{-1} (STD 0.26)	STD	0.3	1.2	0.3	0.3	1.1	0.5	0.3	0.5	0.4	0.4	0.3	0.6
T4 *vf* 10 mh^{-1} Fe 1.9 mgL^{-1} (STD 0.2)	Av.	7.4	6.1	26.0	7.5	4.6	26.0	7.3	0.7	26.0	7.3	0.40	26.0
Mn 0.72 mgL^{-1} (STD 0.13)	STD	0.3	0.1	0.2	0.2	0.3	0.2	0.2	0.1	0.2	0.2	0.1	0.2

T1-T4: Test 1-4, STD: standar deviation

5.3.3. Iron removal under high and low oxygen operation

The results of the pilot plant performance operated at *vf* of 1 mh^{-1} are shown in Figure 5.3. The results show an initial period of respectively three days at high oxygen concentrations and two days at low oxygen concentrations with lower removal efficiencies. The graphs for higher filter rates are similar but do not show the initial period of lower performance. After an initial ripening period, both conditions resulted in considerable removal efficiencies for total Fe. Ibrahim (1997) reported that Fe coated media has much higher efficiencies for Fe removal compared to new media and Sharma (2002) also mentioned that the adsorption capacities of different coated sands were 10 to 55 times higher than that of new sand. The use of old coated gravel thus contributed to the high removal efficiencies.

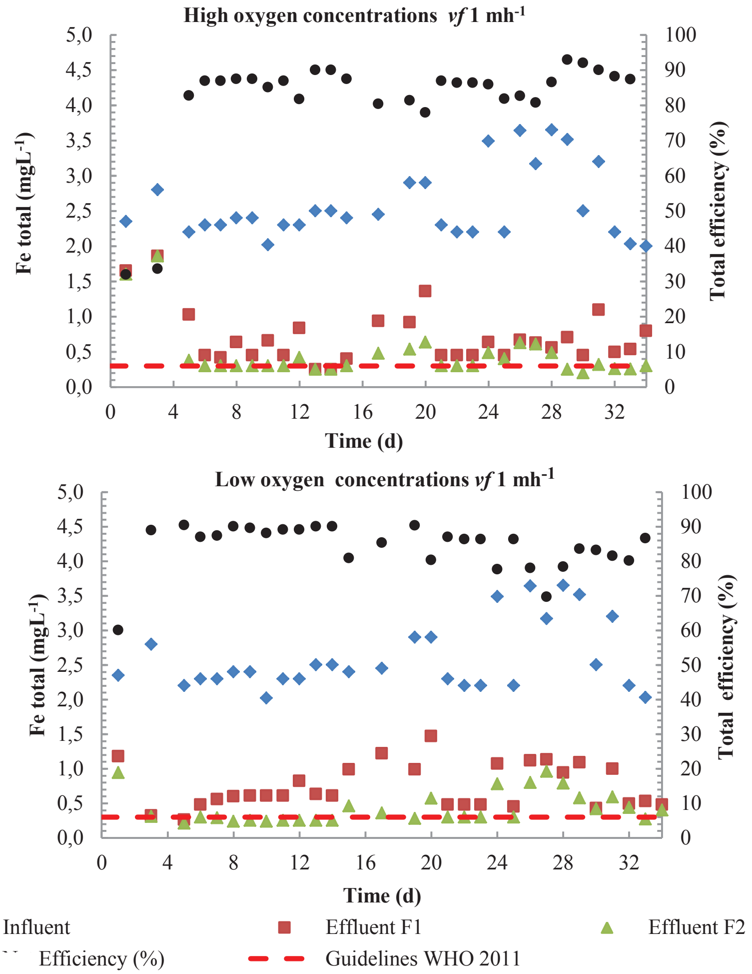
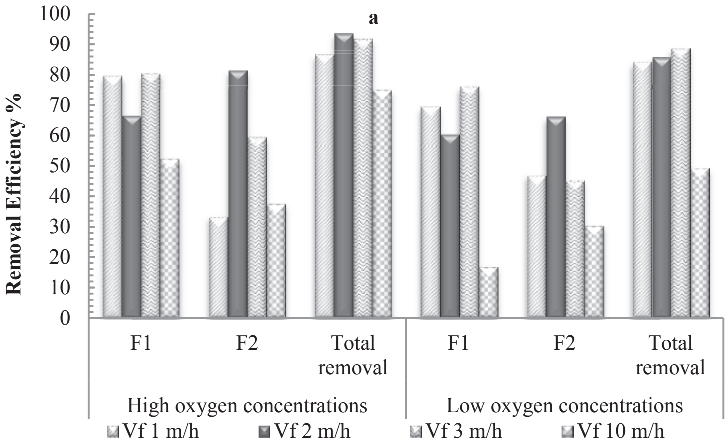


Figure 5.3. Total Fe removal in UGF at high and low oxygen concentrations ($vf = 1\ mh^{-1}$).

Figure 5.4 presents the median removal efficiencies for total Fe (Fig. 5.4a) and Fe^{2+} (Fig 5.2b) of the pilot plant operated at different *vf*. Medium overall removal efficiencies for both total Fe and Fe^{2+} were similar for the low *vf* (1, 2 and 3 mh^{-1}) and the two operation modes, with a slightly better performance for a *vf* of 2 mh^{-1}. Here the overall removal for Fe^{2+} was 95% at high oxygen concentrations and 93.2% at low oxygen concentrations. For a *vf* of 2 mh^{-1} the efficiency was however lower in F1 in comparison to the other two *vfs* and higher in F2. This may have been caused by the higher Fe concentration in this experimental period which may have led to more Fe passing to F2. This higher concentration enhanced the retention in F2 for both operational conditions.

When *vf* was increased to 10 mh^{-1}, the removal efficiency of Fe in both operational conditions was reduced, with a reduction being more pronounced at low oxygen concentrations. Total removal efficiency at high oxygen concentrations was 75% while at low oxygen concentrations it reached only 49%. Fe removal at high oxygen concentrations was higher in F1 than in F2.



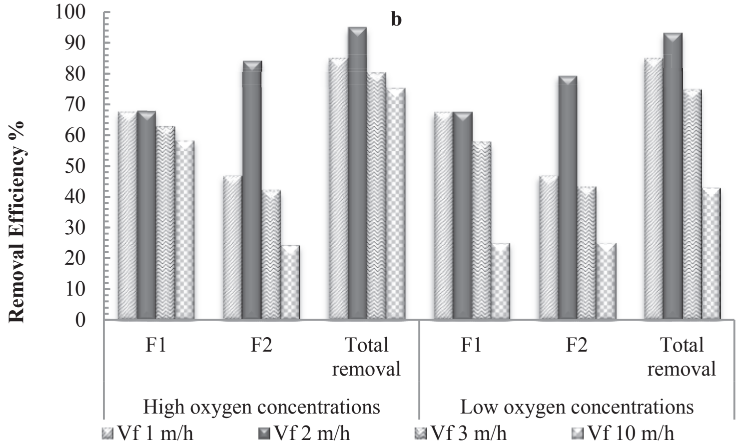


Figure 5.4. Median removal efficiency in UGF: a) total Fe; b) Fe^{2+} (*vf* 1-10 mh^{-1})

5.3.4. Manganese removal at high and low oxygen concentrations

Best Mn removal was obtained for *vf* 2 mh^{-1} (Figures 5.5 and 5.6). A ripening period was observed for both high and low oxygen concentrations of respectively three and four days. Buamah (2009) reported that the ripening of Mn removal on new media is slow, taking several weeks till months. The Fe oxide present within the coating media possibly enhanced the autocatalytic oxidation of adsorbed Mn^{2+} and thereby the formation of Mn oxides (Junta & Hochella 1994) contributing to better removal efficiency in both operation conditions. The use of old coated gravel thus favored the ripening period. Median overall removal efficiencies for both operational conditions were similar: 95% at high oxygen concentrations and 92% at low oxygen concentrations. For *vf* 1, 2 and 3 mh^{-1}, differences in the effluent were observed in the first stage (F1) for the two operational conditions and the Mn removal efficiency in F1 at high oxygen concentrations was higher than in F1 at low oxygen concentrations. Likely at high oxygen concentrations Fe hydroxides contributed to the adsorption of Mn as it has been indicated by Morgan (1964) and Weber (1972). In both operational conditions the two filtration stages contributed to the Mn removal. According to Figure 5.6 the operation with *vf* = 1 mh^{-1} showed few differences in the average removal efficiency of Mn by F1 and F2. However for *vf* = 2 and 3 mh^{-1} the removal efficiency in the second stage (F2) in both operational conditions was better than the first stage (F1), but F2 at high oxygen concentrations showed the best performance.

For *vf* = 10 mh^{-1} total removal efficiency at high oxygen concentrations was 52.2% while at low oxygen concentrations it was only 28.6%.

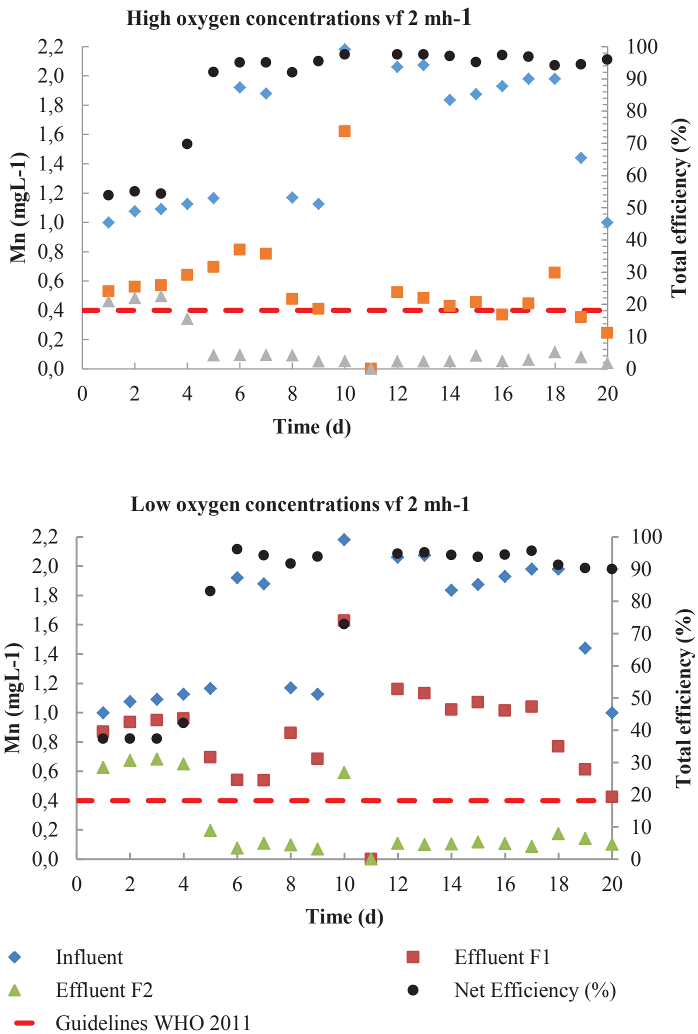


Figure 5.2. Mn removal in UGF at high and low oxygen concentrations (*vf* 2 mh^{-1})

Best removal efficiencies of Mn in F2 for both operational conditions in the pilot experiments can be attributed to the larger surface area of the filter medium available in F2, allowing for a favorable condition for Mn retention, also because large parts of the Fe was already retained in F1, since Fe^{2+} can compete with Mn^{2+} for adsorption sites and affect Mn removal (Po et al. 2004).

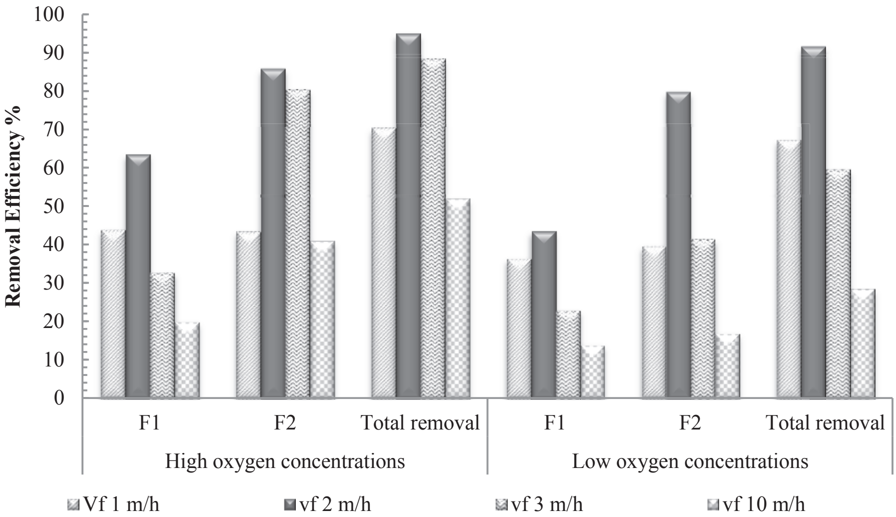


Figure 5.6. Median removal efficiency for Mn in UGF at high and low oxygen concentrations

5.3.5. Head loss development

Head loss development in the pilot filters was different for the filtration stages, operational conditions and filtration velocity. Figure 5.7 shows the head loss development in the first filtration stage for *vf* = 1 and 10 mh^{-1}; for *vf*= 2 and 3 mh^{-1} a summary of the results are shown in Table 5.4. For *vf* = 1 mh^{-1} F1 at high oxygen concentrations had a longer filter run than at low oxygen concentrations being 21 days and 12 days respectively (*hf* max 0.15 m). The shorter filter run at low oxygen concentrations is counter intuitive, and seems to be the result of differences in the mineralogical and morphological shape of the formed Fe hydroxides (Cornell & Schwertmann, 2003). The presence of Fe in the form of gelatinous agglomerates adhering to the gravel and pores was observed (Figure 5.8) and suggests possible biological iron oxidation in agreement with De Vet (2010) who indicated that biological oxidation by *Gallionella spp* was the dominant process for iron oxidation in groundwater at neutral pH in oxygenated water. Although the distinction between chemical and biological iron oxidation is difficult to make, some amorphous iron oxyhydroxides, similar to chemical precipitates, are found to be of biological origin (Emerson and Weis, 2004).

Figure 5.7 shows that for *vf* = 1 mh^{-1} in both operational conditions *hf* was concentrated in the first gravel layer (*h* = 20 cm) located at the bottom of the unit, followed by the second layer (*h* = 20 cm). Increasing *vf* to 2 and 3 mh^{-1} showed that the first and third gravel layers mainly contributed to the *hf*. Increasing *vf* to 10 mh^{-1} resulted in a concentration of the head loss development in the third layer of the gravel bed in both operational conditions, with limitations for the filter cleaning.

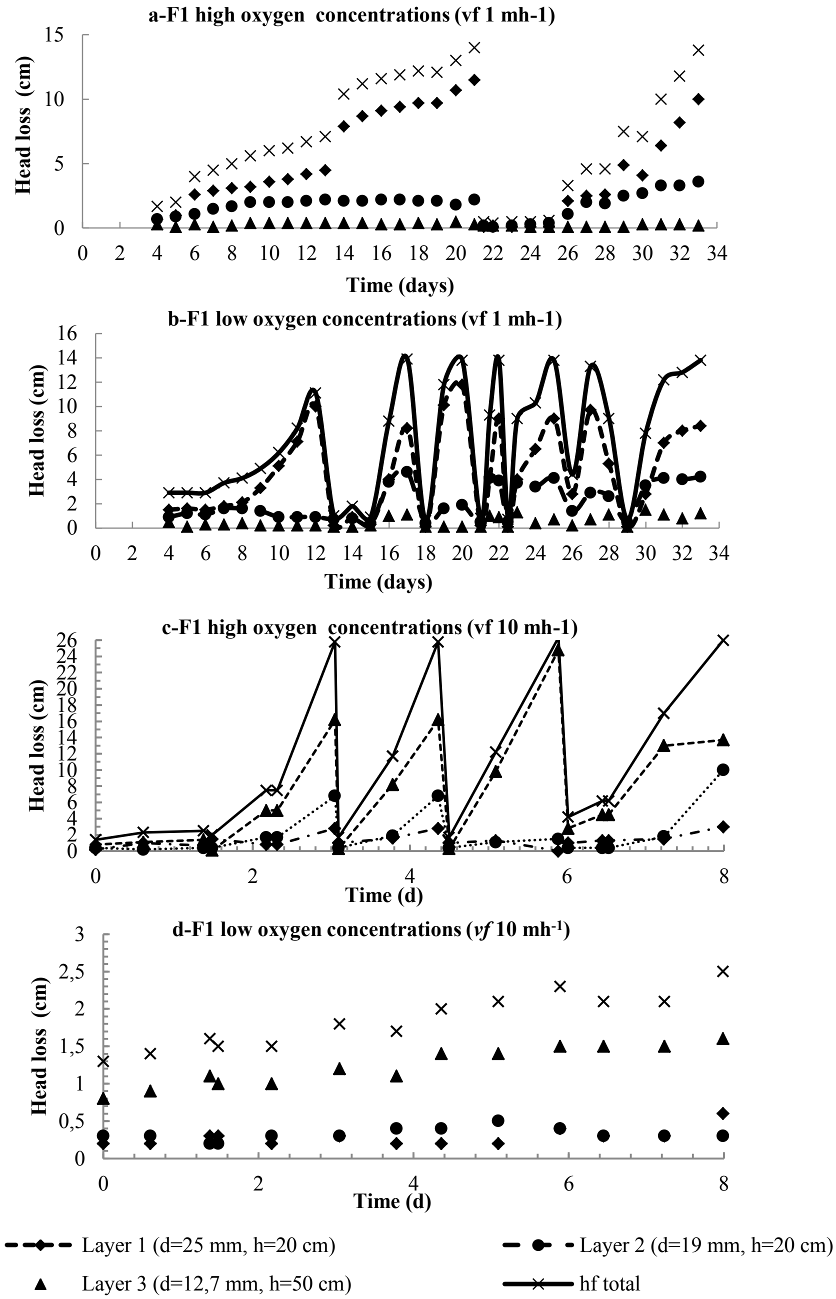


Figure 5.7. Head loss development in the first filter stage (*vf* 1-10 mh^{-1})

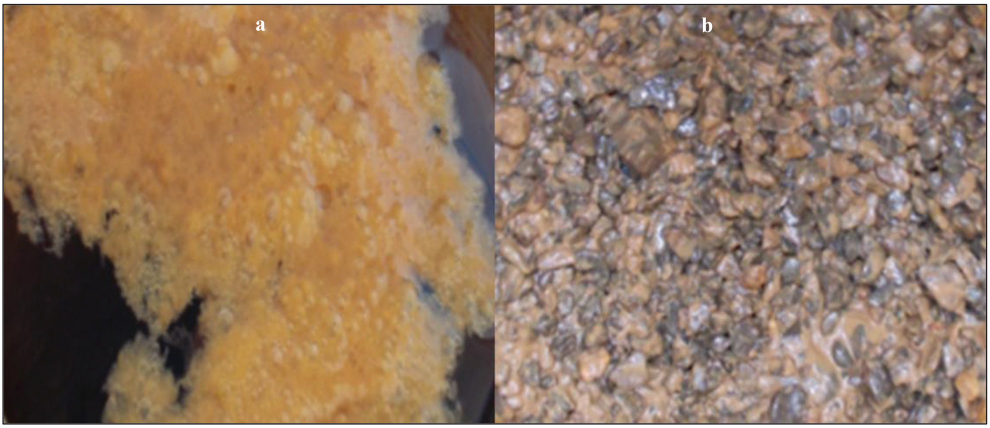


Figure 5.8. Photograph of Fe in the form of gelatinous agglomerates in line at low oxygen concentrations: (a) at the inlet of F1 and (b) in the gravel media of F1.

A synthesis of the results for maximum head loss and filtration run in each filter stage is presented in Table 5.4. These results indicate that: the *vf* = 1 mh^{-1} recorded the longest filter run; *vf* = 10 mh^{-1}, as well, had a long filtration run and a low *hf* in F1, because little Fe was retained; *vf* 2 and 3 m/h at high oxygen concentrations had the same filter run length in F1, but *vf*= 3 mh^{-1} produced more volume of water, which is important to reduce investment costs, but F1 at low oxygen concentrations for both *vf*s, had a shorter filtration run than F1 at high oxygen concentrations; F2 in both operational conditions had longer filter runs than F1 for low *vfs* (1, 2 and 3 mh^{-1}); when *vf* increased to 10 mh^{-1} F2 had a shorter filtration run and an increased *hf,* because biofilm was formed as was shown in Figure 5.8 , but in this case at the inlet of F2.

Table 5.4. Maximum head loss and filtration run in each stage at high and low oxygen concentrations.

vf **(mh^{-1})**	**High oxygen concentrations**				**Low oxygen concentrations**			
	F1		**F2**		**F1**		**F2**	
	hf **max (cm)**	**Fr (d)**	***hf*** **max (cm)**	**Fr (d)**	***hf*** **max. (cm)**	**Fr (d)**	***hf*** **max (cm)**	**Fr (d)**
1	14	12-21	1,5	> 32	14	2-12	9,5	> 32
2	15	1-3	14	12	15	1-2	15	2-4
3	15	1-3	15	14	15	1	14	13
10	26	1-3	26	1.5-4	2.5	> 8	26	0.7- 2

Fr: filtration run (days); *hf:* maximum head loss

5.3.6. Interrelationship between batch and pilot experiments

Batch experiments allowed to define the pH conditions for the operation of pilot filters and revealed the adsorption capacity of Fe and Mn on coated gravel grains, showing little difference between high and low oxygen concentrations for Fe and Mn removal. Small differences for Mn removal at high and low oxygen concentrations were also observed in the pilot filter experiments for low *vf* (1-3 mh^{-1}). However, in the pilot experiments it was observed that Fe hydroxide flocs were formed at the entrance due to the influence of the pre oxidation, these were retained in the filters both at high and low oxygen concentrations contributing to the Fe removal.

The removal of Fe and Mn was higher in the pilot filter than in the batch test, which may be caused by several factors. In pilot filters both Fe and Mn were present and the removal efficiency may be influenced by a contribution of both the Mn oxides and Fe oxides; while in the different batch tests removal of Fe and Mn were tested separately. In addition, in the pilot filters the aeration and the pre-oxidation may have contributed to Fe^{3+} formation which is accumulated in the filter and in turn allows Fe^{2+} adsorption on Fe flocs and Mn oxides. In the batch test at high oxygen concentration the effect of Fe flocs on the gravel was avoided by continuous mixing, and the effect of Fe^{2+} adsorption on Fe^{3+} was very limited, possibly because the amount of Fe^{3+} was not very high. The continuous flow in the pilot filter also facilitated contact with the gravel surface and therewith probably enhancing removal efficiency in comparison with the batch tests.

5.3.7. Practical implications

Operation at low oxygen concentrations makes maintenance difficult due to the formations of Fe^{3+} in the form of gelatinous agglomerates. In order to assure a long filtration run at low oxygen concentrations, it is key to reduce the biofilm formation, which can be achieved by reducing oxygenation at the entrance even more or work under high oxygen conditions. In practice, the first option is difficult to achieve.

Results showed reductions of the filtration run time for *vf* = 2, 3 and 10 mh^{-1} at high oxygen concentrations but also an effect on filter operation and maintenance. Particularly for high *vf,* the head loss increased and the cleaning frequency was considerably higher resulting in a higher loss of water for filter bed cleaning, more work by the operator and higher maintenance costs.

At low oxygen concentrations it was observed that at *vf* = 10 m/h, because the retention of Fe and Mn in F1 was lower, a long filtration run was reached, contributing to reducing cleaning frequency. Therefore a combination of two conditions of operation may be considered: a first stage of pretreatment at low oxygen concentrations to remove part of Fe and Mn by adsorption and oxidation in the gravel, e.g. *vf* = 10 mh^{-1}, followed by a stage at high oxygen concentrations operating at *vf* between 1-3 mh^{-1} to remove the remaining by floc formation.

5.4 Conclusions

This chapter explored the removal of Fe and Mn from groundwater by UGF at high and low oxygen concentrations. The study started with batch experiments to establish the adsorption capacity for Fe and Mn on coated gravel at different pH levels. The second step was to compare the two different oxygen concentrations for Fe and Mn in pilot filters, taking into account the effect of different filtration velocities.

The study used gravel (4.35 mm) from an existing upflow gravel filters which is already in operation for more than 10 years. The batch experiments showed small differences in both

Fe and Mn removal efficiency at high and low oxygen concentrations for a pH range between 5-8 units. In both conditions however the pH had an influence with best retention capacity for Fe at pH 7 and for Mn at pH 8.

Little difference in the removal efficiency for Fe and Mn was found at high and low oxygen concentrations in the pilot study at low filtration velocity (1, 2 and 3 mh^{-1}). For *vf* = 10 mh^{-1}, the removal efficiency of Fe and Mn was considerably lower at low oxygen concentrations and this effect was stronger for Mn.

Head loss in the pilot filters differed with filtration stages, operational conditions and filtration velocity. However, the head loss development was more affected in the line with low oxygen concentrations due to possible formation of Fe hydroxide with a different mineralogical and morphological shape and/or biological iron oxidation and biofilm formation. For low *vf* the filter run time at low oxygen concentrations was shorter than at high oxygen concentrations. However increasing the *vf* to 10 mh^{-1} resulted in longer filtration run times in F1 at low oxygen concentrations, while at high oxygen concentrations the filter run times were shorter.

This consideration together with the Fe and Mn removal results that were obtained suggest that a possible combination of a first stage of pretreatment at low oxygen concentrations at *vf* = 10 mh^{-1} to remove part of Fe and Mn by adsorption and oxidation followed by a stage at high oxygen concentrations operating at *vfs* between 1 and 3 mh^{-1} to remove the remaining by floc formation may be a promising option for Fe and Mn removal at lower cost.

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CHAPTER 6

Impact of upflow gravel filtration on the clogging potential in microirrigation*

This chapter evaluates the effect of six water treatment combinations for the treatment of turbid surface water on four types of emitters by looking at clogging potential and distribution uniformity. The pilot plants use different combinations of upflow gravel filtration, disc filters, and slow sand filtration, fed with natural water from the Cauca River (Cali, Colombia). The outflow of these systems was used to feed four different types of emitters. This chapter analyzes the removal of physicochemical parameters that affect emitter clogging and the distribution uniformity of the lower quarter *(DUlq).* All treatment schemes reduced clogging potential, with the best performance being obtained with upflow gravel filtration followed by slow sand filtration and up flow gravel filters in series. The effect on *DUlq* differed for the four types of emitters. The longest irrigation time was obtained for the pressure compensated emitter and for the Lyn emitter with 600 h of continuous operation until the *DUlq* was below 80%.

*This chapter is based on:

Sánchez, L.D., Valencia-Zuluaga, V., Echeverri-Sánchez, A., Visscher, J.T. & Rietveld, L.C. 2016. Impact of upflow gravel filtration on the clogging potential in microirrigation. Journal of Irrigation and Drainage Engineering. volume 142, Issue 1 (January 2016).

6.1. Introduction

The agricultural sector is responsible for about 61% of the water demand in Colombia, with efficient low water use accounting for only about 40% of the total (Urrutia 2006). Other water-related problems in agriculture include poor water quality and poor governance. One approach to enhance water use efficiency is the introduction of microirrigation, a technique that only supplies water in the root zone of the crops and uses less water than conventional irrigation by gravity or aspersion. The water is supplied frequently, in small amounts, to maintain constant soil humidity, resulting in higher plant productivity (Pizarro 1996; Noble 2007). Microirrigation is mainly applied by farmers with less than 3 ha of cultivated area. These small farmers, however, are important as they are responsible for a considerable portion of crop production in Colombia (Urrutia 2006).

These microirrigation systems face progressive emitter clogging (Capra and Scicolone 2007; Goyal and Ramírez 2007) due to the presence of physical, chemical, and microbiological substances in the water (Nakayama and Bucks 1991; Martínez 2001). This problem is associated with the small emitting diameter that is necessary to guarantee equal water distribution in low-flow conditions, less than 16 Lh^{-1} for drip emitters and 16–150 Lh^{-1} for bubbler and microsprinkler systems (Pizarro 1996). Emitter clogging affects the irrigation distribution uniformity of the lower quarter (*DUlq*) (relation between the average flow received by 25% of emitters and average flow of emitters evaluated) and is directly related to production uniformity and plant growth (Andersson 2005).

Researchers such as Puig-Bargués et al. (2005) have noted that irrigation water must be treated before distribution to reduce the potential of clogging. In Colombia, water treatment before microirrigation is carried out centrally as well as at the individual farm level. Centralized systems often consist of a water intake, a transport main, treatment for sedimentation, storage facilities, and a distribution system. The limited level of treatment through sedimentation makes water quality improvement an individual responsibility at the farm level (Arango 1998). The main systems that are being used in Colombia for water treatment in irrigation systems are disc filters (DF) and hydrocyclones. A disc filter consists of a cartridge of slotted rings that are tightened to each other, leaving small slots permitting the water to pass but retaining particles larger than the slot size (Regaber 2001). Resistance gradually increases until the filters need to be backwashed, which is done automatically by a process that reverses the flow and expands the space between the rings. Hydrocyclones are cone-shaped devices that can be used to remove particles by centrifugal force. The selection of the type of irrigation treatment system is usually established with the help of guidelines that relate the water quality of the source to its preferred treatment option.

An attractive water treatment alternative is the use of upflow gravel filtration (UGF). UGF is successfully employed as a component in multiple-stage filtration systems used for drinking water treatment where it is applied in combination with slow sand filtration (Galvis et al. 1999). UGF is effective in the removal of suspended solids and is easy to operate such as it was presented in chapter 3.

This chapter explores the potential application of UGF for microirrigation, different UGF combinations were tested and compared with other treatment systems that are used in Colombia. Each system was tested by using treated water from four commonly used emitters (Microjet, Lyn emitter, pressure compensated emitter, and drip tape) to analyze the clogging potential and the *DUlq*.

6.2. Materials and methods

6.2.1. Set-up of the pilot treatment system

A pilot system was used comprising two different treatment lines in parallel, testing a total of six system in three periods (Figure 6.1) as the time-consuming water sampling did not allow testing of all six systems in parallel. To avoid possible accumulation of suspended solids on the emitters, during each trial period new systems were used. The treatment systems were operated for a period of 46 days before starting the test with the emitters, permitting the maturation of the treatment systems.

Figure 6.1 shows the pilot system for treatment of irrigation water. The raw water first passes through a dynamic roughing filter (DRF) to protect all the treatment lines from excessive peaks of suspended solids. The treatment lines T3 and T5 have the same treatment system, but in T5, the UGF and the slow sand filter were operated at higher filtration velocity in comparison to T3 (Table 6.1). T4 only comprised a DF (Azud), whereas in T6 the DF was preceded by an UGF as it was expected that this would improve DF maintenance and performance. All the treatment schemes operated continuously, and the effluents were transported to small storage tanks, from where water was pumped to the four different microirrigation systems.

The systems worked continuously for a period of between 11 and 33 days. The operation of an emitter was stopped when the *DUIq* dropped below 80%. The design characteristics of each treatment stage are presented in Table 6.1.

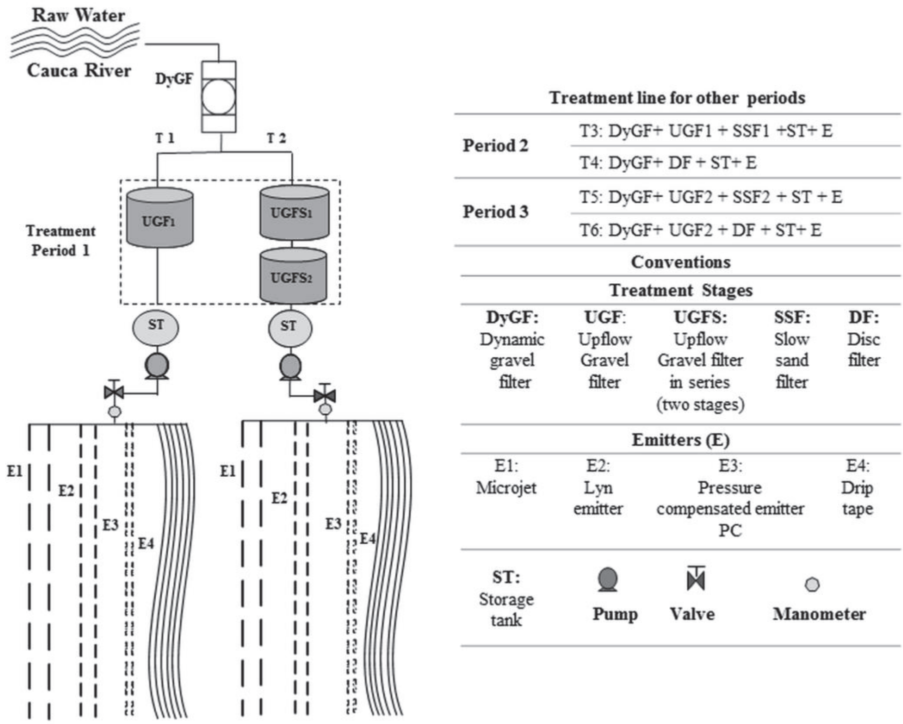


Figure 6.1. Scheme of the pilot plant, where DyGF = dynamic gravel filter; UGF = upflow gravel filter; UGFS = upflow gravel filter in series; SSF = slow sand filtration; DF = disc filter; ST = storage tank; E = emitters

The design and operation characteristics for the emitters are shown in Table 6.2. The design flow for the four emitters together was 0.16 Ls^{-1}, equivalent to an irrigation flow of 0.04 Ls^{-1} per emitter. As no information was provided in the catalogues of the emitters, the value of manufacturing coefficient of variation was assumed to be <0.05 and this was used to design the irrigation module and calculate the minimum flow. Although emitters differ particularly in the required level of filtration (the smallest size of particle that needs to be removed from the water by the filter), the shape of the emitter, and the water distribution in the lateral, operating conditions ensured low head loss in the laterals to minimize the impacts on the flow distribution of the emitters. The maximum head loss allowed for the disc filter is considerably higher than those for the other filters, but at the same time this system has a much smaller surface area and operates at a high filtration velocity. The effect of this difference can be seen in the shorter filter runs for the disc filter.

Table 6.1. Design Features of the Treatment Stages

Feature	DyGF	UGF	$UGFS_1$	$UGFS_2$	SSF	RF
Filtration velocity (mh^{-1}): Period I and II: Period III	2.0 2.0	UGF_1: 0.60 UGF_2: 0.70	0.60 -	0.60 -	SSF_1:0.2 SSF_2:0.4	- -
Maximum head loss (KPa)	-	2.94	2.94	2.94	4.90	29.42
Surface area (m^2)	1.9	2.93	2.93	2.93	2.93	-
Thickness of the filter bed (m)	0.60	1.55	1.50	1.40	1.15	-
			Gravel layers			
bottom layer h_1 (d:19-25 mm) (m)	-	0.30	0.30	0.30	-	-
h_2 layer 2 (d:13-19 mm) (m)	-	0.30	0.60	0.30	-	-
h_3 layer 3 (d:6-13 mm) (m)	-	0.30	0.60	0.40		
h_4 layer 4 (d:3-6 mm) (m)	-	0.30	-	0.40	0.15	-
h_5 upper layer 5 (d:1.6-3 mm) (m)	-	0.35	-	-	-	-
h sand (d_{10}:0.15-0.30 mm) (m)	-	-	-	-	1.0	-
Filtration level (μm)	-	-	-	-	-	130

The Microjet (Aqua-Traxx), a micro-sprinkler that discharges water under different angles, has fewer emitters per line than the other systems, thus having a higher flow per emitter than the other systems. The Lyn emitter (Queen-Gil) consists of a flattened dropper with a sinuous path, while the size of the emitter facilitates operation at a lower water quality, allowing long operation periods. The pressure compensated (Aqua-Traxx) emitter also consists of flattened droppers with a sinuous path, but this maintains a constant flow independent of pressure variations. The drip tape (Queen-Gil) consists of two parallel ducts with a high number of emitters per line and hence a lower discharge per emitter in comparison to the other systems.

Table 6.2. Features of the Four Emitters

Module	Microjet	Pressure compensated	Lyn	Drip Tape
q/Emitter (Lh^{-1})	14	1.6	1.6	0.2
No. Emitter/ lateral	5	45	45	120
Flow/ lateral (Lh^{-1})	70	72	72	24
No. of lateral /manifold	2	2	2	6
Flow /manifold (Lh^{-1})	140	144	144	144
Emitter separation (m)	2.5	0.2	0.2	0.1
Lateral separation (m)	2	2	2	0.5
Lateral length (m)	13.2	9.5	9.5	12.6
Pressure (kPa)	82.4-103	68.6-103	68.6-103	54.9-82.4
Emitter size (microns)	74	125	125	74

6.2.2. Monitoring the water quality and operation of microirrigation

In each of the three periods, water quality was measured daily at the influent and effluent of each system. The water quality parameters measured, as well as the reference values and clogging potential classification, were based on Nakayama and Bucks (1991). The authors classify clogging potential in three categories: low, medium, and high. Only three of the parameters used by them were included, namely total suspended solids (TSS), manganese (Mn), and iron (Fe) concentrations. All parameters were determined according to Standard Methods for the Examination of Water and Wastewater (APHA–AWWA–WPCF 2005).

The testing was started after each treatment line operated for 46 days to guarantee ripening and ensure stable performance. For each treatment line, the efficiency and clogging potential was determined. Efficiency was calculated by the balance between the inlet and the outlet of the treatment, while the clogging potential for each treatment line was estimated by analysis of effluent concentrations, comparing the values with the reference levels indicated by Nakayama and Bucks (1991). The effect of emitter clogging is assumed to be compensated for by an increase in flow in all the other emitters, allowing that head loss in the irrigation line remains constant. Partial or complete clogging reduces the *DUlq* and, as a consequence, decreases irrigation efficiency (Capra and Scicolone 2004). *DUlq* was determined by means of Eq. (6.1), for which the performance of discharged flow at each emitter was measured over time (Merriam and Keller 1978)

$$DUIq = (q_{25}/q_a) \times 100 \qquad (6.1)$$

where *DUlq* = distribution uniformity of lower quarter (%); q_{25} =average flow received by 25% of emitters receiving less flow in the test (Lh^{-1}); and q_a = average flow of emitters evaluated in the field test (Lh^{-1}).

For the evaluation of *DUlq*, Liotta (2006) and Fontela et al. (2002) suggest flow measurements in 16 emitters in each irrigation line regardless of the size of the area irrigated by the emitter. Given the short length of the emitter line in this study, the output of a larger number of emitters was measured to increase reliability in the *DUlq* calculation (Table 6.3). For three systems, the output of all emitters was measured, whereas in the case of the drip tape, this was done for a well-distributed sample of 10% of the emitters. Flow was measured manually in each emitter by the volumetric method (three replicates) in test tubes of 50 mL. Small tubes were used (which were filled between 10 and 20 s) to measure the output of all orifices (except for the drip tape where a sample of 10% of outputs was taken). To minimize possible variations in the results, uniform conditions for each emitter line were assured by maintaining relatively similar hydraulic conditions and uniform flow. Volume and time data were recorded in a readable format and the measured flow were averaged per day. All systems were installed horizontally at a height of 30 cm above ground to prevent any obstruction by particles from, for example, the soil; this setup also facilitated flow measurement.

Table 6.3. Number of emitter

Emitters	Total number of	Number of outputs measured
Microjet	10	10
Lyn	120	120
Pressure compensated	120	120
Drip tape	700	70

The reference value for the minimum acceptable *DUlq* was based on the criteria defined by Pizarro (1996) and Goyal and Ramírez (2007). For the Microjet emitter, Lyn emitter, and the pressure compensated emitter, a minimum acceptable *DUlq* of 0.8 was chosen, taking into account that the space between emitters is less than 2.5 m, the slope is below 2%, and the climate is semi humid. For the drip tape, however, the minimum acceptable *DUlq* is 0.75, according to Pizarro (1996). In this research, the total irrigation time was defined as the time of continuous operation until the *DUlq* limit was reached.

6.2.3. Statistical Analysis

Because the experimental results related to different periods of raw water quality and treatment efficiencies, it was essential to explore whether water quality in the three periods was comparable. First, a homogeneity of variance test (Levene test) and a normality test (Shapiro-Wilk) were done, adopting a significance level (p) of 0.05. However, the assumptions in these tests were not satisfied; therefore, a nonparametric test (Kruskal-Wallis) was needed to identify statistically significant differences between the samples and treatment results. The tests were performed using SPSS statistics 20 software (SPSS, Chicago, IL).

6.3. Results and discussion

6.3.1. Raw water quality

The main parameters that may affect clogging were measured in the raw water during the three irrigation periods (Table 6.4). The Mn concentration in the raw water was low in the three periods and represented a low theoretical clogging potential. The situation was different for TSS, which represented a high theoretical clogging potential. TSS showed a variation among the three periods, with the average in the third period being respectively 29 and 6% higher than that in the second and first period. Higher differences also existed for the total Fe concentration present in the Cauca River. The river receives water from areas with mining activities, resulting in a high theoretical clogging potential in period 1and 3, but only a moderate theoretical clogging potential in period 2. Statistical analysis (Table 6.5) showed that homogeneity of variances and normality was not satisfied for any of the parameters ($p < 0.05$). For that reason, it was necessary to perform the nonparametric test. Results of the Kruskal-Wallis test (Table 6.5) indicated no significant difference in Mn concentration in the 3 periods ($p > 0.05$). For the other parameters, differences were significant, which made it necessary to apply a pairwise comparison test. Results showed significant differences for Fe between periods 1–2 and periods 2–3, and for TSS for the periods 2–3 (Table 6.6).These results made it necessary to take the possible influence of the significantly lower levels of Fe and TSS in the second test period into account.

Table 6.4. Raw water quality

Parameter	Statistics	Period I T1 - T2	Period II T3 - T4	Period III T5-T6
TSS (mgL^{-1})	Minimum	58	37	85
	Maximum	748	691	493
	Average	209.3	158.9	221.9
	Est. Dev.	160.6	121.9	111.3
Mn (mgL^{-1})	Minimum	0.002	0.010	0.0002
	Maximum	0.921	0.020	0.087
	Average	0.10	0.014	0.021
	Est. Dev.	0.239	0.005	0.029
Fe (mgL^{-1})	Minimum	0.49	0.10	0.02
	Maximum	2.89	1.75	3.25
	Average	1.68	0.66	1.61
	Est. Dev.	0.62	0.49	1.02

Table 6.5. Statistical analysis for potential differences in raw water quality

Test	Significance (p)		
	Fe	TSS	Mn
Variances homogeneity	0.000	0.693	0.013
Normality	0.001	0.000	0.000
Non parametric- Kruskal-Wallis	0.000	0.011	0.076

Table 6.6. Results of Kruskal-Wallis test for raw water

Pairwise comparisons between periods	Significance (p)	
	Fe	TSS
Period 1-Period 2	0.000	0.133
Period 2-Period 3	0.000	0.015
Period 1-Period 3	1.000	1.000

6.3.2. Treatment performance

The performance of the different treatment technologies was explored by looking at the water quality parameters shown in Table 6.4 Mn was not tested because its low concentration in the raw water did not represent a high theoretical clogging potential. Table 6.7 shows the results of the performance analysis and the mean treatment efficiencies for each system. All the systems reduced the concentration for TSS and Fe. Tests for statistical analysis were done to evaluate removal efficiencies with data from the three periods for both Fe and TSS. Table 6.8 shows that the homogeneity of variances and normality was not satisfied for both parameters ($p < 0.05$). Hence, the use of a nonparametric Kruskal-Wallis test was necessary to compare the treatments. The results of this test indicated that technologies have different subgroups of efficiencies (Table 9). For Fe removal, efficiency was best for T2, T3, and T5, and between them there was no significant difference ($p > 0.05$). The lowest performance was identified for T4 and T6 (no significant difference between them for $p > 0.05$). TSS removal efficiency was best for T3 and T5 and between them there was no significant difference ($p > 0.05$). The lowest performance was identified for T4, showing a significant difference ($p < 0.05$) with the other treatments.

Table 6.7. Effluent quality of the treatment

Parameters	Statistics	T1 DyGF[(a)] + UGF_1[(b)]	T2 DyGF + $UGFS_1$[(c)] + $UGFS_2$[(c)]	T3 DyGF + UGF_1 + SSF_1[(d)]	T4 DyGF + DF[(e)]	T5 DyGF + UGF_2[(b)] + SSF_2[(d)]	T6 DyGF+ UGF_2 +DF
TSS (mgL^{-1})	Mean	60.1	41.3	6.4	58.2	1.37	19.36
	Est. Dev	68.9	60.6	15.8	66.9	2.29	25.0
	Mean removal efficiencies (%)	82	88	95	72	98	93
Fe (mgL^{-1})	Mean	0.50	0.43	0.09	0.48	0.24	1.22
	Est. Dev	0.28	0.29	0.15	0.47	0.51	0.86
	Mean removal efficiencies (%)	69	72	86	41	79	50

[(a)]DyGF = dynamic gravel filter; [(b)]UGF = upflow gravel filter (UGF1: 0.6 mh^{-1}; UGF2: 0.7 mh^{-1}); [(c)]UGFS1-2 = upflow gravel filter in series first and second stage; [(d)]SSF = slow sand filtration (SSF1: 0.2 mh^{-1}; SSF2: 0.4 mh^{-1}); [(e)]DF = disc filter

Table 6.8. Statistical analysis for treatment removal efficiencies

Test	Significance (*p*) Fe removal efficiency (%)	Significance (*p*) TSS removal efficiency (%)
Variances homogeneity	0.37	0.00
Normality	0.00	0.00
Non parametric- Kruskal-Wallis	0.00	0.00

Table 6.9. Results of Kruskal-Wallis test for treatment removal efficiencies

Kruskal-Wallis results by subgroups of efficiency	Technologies groups for each parameter	
	Fe	**TSS**
I	T4, T6	T4
II	T6,T1,T2	T1,T2,T6,T3
III	T1,T2,T5	T3,T5
IV	T2,T3,T5	

Note: For Fe, subgroup IV is the most efficient, while subgroup I is the less for both. For TSS, subgroup III is the most efficient.

The average removal efficiencies of the systems involving UGF were in line with those reported by different authors (Galvis 1999;Galvis et al. 1999; Di Bernardo and Sabogal 2008). Despite the higher removal efficiency, the theoretical clogging potential of the effluent related to Fe in T5 was medium. For T3, it was low, particularly because the Fe concentration in the raw water was significantly lower in this test period than in the period related to T5. Although Fe was removed, particularly in treatments with UGFS and SSF, removal efficiencies may have been reduced because part of the Fe may have formed complexes with natural organic matter and humic and tannic acids that inhibit the oxidation of organically bound Fe^{2+} by aeration (Theis and Singer 1974; Kawamura 2000).The lowest removal efficiency for both Fe and TSS was observed in T4 because the only removal mechanism was screening through the disc filter with a required level of filtration of 130 μm.

Hence, it may be expected that the smaller particles pass through the system. An additional problem with this system is that it has a very low capacity to store solids, which makes frequent cleaning necessary. The somewhat higher efficiency in TSS removal in T6 compared to T4 is probably a combination of the effect of the UGF and disc filter as the larger particles that manage to pass through the UGF are retained in the DF.

6.3.3. Filtration runs for different treatment lines

Table 6.10 shows a summary of the maximum head losses and filter runs for each treatment stage. The dynamic roughing filter was cleaned every 3 days, to stay close to the maximum head loss recommended by Latorre (1994) of 0.29 kPa. Only in the case of the dynamic roughing filter in T1 and T2, was the head loss a little higher.

The other systems were cleaned when needed without interrupting the flow to the emitters as a small storage tank served as a buffer. The UGF of T1 and the $UGFS_2$ of T2 did not need cleaning as they did not reach the maximum head loss of 2.94 kPa during the run time of the system. In the UGF of T3, the maximum head loss was obtained after a filter run time of 11 days, whereas the *DUlq* limit was not yet reached. The system was then cleaned and the experiment with the emitters was continued. The same approach was followed for T4, T5, and T6, which had relatively short runs between cleanings. The maximum head losses obtained in the UGF-UGFS were consistent with those reported by Galvis (1999), who found <0.1 kPa in UGFS and 2.94 kPa in UGF. The longer filter run of the UGF for T3 in comparison with T5 can be explained by the significantly lower TSS and Fe concentration in the raw water.

Table 6.10. Maximum head loss (MHL) and filter run (FR).

Treatment stage	T1 DyGF[(a)] + UGF_1[(b)]		T2 DyGF + $UGFS_1$[(c)] + $UGFS_2$[(c]		T3 DyGF + UGF_1 + SSF_1[(d)]		T4 DyGF + DF[(e)]		T5 DyGF + UGF_2[(b)] + SSF_2[(d)]		T6 DyGF+ UGF_2 +DF	
	MHL (kPa)	FR (d)	MHL (kPa)	FR (d)	MHL (kPa)	FR (d)	MHL (kPa)	FR (d)	MHL (kPa)	FR (d)	MHL (kPa)	FR (d)
DyGF	0.4	3	0.4	3	0.3	3	0.3	3	0.3	3	0.3	3
UGF_1	0.5	11			1.2	11						
UGF_2									2.9	7	2.9	7
$UGFS_2$			0.2	11								
SSF_1					4.9	7						
SSF_2									3.9	6		
DF							27.5	0.2			27.5	0.8-2.2

[(a)]DyGF = dynamic gravel filter; [(b)]UGF = upflow gravel filter (UGF1: 0.6 mh^{-1}; UGF2: 0.7 mh^{-1}); [(c)]UGFS1-2 = upflow gravel filter in series first and second stage; [(d)]SSF = slow sand filtration (SSF1: 0.2 mh^{-1}; SSF2: 0.4 mh^{-1}); [(e)]DF = disc filter

The filter run time in the slow sand filter units was short in relation with other studies: 20–60 days reported by Schulz and Okun (1984), the minimum of 45 days recommended by Cleasby (1991), the range of 46–178 days reported by Galvis et al. (1999), and 50–70 days as was described in chapter 3. The reasons for these short filter run times was the high concentration of TSS in the influent, 52 mgL^{-1} in T3 while for T5 it was 28.4 mgL^{-1}. These values exceeded the average values of TSS influent (<2 mgL^{-1}) recommended by Galvis (1999) for filter run times over 30 days. In addition, the slow sand filtration was operated at relatively high filtration rates (0.20 mh^{-1} in T3 and 0.40 mh^{-1} in T5). Thus, the slow sand filtration in T3 and T5 contributed to a reduction in the clogging potential of the emitters, but had limitations because of the short filter runs and frequent cleaning of the sand bed, demanding more maintenance time.

The filter run of the disc filter in treatment scheme T4 was 0.17 days (4 h). However, the filter run of the disc filter in T6 was increased to between 0.8 and 2.2 days (18–52 h) because of the effect of the UGF. Taking into account that the application of irrigation in communities often has a duration of, on average, 3 h per day, the disc filter in T4 should be washed after each irrigation cycle. This is consistent with current practice as reported by EIDENAR (2008) and requires about 10 min of cleaning time. However, disc filter cleaning in T6 would only be needed every 6–17 irrigation cycles. This is attractive as it facilitates operation and maintenance and reduces water losses, while UGF cleaning only takes some 45 min (see chapter 3) after 56 irrigation cycles.

6.3.4. Exploring the effect of treatment on performance of the four irrigation methods.

The different treatment methods had an effect on the overall irrigation time for the emitters but considerable differences existed between the effects on the four emitters (Figure 6.2). What emerged was that the different treatment systems had similar effects on the pressure compensated and Lyn emitters with T5 giving the best results followed by T3 and T6. For the Microjet and drip tape the performance was, on average, low, and the effect of the treatment, in general, was limited, compared to the other two irrigation methods, with one exception being the effect of T3 on drip tape. The best performance for the pressure compensated emitter was also observed by Duran-Ros et al. (2009) and Liu and Huang (2009), who identified longer operation times and a better anticlogging effect in this emitter

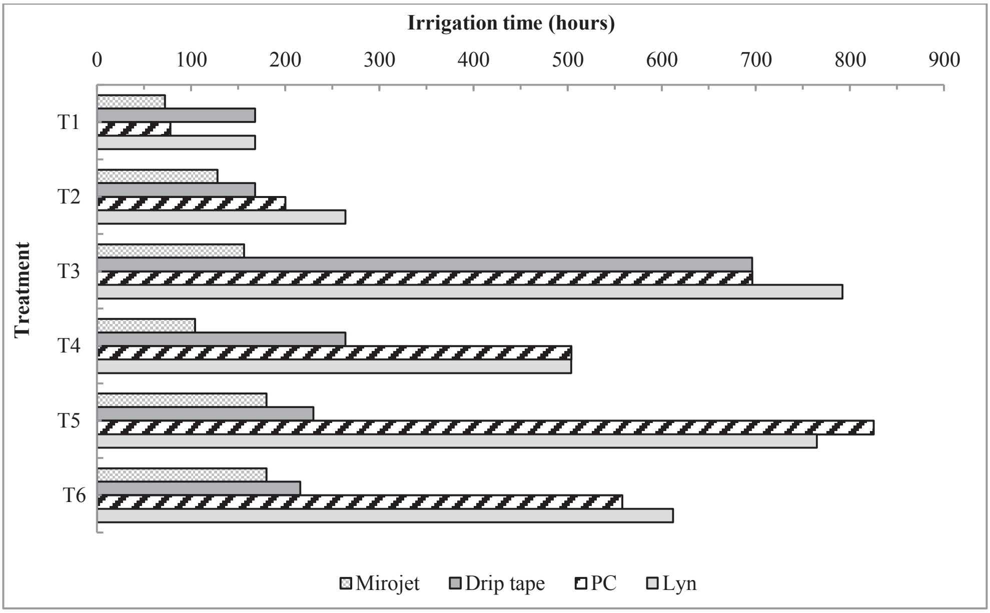
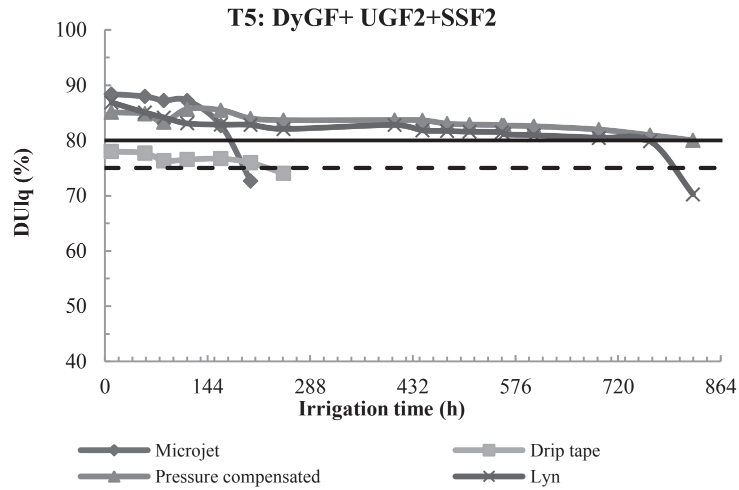


Figure 6.2. Impact of different treatments on irrigation time in the four emitters

The steep drop in *DUlq* in theMicrojet emitter in T6 (Figure 6.3) can be explained by the larger flow in the emitter (Table 6.2), which means that more solids are available for emitter clogging and the higher concentration of TSS and Fe provided by T6 when compared to T3 (Table 6.7). The two systems with the somewhat larger required level of filtration of 125 μm showed a much better behavior with a very gradual clogging process. One particular case was the good irrigation time performance of the drip tape in combination with T3, which may be the result of a lower Fe clogging potential. This particular combination seems attractive for application because it has a lower cost than the other systems and is simple to operate and maintain, although both the *DUlq* and the operating time of drip tape are lower than in the other systems (Figure 6.3). It may be expected that the Lyn and pressure compensated emitters, fed with T3, T5, and T6, can guarantee over 600 h of irrigation time, which corresponds to more than 200 three-hour irrigation cycles before cleaning. These results show that improvement of clogging potential conditions contributed considerably to extending the operation time of the emitters and reduced the cleaning frequency of the emitters.



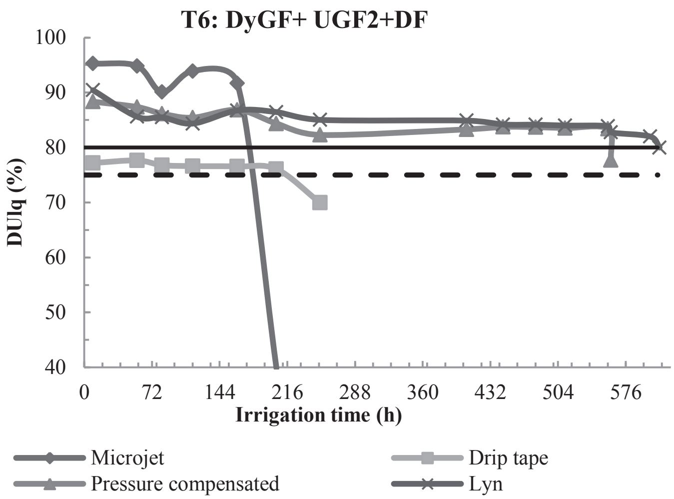


Figure 6.3. ***DUlq*** **performance versus irrigation time in each of the four emitters for T5 and T6.**

6.4. Conclusions

This chapter evaluated six water treatment systems, feeding four emitters, to explore the potential of upflow gravel filtration in irrigation systems to reduce clogging potential and sustain a distribution uniformity of lower quarter above 80% (75% for drip tape). Although all the treatment schemes contributed to reducing the clogging potential by removing total suspended solids, the best performances were observed at the following treatment schemes: An upflow gravel filter (*vf*: 0.60 mh^{-1}) followed by a slow sand filter (*vf*: 0.2 mh^{-1}); an upflow gravel filter (*vf*: 0.70 mh^{-1}) followed by a slow sand filter (*vf*: 0.4 mh^{-1}). Between them there was no significant difference ($p > 0.05$). They produced water with a low clogging potential for the different emitters. Iron was removed particularly in the upflow gravel filter (*vf*: 0.60 mh^{-1}) followed by a slow sand filter (*vf*: 0.2 mh^{-1}), the upflow gravel filter (*vf*: 0.70 mh^{-1}) followed by a slow sand filter (*vf*: 0.4 mh^{-1}), and upflow gravel filters in series with two stages (*vf*: 0.60 mh^{-1}). No significant difference in removal efficiency was observed between these treatments.

The best irrigation distribution uniformity performance was obtained with the Lyn emitter, followed by the pressure compensated emitter (both having a required level of filtration of 125 µm), the drip tape, and finally the Microjet. This clearly shows the influence of the required level of filtration of the emitters as the last two systems have a smaller required level of filtration (74 µm). With the pressure compensated and Lyn emitters, the irrigation time was >600 h fed by an upflow gravel filter (*vf*: 0.60 mh^{-1}) followed by a slow sand filter (*vf*: 0.2 mh^{-1}), an upflow gravel filter (*vf*: 0.70 mh^{-1}) followed by a slow sand filter (*vf*: 0.4 mh^{-1}), and an upflow gravel filter (*vf*: 0.70 mh^{-1}) followed by disc filter, allowing more than three-hour irrigation cycles, corresponding to 6.7 months of irrigation, with a daily frequency.

The best removal efficiency for total suspended solids and iron was obtained for treatments that use an upflow gravel filter followed with slow sand filter and upflow gravel filters in series, but are the most costly treatment systems, whereas the cheapest system, with disc filters, showed the poorest performance. The combination of upflow gravel filters and disc filter is somewhat more costly than the disc filter alone but was much easier to operate than the other systems and had a reasonable performance.

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CHAPTER 7

Conclusions and Recommendations

7.1. Conclusions

This thesis focused on learning more about the performance, the treatment process and the operation and maintenance (O&M) of upflow gravel filtration (UGF) in drinking water supply systems and explored other potential uses of UGF technology (Figure 7.1). This pre-treatment was developed to overcome problems caused by high levels of turbidity, suspended solids and *E-coli* in small water supply systems where the treatment capacity of SSF was exceeded when used as the single barrier. The combination of gravel filtration with SSF is called multistage filtration (MSF). Today UGF is the main pre-treatment used in water supply systems in rural areas in Valle del Cauca-Colombia and is also used in other Latin America countries. UGF was considered in this research because of its ability to maintain treatment simplicity comparable to that of SSF at accessible investment costs, facilitated by the use filter material from local sources. The simplicity in operation and maintenance of the UGF facilitates that it can be sustained by rural communities and small towns. Interest in this technology is growing to help fill the gap in water supply coverage in small communities in developing countries, supported by the new challenge of the sustainable development goals (SDG) established by the United Nations. The interest of UGF application, confirmed in this thesis, extends to the use of coagulants with UGF to improve the performance of MSF during variations in influent water quality and the use of UGF with filter fabric to optimize maintenance and investment costs, and other potential uses such as iron and manganese removal from groundwater and to prevent potential clogging in micro-irrigation. The conclusions starting with the findings of UGF used in multi stages filtration plants, and then the other potential applications are discussed. Figure 7.1 shows a scheme of an overview of UFG use and different applications analyzed during the study (A); thereby, it was possible to obtain a set of results (B) that allowed to revise design criteria and O&M in the UGF (C). This Figure also describes the physical and biological process (D) and the physical-chemical processes in a UGF (E).

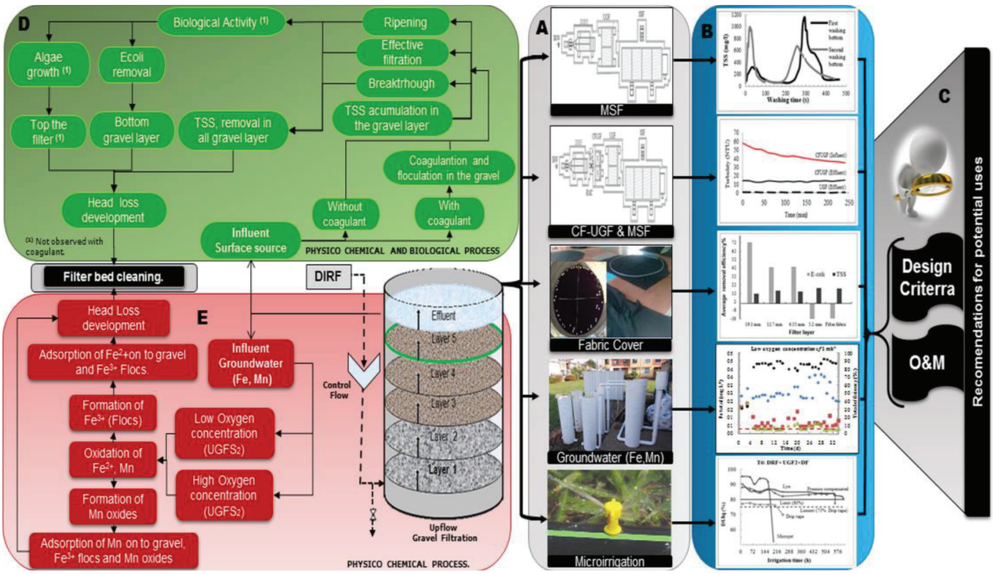

Figure 7.1. Synergistic scheme for the review of UFG use and potential applications.

7.1.1 The performance of upflow gravel filtration in full-scale plants

The analysis of UGFs that are part of four full-scale MSF treatment plants in Valle del Cauca, Colombia, confirm that these systems operate for a long time and are being effectively used, managed and sustained by local water committees in both poor and richer communities. Full-scale applications are showing similar performance (Figure 7.1- part A and B) as those reported in literature and are built to the design criteria presented by Galvis *et al.* 1999 (Figure 7.1- part C) with the exception of the drainage system and flow velocities. In two cases this resulted in lower washing velocities than recommended in literature. Operators follow, to some extent, the recommended O&M procedures but they do not: take samples to monitor water quality, measure head loss, or control the flow velocity. A topic for discussion in UGF has been the filter bed cleaning procedure. Based on the first observations research reported in this thesis showed that shock loads did not influence cleaning efficiency of the filters, implying that this practice can be replaced by just twice draining the UGFs, thus facilitating the work of the operator, but this finding justifies further controlled studies.

The procedures applied for filter bed cleaning are effective despite some limitations found in the drainage systems and low washing velocity (< 10.4 mh^{-1}). About 90% of the retained solids were removed in two drainage cycles; the remaining 10% is probably removed during surface cleaning of the gravel bed. The surface cleaning by orifice proved to be inefficient which may result in the removal of fewer solids. Head loss build up in one week was low, suggesting that is relevant to explore the cleaning cycles in more detail as a lower frequency reduces the operator work, and less water is needed, which may be particularly relevant for pumped systems. Longer periods between cleaning may also have a positive effect on the treatment efficiency by allowing more development of biomass in the filters.

7.1.2 Coagulation-flocculation in upflow gravel filters with multi stage filtration systems

A better understanding of the design variables and operation and maintenance conditions was obtained analyzing a full scale plant combining CF-UGF with MSF (Figure 7.1- A, C). CF-UGF is a relatively new technology that has been applied in some cases with rapid sand filtration. In this thesis CF-UGF with MSF has been considered to address problems caused by variations in turbidity peaks in surface sources with deteriorated basins by deforestation and erosion. The use of CF-UGF with MSF greatly contributed to the removal efficiency of MSF during variations in influent water quality without negatively affecting the biological activity of the treatment system in terms of the efficiency of microorganism removal in the UGF and SSF when coagulant was dosed. This strongly contributed to the operational flexibility of the system as it allowed to dose coagulant only when high influent turbidity peaks occurred. The system operated with coagulant 20% of the time, from turbidity values of 30 NTU.

CF-UGF improved the operation of MSF compared to only UGF, facilitating the performance of the SSF by reducing the load of particulate material to avoid short filter runs and possible interruptions in treatment plant operation. The overall system produced water with turbidity below 1 NTU. This type of solution helps to fulfill the sustainable development goals (SDG) because the technology can be sustained by the local level, its design is simple and the operation and maintenance can be done by operators with low educational level. The cost makes this technology attractive because the production cost of 1 m^3 of water was US $0.05, with a per capita investment of US $18, which are accessible to rural communities in developing countries.

7.1.3 Performance of upflow gravel filtration with fabric cover.

The pilot study of a UGF with fabric cover placed on top, allowed a better understanding of the physical-chemical and microbiological processes occurring within these systems, its benefits related to operation and maintenance, and its potential to enhance subsequent treatment processes. In this type of filters the operation was characterized by a period of ripening, effective filtration and breakthrough (Figure 7.1- D). Best TSS removal efficiency was obtained for an effective filtration period between 30- 39 days. So, the practice of weekly cleaning described in Chapter 2 should be reviewed and adapted to local circumstances. All filter layers contributed to TSS removal (Figure 7.1- D) but the greater accumulation of solids per volume of filter layer occurred in the filter fabric (thickness of 0.0056 m). This result may help to optimize the surface cleaning of these units because filter fabrics are lightweight and relatively easy to remove. The removal efficiency in the filter fabric was 17% for TSS but the fabric did not contribute to *E-coli* removal.

Removal efficiency of *E-coli* was highest in the bottom layer (gravel size of 19.1 mm) and was reduced to the top of the filter. Possibly algae biofilm development on the filter fabric (Figure 7.1- D) facilitated the regrowth of *E-coli*, which raises the question about whether it is necessary or not, allowing such growth of algae and if it is convenient to keep the supernatant level in the filter. The high *E-coli* removal in the bottom layers (Figure 7.1- D) reinforced the idea that in UGF cleaning at longer intervals can improve removal efficiency as it allowed more biomass development in the filters.

All filter layers contributed to particle removal, with a larger number of larger particles being removed in the gravel with diameter of 19.1 and 12.7 mm, whereas small particles < 2 µm were hardly removed. This confirms the discussion of Boller (1993), that these filters have the advantage that larger and heavier particles are first removed at the bottom layer and that sedimentation is the main mechanism for particle removal. Filter fabric contributed to removal of particles less than 25 µm which may provide a better filtration run of SSFs or more irrigation time in emitters when used in combination with micro-irrigation (chapter 6).

TSS removal efficiency by a layer of fabric cover of 0.56 cm is equivalent to a gravel layer of 0.16- 0.25 m with a gravel size between 6.3-3.2 mm at the top of the filter bed. Thus using a filter fabric may allow a reduction in filter bed height without losing removal efficiency, which, in turn, has an impact on the reduction in investment costs.

7.1.4 Performance of upflow gravel filtration for iron manganese removal from groundwater.

Different mechanisms may contribute to Fe and Mn removal in filtration processes. The main mechanisms are: oxidation-floc-formation and adsorption-oxidation. Biological oxidation is the third process which is facilitated by microbiological activity on the filter grains. However, the involved treatment mechanisms are still not fully understood. Iron and manganese removal from groundwater by UGF studied at pilot scale for high and low oxygen concentrations showed that this treatment process has a potential for Fe and Mn removal from groundwater.

Batch and pilot scale experiments (*vf*: 1, 2 and 3 mh^{-1}) showed small differences for iron and manganese removal at high and low oxygen concentrations. For both compounds however, the pH has an influence with best retention capacity for Fe at pH 7 and for Mn at pH 8. In a pilot study using natural groundwater the pH was slightly above 7 which facilitated the removal of both compounds. For low *vf* the filter run time at low oxygen concentration was shorter than under high oxygen concentrations. However for high *vf* (10 mh^{-1}) resulted in longer filtration run times in F1 for low oxygen concentration, while at high oxygen concentration the filter run times were shorter, but high *vf* at high oxygen concentration was not convenient as adequate head loss recovery using normal maintenance procedures was not achieved.

Head loss in the pilot filters differed with filtration stages, operational conditions and filtration velocity. However the head loss development was more affected in the line under low oxygen concentration and a shorter filter run was obtained. This is counter intuitive with literature, and seems to be the result of differences in the mineralogical and morphological shape of the formed iron hydroxides or possible presence of biological mechanisms, being subject to additional studies.

7.1.5 Potential of upflow gravel filtration to be used in microirrigation

Micro-irrigation systems face progressive emitter clogging due to the presence of physical, chemical, and microbiological substances in the water. This problem is associated with the small emitting diameter that is necessary to guarantee equal water distribution in low-flow conditions. Taking into account the advantages of UGF in retaining solids, iron and microorganisms, six pilot water treatment systems with UGF each connected to four emitters were evaluated to study the potential clogging and distribution uniformity in micro-irrigation (Figure 7.1- A). Although all the treatment schemes contributed to reducing the clogging potential by removing total suspended solids and iron, the best performances were observed for treatments that use an UGF followed with slow sand filter and UGFs in series, but are the most costly treatment systems, whereas the cheapest system, with disc filters, showed the poorest performance. The combination of UGFs and disc filter is somewhat more costly than the disc filter alone but showed great potential in micro-irrigation because it is much easier to operate than other systems and had a reasonable performance allowing longer operation of the disc filter.

The best irrigation distribution uniformity performance was obtained with the Lyn emitter, followed by the pressure compensated emitter the irrigation time was >600 h fed by an UGF (*vf*: 0.60 mh^{-1}) followed by a slow sand filter (*vf*: 0.2 mh^{-1}), an UGF (*vf*: 0.70 mh^{-1}) followed by a slow sand filter (*vf*: 0.4 mh^{-1}), and an UGF (*vf*: 0.70 mh^{-1}) followed by disc filter, allowing more than three-hour irrigation cycles per day, corresponding to 6.7 months of irrigation.

7.2. Recommendations

This research deepened the insight in the use of UGF in MSF systems and in other applications, based on literature review, and analysis at laboratory and pilot scale as well as in full scale plants. The UGF technology contributes to a better response to the water quality problems in surface and groundwater sources for small water supply and micro-irrigation systems. The results of this thesis can be useful for researchers, designers, planners, constructors and operators of small water supply systems in rural areas and small towns of developing countries.

With the results of this research, a new treatment plant using CF-UGF was built in a small system in the Jamundí municipality, after a technology selection process. Also a full-scale two stage treatment plant for iron and manganese removal from groundwater by UGF was designed and approved for the rural community of Tuluá municipality in the Valle del Cauca. Construction with finance from the government of Colombia through the departmental water plan is eminent.

The application of UGF is also recommended as a sustainable technology in drinking water supply, but also in irrigation systems particularly useful for rural communities in developing communities due to its capacity to support variation in water quality (surface sources), can be built with local materials and operated by non-specialist operators. The use of UGF systems is an adequate technology to enhance subsequent treatment processes such as SSF but also facilitates micro-irrigation.

Some specific recommendations for design, operation and maintenance of a UGF resulting from this study are the following:

- The limitations in surface cleaning had a negative effect on the hydraulic behavior by the presence of dead zones by the accumulation of solids in the gravel bed. This accumulation was more severe in the systems with limitations in surface cleaning by orifice. Therefore a weir should be included in the design criteria of UGFs to facilitate water drainage during surface cleaning.
- It is also relevant to review the cleaning cycles in more detail to reduce the work load of the operator, water loss and review the effect on treatment efficiency.
- Based on the first observations shock loads did not influence cleaning efficiency of the lowly loaded filters, implying that this practice can be replaced by just twice draining the UGFs, thus facilitating the work of the operator, but this finding justifies further controlled studies.
- To conserve the O&M procedures recommended in literature, low filtration velocity is necessary (vf= 1-3 mh^{-1}) for iron and manganese removal with UGF.

7.3. References

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Summary

Upflow Gravel Filtration (UGF) is an important pretreatment method used in Multi-Stage Filtration (MSF) systems and has been developed particularly to protect Slow Sand Filters (SSF) from receiving high loads of suspended solids. In an UGF water passes through the gravel bed from the bottom where the gravel layer is coars to the top where the gravel is fine. During this passage impurities are retained in the differents filter layers. An important part of the solids are removed with a gradual change in the particle size distribution and other water quality parameters are improved. Algae growth in the top also contributes to this proces.This system acummulates a great volumen of solids at low head loss.

The use of UFG is relevant for rural water supply system in Colombia, because water quality and quantity from surface sources is changing due to the deterioration of watersheds caused by deforestation, erosion, and the discharge of untreated waste water. These changes are intensified by the effects of global climate change. The main problems that occur include the increase in turbidity and suspended solid levels with higher peaks of longer duration. These changes are affecting the existing water treatment plants, causing higher operation and maintenance requirements and even interruptions in their operation. To overcome such problems a better understanding of the performance of UGF systems is needed because this type of gravel filters is used in almost all MSF systems in Colombia.

This research contributes to increasing the knowledge about the treatment process in UGF. The study combined analysis in existing full scale plants and research at lab and pilot filters scale in order to explore possible limitations, identify improvements and test other possible applications of the technology. This includes the potential use with coaugulation and flocculation to overcome longer periods of high loads of suspended solids; the application on a filter fabric on top of a UGF to improve surface cleaning procedures and reduce filter height; the application of the system for iron and manganese removal from groundwater sources; and the potential use in microirrigation.

The relevance, scientific background and problem statement related to UGF performance and potential application in rural communities are presented in **Chapter 1**. UGF is being used and it is sustained by rural communities, e.g. in regions like the Cauca Valley in Colombia it is the main treatment technology used for rural water supply systems (about 54%)**.** UGF is considered to be a promising pretreatment technique because of its ability to maintain treatment simplicity comparable to that of SSF and the accessibility in terms of costs (investment, operation and maintenance) which is facilitated by the use of local material and simplicity of operation and maintenance (O&M).

Chapter 2 describes the results of a study of four full scale upflow gravel filters that are part of full scale multi-stage filtration systems in rural communities of Cali-Colombia. The design criteria, the O&M practices, and the performance of the systems were reviewed. In general design criteria and O&M procedures follow the recommendations as presented in the literature. Performance data showed that removal efficiencies were on the low side when compared to the literature, possibly because of the good influent quality water that was treated during the study. This chapter further analyses cleaning efficiency and shows that an adjustment of the design criteria and O&M procedures is needed to enhance system

performance. This includes drainage system design, surface cleaning, filter bed cleaning and the reduction in cleaning frequency cycles to improve the operation labor, reduce water use and search a better filter efficiency.

Chapter 3 assesses the operational and design aspects of coagulation and flocculation in upflow gravel filters (CF-UGF) in a multi-stage filtration (MSF) plant. This chapter shows that CF-UGF units improve the performance of MSF considerably, when the system operates with turbidity levels above 30 NTU. It strongly reduces the load of particulate material before the water enters the SSF and therewith avoids short filter runs and prevents early interruption in SSF operations. The removal efficiency of turbidity in the CF-UGF with coagulant was between 85 and 96%, whereas the average efficiency without coagulant dosing was 46% (range: 21-76%). Operating with coagulant also improves the removal efficiency for total coliforms, *E-coli* and HPC. The dosing of chemicals did not lead to obstruction of the SSF bed and reductions of microbiological removal efficiency. Filter runs remained between 50 and 70 days for a maximum head loss of 0.70 m. A very important advantage is the flexibility of the system to operate with and without coagulant as needed according to the influent turbidity. It was only necessary for 20% of the time to operate with the coagulant. The CF-UGF unit represented 7% of total construction costs and the O&M cost for the use of coagulant represented only 0.3%.

Chapters 4, 5 and 6 report on different pilot-scale studies. The performance of a UGF with a filter fabric on top of the media in terms of removal of: total suspended solids and *E-coli*, particular size distribution, head loss development and algae growth is reported in **Chapter 4**. The results of the study indicate that in UGF the operation is characterized by a period of ripening, effective filtration and breakthrough. Best suspended solids removal efficiency was obtained for an effective filtration period between 30 and 39 days. All filter layers contribute to TSS removal efficiency. The larger accumulation of solids per volume of filter layer occurred in the filter fabric (thickness of 0.0056 m), but this layer did not contribute to *E-coli* removal. Removal efficiency of *E-coli* was highest in the bottom layer (gravel size of 19.1 mm) and reduced to the top of the filter. A direct relationship was found between the increase in head loss in the filter fabric and algae biofilm development. Suspended solids with particles size larger than 40 μm were completely removed, whereas particles less than 2 μm were hardly removed. The study suggest that applying a filter fabric allows to reduce the filter height without losing suspended solids removal efficiency and that the once-a-week cleaning procedure needs to be revised and adapted to local circumstances as longer intervals between cleanings improves removal efficiency and reduces water losses.

The removal of iron (Fe) and manganese (Mn) in UGF both at laboratory and pilot scale under high and low oxygen concentration at different pH levels is presented in **Chapter 5**. Results at laboratory scale in batch experiments with coated gravel show small differences in Fe^{2+} and Mn removal between high and low oxygen concentration (pH 5-8 units). Removal efficiencies were influenced by the pH with best Fe^{+2} removal (64%) being obtained at pH 7 under high oxygen concentration with the ultimate concentration being reached after 5 hours for an initial concentration of Fe^{2+} of 6 mgL^{-1}. For initial Mn concentration of 2.0 mgL^{-1}, best removal (72%) was obtained at pH 8 reaching the ultimate concentration after 4 hours. Removal efficiencies were higher in the pilot study and indicated that median removal efficiency for total Fe and Fe^{2+} was between 75% - 95% and for Mn between 60-95%, but also in this case the effect of different oxygen concentrations was small. Filtration velocity had an impact with best efficiency being obtained with low filtration velocity (1-3 mh^{-1}).

Chapter 6 presents the effect of six water treatment combinations for the treatment of turbid surface water on four types of emitters by looking at clogging potential and distribution uniformity in microirrigation. The pilot plants use different combinations of UGF, disc filters, and SSF, fed with natural water from the Cauca River (Cali, Colombia). The outflow of these systems was used to feed four different types of emitters. Total suspended solids and iron were the main physicochemical parameters identified that affect emitter clogging and the distribution uniformity of the lower quarter *(DUlq).* All treatment schemes reduced clogging potential, with best performance being obtained with UGF followed by SSF and UGF in series, but are the most costly treatment systems, whereas the cheapest system, with disc filters, showed poorest performance. The combination of UGF and disc filters has a good potential to be used in micro-irrigation being easier to operate than the other systems whilst having reasonable performance and costs. The effect on *DUlq* differed for the four types of emitters. The longest irrigation time was obtained for the pressure compensated emitter and for the Lyn emitter with 600 h of continuous operation until the *DUlq* was below 80%.

Summarizing, this thesis provides a coherent description and analysis of the UGF application, process efficiency, other potential applications and possible settings for operation and maintenance. Several systems already operate for a long time and most systems are managed by local water committees. The design characteristics of the systems follow the literature, with the exception of the drainage system and flow velocities. Adjustment of the design criteria and O&M procedures is needed to enhance system performance. The combination of CF-UGF with MSF greatly contributed to the suspended solids removal efficiency of the system. This allows continuing operating the MSF system in case of longer periods with high turbidity without negatively affecting microbiological removal efficiency and filtration runs of the SSF. This strongly contributes to the operational flexibility of the system as it allows to dose coagulant only when high influent turbidity peaks occur. In UGF all gravel layers contributed to TSS and particles removal but the efficiency was higher in the filter fabric and top gravel layer (gravel size of 3.2 mm). The main algae growth took place in the filter fabric where it contributed to TSS but not to *E-coli* removal efficiency. The highest removal efficiency of *E-coli* occurred in the bottom layer (gravel size of 19.1 mm). The criterion of UGF cleaning frequency must be adjusted and a longer time should be applied to ensure most effective filtration. UGF with coated gravel have potential application for Fe and Mn removal in the range of pH 7-8 from groundwater with high as well as low oxygen concentrations. Little difference was observed in the removal efficiency for Fe and Mn under high and low oxygen concentrations at low filtration velocity (1, 2 and 3 mh^{-1}). Increasing the vf to 10 mh^{-1} under high oxygen concentration was not convenient as adequate head loss recovery using maintenance procedures recommended in the literature was not achieved. However under low oxygen concentration it was observed that although retention of Fe and Mn was lower a long filtration run was reached, contributing to reducing cleaning frequency and a possible combination of a first stage of pretreatment under low oxygen conditions at vf = 10 mh^{-1} followed by a second stage under high oxygen concentration operating at vfs between 1 and 3 mh^{-1} may be a promising option for Fe and Mn removal at lower cost. UGF also contributed to reducing the clogging potential and achieve a good irrigation time by the emitter in microirrigation. The combination of upflow gravel filters and disc filters is promising for use in microirrigation because was much easier to operate than the other systems and had a reasonable performance and costs, allowing more than 200 three-hour irrigation cycles per day, corresponding to 6.7 months of irrigation.

Samenvatting

Opwaartse Grindfiltratie (OGF) is een belangrijke voorbehandelingsmethode die wordt gebruikt in meertraps filtratiesystemen (MFS). Deze methode is met name ontwikkeld om langzame zandfilters (LZF) te beschermen tegen een hoge belasting met zwevende stof. Het water in een OGF stroomt door het grindbed van de bodem, waar de grindlaag grof is, naar de top, waar het grind fijner is. Gedurende dit proces blijven onzuiverheden achter in de filterlagen. Een belangrijk deel van de vaste stoffen wordt verwijderd wat ook een verandering in de verdeling in deeltjesgrootte tot gevolg heeft. Daarnaast verbeteren ook andere waterkwaliteitsparameters tijdens het filtratieproces, waarbij ook algengroei in de top van het filter aan dit proces kan bijdragen. OGF verwijdert dus een groot volume aan vaste stoffen terwijl de filterweerstand laag blijft.

Het gebruik van OGF is relevant voor plattelandswatervoorziening in Colombia, omdat de oppervlaktewaterkwaliteit en -kwantiteit verandert als gevolg van de verslechtering van de stroomgebieden, veroorzaakt door ontbossing, erosie en de lozing van ongezuiverd afvalwater. Deze veranderingen worden versterkt door de effecten van de wereldwijde klimaatverandering. De voornaamste problemen die optreden, zijn de toename van troebelheid en het zwevende stofgehalte, waarbij hogere pieken optreden die van langere duur zijn dan voorheen. Deze veranderingen zijn van invloed op de bestaande waterzuiveringsinstallaties en leiden tot grotere problemen in exploitatie en onderhoud, en zelfs tot het volledig onderbreken van de zuivering. Omdat dit type grindfilters in bijna alle MFS in Colombia wordt gebruikt, is, om dergelijke problemen op te lossen, een beter begrip nodig van de werking van OGF-systemen.

Dit onderzoek draagt bij aan het vergroten van de kennis over het behandelingsproces in OGF. De studie is een combinatie van de analyse van bestaande zuiveringsinstallaties, onderzoek in het laboratorium en met behulp van proeffilters en omvat tevens een verkenning en testen van andere toepassingen van de OGF-technologie. Dit laatste betreft het mogelijke gebruik van coagulatie en flocculatie om problemen van langere periodes van hoge troebeling te overwinnen; het aanbrengen van een filterdoek op het oppervlak van een OGF om oppervlaktereinigingsprocedures te verbeteren en de hoogte van het filter te verkleinen; de toepassing van het systeem voor ijzer- en mangaanverwijdering uit grondwater; en het potentiële gebruik in micro- irrigatiesystemen.

De relevantie, wetenschappelijke achtergrond, en probleemstelling met betrekking tot OGF en de mogelijke toepassing in plattelandswatervoorziening worden gepresenteerd in Hoofdstuk 1. OGF wordt gebruikt en onderhouden door plattelandsgemeenschappen. Het is bijvoorbeeld de belangrijkste zuiveringstechnologie voor plattelandswatervoorziening in het departament Valle del Cauca in Colombia (ongeveer 54%). OGF wordt beschouwd als een veelbelovende voorbehandelingstechniek omdat haar eenvoud vergelijkbaar is met die van LZF, de technologie zeer toegankelijk is vanwege de kosten (investeringen, exploitatie en onderhoud) en de mogelijkheid om lokale materialen te gebruiken en omdat de technologie eenvoudig te bedienen en onderhouden is.

Hoofdstuk 2 beschrijft de resultaten van het onderzoek dat is uitgevoerd in vier bestaande OGF's die deel uitmaken van MFS in plattelandsgemeenschappen in het district Cali-Colombia. De studie omvat de ontwerpcriteria, het onderhoud en de werking van de vier systemen. De studie laat zien dat de ontwerpcriteria en onderhoudsprocedures over het algemeen in lijn zijn met de aanbevelingen die in de literatuur worden gedaan. De verwijderingsrendementen van de vier systemen zijn, in vergelijking met de gekende literatuur, aan de lage kant. Dit komt mogelijk vanwege de goede waterkwaliteit van het oppervlaktewater dat wordt behandeld. Dit hoofdstuk omvat een analyse van het schoonmaken van de filters en toont aan dat een aanpassing van de onderhoudsprocedures in termen van filterefficiency, water gebruik en vermindering van arbeid, nodig is om de werking van het systeem te verbeteren. Dit omvat aanpassingen in het ontwerp van het drainagesysteem, de oppervlaktereiniging, de filterbedreiniging en een verlaging van de reinigingsfrequentie.

Hoofdstuk 3 omvat de evaluatie van het ontwerp en gebruik van coagulatie en flocculatie in OGF in een MFS. Dit hoofdstuk laat zien dat wanneer de waterkwaliteit een hoger troebelingsniveau heeft (boven 30 NTU), coagulatie en flocculatie de werking van MFS aanzienlijk verbetert. Het proces verlaagt de belasting met zwevende stof voordat het water de LZF bereikt. Dit voorkomt korte filtratieperiodes en voorkomt de noodzaak van vroegtijdige onderbreking van de LZF. De verwijderingsefficiëntie van de troebelheid in de OGF met coagulatiehulpmiddel was tussen de 85 en 96%, terwijl het gemiddelde rendement zonder deze toepassing slechts 46% (variatie 21-76%) bedroeg. Toepassing van coagulatie verbetert ook de verwijderingsefficiëntie van totale coliformen, *E-coli*, en verlaagt de heterotrofe kiemgetallen. De dosering van chemische stoffen leidde niet tot verstopping van het LZF of tot vermindering van de microbiologische verwijderingsefficiëntie. De lengte van de filterperiode bleef tussen de 50 en 70 dagen bij een maximaal drukverlies van 0,70 m. Een zeer belangrijk voordeel is de flexibiliteit van het systeem. Het is namelijk mogelijk om met of zonder coagulatie te werken en dit aan te passen op de waterkwaliteit. Het bleek slechts nodig om gedurende 20% van de tijd te werken met coagulatie. De bouwkosten van de OGF-eenheid met coagulatie bedroegen slechts 7% van de totale bouwkosten van de hele zuiveringsinstallatie. De operationele kosten voor het gebruik van coagulatie bedroegen slechts 0,3% van de totale operationele kosten.

Hoofdstukken 4, 5 en 6 hebben betrekking op verschillende proefprojecten. Hoodstuk 4 geeft de resultaten weer van de werking van een OGF waar een filterdoek bovenop het grind is aangebracht. Dit onderzoek omvat de verwijdering van zwevende stof en *E-coli*, verdeling van de deeltjesgrootte, drukverlies over het filter en de ontwikkeling van algengroei. De resultaten van de studie tonen aan dat het zuiveringsproces in een OGF wordt gekenmerkt door een rijpingsperiode, een periode van effectieve filtratie en een filterdoorbraak. De beste verwijdering van zwevende stof trad op gedurende het proces van effectieve filtratie met een tijdsduur van tussen de 30 en 39 dagen. Alle filterlagen dragen bij aan de verwijdering van zwevende stof. De grootste opeenhoping van zwevende stof per volume van de filterlaag vond plaats in het filterdoek (dikte 0,0056 m), maar deze laag draagt niet bij aan de verwijdering van *E-coli*. De verwijdering van *E-coli* was het hoogst in de bodemlaag (gravel afmeting van 19,1 mm) en neemt geleidelijk af in de erboven liggende lagen. Er werd een directe relatie gevonden tussen de toename van het drukverlies in het filterdoek en de ontwikkeling van algen. Zwevende stoffen met een deeltjesgrootte van meer dan 40 micrometer werden volledig verwijderd, terwijl deeltjes kleiner dan 2 micrometer nauwelijks werden verwijderd. De studie suggereert dat het aanbrengen van een filterdoek het mogelijk maakt de totale hoogte van het filter te verlagen zonder vermindering in de verwijderingsefficiëntie van zwevende stof aan te tasten. De studie laat tevens zien dat de huidige wekelijkse schoonmaak van OGF-systemen moet worden herzien en aangepast aan de plaatselijke omstandigheden. Een groter interval tussen de momenten van schoonmaken verbetert het verwijderingsrendement en vermindert waterverliezen.

De verwijdering van ijzer (Fe) en mangaan (Mn) in OGF, zowel op laboratoriumschaal alsmede in proeffilters, onder verschillende omstandigheden met een hoge en een lage zuurstofconcentratie bij verschillende pH-niveaus wordt beschreven in Hoofdstuk 5. Resultaten in het laboratorium met batchexperimenten met grind, bedekt met een laagje waarin ook ijzer en mangaan aanwezig is, vertonen kleine verschillen in Fe- en Mn-verwijdering bij hoge en lage zuurstofconcentraties (pH 5-8). Het verwijderingsrendement werd beïnvloed door de pH. De beste Fe^{2+} verwijdering (64%) trad op bij pH 7 en bij een hoge zuurstofconcentratie en deze concentratie werd bereikt na 5 uur bij een beginconcentratie van Fe^{2+} van 6 mgL^{-1}. De beste verwijdering van Mn (72%) werd verkregen na 4 uur bij pH 8 en een beginconcentratie van 2,0 mgL^{-1}. De verwijdering in de proeffilters was hoger met een mediane efficiëntie voor Fe totaal en Fe^{2+} tussen 75% - 95% en Mn tussen 60-95%, maar ook in dit geval was het effect van verschillen in zuurstofconcentraties klein. De filtratiesnelheid was van invloed waarbij een beter rendement werd verkregen bij een lage filtratiesnelheid (1-3 mh^{-1}).

Hoofdstuk 6 toont het effect van zes behandelingscombinaties voor troebel oppervlaktewater op vier types irrigatiesystemen op basis van een analyse van de potentiële verstopping en de uniformiteit van de waterdistributie. De proefinstallaties maken gebruik van verschillende combinaties van OGF, “disc filters” en LZF, gevoed met natuurlijk water uit de Cauca Rivier (Cali, Colombia). Het gezuiverde water van deze systemen werd gebruikt om vier verschillende type irrigatiesystemen te voeden. De totale hoeveelheid zwevende stof en ijzer waren de belangrijkste fysisch-chemische parameters die van invloed waren op de verstoppingsgraad en uniformiteit in de waterverdeling. Alle behandelingssystemen verkleinden het verstoppingsrisico. De beste prestaties werden verkregen met OGF, gevolgd door LZF en OGF in serie, maar dit zijn tevens de duurste systemen, terwijl het goedkoopste systeem, met disc filters, de slechtste prestatie leverde. De combinatie van OGF met een disc filter biedt echter goede mogelijkheden voor gebruik in micro-irrigatie. Ze zijn makkelijker te bedienen dan andere systemen en leveren een vrij goede prestatie tegen redelijke kosten. De langste irrigatie tijd van 600 uur werd verkregen voor het drukgecompenseerd irrigatesysteem en voor het Lyn-model.

Samenvattend biedt dit proefschrift een samenhangende beschrijving en analyse van de toepassing van OGF, de efficiëntie van het proces, andere mogelijke toepassingen van de zuiveringsmethode en mogelijke verbeteringen in exploitatie en onderhoud. Verschillende OGF-systemen werken al vele jaren en een groot aantal van hen wordt beheerd door lokale watercommissies. De ontwerpgrondslagen van de onderzochte systemen volgen de bestaande literatuur, met uitzondering van de toegepaste drainagesystemen en de stroomsnelheden. Het onderzoek toont aan dat aanpassing van een aantal ontwerpcriteria alsmede de procedures voor exploitatie en onderhoud om prestaties van de OGF te verbeteren, gewenst is. De toepassing van coagulatie in combinatie met OGF draagt in belangrijke mate bij aan de verwijdering van zwevende stof. Deze combinatie maakt het mogelijk om OGF ook te gebruiken in geval van langere periodes met een hoge troebelheid zonder dat dit een negatief effect heeft op de microbiologische verwijderingsefficientie en de duur van de filtratieperiode van de LZF. Dit draagt sterk bij aan de operationele flexibiliteit van het systeem, omdat de dosering van vlokvormingsmiddelen alleen hoeft te worden toegepast wanneer er hoge troebelheidspieken optreden. Alle grindlagen in een OGF dragen bij aan de verwijdering van zwevende stof, maar het rendement was hoger in het filterdoek en in de bovenste grindlaag (gravel grootte van 3,2 mm). De belangrijkste algengroei vond plaats in het filterdoek waar het bijdroeg aan de verwijdering van zwevende stof maar niet aan de verwijdering van *E-coli*. De hoogste reinigingsefficientie voor *E-coli* vond plaats in de onderste laag (grind afmeting van 19,1 mm). De in de literatuur genoemde reinigingsfrequentie voor OGF moet worden bijgesteld en worden verlengd om de meest effectieve filtratie te waarborgen. OGF met grind, bedekt met een laagje waarin ook ijzer en mangaan aanwezig zijn, is goed te gebruiken voor de verwijdering van Fe en Mn uit grondwater met een pH 7-8 met zowel hoge als lage zuurstofconcentraties. Weinig verschil werd waargenomen in de verwijderingsefficiëntie voor Fe en Mn onder hoge en lage zuurstofconcentraties bij lage filtratiesnelheid (1, 2 en 3 mh^{-1}). Het verhogen van de filtratiesnelheid tot 10 mh^{-1}, onder hoge zuurstofconcentratie was geen geschikte mogelijkheid, omdat het schoonmaken van de filters, volgens de in de literatuur vermelde procedure, bij deze toepassing niet tot het herstel van de oorspronkelijk filterweerstand leidde. Echter bij een lage zuurstofconcentratie nam bij deze filtratiesnelheid de verwijdering van Fe en Mn weliswaar af, maar werd wel een aanzienlijk langere filtratieduur bereikt, wat bijdraagt aan het verminderen van de schoonmaakfrequentie. Dit maakt een mogelijke combinatie van een eerste zuiveringstrap onder zuurstofarme omstandigheden bij een filtratiesnelheid van 10 mh^{-1} gevolgd door een tweede trap onder hoge zuurstofconcentratie met een filtratiesnelheid van tussen 1 en 3 mh^{-1} een veelbelovende optie voor Fe- en Mn-verwijdering tegen lagere kosten. OGF kan ook bijgedragen aan het verminderen van het risico van verstopping van micro-irrigatiesystemen en het bereiken van een lange irrigatie tijd. De combinatie van OGF en disc filters is veelbelovend voor gebruik in micro-irrigatie, omdat dit systeem veel gemakkelijker te bedienen is dan andere systemen en een behoorlijke verwijdering van zwevende stof en Fe geeft tegen redelijke kosten. Deze combinatie bleef goed werken gedurende meer dan 200, drie uur durende gietbeurten per dag, overeenkomend met een totale irrigatie periode van 6 maanden en 20 dagen.

Curriculum Vitae

Luís Dario Sánchez Torres

Luis Dario Sánchez Torres was born on 08th of April 1964 in Nuquí-Choco, Colombia. He grew up in Cali where he graduated from High School in 1984. Between 1984 and 1990 he obtained his degree as Sanitary Engineer at the Universidad del Valle in Cali. From June 1990 he works in Cinara Institute, Engineering Faculty at Universidad del Valle. In June 1993 he started his Master studies in Sanitary and Environmental Engineering at the Universidad del Valle in Cali-Colombia. He graduated with a thesis on biomembrane cleaning in a slow sand filter and its effect at the start filtration run in April 1996. In 2000 he was designated as coordinator of the research group on water supply at The Universidad del Valle-Cinara. From August 2010 he has worked on his PhD research under the supervision of Luuk Rietveld and Jan Teun Visscher on upflow gravel filtration for multiple uses. Luis Dario teaches at Universidad del Valle, engineering faculty and has experience in national and international development projects and programs on water supply in rural areas, sanitation and integrated water resource management. He also has experience in technology transfer processes with multidisciplinary groups with an emphasis on: strengthening capacities, institutional strengthening, planning, design, selection and implementation of water and sanitation technologies. He works on different research themes including: water treatment for rural communities, water quality in distribution networks and water use efficiency.

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Acknowledgements

This thesis has been motivated by my experience in the technology transfer program of multi stage filtration systems for improving water quality in rural areas of Colombia, developed by Cinara Institute and IRC- The Netherlands under direction of Gerardo Galvis and Jan Teun Visscher. I am grateful for the support received from the Universidad del Valle and Cinara Institute for this research. I also thank the team of Cinara which provided advice, suggestions and motivation. Special thanks go to Edgar Quiroga, Mariela Garcia, Jorge Latorre and Ines Restrepo. Many thanks also to the staff team of the Research Center at Puerto Mallarino for their laboratory support.

A vote of thanks also goes to Luuk Rietveld and Jan Teun Visscher, for their confidence in the development of this research. Your guidance, patience, time and interest were fundamental to achieve this product. Your visits to the Universidad de la Valle-Instituto Cinara, Cali-Colombia and the different projects included rich discussions which helped to consolidate the results. I am also grateful for the support received from the staff of the Sanitary engineering department at TU Delft.

This thesis has received the support of many people for which I am very thankful. I would like to mention in particular the plant operators: Alfonso Escandón and Eulogio Mósquera (Arroyohondo); Gabriel Murillo and Libardo Ortiz (El Retiro); Ever Tosse (La Sirena) and Leonardo Meneses (Golondrinas). I am also grateful for the support and trust received from the community and the management of the treatment system in the system of Arroyo Hondo, the system in El Retiro where they introduced some modifications in the new upflow gravel filters. In this case special thanks go to Eng. Diego Rengifo and the administrator Pilar Gañan. I also want to mention the community organizations in La Sirena and Golondrinas who facilitated my field work and measurements and the support received from the management and staff of the Terranova water supply system during the pilot study for iron and manganese removal, and specially of Luis Carlos Buitrago (Sanitary Engineer). I am also grateful for the support of other people for my research including Carlos Mejia (FHIS-Honduras, IRC-YZ program), Diana Posso, Ricardo Muñoz, Diego Andres Dominguez and Cristhian Viafara (Programs of Sanitary and Agricultural Engineering, Univalle). Special thanks go to the team in the research station in Puerto Mallerino (water supply research group at Universidad del Valle -Cinara Institute) and particularly to Noel Muñoz Soto, Alberto Benavides, Lina Maria Marin, Andres Vargas, Yuly Paola Artunduaga, Clara Ines Gonzales, Segundo Valverde and Mariano, and to Janet Lucia Jiménez (Mecanizados Industriales) for her trust and support to facilitate my doctoral work through Univalle during these five years.

Writing this thesis would not have been possible without the unconditional support and patience of my wife Claudia Inés Jiménez for which I am very grateful and for the support, love and patience of my daughters Laura and Sara. I love you all.

Thanks also go to my parents Maria Elvia and Bernardino, my brothers Hernán and Arlex, my sisters Mery, Maria Eugenia, Martha, Adiela and Rosaura and my uncles, nephews and cousins for their comprehension and cooperation. Thanks by the support in Delft to my family: Natasha, Arlex, Aylen and David. Special thanks to my mother in law Teresa, brothers in law Janet, Rosa Patricia, Beto and Carlos Julio for your support and finally thanks to all my friends in Cali-Colombia.

List of Tables

The use of upflow gravel filtration (UFG) is relevant for water supply systems in rural areas and small towns in Colombia, because water quality from surface sources is changing due to the deterioration of watersheds caused by deforestation, erosion, and the discharge of untreated wastewater. These changes are intensified by the effects of global climate change. The main problems that occur include the increase in turbidity and suspended solid levels with higher peaks of longer duration. These changes affect the existing water treatment plants, causing higher operation and maintenance requirements and even interruptions in their operation. To overcome such problems a better understanding of the performance of UGF systems is needed because this type of pre-treatment is used in almost all low cost multi stage filtration systems in Colombia.

This research combines analyses in existing full scale plants and research at lab and pilot filter scale in order to explore possible limitations, identify improvements and test other possible applications of the UGF technology. This includes the potential use with coagulation and flocculation to overcome longer periods of high loads of suspended solids; the application of a filter fabric on top of the UGF to reduce filter height and improve surface cleaning procedures; the application of UGF for iron and manganese removal from groundwater sources; and the potential use of UGF in micro-irrigation.

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