OPTIMISATION OF AN AUTONOMOUS VALVELESS GRAVITY FILTER FOR THE COST EFFECTIVE PRODUCTION OF POTABLE WATER FOR RURAL AREAS

B Brouckaert · R Rajagopaul · MJ Pryor

WRC Report No. 919/1/03

Water Research Commission

Optimisation of an Autonomous Valveless Gravity Filter for the Cost Effective Production of Potable Water for Rural Areas

B Brouckaert ● R Rajagopaul ● M J Pryor

Report to the Water Research Commission
by
Umgeni Water

May 2003

Executive Summary

The choice of a treatment process for potable water production is strongly influenced by the raw water quality, desired final water quantity, physical location and the availability of resources for operation and maintenance. Consumers in rural areas are financially disadvantaged to pay for services, placing increased pressure on designers and suppliers of small-scale schemes to reduce both capital and operating costs for affordability.

When conventional type "small-scale plants" are operated by central authorities (water boards or municipalities), operating costs are driven up primarily by the traveling time and transport costs to and from the plant, as well as the wide range of specialised skills that are required to operate the processes from a central base.

These circumstances place restrictions on the designers of rural water supply schemes to provide low cost, low maintenance plants without compromising the quality of water produced. Processes that are inherently self-regulating and simple have an obvious advantage in that they will require fewer skilled operator visits to the rural plant. This allows many of the operating and maintenance tasks to be developed to community level and reduces the reliance of the plant on specialised skills. Examples of these processes include roughing filters, slow sand filters and autonomous valveless gravity filters (AVGF) which backwash automatically by siphoning.

The automated siphoning of the AVGF filter ensures that once the headloss has developed and reached a specific value the unit will stop filtering and initiate a backwash, thereby cleaning the filter regularly and preventing turbidity breakthrough whilst ensuring that the filter media does not become overloaded with suspended matter. This can be significantly better than a manually initiated or timer initiated backwash in areas where there is variable water quality or where the operator is semi-skilled and technical reliability is poor.

Against this background, the automatic backwash filter (AVGF) exhibits numerous advantages.

- a single unit, robust steel construction with no internal moving parts and a minimum of valves thereby reducing mechanical maintenance,
- a siphon actuated, hydraulically controlled backwashing requires no operator intervention,
- no electricity required other than that which may be necessary to obtain water from the source,
- various media can be used for different treatment objectives, and
- the filter is suitable for use with water or gravity powered chemical dosing systems.

Against these advantages, the AVGF filter has a relatively high capital cost due to its robust construction. Furthermore the design may not be ideal for certain applications, and under some conditions the filter may fail.

None-the-less the filter has the potential to operate for extended periods with only a low degree of intervention by a semi-skilled operator, provided that an appropriately skilled supervisor is available to deal with non-routine events.

Re-engineering of the basic design using modern materials and coating technology can reduce costs in the construction of these filters without compromising their robust and simple nature. Should the filter be successfully applied to rural water treatment applications, this type of filter has the potential to significantly reduce both the capital and operating costs of small-scale water treatment plants without compromising water quality.

The critical design parameters for the AVGF are the filtration rate, the headloss at which backwash is initiated (terminal headloss), the head and volume of water available for backwash and the resulting backwash velocity versus time profiles.

The objectives of the project were to :
- assess the suitability of a proprietary autonomous valveless gravity filter for inclusion in low maintenance treatment processes for potable water production from surface waters,
- propose where necessary, design modifications aimed at improving the reliability and reducing the capital cost of a proprietary backwash filter, and
- propose specific operating rules by which the unit should be operated and maintained and to prescribe minimum levels of operator supervision.

The AVGF that was available to the project team was a commercial filter previously operated in a remote Game Reserve under little or no supervision. There were no historical data with respect to design specifications, operating parameters or maintenance. The condition of the filter media and internals were unknown. On the outside, the filter was merely a black box, where raw water entered at one point of the filter and filtered water exited through the other side.

It was therefore necessary to first assess the filter, measure the critical dimensions and renew the media before operating. The influence of important filter operating variables like media size and type and empirical modeling of the media headloss and fluidisation during backwashing was performed using a series of pilot sand filters, specifically constructed during the project

Summary of results

From the various experiments conducted on pilot scale filters and more experiments on the full scale AVGF plant, it was concluded that the AVGF system may be used in rural applications. However, the sustainable water quality and production rate is dependent on a number of operational issues that need to be understood and be given consideration. The findings are presented under these and other operational issues that have been specifically tested and evaluated.

Measurement of the headloss through different nozzle configurations led to modification of the filter.

For potable water, turbidity of the filtered water was identified as the most important criteria, and in some cases this was found to breakthrough prior to the attainment of terminal headloss (which would cause the filter to backwash). This was considered to be poor operation and under these circumstances, either a different size media should be chosen which would cause a more rapid headloss development, or chemical coagulation conditions should be changed which, with the same media, would cause a more rapid headloss development.

Coagulant dosing

In general coagulant is required to achieve acceptable filtered water turbidities. This means that the filter performance will only be as reliable as the dosing system.

Backwash efficiency

Water-only backwash was insufficient to prevent the formation and accumulation of mudballs. However, for raw water turbidities of < 5 NTU, partially mudballed filters were able to operate for extended periods of time (approximately one year in this study) without operator intervention (except coagulant dosing) and without any detectable decline in filter performance.

Filter hydraulics

No valve adjustment is required to maintain a constant filtration rate. This is a significant advantage with respect to plant monitoring requirements.

Backwash velocities are controlled by the head in the backwash tank and the headlosses in the pipes, nozzles and media. The maximum backwash rate is determined by the height of the backwash tank above the media compartment. Backwash rate (during backwashing) decreases relative to the level in the backwash tank and frctional losses in the pipes.

Media selection

It is imperative that the media selection is compatible with the available backwash rates for a given filter. Dual media filters had the advantage of longer run times with less risk of breakthrough at higher influent turbidities than mono-media sand filters.

At low turbidities anthracite was more susceptible to mudballing than sand, however the mudballing was entirely restricted to the anthracite layer in dual-media filters whereas the mudballs in the mono-media filters gradually moved down towards the filter floor. Therefore it may be easier to manage mudballs in dual-media filters. Increasing the fraction of sand in dual media filters delayed turbidity breakthrough and shortened the run time.

Terminal headloss

The AVGF hydraulic design set the terminal headloss for the initiation of backwash, at 1,5 m. The fixed terminal headloss contributed to accumulation of solids in the filter, resulting in turbidity breakthrough for raw water turbidities greater than 16 NTU. Under low raw water conditions, excessively long filter run times (>60 hours) were experienced.

Raw water turbidity

No turbidity breakthrough was observed in any of the filter or media combinations for a raw turbidity of <10 NTU. The dual media filter (DM#2) reached terminal headloss without breakthrough for raw water turbidities up to 26 NTU. The 0.5mm sand operated with similar performance under raw water conditions up to 40 NTU.

Backwash volumes

The actual backwash volume of the AVGF evaluated in this study is 2,2 m^3/m^2 which is ssignificantly less than the minimum recommended in the literature. The effective backwash volume, defined as the volume required to reduce the backwash water turbidity to 10 NTU, ranged from 2,3 to 3,1 m^3/m^2 for monomedia filters and 2,8 to 3,6 m^3/m^2 for dual media filters.

Mudballing

Mudballing was a common occurrence during the operation of the AVGF. The extent of mudballing and their location in the filter depended, to a large extent, on the filter media (dual or mono media), coagulant type and dose (inorganic or organic).

Power requirements

In general no electrical power may be required if the placement of the filter ensures sufficient head at the raw water inlet and filtered water delivery.

Design considerations for the AVGF

After analyzing the components and dimensions of the AVGF and monitoring the filtration process under controlled conditions on a pilot plant scale and fullscale AVGF, some modification to the design and other considerations for design were suggested. Implementation of these suggestions will be influenced by cost and other factors like raw water variability, infrastructure, readily available technically competent operators. Although these are presented in some detail in chapter 5, the main conclusions are restated below:

- It is possible to provide facilities for manual auxiliary backwash and air scour.
- Additional tappings on the backwash pipe for adjustments to the terminal headloss.
- Dual media provides more protection against premature turbidity breakthrough at high raw water turbidities (> 15NTU) than monomedia filters.
- The volume of backwash water required is $3.1m^3/m^2$ for monomedia filters and $3.6m^3/m^2$ for dual media (anthracite/sand) filters.
- Filter nozzle density should be atleast 50 nozzles/m^2.
- The backwash tank should be covered.

Recommendations

The application of the AVGF for the supply of drinking water in the rural areas, considering the lack of resources and infrastructure and the level of readily available technical expertise, is a challenging prospect. Experience has shown that the AVGF cannot operate efficiently without operator intervention.

Recommendations arising from the technical investigations and operating experience of the AVGF plant have been dealt with in detail in chapter 6. Here, the more practical recommendations are presented.

- In situations where the inflow turbidity is greater than 15NTU and large variations in raw water turbidity is anticipated, eg, a river abstraction, a roughing filter or clarifier should be installed upstream of the AVGF. This would ensure a smoother solids loading profile, in addition to keeping the solids low enough for the filter to handle.
- Greater emphasis should be placed on ensuring that the material of construction of the filter should be corrosion resistant. This is most critical to the internals of the filter, where rust and scale deposits contaminate the final water and may also block the nozzles. Although

mild steel may be preferred for its strength and cost, suitable coating of the insides with a corrosion resistant material should be considered.

- Due the inherent inadequacy of the 'water only' backwashing system, mudball formation and associated operating problems, will require more frequent access to the filter media. Filter media handling during loading of fresh media, inspection and removal of spent media can be significantly improved by increasing the size of the manholes. A thicker and more durable rubber gasket should be used to provide a better seal without damaging the nuts and bolts through excessive tightening.

- To increase the efficiency of backwashing and thus reduce mudballing, , the use of air scour is recommended. A valve has to be connected on the pipe that connects the backwash water tank to the underdrain compartment to prevent the air from bypassing the media and escaping through backwash tank into the atmosphere.

- Filling of the filter bed with water before loading the media, will reduce air entrainment and consequent filter binding. When using anthracite as a filter media, it is imperative that the anthracite is allowed to soak in water for at least 24 hours before backwashing to prevent media loss. A further advantage of filling the bed with water first, is that the filter can also brought into service in a relatively shorter time.

- The inclusion of a chute for the removal of wet filter sand from the filter, will make the filter more ergonomically acceptable.

- Nozzle damage during sand inspection for mudballs or sand removal is a distinct possibility and should be avoided. When sand levels reach a few centimeters above the nozzle caps, extreme care should be exercised and the use of spades should be avoided.

- For safety reasons, the vertical fixed ladder mounted on the side of the filter for access to the top of the filter, should have hooped guards to prevent accidents due to slipping and falling off.

- Because the filter operation is not as labour intensive as manually backwashed rapid filters, they are recommended in situations where there may be a shortage of appropriately skilled labour.

Acknowledgments

The research in this report emanates from a project funded jointly by the Water Research Commission and Umgeni Water and entitled:

Research on the optimization of an autonomous valveless gravity filter

for the cost-effective production of potable water to rural areas

The Steering Committee responsible for this project consisted of the following persons:

Dr I Msibi	Water Research Commission
Dr G Offringa	Water Research Commission
Mr C D Swartz	CD Swartz Consulting
Mr K Naidoo	Rand Water
Mr A L Batchelor	Environmentek, CSIR
Prof J Haarhoff	Rand Afrikaans University
Mr C Brouckaert	University of Natal
Ms C Motau	Eskom
Mr V Botes	Mhlatize Water
Mr W J Voortman	Voortman & Associates
Prof F A O Otieno	Technikon RSA
Mrs E Engels / Mrs C M Smit	Water Research Commission (Committee Services)

The financing of the project by the Water Research Commission, the provision of laboratory, pilot-plant and office facilities by Umgeni Water and the contribution of the members of the Steering Committee is gratefully acknowledged.

The employment of In-service trainees by Umgeni Water is acknowledged. This has contributed to persons being able to work on the project and gain an understanding of water treatment, especially filtration.

The assistance of the following persons during the course of the project is acknowledged, and without their commitment and dedication the quality of data from the pilot plants would not have been achieved.

R. Rajagopaul.. Chief Technician Umgeni Water

M I Maishi .... In-service Trainee / Contract Technician

N Moodley .... In-service Trainee

M N Nkabinde .... In-service Trainee

M H Shabane .... In-service Trainee

M T Mthiyane... In-service Trainee

X P Ngcobo.. In-service Trainee

T S Kistan .... In-service Trainee

T Zondi .... In-service Trainee

P Pillay.. In-service Trainee

Table of Contents

List of Figures

Section 5

Section 10

Section 11

List of Tables

Glossary

Alum	Aluminium sulphate
AVGF	Autonomous Valveless Gravity Filter
DM	Dual media filter
gpm	Gallons per minute
gpm/ft^2	Gallons per minute per square foot
mg/L	Milligrams per litre
m/h	Metres per hour
min	Minutes
m^3/m^2	Metres cubed per metres squared filter area
NTU	Nephelometric Turbidity Units
PEF	Umgeni Water Process Evaluation Facility
WRC	Water Research Commission

1. Introduction

The choice of a treatment process for potable water production is strongly influenced by the raw water quality, desired final water quantity, physical location and the availability of resources for operation and maintenance. The complexity of the treatment process required to treat raw water to potable standard is not significantly affected by the capacity of the plant. Economies of scale between large municipal water treatment plants and small-scale plants can influence the process selection and therefore the quality of the water supplied to a consumer. In most instances the consumers in rural areas are reluctant to pay for services, placing increased pressure on small-scale schemes to reduce both capital and operating costs in order that they remain affordable.

When conventional type "small-scale plants" are operated by central authorities (water boards or municipalities), operating costs are driven up primarily by the traveling time to and from the plant, as well as the wide range of specialised skills that are required to operate the processes from a central base.

These circumstances place restrictions on the designers of rural water supply schemes to provide low cost, low maintenance plants without compromising the performance and quality of water produced. Processes that are inherently self-regulating and simple have an obvious advantage in that they will require fewer skilled operator visits to the plant at less frequent intervals. This allows many of the operating and maintenance tasks to be delegated to community level and reduces the reliance of the plant on specialised skills. Examples of these processes include roughing filters, slow sand filters and the autonomous valveless gravity filter, (AVGF).

1.1. Background / statement of problem

The 'self initiating' siphoning mechanism of the AVGF backwash cycle ensures that once the headloss has developed and reached a specific value, the filtration cycle will stop and a backwash initiated. This autonomous backwashing mechanism has significant advantages over other manually initiated or timer initiated backwash systems in areas where the raw water source has a variable water quality or where technical operating skills are lacking.

Against this background it is believed that the autonomous valveless gravity filter (AVGF) exhibits numerous advantages.

- a single , robust steel structure with no internal moving parts and a minimum number of valves reduces mechanical maintenance,

- a siphon actuated, hydraulically controlled backwashing cycle requires no operator intervention,
- no electricity is required if the head requirements for gravity feed are met.
- various media can be used for different treatment objectives, and
- the filter is suitable for use with water or gravity powered chemical dosing systems.

While the AVGF has a number of advantages for potable water production in a rural environment, capital costs are high due to its robust construction. Furthermore, there may be certain situations where the system is not suitable and under some conditions the filter may fail. The filter is similar in principle to other gravity filters. The significant difference is in the automatic initiation of the backwash cycle based on a preset terminal filter headloss. Problems arise when turbidity breakthrough precedes terminal headloss. The optimization of the AVGF and sustained final water quality depends to a large extent on the understanding of operating variables that include raw water quality, chemical pretreatment, solids loading, filter runtime, media type and size. Another important consideration is the backwash rate, frequency and duration. The use of a 'water only' backwash strategy may also pose a problem , especially in a direct filtration application. In general, for an efficient AVGF system,

- Filtration performance must be adequate
- Requirements for adequate backwashing must not exceed the head and volume available.

1.2. Objectives

The objectives of the project were to :
- assess the suitability of a proprietary AVGF for inclusion in low maintenance treatment processes for potable water production from surface waters,
- assess the suitability of a proprietary automatic backwashing filter for inclusion in low maintenance treatment processes for the removal of iron and manganese from groundwater supplies,
- propose where necessary, design modifications aimed at improving the reliability and reducing the capital cost of a proprietary backwash filter, and
- propose specific operating rules by which the unit should be operated and maintained and to prescribe minimum levels of operator supervision.

1.3. Approach

The AVGF that was available to the project team was a commercial filter previously operated in a remote Game Reserve under little or no supervision. There were no historical data with respect to design specifications, operating parameters or maintenance. The condition of the filter media and

internals were unknown. From the outside, the filter was merely a "black box", where raw water entered at one point of the filter and filtered water exited through the other side.

It was therefore necessary to first assess the filter, measure the critical dimensions and renew the media before operating. The influence of important filter operating variables like media size and type and empirical modeling of the media headloss and fluidisation during backwashing was performed using a series of pilot sand filters, specifically constructed during the project.

The experimental approach adopted in meeting the objectives of the project are listed:

- Operate filter to gain operating experience.
- Undertake a detailed analysis of hydraulics
- Accumulate data to predict long term performance
- Design ,commission and operate pilot plant sand filters to investigate
  - Variation of design parameters
  - Media specifications
  - Backwash rates

Based on the results of the pilot- and full-scale evaluation of the performance of the AVGF specific design variables were considered and the effect of optimizing these are discussed in **Chapter 5**. The experience gained in operating this type of filter for direct filtration of surface water and the need for regular filter observation and maintenance is summarized. Proposals for the efficient operation of the AVGF are made in **Chapter 6**.

2. Literature Review

Filtration is one of the principle unit operations used in the treatment of potable water. The ability to design filters and to predict their performance must be based on an understanding of the variables that control the process and knowledge of the pertinent mechanism or mechanisms responsible for the removal of particulate matter from raw water. AVGF's are a particular example of rapid granular media filtration with unique design and operating constraints. Since these are proprietary items, there is little information on their design and operation in the water treatment literature. This Chapter presents general aspects of rapid filtration operation and design which are relevant to the results and discussion of AVGF performance and design presented in this report.

The filtration operation is comprised of two phases: filtration and cleaning (commonly called backwashing). During filtration, water passes through a filter medium and particulate matter either accumulates on the surface of the bed or is collected through the depth of the media. Filters have been found to be effective for removing particulates of all size ranges including algae, colloidal humic substances, viruses and colloidal clay particles, provided proper design parameters are used.. The most common filters for water treatment are slow and rapid sand filtration (Cleasby, 1990; Kawamura, 2000).

Table 2.1   Variables in the design of granular media filters

Variable	Significance
1. Filter Media Characteristics a. Grain size and uniformity b. Grain size distribution c. Grain shape and density	Affect the particle removal efficiency and headloss development as well as the backwash rate requirements (Cleasby, 1990)
2. Filter bed porosity and collector area	Determines the quality of the filtrate (Amirtharah, 1978a; Kawamura, 1975a)
3. Filter bed depth	Determines the collector surface area and floc storage capacity of the bed (Cleasby, 1990)
4. Filtration rate	Affects the rate of headloss development, filter run time and volume produced (Cleasby, 1990).
5. Backwash rate and duration	Affects the bed expansion and removal of entrained particles for continued reliable operation (Amirtharajah, 1978a)

4

6. Maximum available headloss	Determines the length of filter runs, and the case of valveless filters, the height and therefore the cost of the filter shell (Kawamura, 1975a)
7. Influent water characteristics	These can be controlled to some extent by pre-treatment, and will impact on the ability of the filter to retain particles or result in premature breakthrough of suspended solids from the filter (Kawamura, 1975a; Cleasby, 1990).
a. Suspended solids conc.	
b. Floc strength	
c. Floc or particle charge	
d. Fluid properties	

The principal variables that should be considered in the design of filters are identified in **Table 2.1**. In the application of rapid gravity granular media filtration, the design of filter media (size of media, media properties), the nature of the particulate matter in the influent to the filter (and pre-treatment requirements), the filtration flowrate and effective backwashing of the filter are the most important process variables.

2.1. Filter media characteristics

In practical applications the media used for filtration are neither uniform nor spherical and consist of a range of particle sizes. Combinations of different types of media can also be used in the same filter to promote depth filtration and produce longer run times.

2.1.1. Media Properties

Common types of filter media used in granular bed filters are silica sand, anthracite coal and garnet or ilmenite (Cleasby, 1990). These may be used alone or in dual or triple media combinations. Other types of media have been used for special applications. For example, granular activated carbon is used for taste and odour reduction and serves for both filtration and adsorption. A number of properties of filter media are important in affecting filtration performance and defining the media. These together with effective selection for a particular application improve the reliability of filtration for particle removal. A comprehensive review of the South African filter media characteristics and trst methods is given by Ceronio and Haarhoff (1994).

2.1.1.1. Density

The density of granular media does not directly affect the performance of filtration, but is a critical parameter in determining the backwashing / fluidisation behaviour of filter grains and in selecting compatible media sizes for multimedia filters. Typical densities are shown on **Table 2.2**.

Table 2.2	Density and sphericity of common filter media		
Material	**Density** (kg/m³)	**Sphericity** (shape)	**Reference**
Quartz sand (UK)	2650	0.85	
Anthracite (UK)	1400	0.70	(Ives, 1990)
Hydroanthracite	1700	0.65	
Silica sand	2650	0.7 – 0.8	(Cleasby, 1990)
Anthracite coal	1450 – 1730	0.46 – 0.60	
Sand 0.5 – 0.6 mm (Brits, SA)	2623 – 2629	0.52 – 0.69	
Sand 1.0 – 1.18 mm	2623 – 2629	0.48 – 0.66	
Sand 1.4 – 1.7mm	2623 – 2629	0.42 – 0.58	
Anthracite 1 – 1.18 mm (Brits, SA)	1443	0.66	(Ceronio and Haarhoff, 1994)
Anthracite 1.4 – 1.7 mm	1443	0.68	
Anthracite 2 – 2.36 mm	1443	0.67	

2.1.1.2. Sphericity

Many definitions for the media grain shape have been proposed (Cleasby, 1990; Ceronio and Haarhoff, 1994; Ives, 1990). The settling velocity of individual grains of a uniform size is a useful characteristic to determine the stratification behaviour of the media (if water alone is used for backwash), and to determine the equivalent hydraulic diameter of a sphere with the same settling velocity. This leads to the determination of hydraulic shape, or sphericity of the grains by determining the ratio of the equivalent hydraulic size to the mean sieve size (Ives,1990). In contrast the sphericity is also defined as the ratio of the surface area of an equal volume sphere to the surface area of a grain. The volume of individual grains is measured to determine the equivalent spherical diameter. The pressure drop of water flow through a bed of uniform-sized particles can be related to the sphericity by the Kozeny equation (Amirtharajah, 1978a). **Table 2.2** shows the range of sphericities for different media.

$$\frac{h}{l} = 25 \frac{v}{g} V \frac{(1-\varepsilon)^2}{\varepsilon^3} \left(\frac{6}{\psi}\right)^2 \sum_i \frac{x_i}{d_i^2} \qquad \dots\dots\dots\dots\dots \quad (2.1)$$

h Headloss, m

l	Bed height, m
v	Kinematic viscosity, m^2/s
V	Superficial velocity, m/s
ε	Porosity
ψ	Surface area sphericity
d_i	Geometric mean mesh size for the ith mass fraction, m
x_i	ith mass fraction

2.1.1.3. Porosity

The fixed bed porosity is defined as the ratio of the void volume to total fixed bed volume (Ceronio and Haarhoff, 1994). This is important as it affects the minimum fluidisation velocity, the fixed bed headloss and the solids holding capacity of the medium. The porosity is used to determine the mass of media to fill a given volume of filter and is not a property of the media grains themselves but rather a function of the media grain shape. Porosity tends to decrease with increasing sphericity (Wen and Yu, 1966). The porosity of freely settled grains will be the highest (Cleasby, 1990) but can be reduced by vibration of the container (Ives, 1990) and consequently can depend on laboratory technique.

2.1.1.4. Media size distribution

Sieve analysis of media will provide information relating to the size distribution of the media and mean grain size. The effective size d_{10} and uniformity coefficient UC are used to characterise the grain size distribution for a given media sample, where (Ives, 1975)

$$U.C. = \frac{d_{60}}{d_{10}} \qquad \text{................................} \qquad (2.2)$$

d_{10}	Mesh size through which 10% of the sample mass passes
d_{60}	Mesh size through which 60% of the sample mass passes

This is determined by a standard sieve test (e.g SABS Standard Method 829 (SABS, 1976) and ASTM Standard Test C136 – 84a (ASTM, 1985)), which measures the weight of sample passed through successively finer sieves. The d_{90} size also be determined by sieve analysis or can be calculated from the d_{10} and UC (Cleasby, 1990).

$$d_{90} = d_{10}\left(10^{1.67 \log UC}\right) \qquad \text{................................} \qquad (2.3)$$

d_{10} d_{90}	Mesh size that passes 10% and 90% of the sample respectively

UC	Uniformity coefficient

2.1.1.5. Minimum fluidisation velocity

The point of incipient fluidisation, or minimum fluidisation velocity V_{mf}, is the superficial fluid velocity required for the onset of fluidisation. V_{mf} is a function of grain size, shape and density, water temperature and fixed bed porosity. Wen and Yu (1966) developed the following correlation for V_{mf} using an approximate relationship between grain sphericity and fixed bed porosity:

$$V_{mf} = \frac{\mu}{\rho\, d_{eq}}\left(33.7^2 + 0.048 Ga\right)^{0.5} - \frac{33.7\mu}{\rho\, d_{eq}} \quad \text{..............} \qquad (2.4)$$

V_{mf}	Minimum fluidisation velocity
μ	Water viscosity
ρ	Water density
d_{eq}	Equivalent media diameter

The Galileo number Ga is defined by :

$$Ga = d_{eq}^3\, \frac{\rho(\rho_s - \rho)g}{\mu^2} \qquad \text{.............................} \qquad (2.5)$$

ρ_s, ρ	Density of the media and water respectively (units)
μ	Water viscosity (units)
g	Gravitational acceleration constant 9.81 (m/s^2)
d_{eq}	Equivalent media diameter

This procedure however assumes that the filter bed has identical sized particles. In reality, for a bed containing a particle size distribution, smaller grains become fluidized at lower superficial velocity than larger grains. Generally the equivalent diameter is not conveniently available, and the d_{90} from a sieve analysis may be used as an acceptable approximation (Cleasby, 1990).

Leva (1959) developed an alternate semi-empirical correlation for graded media based on the d_{60} size.

$$V_{mf} = \frac{0.00381(d_{60})^{1.82}\left[\gamma(\gamma_m - \gamma)\right]^{0.94}}{\mu^{0.88}} \qquad \text{.........................} \qquad (2.6)$$

V_{mf}	Minimum fluidisation velocity in gpm/ft^2 *(need to show unit conversions)*

8

d_{60}	Particle diameter in mm
γ, γ_m	Fluid and specific media weights in lb/ft³
μ	Viscosity of water in centipoise

Since the d_{60} media size is usually available, **Equation 2.6** is often more convenient to use than **Equation 2.4**.

2.1.2. Media Design

The selection of media for filtration should primarily be based on the degree of purification required, as well as the durability of the media (Kawamura, 1975a). However, the length of the filter run and the ease with which suspended matter is removed from the media during backwashing are equally important factors. The finer the filter media, the better the water quality produced, but the length of the filter run is reduced (Baumann, 1978).

In general, the quality of the filtered water is a function of the surface area of the media and of the pore size between the media grains (Kawamura, 2000). The efficiency with which influent particles are removed by the filter is roughly correlated to the total surface area of the grains. The total collector surface per unit cross-sectional area of filter may be estimated from **Equation 2.7** (Amirtharajah, 1978a).

$$\sum A = \frac{6(1-\varepsilon)}{\psi}\left(\frac{l}{d}\right) \qquad \text{...} \qquad (2.7)$$

$\sum A$	Collector surface per cross sectional area
ε	Porosity
ψ	Sphericity
l	Bed height, m
d	Average grain size, m

$\sum A$ typically $\cong 2800$ for conventional filters, however, greater bed depths are usually used in direct filtration.

Therefore, a bed of fine sand will tend to have higher removal efficiency than the same volume of coarse sand, since fine grains have a higher specific surface area. Conversely, a bed of coarse sand must be deeper than a bed of fine sand to be able to achieve the same removal efficiency. Kawamura (1975a) states however that the use of fine media does not provide a reasonable length of filter run at filtration rates greater than about 7.5 m/h (180 m/day), mainly due to deposition of

9

suspended matter in the top 5cm of media. This can under certain conditions create negative head and in the upper layers of the filter bed, which is undesirable due to air binding, but also forms a compacted layer of mud-sand mixture at the top of the media. This is likely to contribute to mudball formation in filters without adequate auxiliary backwash.

There are many possible combinations of filter media size, d, and depth, l. Typical values of l and d_e (d_{10}) for various applications are shown in **Figure 2.1** (Kawamura, 1975; Montgomery, 1985). The group of diagonal lines in **Figure 2.1** was established base on a surface area of 2800 m^2 and various porosities that are typical of high rate filters. The *uniformity coefficients* of media in **Figure 2.1** were 1.3 – 1.5 Kawamura (1975a) indicates that some investigators have claimed that uniformity coefficients even smaller than 1.3 should be used.

Kawamura (2000) indicates that l / d ratio should differ depending on the filter configuration, application and size of the media used. When the size of the media exceeds 1.5 mm the space between the grains becomes large compared to regular filters and poor filter performance could result. The general design guide for filter beds are as follows :

l/d_e

> 1000	for rapid sand filters and basic dual media filters	
= 1250	for regular tri-media filters	
> 1300	for coarse media filters where d is 1.2 – 1.4 mm and l is less than 2 m	
>1500	for coarse media where d is greater than or equal to 1.5 mm	

In summary therefore some designs specify the use of monomedia, preferably of a narrow size distribution to eliminate the risk of intermixing. The finer the media, the better the filtrate quality and the lower the backwash rates required. However, the smaller the media size the greater the rate of headloss development and the tendency to mudball (Kawamura). Coarse monomedia filters however require deeper beds and higher backwash rates but may be less likely to mudball than fine or dual media filters. This allows deeper penetration of the floc in coarse sand filters, resulting in slower headloss development, longer run times and more efficient use of the floc storage volume of the bed (Monk and Willis, 1987). However, greater penetration of the floc implies greater risk of breakthrough and coarser, deeper beds require higher back wash velocities and available heads.

10
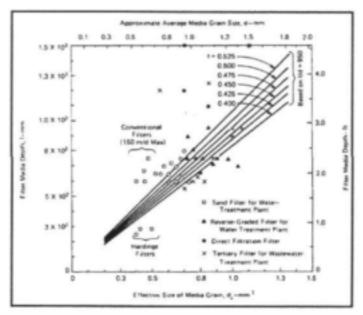
Figure 2.1   Relationship between depth of media and size of media (f = fixed bed porosity)

2.1.3. Dual Media

The disadvantages of using smaller sand sizes can be offset by using a combination of sand and anthracite Some designs utilise dual media where the reverse grading of the media results in longer run times compared to fine monomedia sand while reducing the risk of breakthrough compared to coarse monomedia filters (Kawamura, 2000). Dual media filters consist of a layer of relatively coarse anthracite on top of a layer of relatively fine sand. The lower density of the anthracite allows this reverse grading of the media to be preserved during fluidisation and re-stratification of the filter bed. Dual media filtration is widely used in the USA (Kawamura, 2000; Monk and Willis, 1987) and other parts of the world but as far as the authors know, it is little used in South Africa. The advantages of dual media include:

- The coarse top layer allows greater penetration of floc into bed. Consequently, there is better utilization of the floc storage capacity of the filter. This result in slower headloss development (half or less than that for monomedia sand of the same size as used in the dual media filter) and longer filter runs (Cleasby, 1990).
- Delays in turbidity breakthrough in dual media filters as compared to monomedia filters have been reported (Robeck *et al.*, 1964)

Disadvantages include:

- Anthracite is more expensive than sand.
- Care must be taken to choose compatible media sizes to prevent excessive intermixing of sand and anthracite (Amirtharajah, 1978a).

11

To achieve a sharp interface between sand and anthracite, the largest anthracite grains (d_{90} anthracite) must fluidize at approximately the same velocity as the smallest sand grains (d_{10} sand). Problems arise when the sand fluidizes first, penetrating the anthracite layer. Sand grains which are less than one fifth of the size of anthracite grains will fill the pores between the larger grains resulting in a region of vary low permeability at the interface. This may clog excessively and be difficult to clean (Cleasby, 1975). Amirtharajah (1978a) presents the following criteria for compatibility the media based on the minimum fluidization velocity of the d_{90} size of the anthracite and the d_{10} size of the sand:

$$\frac{d_{c,90}}{d_{s,10}} = \left(\frac{\psi_s}{\psi_c}\right)\left[\frac{\rho_s - \rho}{\rho_c - \rho}\right]^{0.52}$$ .... (2.8)

$d_{c,90}$	90 % passing size for anthracite
$d_{s,10}$	10 % passing size for sand
ψ_s, ψ_c	Surface area sphericities of sand and anthracite, respectively
ρ_s, ρ_c, ρ	Density of sand, anthracite and water, respectively

Amirtharajah calculated a size ratio of ~ 2.7 based on the sphericity of sand and anthracite being 0.85 and 0.7 respectively and the density of sand and anthracite being 2650 kg/m^3 and 1500 kg/m^3 respectively.

If the sphericities of the media are not available than the following relationship adapted from Kawamura (2000) can be used.

$$\frac{d_{s,10}}{d_{c,10}} = \left[\frac{\rho_c - \rho}{\rho_s - \rho}\right]^{0.667}$$ .... (2.9)

$d_{c,10}$	10 % passing size for anthracite

In practice, a moderate amount of intermixing is usually allowed and a size ratio of $\frac{d_{c,90}}{d_{s,10}} \sim 3$ is recommended (Baumann, 1978). Typical specifications are shown in **Table 2.3**. Typical properties of SA filter media including anthracite were shown in **Table 2.2** (Ceronio and Haarhoff, 1994).

Table 2.3   Typical specifications for dual media

Reference	Sand		Anthracite		Headloss
	d_{10} (mm)	U.C.	d_{10} (mm)	U.C.	
US practice (Cleasby,1990)	0.45 – 0.55	1.3 – 1.5	0.90 – 1.10	1.5	0.6 – 0.9 m total, anthracite = 10 – 70 % of total

2.2.   Pretreatment for rapid filtration

Effective operation of rapid granular media filters requires some form of pre-treatment. This is opposed to the treatment of water in slow sand filters where, provided the water turbidity is sufficiently low, a consistent filtered water quality can be maintained without chemical pre-treatment. The nature as well as the quantity of the suspended matter in the raw water (or pre-treated water) will impact on the filter performance (AWWA, 1998; Kawamura, 1975a; Baumann, 1978 and Cleasby, 1990). Baumann indicated that as filtration rates increased over the last decades, more and more filter runs had to be terminated due to solids breakthrough since the higher rates pushed solids deeper into and through the bed. Thus research effects have shifted to improving water coagulation, flocculation and sedimentation.

Chemical pre-treatment usually involves the use of metal salts or cationic polymers as primary coagulants (Martin, 1998). Pre-treatment may also include oxidation of soluble forms of iron and manganese, or oxidation for the removal of algae and related taste and odours. The most common pre-treatment is alum flocculation and sedimentation with or without chlorination. For dual-media filters however, alum coagulation with short-time settling or without settling, and a small amount of polyelectrolyte addition just ahead of filtration are common (Kawamura, 1975a).

Assuming that adequate coagulation is feasible, the need for clarification needs to be assessed. The primary advantage of direct filtration (coagulation, flocculation and filtration only) is the elimination the capital and operating costs associated with clarification. Higher solids load on the filter will however shorten filter run times and increase the amount of product water required for filter washing. Sedimentation is recommended before rapid filtration when the turbidity exceeds 5 NTU (Kawamura, 1975a; AWWA Committee Report, 1980). The higher storage capacity of dual- and mixed-media filters has however made filtration of water with higher turbidities practicable. Cleasby et al. (1984) argued that direct filtration could be used successfully for low algae concentrations and raw water turbidities of up to 12 NTU using alum only or 16 NTU using cationic polymer only. For high algae concentrations these authors recommended turbidity limits of 7 NTU with alum and 11 NTU with polymer.

Typical alum doses for direct filtration are 5 to 10 mg/L. These are usually lower than the optimum dose determined by standard jar tests. This is because the goal in direct filtration is to produce small

pinpoint floc which are barely visible but filter effectively (Cleasby, 1990) whereas the goal in jar tests is to produce settlable floc. Kawamura (1975a) suggested that proper alum dosing can only be established by pilot-scale studies or actual filter studies with turbidity monitors.

Kawamura reports that reverse graded filters provide twice or even three times longer filter runs than are obtained from conventional filters, other conditions being equal. This was based on filter rates of 10 – 15 m/h (which iare 2-3 times faster than those of conventional filters), settled water turbidities of 3 – 6 NTU and very low doses of polyelectrolyte filter aid added just ahead of the filters.

2.3. Filtration rates, headloss development and filtrate quality

During the operation of a filter, the accumulation of solids in the media will result in progressive headloss development across the media. An increase in the rate of filtration will increase the solids loading on the filter and may result in more rapid headloss development. Depending on the media selection this will cause particles to be drawn deeper into the filter media and may result in premature particle breakthrough. Ideally a filter should reach the point of exhaustion of its solid's holding capacity at the instant that all the available head is used up.

2.3.1. Typical filtration rates and run times

Slow sand filters, designed for filtration rates of 0.1 to 0.2 m/h, were initially replaced by rapid sand filters that operated at rates of 2.4 to 5.0 m/h (Cleasby, 1990). The 5.0 m/h rate became widely accepted in the US as an upper limit for filtration. However, experience has shown that filters can be operated up to 25 m/h and in some cases higher without deterioration in filtrate quality provided there is adequate pretreatment (Kawamura, 2000). Filtration rates of up to 10 m/h appear reasonable with conventional pre-treatment and alum as the only coagulant. Higher rates require the addition of polymeric filter aids to maintain filtrate quality (Cleasby, 1990). Robeck *et al* (1964) compared the performance of pilot filters at 5 to 15 m/h filtering alum coagulated water through single medium and dual media filters. Pretreatment included the use of activated silica as a flocculant aid and a polyelectrolyte as a filter aid. It was concluded that with proper coagulation, the effluent turbidity and bacterial performance were as good at 15 m/h as at 10 or 5 m/h.

Haarhoff (1997), however, points out that a designer can only adopt a high filtration rate on the assurance of optimal pretreatment. In practice this will require a high level of skill, on-line monitoring and automated chemical dosing systems. In South Africa the inherent difficulties in meeting these requirements has led to conservative filtration rates of less than 10 m/h and filtration rates of around 5 m/h are commonly encountered.

Whilst the head loss increases at a faster rate per hour at higher filtration rates, the production volume per unit time also increases Cleasby (1990). Optimisation of filter operation depends therefore on the effect of filtration rate on filtrate quality, net production and costs. The unit filter run volume (UFRV) is the actual throughput of a filter during one filter run. Therefore depending on the volume of water required for backwashing, the net production of the plant can be calculated. Cleasby indicates that whilst allowing a backwash volume of 4 to 8 m^3/m^2 per wash the production will remain high provided the unit filter run volume exceeds 200 m^3/m^2. (i.e. 98% production efficiency). According to Kawamura (2001), 98 % efficiency or greater reflects good plant performance while less than 95 % water recovery indicates room for improvement, even in for direct filtration processes.

Kawamura further suggests that the goal for conventional water treatment should be to achieve a unit filter run volume of 600 m^3/m^2, but only 200 m^3/m^2 is expected for direct filtration. A unit filter run volume of 200 m^3/m^2 translates to a filter run time of 42 hours at 5 m/h. Kawamura (1975a) indicates that there are diminishing advantages in operating filters at rates higher than 25 m/h because of the frequency of filter washing, the quantity of backwash water required and the need for special supervision, filter aids and filter pre-conditioning (or both) and special consideration needed for the selection of media. The key issue therefore in considering the impact of high filtration rates on production is not the run length, but rather the gross production per run and the filtrate quality.

2.3.2. Headloss development

Most conventional type gravity filters are designed to allow 2 –3 m available head loss before filter washing. 3 m is common when a deep-water depth above the filter media surface is provided (Kawamura, 1975a; Montgomery, 1985). Certain other types of filters such as proprietary valveless (AVGF) or mono-valve package filters do not provide more than 1.8 m of available headloss.

Generally a 3 m headloss in gravity filters will permit a reasonably long filter run. Kawamura, however, believes that 1.8m of headloss is not necessarily inadequate, especially for reverse graded filters such as anthracite-sand dual-media filters. Kawamura reports that breakthrough quite often starts when the headloss reaches about 1.2 m, but the turbidity only exceeds 1 NTU after about 2 m headloss. The fact that early breakthrough is hardly noticed depends on strict pre-treatment control. In the absence of good control, in some instances breakthrough can occur with less than 1 m headloss. Kawamura points out that breakthrough can occur at any time regardless of headloss because of (1) poor coagulation control and (2) hydraulic shocks. It was concluded that both conventional and reverse graded dual-media filters have a tendency toward turbidity breakthrough

when headloss exceeds 1.3 m and many dual-media filtration plants set the terminal headloss in the vicinity of 1.5 m or within 48 hr of filter run, whichever comes first.

The rate of headloss development depends on the filtration rate and media characteristic as well as the characteristics of the floc. Cleasby & Baumann (1962) demonstrated that lower filtration rates can lead to an exponential rate of headloss development caused by partial capture of solids in a surface cake or by solids capture in a narrow layer at the top of the media (**Figure 2.2**).
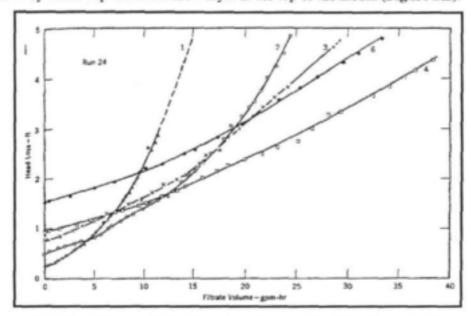
Figure 2.2   **Total head loss and filtrate volume for varying filtration rates in gpm/ft^2** (Cleasby, 1990)

Note: 1 gpm/ft^2 = 2.44 m/h; 1 gpm.h = 0.227 m^3

Increasing the filtration rate reduced the exponential tendency by encouraging greater penetration into the media and a greater production of filtered water was achieved for a given headloss.

2.4.   Filter backwashing

Periodic filter washing is required to remove filter deposits clogging the media and to avoid turbidity break through when the floc holding capacity of the filter is exceeded. Failure to clean a filter adequately can lead to the formation of mudballs, which will accumulate in the bed and result in media clogging (Cleasby, 1990). Clogged areas of the bed will tend to contract as the headloss increases. This shrinkage can cause cracks on the filter surface and at the filter walls, which may lead to short-circuiting of the media during filtration and a subsequent decline in filtered water quality. The most common backwash systems currently in use are discussed below.

2.4.1. Filter Backwash Options

2.4.1.1. Upflow water wash without auxilliary scour

This system relies on the expansion of the bed and drag forces on attached suspended particles to affect cleaning. It is now widely accepted that using water alone is a weak washing method that does not solve all dirty filter-related problems. Amirtharajah (1978b) argued that this is due to the lack of abrasion occurring between the grains in a fluidised bed.

Extensive experience in the USA has shown that high rate water wash alone is generally successful in filters which remove iron from groundwater or remove colour from otherwise high quality waters (Martin, 1998). However, it is generally inadequate for filters removing large amounts of suspended solids or using polymers. Kawamura (2001) states that water backwash without auxiliary wash cannot maintain filter beds in a reasonably clean condition for longer than several months if coagulants are used. Water only backwash without some form of auxiliary backwash is now seldom if ever employed in modern treatment plants (Haarhoff, 1997)

2.4.1.2. Upflow water wash with air scour

Air scour systems supply air to the full area of the filter from orifices under the filter medium (Cleasby, 1990). Air scour has been used alone (consecutive air and water wash) or together with low-rate water backwash in an unexpanded or slightly expanded bed (simultaneous air and water wash). Both types of air scour have been found to be very effective in preventing filter mudballing (Martin, 1998).

When consecutive air and water is used, most of the scouring occurs near the bed surface. Agitation deeper in the bed is only observed in the first minute or so while discrete air bubbles moves through the mixture of water and sand in the bed. Gradual displacement of water from the pore spaces results in compaction of the bed and the formation of fixed channels through which air travels directly to the bed surface (Haarhoff and Malan, 1983). As a result, the system is not very effective in cleaning the lower sections of the filter bed and there have been a few cases of significant mud accumulation in the lower regions of filters using air scour (Kawamura, 2001).

By contrast, simultaneous air and water flows in the correct ratios results in the formation and collapse of pockets of air throughout the filter bed, or "collapse pulsing" of the filter bed (Amirtharajah, 1993). This has been established as the most effective filter cleaning method as it produces the greatest amount of abrasion between the media grains throughout the depth of the bed. Air scour alone followed by low rate water wash is typically used in monomedia filters with 0.6 to 1.2 mm effective size media (Martin, 1998). Air scour alone followed by high rate water backwash

is used in dual and multimedia filters. Simultaneous air and water wash is usually reserved for deep bed coarse grained filters (effective size 1 – 2 mm).

2.4.1.3. Upflow water wash with surface/ sub-surface wash

Surface wash systems have been extensively used to improve the effectiveness of fluidised-bed backwashing in the USA (Haarhoff, 1997) but is little used elsewhere in the world (Haarhoff and Malan, 1983). As far as the authors of this report are aware, surface and sub-surface wash are rarely used in South Africa if at all.

Surface wash is a mechanism, which provides jets of water about 5cm above the fixed bed media surface to increase the agitation of the media during backwash and thus assist the release of attached particles (Cleasby, 1990). Fixed systems discharge auxiliary washwater from equally spaced nozzles in a pipe grid while rotary arms have pipe arms which swivel on a central bearing (Martin, 1998). Rotary systems provide a better cleaning action but are more likely to fail due to mechanical problems. Subsurface wash systems are sometimes used in dual or multimedia filters and have jets below the surface of the fixed bed. Dual arm rotary systems have one arm above and one below the media surface. Nozzle plugging can be a problem with sub-surface systems. Surface wash systems are relatively simple to install (as compare to air scour) and only require a source of pressurised water in conjunction with a set of nozzles. Maintenance is also relatively easy as the system is located above the media surface.

The effectiveness of surface wash has been found to be comparable to consecutive air and water wash (Cleasby *et al.*, 1977). Both systems may be ineffective in cleaning certain areas of the bed (Kawamura, 2001; Martin, 1998), especially if mudballs sink into the bed and away from the zone of maximum agitation.

2.4.2. Fluidised bed backwash

Filter backwash is a process in which the flow through the media is reversed to dislodge deposits accumulated during filtration. Although clogged filter beds behave somewhat differently to clean beds, analysis of filter backwashing is usually based on clean bed fluidisation phenomena.

At low backwash velocities, the grains in a clean bed remain undisturbed and the media remains at rest. As the velocity increases a point is reached where grains begin to move relative to each other, starting with the finest size (Kawamura, 1975b). A bed of non-uniform sized grains is completely fluidised when the drag of the washwater equals the buoyant weight of the largest grains (Gregory

and Zabel, 1990). This relates to the minimum fluidisation velocity of the bed described in **Section 2.1.1.**

Further increases in velocity result in the space between grains becomes greater, increasing the porosity and resulting in bed expansion. Ultimately the grains of media move independently and unhindered by other particles. This backwash velocity is approximately equal to the terminal settling velocity of the media grains in a free settling situation.

2.4.3. Backwash rates

In practice the backwashing rate must be less than the terminal settling velocity of the media to avoid loss of media. At the same time the backwash velocity should be higher than the settling velocity of the floc entrained in the media (Kawamura, 1975b). The terminal velocity of most suspended matter including alum floc ranges from 1.5 to 15 m/h. Kawamura therefore recommends a minimum backwash rate of 18 m/h.

Kawamura (1975b) developed a theoretical model of the rate of grain collisions or rubbing action between media grains which predicted that the maximum rubbing occurs when the backwash rate is 0.1 times the terminal settling velocity. This rate is called the appropriate backwash rate.

$$V_b = 0.1 \ U_t \qquad \text{..............................} \qquad (2.10)$$

V_b Optimum backwash velocity (m/min)

U_t Terminal velocity (m/min)

Where U_t is calculated by (Allen, 1909)

$$U_t = \left\{ 0.018 \frac{(\rho_s - \rho)^2 \cdot g^2}{\rho \, \mu} \right\}^{\frac{1}{2}} \cdot d \qquad \text{.........................} \qquad (2.11)$$

$\rho_s \, \rho$ Density of media and water respectively

G Gravitational acceleration constant 9.81 (m/s^2)

μ Water viscosity

D Particle diameter (mm)

The appropriate backwash rates at 20°C are therefore

- anthracite (s.g. = 1.55) $V_b = 0.47 \ d_{60}$
- sand (s.g. – 2.65) $V_b = d_{60}$

Where the backwash water is expected to be at a different temperature a correction is required:

$$V_b = V_{b,20^\circ C} \cdot \mu^{\frac{1}{3}}$$ .... (2.12)

μ Water viscosity (cP)

Figure 2.3 indicates the appropriate backwash rate for filter sand and anthracite coal media of various specific gravities and sizes at 20°C. **Figure 2.4** shows the appropriate backwash rate for regular filter sand, anthracite coal for various media sizes and water temperatures.

The theoretical basis of **Equation 2.10** has been questioned. Amirtharajah (1978b) contests that no grain abrasions occur in fluidized beds therefore they cannot play a role in detaching deposits. He showed experimentally that maximum detachment occurs under conditions of maximum fluid shear. This corresponded to expanded bed porosities of 0.7 – 0.75 for beds of mono-sized particles or similar porosities in the top few centimeters of a graded bed. In practice, both approaches predict similar optimum backwash velocities. Because of its convenience, **Equation 2.12** has been used to calculate optimum velocities in this investigation.
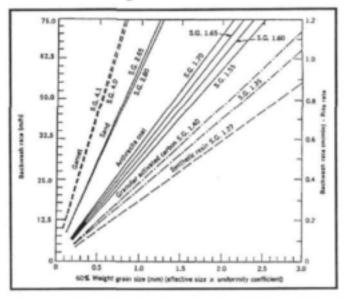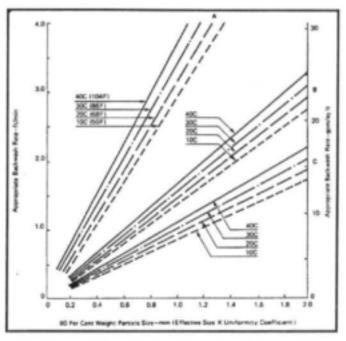
Figure 2.3   Appropriate filter backwash rates at water temperature 20°C

                             (Kawamura, 2000)

A - filter sand (sg = 2.60); B - anthracite (sg = 1.50)
C - spherical resin (sg = 1.27)

Figure 2.4   Appropriate filter backwash rates for filter media (Kawamura, 2000)

Note that washing a fine media at its optimal rate is not necessarily better than washing a coarser media at a non-optimal rate as the drag force and hence cleaning action is larger on coarser rather than finer particles.

2.4.4. Backwash volume

The optimum backwash volume depends on a number of factors including the strength and amount of filter deposits, the type, size and depth of the media, wash rates and use of auxiliary backwash, the distance between the expanded media and the backwash trough and the subsequent filter start-up strategy. Kawamura (2000) recommends termination of filter washing when the backwash turbidity drops below 10 to 15 NTU.

Design recommendations for backwash volume vary from 3-4 m³/m² (Amirtharajah, 1978a) to 6 m³/m² (Kawamura, 2001). The recommendation of 3-4 m³/m² is based on a trough height above the media of 0.91m. Larger spacing between troughs or a larger height above the filter media would increase the washwater requirements.

2.4.5. Criteria for initiating a backwash

Filters should be backwashed as soon as any of the following occur (Haarhoff, 1997)

21

1. terminal headloss is reached.
2. filtrate turbidity deteriorates beyond an acceptable limit.
3. filter run length exceeds an acceptable limit.

For the AVGF, backwash on terminal headloss is built into the design but the need to comply with the other criteria must also be taken into account.

2.4.6. Problems resulting from inadequate filter washing

Mudballing is a common filter problem associated with inadequate backwash. Mudballs are composed of filter grains and solids removed by the filter compacted together. Inadequate cleaning leaves a thin layer of compressible mud around each media grain (Cleasby, 1990). As the pressure drop in the filter increases during filtration, the coated grains are squeezed together, sometimes resulting in cracks in the bed surface, usually close to the walls. This can result in short circuiting of the treated water and deterioration of the filtrate quality (Martin, 1998).

Each time the filter is washed the particles of mud become more compact and build up in the filter bed forming greater masses (Baylis, 1954). According to Cleasby (1990), mudballs are formed when heavier deposits of solids near the surface of the sand breaks into pieces during backwashing. These can range in size from pea size to 2.5 – 5 cm or more. Baylis (1954) reported that mudballs when newly formed tend to remain at or near the surface of the bed after washing. Eventually they attain a specific gravity great enough to cause them to sink to the bottom of the sand bed during washing or to the sand – anthracite interface in dual media filters (Martin, 1998). The presence of the impermeable sub-surface regions increases the filtration rate through the remaining active portion of the filter bed with potential detriment to filtered water quality. They may also result in higher rates of headloss development (Cleasby, 1990).

Mudball problems are more like to occur when polymers are used as coagulants or filter aids which increase the attachment forces between filter grains and deposited floc (Cleasby, 1990). Finer media also appears to more prone to mudballing than coarser media. Kawamura (1975a) found that ten times as much mud accumulated in 0.45 mm effective size filters compared to 0.72 mm sand filters over a period of 4 years. The two sets of filters were operated under the same conditions and backwashed at the same rates over the study period. The sand grains encapsulated in mudballs found in the upper regions of the filters were found to be mostly much smaller than the effective size of the sand bed. Kawamura therefore suggested that the fine fraction of filter media forms the core of small mudballs which gradually grow into much larger masses if backwash is inadequate.

Air scour and surface wash can prevent the formation of mudballs, but previously formed mudballs have to be removed by hand or broken down by soaking in acidic solution (Martin, 1998).

2.4.7. Assessing the efficiency of backwash

Most plant operators evaluate filter performance based on the quality of the filtered water and filter run length under certain operating conditions. This method does not provide a complete picture of filter conditions (Kawamura, 2001).

Observation of the surface of the filter before, during and after backwashing will provide some clues about the state of the media (Kawamura, 2000; Chipps *et al.*, 2001). Lumps and cracks on the surface of the bed prior to backwashing indicate excessive mud accumulation and probable mudballing as do lumps, mudballs, worms and debris on the surface after backwashing (Kawamura, 2000). Mounding, concaved areas and shifts in the media can indicate backwash flow mal-distribution or scouring by the filter effluent. "Boils" and other indications of flow uneven washwater distribution may also be observed during backwashing.

Kawamura (2000) indicates that core sampling can provide substantially more information than visual inspection. The analysis should include a filter grain gradation profile and floc retention profile across the depth of the media. Since core sampling also measures the depth of the bed at coring points filter problems associated with loss of media or mounding of gravel (should a gravel underdrain be used) can be detected.

Chipps *et al.* (2001) recommended looking for trends in the clean bed headloss (normalised for flow and temperature) at the beginning of each filter run as an alternative method of monitoring backwash effectiveness. Changes in the normalised headloss over time may indicate media losses or problems developing within the filter bed, and should be further investigated.

Visual inspections and core sampling are difficult for many package plant type filters including pressure filters and AVGF's where the filter media is contained in a pressurized shell. Regular inspection may be possible, but often the media has to be completely removed and replaced with clean media if adverse plant operation is observed.

The efficiency of washwater usage can be assessed by measuring the backwash turbidity profile (backwash turbidity as a function of backwash time). A relatively low, flat backwash turbidity profile is indicative of ineffective backwashing while a high, sharp turbidity peak indicates is characteristic of effective washing (Kawamura, 2000). Kawamura recommends termination of backwash when the turbidity drops to 15 to 10 NTU as further washing will waste water and can

also negatively impact filtrate quality at the beginning of the next run by leaving the filter media too clean.

2.5. Rapid filters for small and developing communities

Most filtration plants in the developing world use sand as the filter media (Schulze and Okun, 1984). However, conversion to dual media can allow substantial increases in filtration rate can be achieved (10 to 15 m/h) without sacrificing run time or filtrate quality. Dual media filters traditionally use sand as the fine layer and anthracite coal as the coarse layer, however, other types of non-conventional filter media made from indigenous materials have been tested in various parts of the world. Unconventional coarse media including indigenous coals, crushed coconut shells, pea gravel, berry seeds and kernels from stone fruits such as apricots have all been found to be suitable for use in dual media filters.

The more efficient use of floc storage capacity in dual media filters is particularly advantageous when direct filtration is considered. The relatively low cost of direct filtration makes it an attractive option to conventional treatment in developing countries (Shulz and Okun, 1984). Capital costs are reduced by the elimination of of settling basin structures, sludge removal equipment and, sometimes, flocculation structures and equipment. There is also the potential for reduced chemical consumption, which results in lower operating costs. However, the application of direct filtration is limited to raw waters that are relatively low in colour and turbidity.

Washwater pumps are generally not recommended for small plants because of their high horsepower requirements and cost. Options for backwashing without a backwash pump include.

- An elevated backwash tank
- Taking backwash water from a high pressure distribution tank.

Fixed grid type of surface wash systems are recommended over air scour for developing countries because of their simplicity in design and lack of moving parts and lower energy requirements. Monk *et al.* (1984) reported the successful use of a fixed nozzle surface wash system in the Mohara Water Treatment Plant in Bangladesh. Auxiliary wash water was supplied from a high lift pump station force main.

2.6. Rapid filtration for iron and manganese removal

Iron and manganese exist in nature in both the insoluble (Fe^{+3} and Mn^{+4}) and soluble (Fe^{+2} and Mn^{+2}) oxidation states. The insoluble iron and manganese are readily removed by coagulation,

sedimentation and filtration. However, the removal of soluble iron and manganese poses a more serious problem.

In ground and surface waters supplies, soluble iron generally exists as divalent ferrous salt. Soluble manganese, with few exceptions, exists in the divalent manganous state. Groundwater treated for iron and manganese removal by oxidation, precipitation and filtration generally does not need other chemical pre-treatment (Cleasby, 1990). Iron and manganese also appear in surface water as organic complexes. These complexes are formed by the combination of iron and manganese ions surrounded by negatively charged organic units called ligands and commonly referred to as "organically bound" iron and manganese. An oxidation process is the most effective means of removing soluble iron and manganese. During oxidation, soluble divalent iron and soluble divalent manganese are converted to insoluble states.

These oxidation techniques are:
- *Air Oxidation/Aeration* - Aeration in some situations is satisfactory although it is a relatively slow process. When high concentrations of manganese are present, as organic complexes, air oxidation is generally ineffective.
- *Chlorine and Hypochlorites* – Chlorine and hypochlorites must first react with water to form hypochlorous acid. Soluble iron and manganese are more rapidly and completely oxidised by hypochlorite addition than by aeration. More than the stoichiometric amount of chlorine is required, which leads to the possibility of forming taste and odor problems. Hypochlorous acid does not efficiently oxidize manganese and organically bound iron and manganese.
- *pH* - Adjustment of pH is an effective means of iron and manganese removal. The pH of the water affects the rate of oxidation of Fe and Mn prior to coagulation. Iron is relatively easily oxidised under ambient pH conditions, but this oxidation is more rapid at a pH above 8.5. The oxidation of manganese is however relatively slow under ambient pH conditions. Aeration is often performed to increase the oxidation rate. This is only partially successful however and an elevated pH of >9.5 may be required to achieve full oxidation in reasonable time.
- *Potassium Permanganate* - Potassium permanganate may be used in either of two conventional systems. It is utilized in pressure systems as well as the more conventional gravity systems. The use of $KMnO_4$ for the removal of iron and manganese is very attractive because the reactions are complete, rapid, and require only a minimum quantity of chemicals.
    - Gravity Systems - The oxidation of both of these contaminants using $KMnO_4$ in gravity systems presents no real problem because the oxidative rates are fast. If the

demand is met, all of the ferrous iron and manganous manganese is oxidized and the resultant precipitates are coagulated and removed through settling and filtration.

  o Pressure Systems - The most popular type of pressure filter used in conjunction with potassium permanganate is the manganese zeolite or manganese-treated greensand filter.

- **Manganese Dioxide / Greensand** - Manganese dioxide affixed to zeolite or greensand is effective in removing soluble iron and manganese in either regenerative-batch or continuous processes. Manganese greensand is a mineral capable of exchanging electrons and thereby oxidizes iron and manganese to their insoluble and filterable states. The greensand has the ability to do both the oxidation and the filtration. However, its oxidative capacity is limited and eventually the bed must be regenerated.

  The above process can be time-consuming, however, and Davis and Fackler describe a more efficient process referred to as continuous regeneration. A solution of potassium permanganate is continuously fed into the raw water line ahead of the filter to reduce the amount of soluble iron and manganese going to the filter. The greensand than acts as a buffer. If the feed of $KMnO_4$ does not oxidize all of the soluble iron and manganese, the greensand will ozidize and filter these minerals. If the feed of $KMnO_4$ is in excess of the demand, the excess pink color is used up in regenerating the greensand. If an excess of $KMnO_4$ is continuously fed, the pink color will eventually break through.

The manganese-treated greensand is usually capped with anthracite to remove the bulk of the insolubles so as not to blind the greensand.

Some ground waters containing iron can be treated by pressure aeration and/or chemical oxidation and then filtered directly using pressure filters. This approach has received considerable application for small communities. Not all iron bearing water responds successfully to this method of treatment, and prior pilot-scale testing is usually required.

Mineral deposits can however develop on the grains of the filter medium, causing them to change in shape, size and density. Calcium carbonate deposits are common, as are iron and manganese deposits on the filter medium of iron and manganese removal plants. All these deposits can be subject to later leaching into the filtered water, if the pH is changed in a direction to increase solubility. The increased grain size due to such deposits results in an increase in bed depth and can impair filtration efficiency. More significantly, however, the increased grain size and density may impair the adequacy of backwashing. Thus cleaning the deposits from the media or replacing the media may become necessary.

2.7. Autonomous valveless filters/ Autonomous valveless gravity filters

Valveless filters are generally proprietary designs and there is a very little information about their design criteria in the literature. The designers and suppliers of AVGF's claim the following advantages:

- regular backwash of the sand filters on terminal headloss,
- backwash pumps and electronic control systems for filter control are not required,
- operator intervention is not required (to initiate or control backwash),
- the declining backwash rate results in good re-stratification of the media,
- turbidity breakthrough is unlikely due to the low headloss permitted, and
- after a backwash, the initial volume of poor quality filtrate goes to the backwash reservoir rather than to service (i.e. built-in filter to waste).

The major disadvantages of AVGF's are that backwashing of sand filters using water alone is a relatively weak cleaning process (as discussed in **Section 2.4.1.1**) and that, as with other rapid filters, coagulant is generally required to achieve filtered turbidities of < 1 NTU.

3. Investigation and methodology

3.1. Overview of the investigation

The project investigated the operation of a prototype autonomous valveless gravity filter. A full-scale filter was purchased by Umgeni Water and installed at the Umgeni Water Process Evaluation Facility in Durban. In order to fully evaluate its performance and simulate its operation under different operating conditions whilst using various combinations of media, pilot-scale filters were constructed and operated in parallel to the full-scale filter. This chapter describes the equipment used and the methodology followed in collecting data for the project.

After commissioning, problems with filter operation and poor filtered water quality were observed. As a result, the first phase of the project involved a complete overhaul of the filter, which included de-rusting and replacement of some of the filter nozzles after consultation with the local agents for the filter. The sand in the filter was replaced (with locally available sand) without paying particular attention to the backwash hydraulics and the correct media selection.

Operation of the filter continued for a period of 7 months, during which time the primary objective was to improve the filtered water turbidity and investigate the use of motive water regulated dosing pump for potential application with this type of filter.

It became apparent that further work was required to optimize the operation of the filter by performing parallel laboratory scale experiments with different media. The operation and control of the pilot-scale columns required an understanding of the hydraulics of the full-scale filter to be able to manually control and backwash the filters in a manner that reasonably represents the full-scale hydraulics.

This section reports on the modeling of the hydraulics of the AVGF, the filtration performance of the AVGF (including the optimization and operation with and without coagulant), and the experiments conducted on a pilot-plant scale. The performance of backwashing of the pilot filters was monitored and analysed by considering the formation of mudballs (by visual inspection of the pilot-scale filters) and by measuring the backwash turbidity.

```
                    ╱─────────────────────────────────────────╲
                   ◁      Commissiong and Trouble Shooting      ▷
                    ╲─────────────────────────────────────────╱

         ┌───────────────────────────────────────────────────────┬───┐
         │                                                        │ F │
         │        Commission AVGF and operate with polymer        │ U │
         │                          ↓                             │ L │
         │        Problems with filter media and mudballing       │ L │
         │                          ↓                             │   │
         │          Replace media with 0.8 mm sand                │ S │
         │                          ↓                             │ C │
         │    Filtration performance with and without coagulant   │ A │
         │                          ↓                             │ L │
         │       Observations of inadequate backwash rates        │ E │
         │                          ↓                             │   │
         │        Investigation into backwash hydraulics          │   │
         │                          ↓                             │   │
         │     Changed the AVGF Nozzles to suit hydraulics        │   │
         │                          ↓                             │   │
         │  Model backwash hydraulics for pilot filters simuation │   │
         └───────────────────────────────────────────────────────┴───┘
                                   ⇩

                    ╱─────────────────────────────────────────╲
                   ◁              Media Selection               ▷
                    ╲─────────────────────────────────────────╱

         ┌──────────────────────────────────────────────────────────┬─┬─┐
         │                                                          │P│S│
         │ Evaluate different media in pilot filters, high influent │I│C│
         │ turbidity and dose conditions.                           │L│A│
         │ Select dual media design for AVGF.                       │O│L│
         │                                                          │T│E│
         └──────────────────────────────────────────────────────────┴─┴─┘
                                   ⇩

              ╱─────────────────────────────────────────────────────╲
             ◁  Investigate impact of backwash limitations on long   ▷
             ◁                 term performance.                     ▷
              ╲─────────────────────────────────────────────────────╱

    ┌─────────────────────────────┬───┐   ┌──────────────────────────────┬───┐
    │ Observations of mudballing  │ P │   │                              │ F │
    │ at low turbidities,         │ I │   │                              │ U │
    │ inspection of media         │ L │   │    Dual Media for ABGF       │ L │
    │            ↓                │ O │   │            ↓                 │ L │
    │ Investigate effect of       │ T │   │                              │   │
    │ skimming media to remove    │   │   │ Inspection of media,         │ S │
    │ fines                       │ S │   │ evaluate long                │ C │
    │            ↓                │ C │   │ term performance             │ A │
    │ Increase depth of           │ A │   │ (15 months)                  │ L │
    │ hypothetical backwash tank  │ L │   │                              │ E │
    │            ↓                │ E │   │                              │   │
    │ Evaluate long term          │   │   │                              │   │
    │ performance (1 month)       │   │   │                              │   │
    └─────────────────────────────┴───┘   └──────────────────────────────┴───┘
                                   ⇩

              ╱─────────────────────────────────────────────────────╲
             ◁    RECOMMENDATIONS FOR DESIGN AND OPERATION            ▷
              ╲─────────────────────────────────────────────────────╱
```

Figure 3.1 Overview of the investigation

Finally, after specifying the size of dual media required for the AVGF, the full-scale and pilot-scale filters were operated in parallel under similar conditions for an extended period, to assess the suitability of the filter for operation without significant operator intervention. Data was analysed statistically to determine whether the filters' performance declined due to inefficient backwash and the accumulation of mudballs. Figure 3.1 gives an overview of the investigation.

3.2. Apparatus

3.2.1 Autonomous valveless gravity filter (AVGF)

The "Autonomous valveless gravity filter" (AVGF), has the potential to ensure more reliable operation with less operator intervention than many other types of filters currently installed or being considered for rural treatment schemes. The AVGF is unique in that backwash is initiated automatically without instrumentation, valves or electronic controls once a specific headloss across the media has been reached.

The design specifications of the AVGF evaluated in this investigation are summarized in Table 3.1.

Table 3.1 Design specifications of the Autonomous Valveless Filter evaluated in this investigation
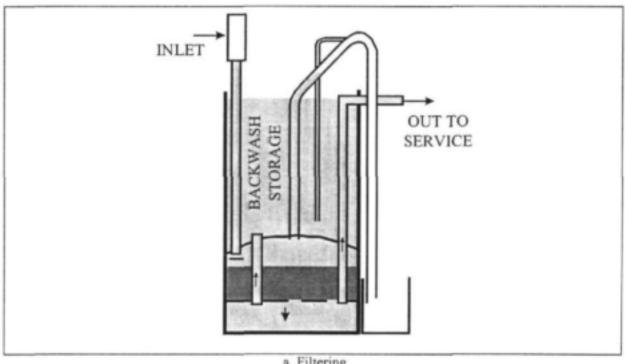
Dimensions	
Diameter	1.2 m
Height of filter compartment	1.3 m
Height of backwash tank	2.4 m
Total height of unit (from base to top of raw water reservoir)	6.4 m
Material of construction	Mild Steel
Capacity	10 m³/h
Filtration rate	10 m/h
Head availabe for filtration (including losses in pipes and nozzles)	~ 2.3 m
Terminal headloss	1.5 m
Filter media	
Design media*	0.7 mm silica sand

Design bed depth*	0.6 m
Backwash parameters	
Backwash volume	2.5 m^3 (2.2 m^3/m^2 filter area)
Backwash rates	45m/h (max) , 35m/h (Avg) , 25m/h (min)
Duration	3 min 45 sec
Head available for backwash (including losses in pipes and nozzles)	3.6 m
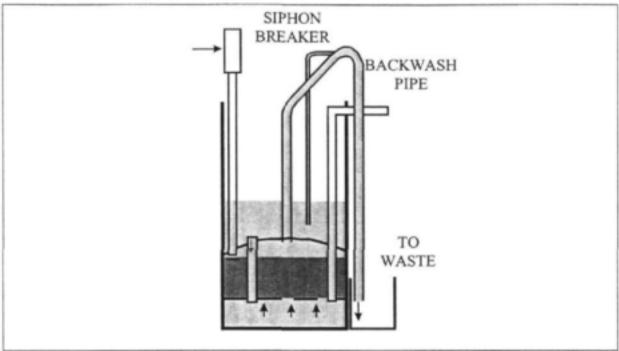
* These are the media specifications this filter was originally designed for. (Walker, 2000). See tables 3.2 – 3.5 for alternate designs evaluated at full scale in this study.

3.2.1.1. Operation

The raw water enters the filter-bed compartment just above the bed. It flows downward through the sand and filter nozzles into the collector chamber and upward through the effluent duct and out to service (**Figure 3.1a**).

a. Filtering

b. Backwashing

Figure 3.1   Operation of the AVGF

As the filter-bed collects dirt during the filter run, the headloss increases, and the water level slowly rises in the inlet pipe and in the backwash pipe. Just before the water reaches the top of the U bend, water flows through a subsidiary pipe and discharges through a venturi into the effluent tank. This creates a suction which evacuates the air from the large downward leg of the backwash pipe. This action draws the water over the U bend., initiating backwash.

The backwash pipe carries approximately four times as much water as the inlet pipe. The siphoning effect causes the water to be drawn from the backwash storage chamber down through the ducts into the collection chamber, from whence it rises upwards through the strainers or nozzles, the sand bed, and the filter compartment into the backwash pipe.. (**Figure 3.1b**). The backwash action continues until the level of the water in the backwash storage space drops below the end of the siphon breaker. This admits air to the top of the backwash pipe, and backwashing stops.

The inlet water resumes its downward gravity flow automatically, flushing some of the remaining detached floc out of the media and into the underdrain.. Consequently, the filtrate quality directly after backwashing is typically poor compared to the rest of the run and may not meet finished water turbidity standards (See **Section 2.3.3**). In the AVGF, the filtrate cannot go out to service until the backwash tank is full which means that the initial volume of poor quality water is diverted to and stored in the backwash tank for use in the next backwash. Hence, a filter-to-waste period equivalent to the volume of the backwash tank is inherent in the design of the AVGF.

Backwash can also be initiated manually by running water directly from the backwash tank into the venturi. The suction generated evacuates the air from the backwash pipe to establish the siphon between the filter compartment and the effluent tank.

3.2.1.2. Modifications for viewing backwashing

A window was installed in the side of filter bed so that the state of the filter bed could be viewed during backwashing. Unfortunately, the window created a dead-zone in which severe mudballing occurred, so that the view through the window was not representative of the rest of the filter bed. However, the height of the bed and the point at which it began to expand during backwashing could still be viewed.

A calibrated sight glass on the backwash tank allowed the measurement of the rate at which the level drops during backwash. These measurements were used to calculate the backwash rates (**Section 3.5**).

3.2.1.3. Filter nozzles

Upon investigation, it was found that the AVGF had only 22 nozzles, (19,5 nozzles per m²). Umgeni Water typically uses ≥ 50 nozzles per m² in its designs as poor flow distribution and dead zones between nozzles may promote mudballing. The original nozzles used in the filter were quite large (diameter = 105 mm) which would have to some extent compensated for the low number density (Larkin, 2000). However, in the initial phase of the project, the original nozzles (which are apparently no longer produced) were replaced with smaller (diameter = 60 mm) disc type strainers supplied by a local supplier as "AGF nozzles".

The disc type strainers each consisted of layers of plastic lamellae with radial ridges less than 1 mm in height on their surfaces. These ridges created tiny flow channels into the center of the nozzle head. Unfortunately, this strainer design appeared to be particularly susceptible to clogging by media grains which would get lodged between the lamellae. Furthermore, smaller nozzle head probably resulted in a more uneven flow distribution and the headloss across the strainers was significantly higher than the original nozzles, which resulted in a decrease in backwash rates (See **Section 4.2**).

Based on recommendations from a local supplier of filtration equipment (Larkin, 2000) it was decided to drill extra holes in the floor of the filter to increase the number of nozzles to 39 (34,5 per m²). The number of holes which could be added while maintaining an even distribution was limited

by the layout of the existing nozzles. Standard KSH type C2 nozzles with 0.2 mm slots were selected to replace the strainers after their compatibility to the media and other criteria was determined. These included minimum sand size to be used, susceptibility to clogging and headloss across the underdrain to match the original nozzles (taking into account the increase in number of nozzles). The C2 nozzles were standard filter nozzles with hollow plastic heads that allowed water flow via 0.2 mm wide vertical slots.

3.2.2 Pilot filters

Four PVC pilot filters (200 mm diameter) were used to simulate filtration and backwashing of the AVGF for various media designs and for an increase in the height and volume of the backwash tank. The filter media was supported on a PVC orifice plate drilled with 5 mm holes and covered by a wire mesh to prevent sand passing through the orifices. A scale marked on the front of each column was used to estimate bed height (in mm). Each column was fitted with nine manometer ports for measuring the headloss profile across the filter bed and a sampling point above the filter bed surface where samples were collected for coagulated water turbidity analysis. The manometer board scale was marked in intervals of 2 mm (**Figure 3.2**). As the filter clogs up during filtration, the water level above the filter surface rose to increase the driving force for filtration. Headloss development was monitored using the manometer tubes.
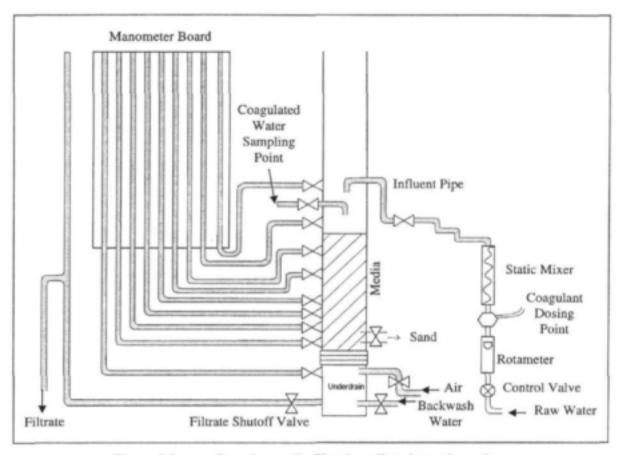
Figure 3.2   Granular media filtration pilot plant schematic

The influent line to each filter included a coagulant dosing point with dosing pump, static mixer, rotameter and diaphragm valve for flow control. The filters shared a common raw water pump supplying Wiggins raw water from the water storage tank. Using a recycle line, a fraction of the flow from the pump discharge returned it to the tank. Samples for raw water quality were then collected at the discharge of the recycle line.

A separate pump provided domestic water for backwashing. The backwash flow was regulated by the control valve and measured using the backwash rotameter. The AVGF backwash rates were simulated by manually varying the flowrates according to a pre-determined schedule. The clear perspex walls of the filter columns allowed direct observation of filter backwash. A compressor was used for air supply for air scour and collapse pulsing backwash when required.

35

3.3. Filter media

During the course of the project different media configurations were tested in the pilot- and full-scale filters. The relevant media properties and designs are presented in this section.

3.3.1. Media Properties

The silica sand used in the laboratory and full-scale experiments and the anthracite used in the dual media trials were obtained from B.&E. Silica (Brits). The anthracite used was purchased from LPM Chemicals. While it was possible to order various different size ranges of silica sand, anthracite could be obtained in only one size range and LPM was unable to supply the size distribution data. Size analysis of the anthracite revealed that the d_{90} of the anthracite was too large for use with 0,5 mm sand. However, removing the size fraction greater than 1.7 mm would bring the size ratio of the two media within the recommended limit (See **Section 3.3.2**). The project team was unable to identify a local company willing to separate / screen a small sample (mass < 1000 kg) of anthracite, therefore it had to be sieved manually. The anthracite was sieved by hand using a 1 m^2 stainless steel mesh in a wooden frame. This took several months to complete and in the interim, a number of filtration experiments were carried out using the unsieved anthracite. Despite the coarser particle size, intermixing did not appear to be a problem in the limited laboratory trials carried out with unsieved anthracite.

Samples of media used in the laboratory filters and the AVGF were characterised in terms of size distribution, density, surface area sphericity, minimum fluidisation velocity and fluidised bed expansion. The terminal velocity and Kawamura's optimum backwash velocity based on the size and density of the media (**Figure 2.4**) were also calculated for each sample. The results are summarised in **Table 3.2**.

3.3.1.1 Size distribution

The size distribution of each of the effective sand sizes investigated (0,5, 0,7 and 0,8 mm) and the anthracite were determined by sieve analysis using a mechanical shaker. Representative sampling techniques were used and multiple determinations were made for each sand size. Because of the narrow size distributions of the media samples, $\sqrt{2}$ sieve series did not provide a good resolution of the size distribution, therefore $\sqrt[4]{2}$ series were selected. The mass of sand in each size fraction was summed for replicate determinations and the totals in each fraction used to determine the average size distribution. The d_{10}, d_{60}, d_{90} and U.C. of each effective size were calculated from its average cumulative distribution. The size distribution results are presented in **Figure 3.3**.
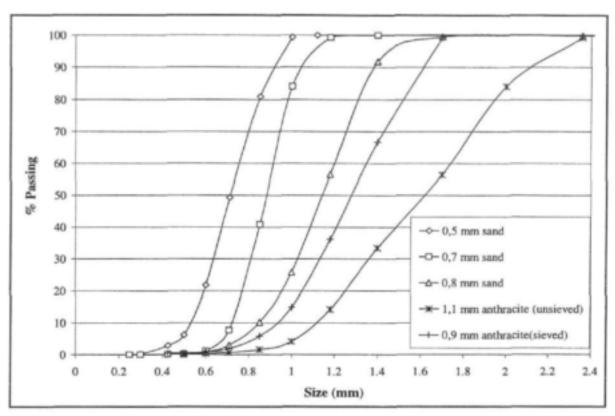
Figure 3.3   Media Size Distributions

3.3.1.2 Density

Samples of sand and anthracite were rinsed several times in deionised water to remove dust and clay and then dried overnight at 110 °C. Samples were stored in a dessicator while cooling. 200 mL volumetric flasks were weighed to 1/100th of 1 g and then half filled with deionised water. Weighed samples of media (~ 50 g for anthracite and ~ 100 g for sand) were transferred to the volumetric flasks which were then sealed and shaken to eliminate air bubbles. Dry anthracite has a very high affinity for air bubbles therefore the samples were left to stand overnight before proceeding. Once the air bubbles had been eliminated, the flasks were filled to 200 mL and the final mass of the flask, water and media and the temperature were determined. The density of the media could then be determined by calculating the volumes of the water and media in the flask.

3.3.1.3 Surface area sphericity

Surface area sphericities of the various media samples calculated from clean bed headloss measurements using **Equation 2.1**

3.3.1.4 Minimum fluidisation velocity and fluidised bed expansion

The minimum fluidisation velocity for each size and type of media was determined experimentally by measuring the steady state clean bed headloss as a function of backwash velocity. The minimum fluidisation velocity is defined to be the velocity at which the sub-fluidised bed headloss (linearly

dependent on backwash rate) and fluidized bed headloss (constant) intersect as shown in **Figure 3.4**
The expanded bed heights were also measured as a function of backwash rate. Quadratic regression
lines fit to the fluidized bed height data were used to estimate bed heights during backwashing in
Section 4.3.2. The minimum fluidisation velocity was also calculated using **Equation 2.4** and **2.6** as
comparisons. **Equation 2.6** tended to agree better with the measured minimum fluidisation
velocities than **Equation 2.4.**

The graph contains the following annotations:

$$y = -5.23E\text{-}05x^2 + 6.90E\text{-}03x + 3.52E\text{-}01$$

$$y = 0.0122x + 0.0087 \qquad R^2 = 0.9893$$

$$y = 9E\text{-}05x + 0.4185 \qquad R^2 = 0.0369$$

Legend:
- ○ Fluidized
- □ Sub-fluidized
- × Fluidised bed height
- ✕ Sub-fluidised bd height

Axis labels: Headloss, Bed Height (m) (vertical); V (m/h) (horizontal)
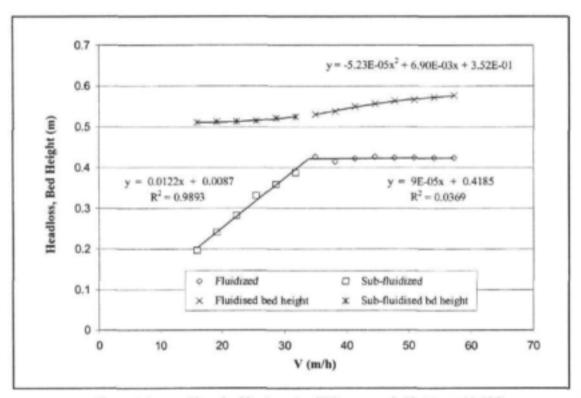
Figure 3.4   Clean bed backwash of 0,7 mm sand, 22,4 kg at 23,5°C

3.3.1.5 Terminal velocity U_t and Optimum backwash rate, V_b

The terminal velocities and optimum backwash rates at 20 °C for each media size and density were
estimated using **Equations 2.9** and **2.10**.

Table 3.2   Media Characteristics

	Sand			Anthracite	
	0,5 mm	0,7 mm	0,8 mm	0,9 mm (sieved)	1,1 mm (unsieved)
D_{10} (mm)	0,53 mm	0,72 mm	0,85 mm	0,91 mm	1,11 mm
D_{60} (mm)	0,75 mm	0,91 mm	1,2 mm	1,35 mm	1,73 mm
D_{90} (mm)	0,91 mm	1,04 mm	1,38 mm	1,60 mm	2,10 mm
U.C. (Eqn 2.2)	1,42 mm	1,36	1,40	1,48	1,56
Density, ρ_s, ρ_c	2642 kg/m³	2642 kg/m³	2642 kg/m³	1545 kg/m³	1474 kg/m³
Sphericity, ψ	0,90	0,86	0,76		0,67
Minimum fluidisation velocity, V_{mf}	22, 8 m/h at 24 °C	33,6 m/h at 23,5 °C	49,5 m/h at 24 °C		31,6 m/h at 25 °C
V_{mf} (Eqn 2.4)	28,5 m/h	35,3 m/h	54,7 m/h	25,5 m/h	37,0 m/h
V_{mf} (Eqn 2.6)	22,6 m/h	31,8 m/h	53,2 m/h	21,4 m/h at 20 °C	32,9 m/h
Terminal velocity at 20 °C, U_t (Eqn 2.9)	451 m/h	547 m/h	721 m/h	388 m/h	454 m/h
Optimum backwash rate, V_b (Eqn 2.8)	45 m/h	55 m/h	72 m/h	39 m/h	45 m/h

3.3.2. Media specification

3.3.2.1 Monomedia filters

The AVGF under evaluation at the PEF was designed to operate with a 0,6 m filter bed of 0,7 mm silica sand (Walker, private communication, 2000). It appears that the filter did contain 0,7 mm sand when it arrived at the PEF but the media was subsequently replaced with 0,8 – 0,9 mm sand which is the same media used in the filters at the Umgeni Water Wiggins Treatment Works and was therefore readily available on site. It was discovered later that the use of the coarser media was incompatible with the available backwash rates in the AVGF.

The first laboratory scale experiments were conducted using 22,5 kg of 0,8 mm sand. When the sand was initially loaded into the filters, the bed height was approximately 0,6 m, but after several backwashes it settled down to about 0,54 m. Since the media headlosses during backwashing and hence the backwash rates in the AVGF (Section 4.1) are a function of the weight of the media, it was decided to use the same mass of media for the laboratory trials with smaller sand. In the last set of laboratory trials (long term filter performance), a further 5 kg of sand was added to the 0,7 mm sand filter to give a total media mass equivalent to 1 metric ton of media in the AVGF. The media

designs for the mono media filters are summarised in **Table 3.3**. The exact mass of 0,8 to 0,9 mm sand in the AVGF was not recorded but is was assumed to be between 800 and 1000 kg.

Table 3.3   Monomedia Filter Bed Specification

	Pilot-scale filters				AVGF
	0,5 mm sand	0,7 mm sand	0,7 mm sand with sand added	0,8 mm sand	0,8 - 0,9 mm sand
Mass of media	22,5 kg	22,5 kg	27,5 kg	22,5 kg	800 – 1000 kg
Bed height	0,51 m	0,52 m	0,64 m	0,54 m	~0,6 m
Minimum fluidisation velocity, Vmf (20 °C)	21 m/h	31 m/h	31 m/h	53 m/h	53 m/h*
Fluidised bed headloss, Δh_{media}	0,42 m	0,42 m	0,51 m	0,51 m	0,46 m (assumed)
Contactor area, ΣA (**Equ 2.6**) m^2/m^2	2405	2065	2523	1561	
l/d$_e$ (**Section 2.1.2**)	962	722	882	635	

Minimum fluidisation velocities were corrected for temperature using **Equation 2.6** Calculated contactor areas and bed depth to effect size ratios indicate that the filter beds were somewhat shorter than recommended practice (**Section 2**) especially for direct filtration.

3.3.2.2 Dual Media

Initial laboratory scale trials (DM#1 and DM#2) utilised 0,5 mm sand and unsieved anthracite as received from the suppliers. Media size analysis revealed that the d_{90} of the anthracite was 4 times the d_{10} of the sand compared to the recommended ratio of 3. Consequently, all the anthracite used in subsequent laboratory scale experiments and in the AVGF itself was screened to remove the fraction greater than 1,7 mm. The calculations for compatibility of the sand and anthracite are summarised in **Table 3.4**

Table 3.4   Compatibility of sand and anthracite.

Media	0,5 mm sand 1,1 mm anthracite	0,5 mm sand 0,91 mm anthracite
d_{c90}/d_{s10}	4,0	3,0
$\left(\dfrac{\psi_s}{\psi_c}\right)\left[\dfrac{\rho_s-\rho}{\rho_c-\rho}\right]^{0.52}$ (**Eqn 2.8**)	2,6	2,4

The filter bed design for the dual media filters investigated in this study are summarised in **Table 3.5**.

Notes:

- The media depths for DM#1 were selected based on the recommendations of Dr Isias, a granular media filtration expert for the Saudi Arabia Oil Company who visited the PEF in 1999 (personal communication).

- DM#2 was the original 0,5 mm monomedia bed with a layer of anthracite added.

- DM#3 was the laboratory scale version of the full-scale AVGF. It was originally intended to initially operate DM#3 and the AVGF as described in **Table 3.4** and later add 0,1 m of sieved anthracite to both. However, since the filters performed satisfactorily with the shorter beds, they were left as they were.

- DM#4 was the same design as DM#3 but both the sand and anthracite layers were initially skimmed to remove fines. (See **Section 4.3.1**) The effect of skimming on the media size distribution was not determined.

- When anthracite is added to a filter, it should be left to soak for at least 24 hrs before backwashing. Should the filter be backwashed immediately after media installation, entrained air in the anthracite will result in loss of media.

Table 3.5   Dual Media Filter Bed Design

		Pilot-scale filters				AVGF
		DM#1	DM#2	DM#3	DM#4	Dual Media
Media sizes	Sand	0,5 mm			0,5 mm	
	Anthracite	1,1 mm			0,9 mm	
Media mass	Sand	17,5 kg	22,5 kg	17 kg	17 kg	600 kg
	Anthracite	5,1 kg	1,9 kg	3,1 kg	3,1 kg	110 kg
Bed height	Sand	0,39 m	0,51 m		0,41 m	
	Anthracite	0,23 m	0,08 m		0,12 m	
Minimum fluidisation velocity, Vmf (20°C)						
Fluidised bed headloss, Δh_{media}		0,37 m	0,46 m		0,34 m	
Contactor area, ΣA (**Eqn 2.7**) m²/m²		2359	2597		2204	
l/d_c (**Section 2.1.2**)		943	1034		905	

3.4. Modelling the AVGF Backwash Hydraulics

After the AVGF had been operating for some time with 0,8 mm sand and alum coagulation, it was found that the backwash rates had dropped below the design rates. It was also established that the design rates were in any case too low to fluidise 0,8 mm media and that the media had to be replaced. Different media sizes and designs were to be evaluated in the pilot filters. However, media headlosses play a role in determining backwash rates and the effect on the backwash of changing the media was unknown. Therefore the backwash headloss characteristics of the various filter components were determined and used to construct an unsteady state numerical model which can be used to predict the effect of various design changes on the backwash rates. Details are documented in the **Appendix 1**.

4. Results and Discussion

4.1. Preliminary Results At Full Scale

4.1.1. Turbidity removal from Wiggins raw water

After the AVGF had been commissioned at the PEF, it was initially operated with polymeric coagulant of a blended polymeric coagulant (di-alyl,di-methyl ammonium chloride and poly-aluminium chloride). Filtrate turbidities of < 1 NTU were consistently achieved but within 3 months, the media was severely mudballed and had to be replaced.

Polymeric organic coagulants tend to result in stronger interfacial adhesion forces than inorganic metallic coagulants (alum and ferric) and are therefore more likely to result in rapid headloss development and mudballing especially in the absence of auxiliary backwash (Martin, 1998). The results of this study confirm that polymers are unsuitable for use as primary coagulants for filtration with water only backwash.

After the AVGF was started up again with 600 mm of 0,8 –0,9 mm effective size sand, it was initially operated at 7 – 7,5 m³/h without any coagulant. Turbidity removal was poor with filtrate turbidities of < 1 NTU seldom being observed (**Figure 4.1**). During this period the backwash flowrates were measured regularly and it was noted that the bed expansion appeared to be less than 10% during backwashing. This contrasts to recommendations of at least 25% bed expansion for effective backwashing with water only (Amirtharajah, 1978b).
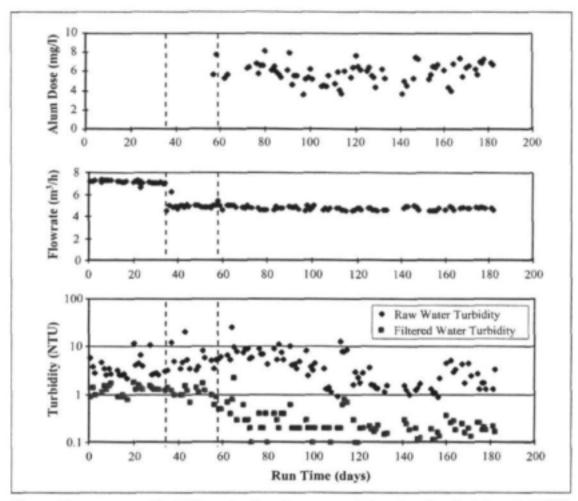
Figure 4.1   **Turbidity reduction, flowrate and alum dose for the full-scale AVGF**

The next phase of the project was to look at the effect of using alum as a coagulant. A *Dosatron* hydraulically powered dosing pump was installed to operate in-line with the AVGF (Discussed further in **Appendix 2**). When the *Dosatron* was initially installed, the increased headloss in the feed line meant that feed rate had to be reduced to 4,5 m³/h to prevent the raw water header tank overflowing. The filter was operated for a period at the lower flowrate without coagulant addition to look at the effect of reducing the flowrate on particle removal.

The filtrate turbidity did not improve, however (**Figure 4.1**).

In direct filtration, the aim is to produce small filterable floc rather than large settlable floc so that the alum doses used are typically smaller than those used for conventional treatment. Cleasby (1990) documents direct filtration studies where typical doses of 2 – 10 mg/l of alum were used. A dose of 5 mg/l as alum was selected as an appropriate dose. The dosing system was set to produce a dose between 4 and 8 mg/L, however, measured doses varied up to 30 % from the set dose (See **Appendix 2** for details). The dose was not varied in response to changes in raw water turbidity.

During alum dosing the turbidity removal of the filter immediately improved and turbidities of 0.5 NTU were consistently obtained. The raw water turbidity ranged from 1.5 NTU to 10 NTU during this period. The dose of 5 mg/L would probably have not have been adequate to achieve < 1 NTU filtrate turbidity at significantly higher raw water turbidities.

4.1.2. Backwashing frequency and cleaning efficiency

A level sensor was initially installed in the dirty water outlet tank, to detect the increase in the level in the tank during backwashing. Subsequently a pressure transducer was installed above the filter bed to measure the head loss development during a filtration cycle. These measurements were recorded on a weekly chart recorder.

Figure 4.2 shows the length of the filter run and raw water turbidity. During the first 60 days operation the flowrates through the filter were higher showing on average a shorter filter run of approx. 20 hours. Once the flowrate had been reduced to allow for chemical dosing, the average length of the filter run increased to between 20 and 60 hours. **Figure 4.2** indicates that as the turbidity of the raw water decreased, the run length increased. Similarly an increase in turbidity appeared to shorten the filtration length and cause more frequent backwashing. This can be attributed to a slower or more rapid accumulation of deposited particles in the filter media at lower or higher influent turbidities respectively.
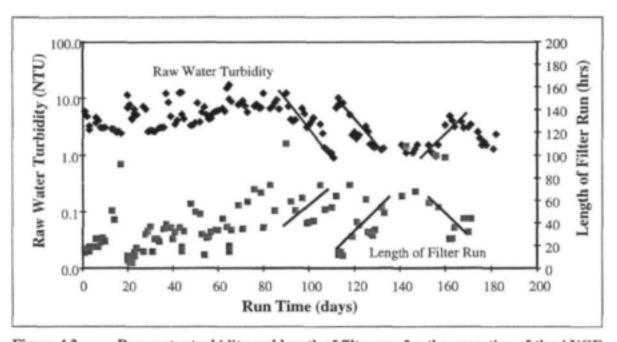

Figure 4.2   Raw water turbidity and length of filter run for the operation of the AVGF

45

Figure 4.1 shows however that there was significant variation in coagulant dose (discussed in **Appendix 2**). At certain times (especially over the weekends) the alum solution in the chemical tank was found to run dry, resulting in longer run times. This will have contributed to a variation in the observed filter duration.

4.2. Autonomous valveless gravity filter hydraulics

The initiation, intensity and duration of backwashing in the AVGF is completely dependent on its hydraulic design. Failure or improper specification of any of the hydraulic components will severely compromise the filter operation. This section presents the results of a detailed investigation into the filter hydraulics with emphasis on backwash. The results obtained provided a number of important insights into the operation of the filter and its limitations. These have led to a number of recommendations for the design and operation of AVGF's. (**Chapters 5** and **6**). The hydraulic model developed in the course of the investigation has also made it possible to predict the effect of various design changes on the filter operation.

4.2.1. Hydraulic model of backwash in the AVGF

The characterization of headlosses in the various components of the AVGF and the development of a model to predict the backwash velocity profiles were described in **Section 3.5** and **Appendix 1**. The model was developed primarily to:

- Assess the effect of various components of the filter on the backwash rates (Results presented in **Chapter 5**).
- Predict the effect of various design changes on backwash rates in order to simulate them at laboratory scale.
- Determine why the filter was not achieving the design backwash rates after 7 months of operation with 0.8 – 0.9 mm sand.
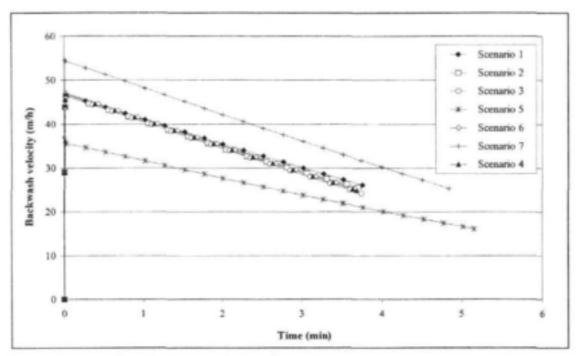
46

Figure 4:3   *Backwash velocity predictions for various scenarios*

Figure 4.3 shows the model results for different masses of media in the pilot filters and the AVGF. Each of the modeled scenarios is listed in **Table 4.1**. Each of the backwash rate profiles shown were simulated in the pilot filters during the investigation, with the exception of Scenario 5 (for this scenario, the measured rates were used instead of the predicted rates – See **Figure 4.3**). Scenario 5 attempts to simulate the situation in the AVGF before the old media and the strainers were removed. Scenario 6, is the actual dual media design used in the AVGF and Scenario 7 simulates the effect of increasing the backwash tank depth by 1 m for the same media. Descriptions of the C2 nozzles and disc type strainers were provided in **Section 3.2.1.3**.

Table 4.1   **Design parameters for modeling backwash velocities**

	Media	Mass	Nozzles	Feed Rate	Backwash Volume
Scenario 1	0.8 mm sand	814 kg,	39, C2 type	7.5 m³/h	2.2 m³/m²
Scenario 2	0.7 mm sand	814 kg	39, C2 type	7.5 m³/h	2.2 m³/m²
Scenario 3	0.5 mm sand	814 kg	39, C2 type	7.5 m³/h	2.2 m³/m²
Scenario 4	Anthracite 0.5 mm sand	180 kg 612 kg	39, C2 type	7.5 m³/h	2.2 m³/m²

47

Scenario 5	0.8 sand	1000 kg	22 clogged disc type	4.5 m³/h	2.2 m³/m²
Scenario 6	Anthracite 0.5 mm sand	160 kg 110 kg	39, C2 type	7.5 m³/h	2.2 m³/m²
Scenario 7	Anthracite 0.5 mm sand	160 kg 110 kg	39, C2 type	7.5 m³/h	3.2 m³/m²

The model results for scenarios 1- 4 and 6 agree reasonably well with the design backwash rates for the AVGF (45 m/h maximum, 35 m/h average, 3,5 min duration). The variation in bed composition and media headlosses does not in fact have much impact on the backwash rates. However, the model predicted a substantial decrease in backwash rates when clogged disc type strainers were. Increasing the backwash tank height increased the rate and duration of backwash (Discussed further in **Chapter 5**).

Direct measurements of the backwash rates and headlosses in the AVGF with dual media and C2 type nozzles agreed well with the model predictions once the maximum backwash rate was achieved (**Figure 4.4**) but the measured rates were lower than the predicted rates for the 0.8 mm sand and clogged disc type strainers (**Figure 4.5**). Possible explanations include:

1. The headloss characteristic for clogged disc type strainers was determined from a single nozzle removed from the filter (See **Appendix 1**). The average headlosses across strainers in the filter may have been higher than measured. Removing and testing the strainer may also have dislodged some of the material originally clogging it.

2. Sludge, algae, rust or other debris may have resulted in blockages and constrictions in the various pipes in the AVGF, which would have increased the empty filter headlosses.
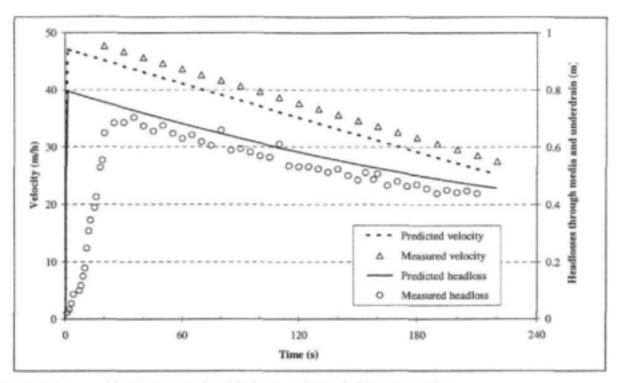
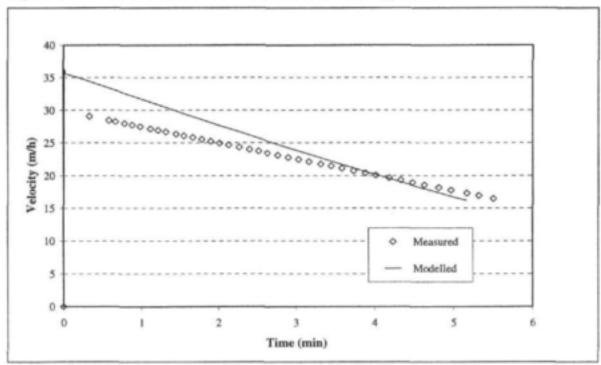**Figure 4.4   AVGF backwash with dual media and C2 type nozzles**

Figure 4.5 AVGF backwash with 0.8 mm sand and clogged disc type strainers

The backwash rates had not decreased after 7 months of operation with dual media and the new nozzles. This tends to endorse the choice of the C2 type nozzles. The choice of dual media may also have helped to prevent clogging of the nozzles by restricting mudballing to the anthracite layer (See **Section 4.4.1**).

4.2.2. Initiation of backwash

The main discrepancy between all the model results and the observed behaviour was the time required to achieve the maximum backwash rate. According to the model, this should have occured within seconds. However, measured velocity profiles based on changes in the backwash tank level indicated that it took at least 20 s for the maximum velocity to be reached.

The headloss required to accelerate the fluid in the filter from zero to its maximum backwash rate was found to be very small (**Appendix 1**). The backwash rate should therefore have peaked almost immediately after water begins to flow from the backwash reservoir. This is based on the assumption that all air was purged from the backwash effluent pipe before the level in the backwash tank begins to drop, i.e. the siphon is initiated using the incoming feed water only. This, however, is not necessarily entirely true. Air pockets in the backwash pipe will reduce the flow area available to water, resulting in higher headlosses than have been calculated. Therefore only once the air has been evacuated will the maximum velocity be obtained.

The level in the backwash reservoir may also decrease slowly due to the decrease in filtrate flow while the siphon is still being generated and before the flow reverses. This will result in a slower start to the backwash and reduce the maximum backwash rate achieved.

The headloss across the underdrain and the media were measured during normal filtration using a differential pressure probe. The profile of headloss before, during and after a backwash is shown in **Figure 4.6**. It is apparent that there was a period of gradual headloss decrease before the flow through the media reverses and any form of backwashing can take place. **Figure 4.6** was subdivided into six regions for ease of analysis:

a) High headloss at the end of the filtration cycle before backwashing.
b) Filtration rate drops as feed water flows through priming system.
c) More rapid reduction in filtration as water starts flowing though backwash pipe.
d) Upflow through media begins during final generation of siphon.
e) Initial high backwash rate, reducing as the level in reservoir drops.
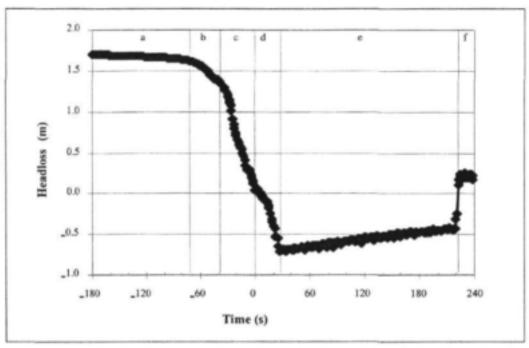f) Start of following filtration cycle

Figure 4.6   Headloss across the media and underdrain during backwashing

4.3.   Filtration performance

4.3.1.   Effect of media design on filtration performance

Pilot plant studies using the backwash velocity profile measured in the AVGF (**Figure 4.3**) showed that the observed velocities were totally inadequate to achieve fluidisation of the 0.8mm filter media used in the AVGF, resulting in rapid deterioration of the media. It was therefore decided to investigate the use of finer sand and/or dual media (sand and anthracite). Three different sand sizes (0.8, 0.7 and 0.5 mm) and two dual media configuration have been investigated. The relevant media properties were presented in **Tables 3.2 , 3.3** and **3.4.**

Data obtained in the pilot filters for different media designs, raw water conditions and coagulant doses has been evaluated and compared on the basis of the following:

- Total volume filtered per filter run
- Filtered water turbidity
- Rate of head loss development
- Head loss at breakthrough
- An estimation of the bed depth required bed depth to prevent turbidity breakthrough

51

Tables **4.2** to **4.4** summarise the purpose and results of several filter runs at varying coagulated water turbidities and alum doses. Dual media was selected for use in the AVGF on the basis of the Series 1 to 3 experiments in the pilot filters. Thereafter, the AVGF and pilot filters were operated in parallel.

Table 4.2. Summary of Filtration Experiments

	Aim	Filters
Series 1 - 3	Compare performance of different media. Selection of media for AVGF	0.5 mm sand 0.7 mm sand 0.8 mm sand DM#1 DM#2
Series 4	Compare performance of full scale and pilot filters Attempt to reduce mudballing in the pilot filter by limiting run time to 24 h	AVGF, DM#3
Series 5	Operate at reduced dose in an attempt to reduce mudballing	
Series 6	Extended operation of pilot filters. Parallel operation of AVGF	AVGF 0.7 mm sand DM#3
Series 7	Operate AVGF until performance deteriorated	AVGF

Table 4.3. Operating conditions (AVGF and pilot filters)

	Filtration Rate	Water Temperature	Alum Dose	Coagulated Water	
				Turbidity	pH
Series 1	6.6 m/h	22 – 24 °C	29 mg/L	80–120 NTU	6.8
Series 2	6.4 – 6.6 m/h	22 - 24 °C	20-21 mg/L	27–40 NTU	7.0
Series 3	6.3 – 6.9 m/h	20 – 23 °C	14 –16 mg/L	15–26 NTU	7.2
Series 4	6.3 – 6.9 m/h	20 – 21 °C	9 – 10 mg/L	2-8 NTU	7.4
Series 5	6.1 – 6.5 m/h	21 – 24 °C	3 – 5 mg/L	1 – 5 NTU	7.5
Series 6	6.0 – 6.9 m/h	21 – 26 °C	5.5–6.5 mg/L	1.5 – 6 NTU	7.5
Series 7	5.8 – 6.4 m/h	18 – 26 °C	0 – 8 mg/L	1 – 40 NTU	7.3 – 7.8

Table 4.4 **Filtration Results**

	Filter	Headloss at turbidity breakthrough	Run time to terminal headloss of 1.5 m	Filtrate turbidity before breakthrough
Series 1	0.5 mm sand	0.4 – 0.7 m	7 - 10 h	0.2 – 1.7 NTU
	0.8 mm sand	0.1 – 0.4 m	10 – 20 h	0.5 – 2 NTU
	Dual media #1	0.8 m	11 h	0.1 NTU
Series 2	0.5 mm sand	> 1.5 m	6 h	0.2 – 0.6 NTU
	0.7 mm sand	0.85 m	14 h	0.34 – 1.2 NTU
	0.8 mm sand	0.5 m	22 h	0.5 – 1.2 NTU
	Dual media #1	< 1.1 m	13 – 16 h	0.1 – 0.5 NTU
Series 3	0.5 mm sand	> 1.5 m	10 h	0.1 – 0.8 NTU
	0.7 mm sand	> 0.8 m, < 1.3 m	12 – 20 h	0.3 – 1.1 NTU
	0.8 mm sand	> 0.5 m; < 1.0 m	15 – 24 h	0.3 – 1.0 NTU
	Dual media #1	1.2 m	19 – 24 h	0.1 – 1 NTU
	Dual media #2	> 1.5 m	12 – 14 h	0.1 – 0.5 NTU
Series 4	Dual media #3	> 1.5 m	19 – 23 h	0.1 – 0.5 NTU
	AVGF	> 1.5 m	14 – 19 h	0.1 – 0.6 NTU
Series 5	0.7 mm sand	> 1.5 m	56 – 58 h	0.6 – 0.9 NTU
	Dual media #3	> 1.5 m	48 – 52 h	0.3 – 0.6 NTU
	AVGF	> 1.5 m	43 – 45 h	0.2 – 0.5 NTU
Series 6	0.7 mm sand	> 1.5 m	19 – 23 h	0.18 – 0.8 NTU
	Dual media #3	> 1.5 m	19 – 26 h	0.18 – 0.8 NTU
	AVGF	> 1.5 m	11 – 22 h	0.14 – 2 NTU
Series 7	AVGF	> 1.5 m	4 - 48	0.1 – 34 NTU

4.3.1.1. Filter breakthrough at high influent turbidities

It was found that as the raw water and coagulated water turbidity increased, the headloss at which breakthrough started decreased and at higher turbidities, breakthrough occurred before terminal headloss. (Series 1 to Series 3). **Figures 4.7** and **4.8** summarise the pilot filter data in terms of filter breakthrough and filter run volume as a function of influent turbidity.
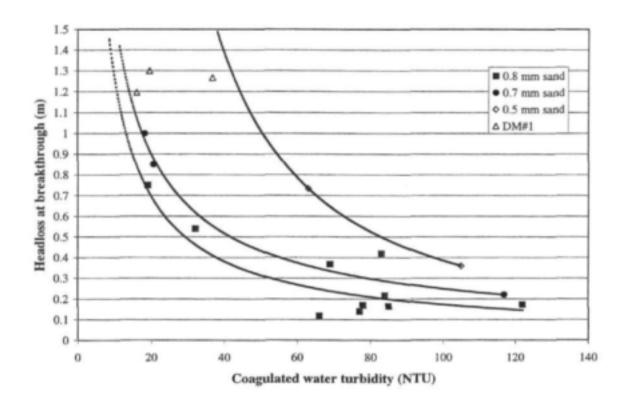
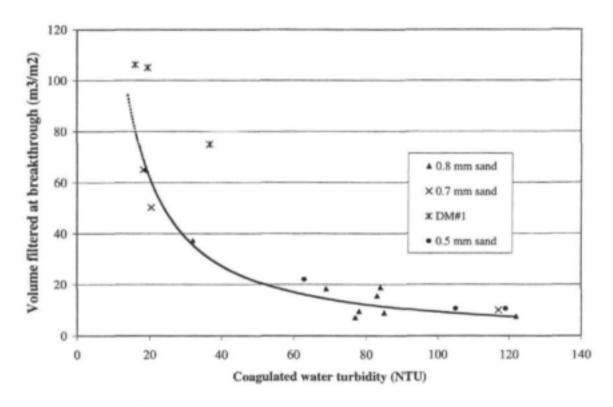**Figure 4.7 Headloss at breakthrough as a function of influent turbidity**

Figure 4.8 Volume filtered at breakthrough

The headloss at breakthrough increased with increasing contactor area (**Tables 3.3 and 3.4**). Thus headloss at breakthrough was found to be greater for the dual media filters than the 0.7 mm sand filter (Series 3) and increased with the depth of the 0.5 mm sand (**Table 3.4**; Dual media #2 > Dual media #1) since the finer sand grains contribute most of the contactor area while the anthracite provides additional floc storage volume (Cleasby, 1990).

The headlosses at breakthrough for the 0,7 and 0,8 mm sand filters are significantly lower than values reported in the literature. This was assumed to be as a result of the high influent turbidity and relatively short beds. Kawamura (1975a) reports that breakthrough quite often starts when the headloss reaches about 1.2m, but in some cases turbidity can breakthrough at headlosses less than 1m. (Kawamura also indicated that turbidity breakthrough can occur at any time depending on (1) poor coagulation control and (2) hydraulic shocks.)

Breakthrough did not occur in any of the filters at influent turbidities of < 10 NTU (Series 4 and Series 5).

4.3.1.2. Filtrate turbidity before breakthrough

The dual media filters exhibited long improving periods (i.e. after filter ripening, the filtered water turbidity continued to improve as the filter run progressed) and generally produced better filtrate quality than the 0.7 mm sand.

Figure 4.9 shows filtrate turbidity as a function of volume filtered since the last backwash for series 4 – 6. Filtrate turbidities of less than 1 NTU were almost always achieved by the time the backwash tank had filled up and filtrate was flowing out to service. Turbidities of greater than 1 NTU were associated with dosing disruptions. The low backwash rates at the end of backwash may have contributed to the relatively good initial filtrate quality by reducing the number of backwash remnant particles remaining in the filter when backwash terminated (Amburgey, et al, 2002).
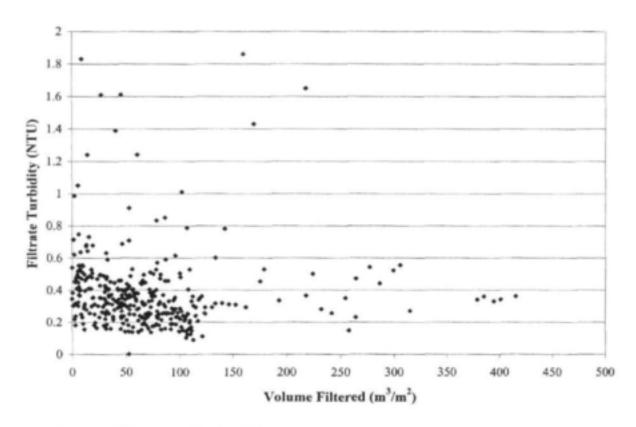
Figure 4.9 Filtrate quality in AVGF

4.3.1.3. Filter run volume to terminal headloss

Figure 4.10 shows the filter run time to terminal headloss as a function of influent turbidty for the various filters.
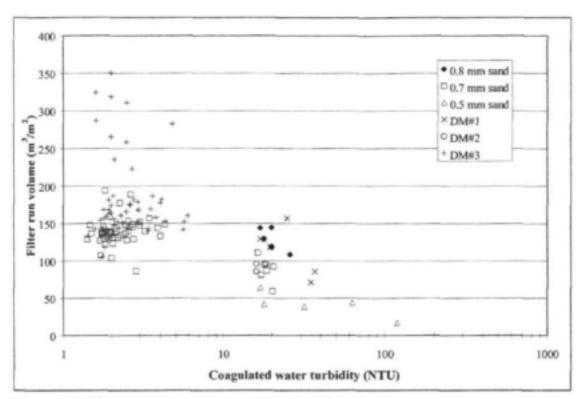
Figure 4.10 Filter run volume to terminal headloss (1.5 m)

The full-scale filter run time to terminal headloss was found to be shorter than the laboratory-scale filter (Series 4) with the same media configuration (**Table 4.3**). A similar trend is observed at lower turbidities and correspondingly lower coagulant doses where the operation resulted in run times > 40 hours (Series 5). Possible explanations include:

- Relatively poor flow distribution in (despite the increase in nozzle density) appeared to result in greater intermixing of the sand and anthracite in the AVGF than in the pilot filters

- Differences in coagulation control: the AVGF was typically left unattended for extended periods where slight fluctuations in the coagulant dose could have occurred. In contrast the pilot-scale filters were supervised more regularly and a more consistent coagulant dose and flowrate were maintained.

4.3.1.4. Estimation of the bed depth to prevent turbidity breakthrough

Given that the total floc holding capacity of a filter is a function of bed volume, it should possible to estimate to the bed height required to ensure terminal headloss is reached before breakthrough, based on the volumes filtered to achieve breakthrough and terminal headloss.

Figures **4.11** and **4.12** show Series 2 filter run data for the 0.8 mm sand, 0.7 mm and dual medial filters. Series 2 could be regarded as the worst conditions an AVGF could be expected to cope with for short periods of time.
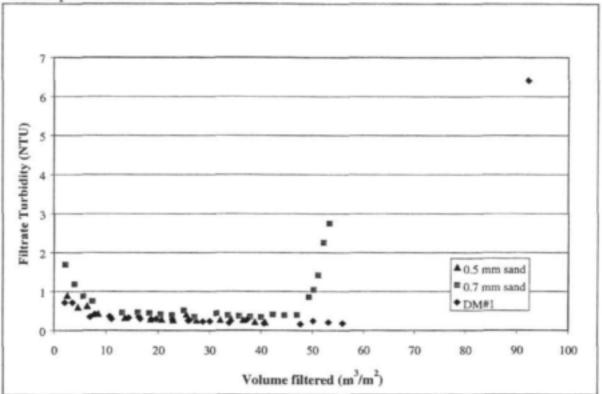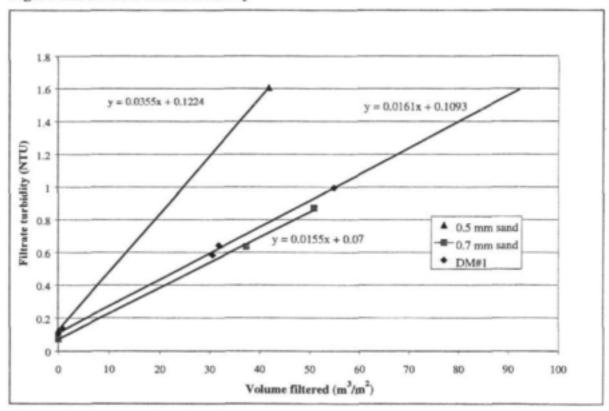

Figure 4.11 Series 2 filtrate turbidity

Figure 4.12 Series 2 – headloss development

Figure 4.12 Series 2 – headloss development

Assuming constant influent conditions and that floc holding capacity is proportional to bed depth, then the bed depth required to prevent breakthrough before terminal headloss should be equal to ratio of the volume filtered to a specified terminal headloss and breakthrough respectively, multiplied by the bed depth at which they are measured.

Table 4.5 summarizes calculations of the required bed depth to prevent breakthrough before terminal headlosses of 1.5 m and 1.0 based on **Figures 4.11** and **4.12**. All three filters achieved terminal headloss around 90 m³/m² but breakthrough occurred at around 35 m³/m² in the 0.8 mm sand, 50 m³/m² in the 0.7 mm sand and between 60 and 90 m³/m² in the dual media filter (breakthrough happened overnight therefore the exact time at which it occurred could not be determined). For the purposes of the calculation, breakthrough in the dual media filter is a assumed to have occurred at ca. 60 m³/m² (ca. 1.1 m headloss).

Table 4.5   **Determining the bed height required to prevent breakthrough**

Pilot Filter	0.8mm sand	0.7 mm	Dual Media #1
Bed depth (m)	0.54	0.52	0.62
Volume filtered at breakthrough (m³/m²)	35	50	60
Volume filtered to 1.5 m headloss	90	90	90
Bed depth to prevent breakthrough	1.4	0.94	0.93
Volume filtered to 1.0 m headloss	60	60	60
Bed depth to prevent breakthrough	0.92	0.62	0.62

The bed depth requirements of the 0.7 mm sand and dual media are very similar but a much deeper 0.8 mm sand bed is required to achieve the same result.

4.3.2. Extended operation of the AVGF and pilot-scale filters

Once the selection of dual media for the full-scale AVGF had been finalized, the focus of the investigation shifted to studying the performance of both the full and pilot scale filters (DM#3 and 0.7 mm sand) over the long term with the minimum operator intervention. At this point, the raw water turbidity had dropped below 10 NTU (Series 4 and 5 in Table 4.2) and premature breakthrough no longer occurred. However, a new problem soon became evident. Mudball

formation was observed in the top sections of the pilot filter beds after two or three consecutive runs without air scour. The mudball clusters continued to grow in size and sink deeper into the filter over subsequent runs. Mudballing was also observed in the full scale filter after one month of operation. Three different strategies were adopted to try to eliminate or reduce the accumulation of mudballs in the pilot filters.

1. Increasing the backwash volume to 3,2 m^3/m^2 and the peak backwash rate to 55 m/h
2. Skimming both the sand and anthracite layers to remove fines (DM #4)
3. Backwashing the filters once a day regardless of headloss development.

None of these strategies proved effective and based on the Series 4 experiments, it was concluded that some mudballing of the filter media was inevitable in the absence of auxiliary backwash, even if the backwash design was improved. However, at least initially, visual observation of mudballs in the pilot filters and the full scale AVGF did not correlate to any obvious decline in filter performance.

4.3.2.1. Pilot Filter Filtration Performance

In the series 5 experiments the two pilot filters (0.7 mm sand and DM#3) were operated as consistently as possible for 65 consecutive runs without auxiliary backwash, totaling a cumulative run time of approximately 1 month. Filtration performance was closely monitored and the data analysed to determine whether there were any statistically significant trends in performance, which could be attributed to mudballing of the media The pilot-scale filters were backwashed and the filter media was cleaned as best possible with air scour to represent a clean media at the beginning of the operation.

Figures 4.13 and 4.14 show the performance of the pilot-scale columns containing 0.7mm sand and dual media (DM#3 Table 3.4) respectively. . At this point in the investigation, it was clear that one of the recommendations that would come out of the project would be that the backwash volume should be increased. Therefore, the pilot plants were backwashed with 3,2 m^3/m^2 (Scenario 7 in Figure 4.3) instead of 2,2 m^3/m^2 for this series of the experiments. The filtered water turbidity presented in Figures 4.13 and 4.14 is the turbidity after the first 3.2 m^3/m^2 of each run which would be diverted to the backwash tank in the hypothetical full scale version. The filtrate turbidty consistently lower than 1 NTU (SABS standard for potable water). This is significant as, during the filtration period the coagulant dose was measured but not deliberately adjusted to allow for changes in raw water turbidity.

The unit filter run volume is presented and was relatively constant for the filter containing 0.7mm sand (ca. 140 m^3/m^2) but tended to be slightly shorter than the unit filter run volume of the dual media filter (ca. 160 m^3/m^2). Unit filter run volumes for both filters were significantly lower than those recommended by Cleasby (1990) and Kawamura (2000), that is a minimum of 200 m^3/m^2 for direct filtration and 600 m^3/m^2 for conventional treatment). The filter run volumes are however, limited by the relatively low terminal headloss in the AVGF. This issue will discussed further in **Chapter 5**.

Since there was no obvious deterioration in filter performance over the monitoring period, a multiple linear regression analysis of the data was undertaken to see if any of the variability in filtrate quality and filter run volume could possibly be correlated with changes in the filter bed over time as opposed to fluctuations in dose, filtration rate, influent turbidity and temperature. Details of the analysis are presented in **Appendix 3**.

The overall conclusion drawn from the statistical analysis is that mudballing did not result in a decline in performance in the period over which the two laboratory filters were monitored. This may have been because the conditions of low influent turbidity and relatively short run times (< 24 hours) did not particularly challenge the filters so that loss of floc storage capacity and probable increase in channelling and worm hole formation did not pose a problem and would not necessarily have been the case if there had been a large surge in influent turbidity.
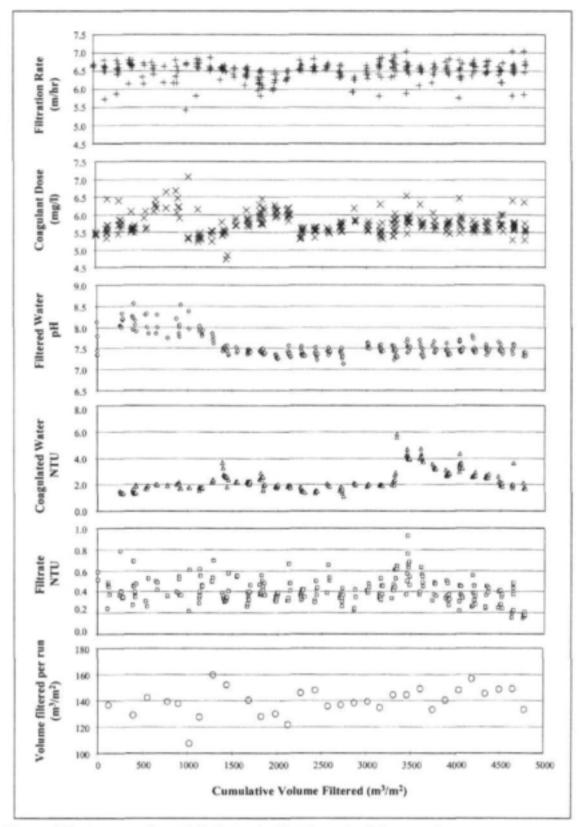
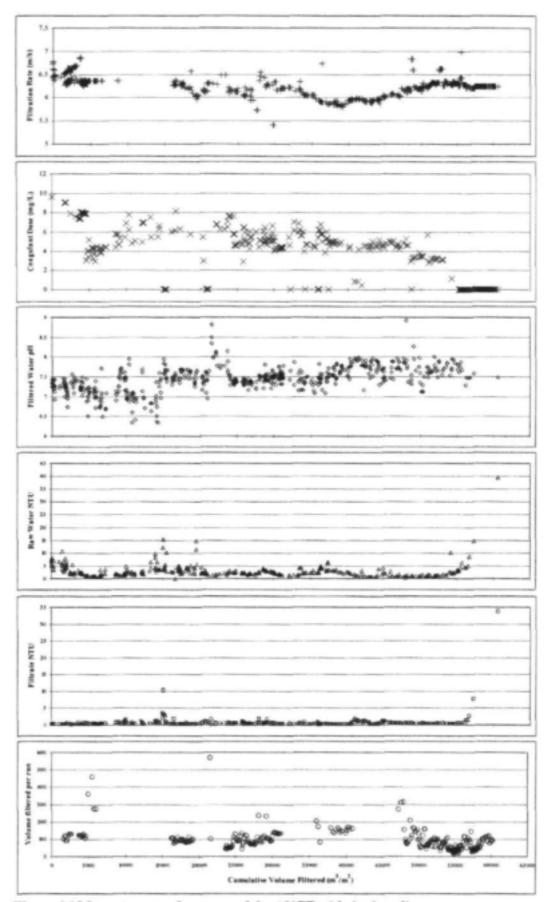**Figure 4.13**   Extended pilot-scale filtration using 0.7mm sand

Figure 4.14 Extended pilot-scale filtration using dual media (DM#3)

63

Figure 4.15 Long term performance of the AVGF with dual media

64

4.3.2.2. Long term performance of the full-scale AVGF

The AVGF was operated for 15 months (October 2000 to December 2001, Series 4 to 6 in **Table 4.2 - 4.4**). The filter performance over this period is shown in **Figure 4.16**. Satisfactory performance was achieved for the first 12 months of operation provided that the dosing system was functioning properly.

After approximately a year of operation (ca. 53 000 m^3/m^2 cumulative run volume), the run times began to drop below 10 hours (ca. 60 m^3/m^2 run volume) despite low raw water turbidities and reductions in dose. Eventually alum dosing was stopped completely at 55 000 m^3/m^2 cumulative run volume. Run times and filtrate turbidity increased slowly thereafter until the raw water turbidity increased above 15 NTU where upon the run times dropped again initially and then continued increase in spite of further increases in turbidity. The slow response of the of the filter to changes in dose and raw water quality, were assumed to be due to the large amount of accumulated floc in the filter which would initially have absorbed the uncoagulated influent particles.

4.4. Backwash performance

One of the key issues which will determine the suitability of the AVGF for use in rural areas is whether or not the filter media can be adequately cleaned by water only backwash. If the AVGF cannot operate indefinitely without intervention to restore the filter bed to an acceptable state then the feasibility of this process may depend on the nature and frequency of the intervention required to maintain satisfactory performance.

Typical methods for assessing backwash efficiency were presented in **Section 2.4.5**. There are two aspects to backwash efficiency

- The efficiency of the backwash regime in detaching deposited from floc media grains
- The efficiency of wash water usage

In this study, backwash performance was assessed in terms of the following:

- The formation, accumulation and distribution of mudballs. Understanding how, why and where mudballs form and accumulate will aid in determining strategies to minimise them.
- Volume and rates of backwash, which are effective in cleaning the media.
- Efficiency of backwash in removing floc from the media
- Effect of mudballing on the long term filter performance.

65

4.4.1. Visual observations and inspection of the media

For the purposes of this discussion, a mudball is considered to be any structure consisting of media grains "glued" together or coated with floc, which is not completely broken down by backwashing. During filtration, the media grains near the top of the bed become cemented together with floc to form a long range composite structure which has to be broken down during backwashing. Mudballs result when the disintegration of the clogged bed structure is incomplete. At least two different types of mudballing phenomena were observed during the course of this study.

On several occasions during operation of the pilot-scale filters, for doses of 15 – 20 mg/L, raw water turbidites of 14 to 20 NTU and run times of 18 hours or less, large chunks (up to 10 cm across) of clogged media were observed sinking into the 0,7 and 0,8 mm sand filters as the top sections of the bed began to fluidise. These chunks were slowly eroded away as the backwash progressed but residue from the original structures was visible through subsequent filter runs. This type of mudballing appeared to be typical when the maximum backwash rate was close to the minimum fluidisation velocity of the bed.

No similar behaviour was observed in the dual media filters or the 0,5 mm sand filter under the same operating conditions. This was assumed to be due to their lower minimum fluidisation velocities. However, small individual pebble like mudballs could sometimes be seen close to the filter walls during backwashing. When the raw water turbidity dropped below 10 NTU, clusters of ca. 1 cm mudballs began to appear in the anthracite layer close to the interface between the anthracite and the sand. These were often only really visible during backwashing when they were exposed by the flow channelling past them at the wall. **Figure 4.16** (a) shows a large cluster of mudballs close to the sand- anthracite interface at a backwash rate of approximately 40 m/h. The actual size of this agglomerate was possibly larger than the portion visible through the column wall.

The anthracite was subsequently removed from the laboratory filter (containing DM#3) and examined more closely. Numerous clusters of 10 to 50 anthracite grains were found glued together by small kernels of mud and fine sand grains. Individual fine sand grains were dispersed throughout the anthracite layer but did not constitute a significant fraction of the volume of solids in the anthracite layer. The same phenomenon was observed in anthracite excavated from the AVGF after it had been operating with dual media for a few weeks only. The concentration of sand grains in the anthracite layer in the AVGF appeared to be greater than in the pilot filters. Poor flow distribution resulting from the internal structure of the filter compartment seems to have played a role.
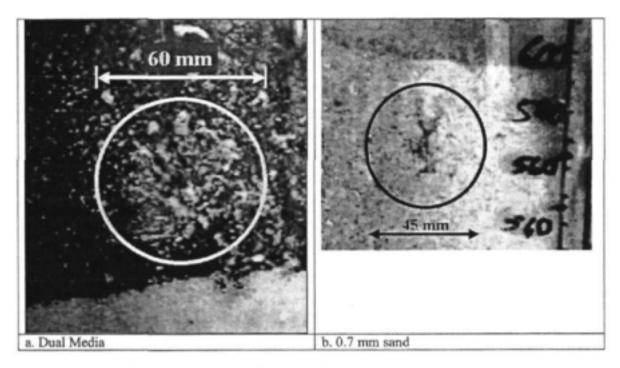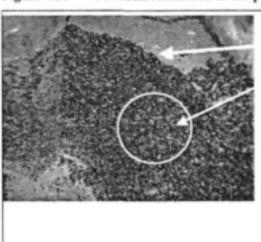
| a. Dual Media | b. 0.7 mm sand |

Figure 4.16   Mudball formation in the pilot filters

1. Incomplete removal of floc from top layer of fluidised media

2. Developing mudballs (anthracite grains cemented together by patches of floc)

3. Large mudball from the sand-anthracite interface

Figure 4.17 Sludge and mudball accumulation in the AVGF

It was thought that slight overdosing of coagulant may have played a role in promoting the mudball formation. However, lower doses resulting in long run times also seemed to cause problems. At doses of 4 mg/L and run times of up to 3 days clogged sections of media of up to 5 cm in diameter were observed in both the monomedia and dual media filters during backwashing. **Figure 4.16(b)** shows evidence of mudball formation in the 0.7 mm sand under these conditions.

The media in the pilot-scale filters was cleaned with air scour and the backwash rates and volume were then increased to see if this would improve the situation. However, once again mudballs re-appeared in the filters within a few runs. At this point it became clear that mudball formation was inevitable and the focus shifted to looking at what would happen to the filter media over the long term and how the filter performance would be affected. The 0.7 mm sand and DM#3 filters were operated for a total cumulative run time of approximately one month with the increased backwash rates and the accumulation of mudballs over time observed through the perspex filter walls.

While the agglomerates of mudballs in the 0.7 mm sand filter slowly sank deeper and deeper into the filter as the experiment progressed, mudballing in the dual media filter appeared to be limited to within the anthracite layer and accumulated at the sand interface. In some areas, the anthracite at the interface gradually fused into solid structures up to 10 cm in length, which resulted in dramatic channeling effects during backwashing. At the end of the monitoring period, the filter media was removed from the laboratory filters and the structure and distribution of mudballs, and the general state of the media examined further.

Apart from the mudballs, the 0.7 mm sand was relatively clean below the first 1 cm from the bed surface. The top surface of the bed was however coated with a thin layer of sludge. By contrast, sludgy areas were distributed throughout the entire anthracite layer. Near the top of the bed, small clusters of anthracite were loosely glued together by patches of relatively soft floc. The fine sand observed previously did not appear as prevalent as before, possibly because it had been washed out by air scour in the interim. Deeper in the anthracite, the mudballs became larger, harder and contained increasing amounts of relatively coarse sand. It is thought that this sand may have been carried into the anthracite layer by high velocity channels (occurring during backwashing) due to mudballing at the interface. Mudballs at the interface were entirely coated with sand but their sand content decreased towards their centres where the material was similar to the mudballs found near the bed surface. The sand below the interface was remarkably clean and no evidence of mudballing was apparent. Since the mudballs are composed primarily of anthracite and floc, they have a lower bulk density than the sand and therefore do not sink into the sand layer when the bed is fluidized.

Similar observations were made when the anthracite layer was excavated from the AVGF after approximately 7 months of operation. The state of deterioration of the anthracite was more advanced than in the DM#3 as the AVGF had been running for longer with lower backwash rates and backwash volume. The interface between the sand and anthracite was entirely composed of mudballs of various sizes and relatively coarse sand was found throughout the anthracite layer. However, the sand below the interface was clean and free of mudballs.

After 15 months of operation the anthracite layer in the AVGF was found to be relatively thin in the middle of the filter while it had piled up against sides of filter. Mudballs in the regions close to the walls and in particular, between the walls and the internal pipework were larger than those in the centre of the filter and in some places large areas of media had become fused together in a cake structure rather than clusters of discrete mudballs. It appears that at this point, increasingly poor fluidisation near the walls was resulting in the deposition of anthracite at the edges of the filter while most of the backwash flow was channeled through the centre of the filter. This phenomenon may have been linked to the ultimate decline in filtration performance discussed in Section 4.3.
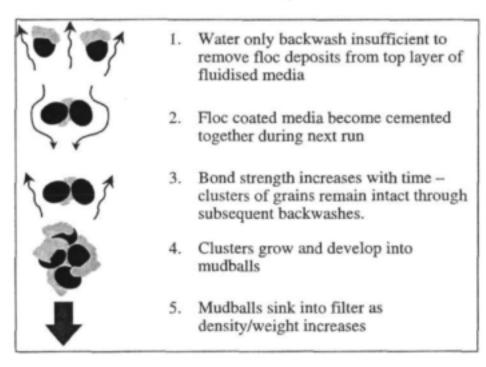

1. Water only backwash insufficient to remove floc deposits from top layer of fluidised media

2. Floc coated media become cemented together during next run

3. Bond strength increases with time – clusters of grains remain intact through subsequent backwashes.

4. Clusters grow and develop into mudballs

5. Mudballs sink into filter as density/weight increases

Figure 4.18 Proposed mechanism of mudball formation

The observations of mudballing in dual media filters tend to suggest that mudballing of the anthracite is actually initiated near the surface of the bed and the mudballs sink and accumulate sand as they become larger and heavier. The same mechanism probably applies to mudball formation in monomedia filters. It is likely that these mudballs start off as fairly small structures, possibly individual grains to which floc remained attached. In fact the top layers of the fluidised pilot-scale filter beds were observed to be almost entirely made up of such grains. The partially cleaned grains would settle back after backwashing and become compacted together. The remaining floc deposits will act as nuclei for the formation of new inter-grain floc bonds. Since there is a larger amount of, and denser floc in the deposits than would otherwise be present at the beginning of a filter run, intergrain bond formation results in stronger and denser bonds which have a higher probability of surviving the next backwash. Once grains have become irreversibly bonded together, the crevices between them tend to fill up with floc, which is not flushed away during backwash so that the

surfaces of the mudballs become smooth and entirely coated with floc. Eventually, small individual mudballs become bonded together into larger structures. This process is illustrated in **Figure 4.18.**

4.4.2. Backwash turbidity

Cleaning of the filter during backwashing can be conceptually broken up into two steps:

1. Detachment of floc from the media, and
2. Flushing of the detached material out of the filter and transport to the backwash trough or backwash pipe.

Ideally, the floc should be instantaneously detached as backwash is initiated and the resulting mobilised sludge be washed out in plug flow. In practice the volume of water required for effective cleaning is increased by a number of factors:

- Most detachment of floc occurs within the first few seconds of backwash at properly designed backwash rates, however, it is not entirely instantaneous. The lower the backwash rate, the slower the breakdown of the floc-media composite.
- Uneven velocity distribution within the media and the freeboard below the backwash trough/pipe results in dispersion and backmixing.

The volume of wash water required the to flush the bulk of the detached floc out of the bed is the fraction of the total wash water which is effectively cleaning the media. An additional volume increment is required to flush the floc into the backwash trough to ensure it is permanently removed from the filter.

If the backwash is stopped too soon, the media may still be dirty and detached floc, which has not yet been flushed out will settle back onto the bed as a layer of sludge which gels within minutes. Fluid shear above the media is much less than within the media and the sludge layer cannot be flushed out of the filter without auxiliary backwash to break it up. However, washing filters for too long, wastes wash water and tends to increase filtrate turbidity at the beginning of the next run and prolong filter ripening.

The optimum backwash volume can be determined by analyzing the filter backwash turbidity as a function of backwash duration. Grab samples of backwash turbidity were collected from four different pilot-scale filter set ups for operating conditions involving both high and low influent turbidity (**Table 4.2 - 4.4**). The raw water feed was turned off during backwashing for these

70

experiments. The results were analysed to determine the efficiency of backwash water use in terms of both detaching floc from the media and flushing it out of the filter.

Effective backwash was assumed to have ceased when ceased when the backwash turbidity dropped below 10 NTU. The was based on Kawamura's recommendations (Kawamura, 2000) for determining when to terminate backwash. It must be noted however, that low turbidity backwash effluent does not necessarily imply a clean filter bed. In particular, the declining backwash rate means that the last remaining filter deposits, which are obviously the most difficult to remove, will face declining shear forces resulting in declining detachment rates.

Kawamura's recommendations for optimum backwash rates are based on a theoretical analysis of the maximum cleaning action on the average grain size (d_{60}). However, the bulk of floc removal during filtration occurs in the upper layers of the filter bed where the finest grain sizes are found. Furthermore, the results of this study and others reported in the literature (Kawamura, 1975a) suggest that the finer grain sizes play a major role in mudball formation. Therefore it is possible that backwash efficiency may not be significantly impacted if backwash velocity towards the end of backwashing drops below the optimum backwash rate or even the minimum fluidisation velocity for the average grain size provided that conditions remain close to optimal for the finest grain sizes.

Figures 4.19 - show backwash turbidity leaving the filter bed and backwash rate as a function of backwash volume for a number of different conditions. The minimum fluidisation and optimum backwash velocities (**Equation 2.4** and **2.9**, respectively) for both the d_{10} and d_{60} sizes are also shown for comparison. Where backwash was terminated before the turbidity reached 10 NTU or data was missing, the turbidity profile was extrapolated to 10 NTU using the relationship

$$\ln(\ln (\text{turbidity})) = A(\text{backwash volume}) + B$$

The coefficients A and B were found by regression on the declining leg of the backwash turbidity profile.
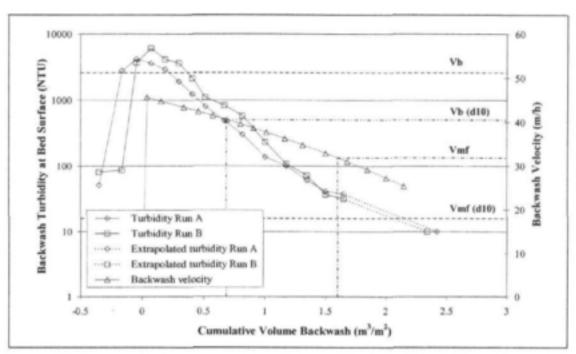
Figure 4.19 Backwash of 0,7mm sand for high influent turbidities
Backwash rate 46 – 25 m/hr; Volume 2,2 m^3/m^2;
Run A: influent 20 NTU; 20 mg/L Alum; Vol. filtered 102 m^3/m^2
Run B: influent 20-120 NTU; 15-30 mg/L Alum; Vol. filtered 31 m^3/m^2
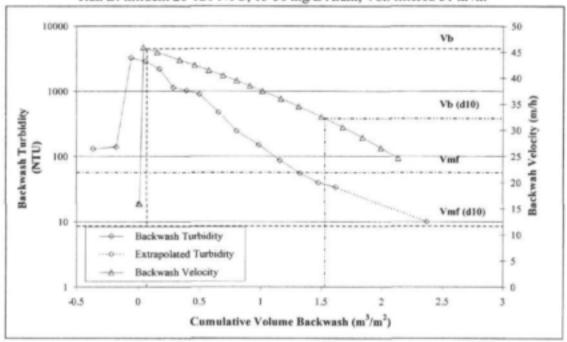

Figure 4.20. Backwash of 0.5mm sand for high influent turbidities
Backwash rate 46 – 25 m/hr; Volume 2,2 m^3/m^2;
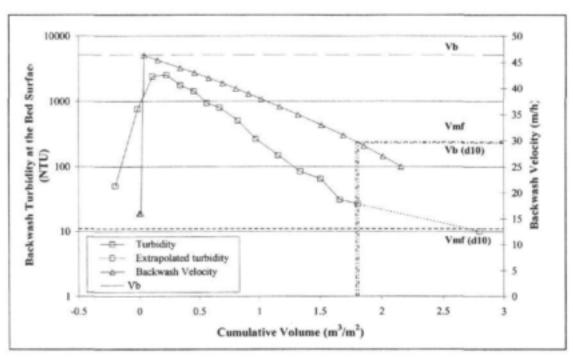influent 20 NTU; 20 mg/L Alum; Vol. filtered 102 m^3/m^2

Figure 4.21 Backwash of DM#1 for high influent turbidities
Backwash rate 46 – 25 m/hr; Volume 2,2 m³/m²;
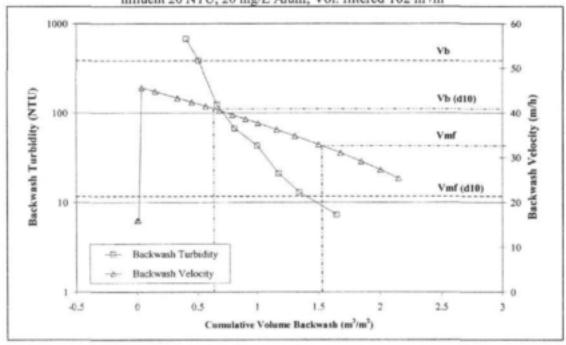influent 20 NTU; 20 mg/L Alum; Vol. filtered 102 m³/m²

Figure 4.22 Backwash of 0,7mm sand for low influent turbidities
Backwash rate 46 – 25 m/hr; Volume 2,2 m³/m²;
influent 2-7 NTU; 5 mg/L Alum; Vol. filtered 409 m³/m²
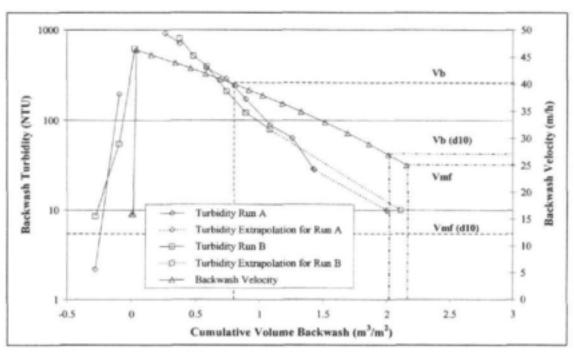
73

Figure 4.23 Backwash of DM#3 for low influent turbidities
Backwash rate 46 – 25 m/hr; Volume 2,2 m³/m²;
influent 2-7 NTU; 5 mg/L Alum; Vol. filtered: Run A 397 m³/m²; Run B 286 m³/m²
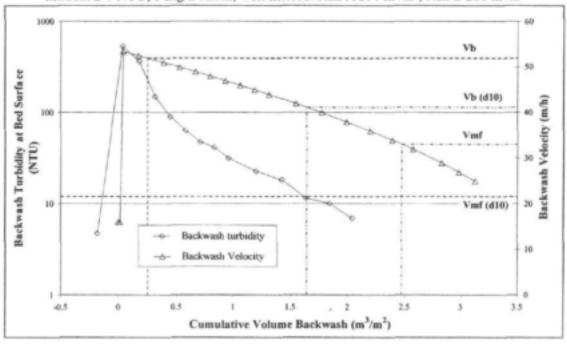

Figure 4.24   Backwash of 0,7mm sand for low influent turbidities
Backwash rate 55 – 25 m/hr; Volume 3,2 m³/m²;
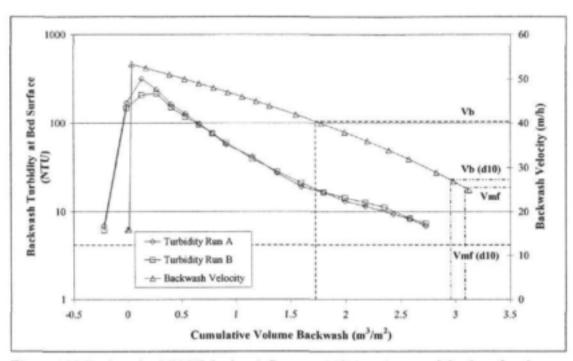influent 1-3 NTU; 6 mg/L Alum; Vol. filtered 155 m³/m²

74

Figure 4.25 Backwash of DM#3 for low influent turbidities – increased backwash volume
Backwash rate 55 – 25 m/hr; Volume 3,2 m^3/m^2;
influent 1-3 NTU; 6 mg/L Alum; Vol. filtered 155 m^3/m^2

Based on the limited number of results obtained, the effective backwash volume appears to depend mainly on the amount and strength of (detachable) floc in the bed and the type of the filter bed. The dual media filters always required approximately 0,5 m^3/m^2 more backwash volume to reach 10 NTU than the monomedia filters. Estimation of the pore volume of the expanded anthracite layer indicated that the travel time from the sand-anthracite interface could not have contributed much more than 1 % of this difference, indicating a slower release of floc from the anthracite grains than from the sand.

Depending on the expanded bed height, the backwash volume required to carry floc from the surface of the filter bed to the "point of no return" in the AVGF backwash pipe, was approximately 0,6 to 0,8 m^3/m^2. Based on the results presented above, the maximum effective backwash volumes were estimated to be 2,5 m^3/m^2 and 3,1 m^3/m^2 respectively for monomedia sand and dual media filters operating at low influent turbidities, and 3,1 m^3/m^2 and 3,6 m^3/m^2 for high influent turbidities. While these values are relatively low compared to typical design specifications for conventional filters (**Section 2.4.4**), it appears that additional wash water would be wasted unless the efficiency of floc detachment is increased.

There did not appear to be any particular trends in volume of effective backwash with respect to the optimum and minimum fluidisation velocities and more experimentation is needed in this area.

Ideally, the average backwash rate should be the optimum rate $(V_b(d_{60}))$ and the minimum rate should not be less than $(V_b(d_{10}))$. However, efficiency of backwash is probably not very sensitive to the backwash rate in the region of the optimum (Amirtharajah). Therefore, there may not be much advantage in using such high rates. The backwash rate should however, not drop below the minimum fluidisation rate of the bed. Possible design modifications to the AVGF backwsh hydraulics and their implications will be discussed in **Chapter 5**.

4.4.3. Material Balance

Material balances were carried out on the backwashing of the 0,7 mm sand filter and DM#3 during and after the long term pilot-scale experiments. The filters were initially backwashed as usual using the simulated AVGF results. The backwash water was collected in 120 L drums and analysed for turbidity and suspended solids. The filters were then backwashed with simultaneous air and water followed by high rate backwash until all the floc and mudballs had been removed (this was based on visual judgement). The backwash effluent generated by this step was also analysed for turbidity and suspended solids. The mass of floc remaining in the filter after water only backwash and percent efficiency of detachment were then calculated. The results are summarised in **Table 4.6**

Table 4.6   Efficiency of detachment of floc based on material balance

Cumulative volume filtered before air scour	Filtration Conditions	Back wash	% Efficiency of detachment	
			0,7 mm sand	DM #3
1200 m³/m²			36 %	
5000 m³/m²	1 – 5 NTU 5 mg/L	3,2 m³/m² 55 – 25 m/h	28 %	24 %

4.4.4. Headloss profile after backwash

It was originally thought that the accumulation of mudballs in various parts of the filter would be reflected in the downflow headloss profile measured directly after backwash. Instead it was found

that the total initial headloss and headloss profile remained essentially the same but that the settled bed height increased. This is illustrated for DM#3 in **Figure 4.21** (4.3.4.1). Similar results were obtained for the 0,7 mm sand filter. The initial headloss tended to decrease with increasing temperature and vise versa but showed virtually no trend when it was corrected for temperature. The initial bed height, however, showed a consistent increase with cumulative volume filtered independent of the trend in temperature. Increases in the settled bed height over time were also observed in the AVGF. When the laboratory filters were air scoured to remove mudballs, the settled bed heights returned to their previous levels.
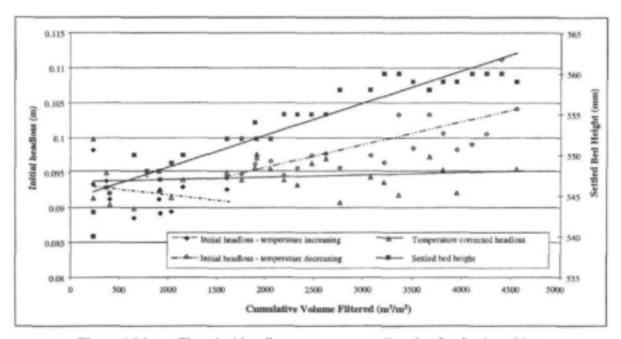
Figure 4.26   Clean bed headloss measurement directly after backwashing

The increasing bed heights can possibly be explained terms of the fact that clogging in some regions of the media will result in higher backwash velocities and porosities in the fluidised sections. As the backwash rates decrease, the loose media will settle out at higher porosities to compensate for the decrease in flow area. The clusters of mudballs themselves often appear like coarse gravel with large pore spaces in between. This phenomenon is likely to promote channelling and ultimately filter breakthrough

5. Design of the autonomous valveless gravity filter

The full-scale filter that was commissioned at the Umgeni Water Process Evaluation Facility is a proprietary design and has been shown to have certain limitations. Some of these limitations are inherent in the design of the AVGF but others can and apparently have been improved upon in later valveless filter designs (Walker, 2000; Larkins, 2000). This Chapter presents the specific limitations of the AVGF and discusses possible design modifications which could improve its performance and reliability.

5.1. Limitations of AVGF operated at PEF

- *Backwash volume*: At 2,2 m^3/m^2, the backwash volume is insufficient and the available backwash rates limit the effective sand size which can be used in the filter to 0,7 mm or less. Even for 0,7 mm sand, the bed is only partially fluidised for almost half of the backwash.

- *Material of construction*: The construction is from mild steel and has since its installation been painted on the inside. Nonetheless corrosion has occurred and there are areas where pitting and blistering is occurring. This may be aggravated by a lack of stabilisation of the water but the Wiggins water is relatively non-corrosive compared to other waters in South Africa. The selection of suitable materials of construction to prevent long term problems occurring either - Iron in the final water or scale and metal flakes in the backwash water which may either block nozzles or become entrained in the sand bed.

- *Open backwash water reservoir*: The top of the filter (i.e. the top of the backwash reservoir) is open to atmosphere and is not covered. A large amount of algae growth has been observed in this reservoir which is detrimental to the operation of the filter. Algal cells in the backwash water can become entrained in the filter media during backwashing and will contribute to head loss in the media. Where possible, it is recommended that the top of the filter be covered so that the water is not exposed to light.

- *Uneven flow distribution*: A number of factors could have contributed to uneven flow distribution. The low nozzle density, and relatively low nozzle headlosses during backwashing would not have favoured even backwash flow distribution. Furthermore, the internal geometry of the filter compartment resulted in dead zones in certain areas. For example, the feed water drained into a steel box at the back of the filter on the inside. The top of the box was below the top of the media, resulting in a dead zone during backwashing. When the filter was opened, a pile of dirty media which had accumulated on top of the feed box was found. The relatively uneven flow distribution in the AVGF appeared to have contributed to greater intermixing of sand and anthracite in the full scale filter than in the pilot filters (**Section 4.3.1**).

- *Auxiliary backwash:* This is a 'water only' backwash system. Some AVGF's can be manually air-scoured (without simultaneous water flow) by connecting a compressor / blower to the underdrain. During air scour, the downcomers between the underdrain and the backwash tank have to be shut off to prevent the air by-passing the media. The downcomers in the AVGF evaluated in this study are not fitted with any valves and cannot be accessed from outside the filter.

5.2. Design Parameters

5.2.1. Media selection

The filtration results suggest that the use of dual media has significant advantages over the use of monomedia especially at influent turbidities > 15 NTU. At these turbidities, the backwashing of the dual media filters also appeared to be adequate. However, the anthracite layer appears to be particularly prone to mudball formation. Intermixing of the sand and anthracite could be a factor even when the ratio of sizes of the two media agrees with recommendations in the literature. Specifying narrower size distributions and a sharper interface between the two media, to compensate for the relatively inefficient backwash regime, may reduce mudball formation.

5.2.2. Terminal headloss

Both the filtration and backwashing results illustrate the potential disadvantages of backwashing on terminal headloss only. At high turbidities, turbidity breakthrough can occur before terminal headloss is reached while at low turbidities, excessively long run times may promote mudball formation. Monk and Willis[5] suggest that irreversible deterioration of filter media can occur during filter runs > 36 h, even for systems with auxiliary backwash. For water only backwash, the maximum run time should be < 36 h.

Pre-sedimentation of the coagulated raw water would decrease the solids loading on the filters and hence reduce the risk of breakthough but would increase the cost and complexity of the plant and also increase the risk of extended run times. Alternately, the terminal headloss could be decreased. For example, setting the terminal headloss at 1.0 m would decrease the risk of breakthrough in dual media filters DM#1 and DM#3 at influent turbidities of < 40 NTU. Decreasing the terminal headloss also reduces the run time. This would be an advantage at low alum doses and low coagulated water turbidities (Series 5 in **Section 4.2**). However, under these conditions, the headloss development in the laboratory filters was found to be non-linear with time, increasing in rate

towards the end of the run. Therefore decreasing the terminal headloss to 1 m would only have decreased the run time in Dual Media #3 from 50 h to 40 h.

In order to maintain filter performance over a broad range of raw water turbidities and coagulant doses whilst ensuring that the filter backwashes regularly, it is desirable to be able to adjust the terminal headloss as required. The position at which the self-actuating primer system connects to the backwash pipe determines the headloss at which the filter backwashes. Multiple ports on the backwash pipe with individual isolation valves could be installed to provide this flexibility. Decisions to adjust the terminal headloss could only be made by skilled personnel and should be based on breakthrough and run time data collected on site.

5.2.3. Backwash rates and volume

It may be possible to increase the efficiency of backwashing by increasing the volume and rate of backwash. The AVGF under evaluation in this study has a relatively small backwash reservoir of 2.2 m^3/m^2 compared to design recommendations of 3 m^3/m^2 to 4 m^3/m^2. **Figure 5.1** shows the volume of backwash available for which the media in the filter is fluidised, as a function of the minimum fluidisation velocity.

As described in the previous sections, the rate of backwash is governed by the head available for backwashing, which is determined by the instantaneous level in the backwash tank, the friction losses in the pipes, nozzles and filter media as well as the head required for acceleration of the fluid and the media. The backwash headlosses through the various filter components are presented in **Figure 5.2**.
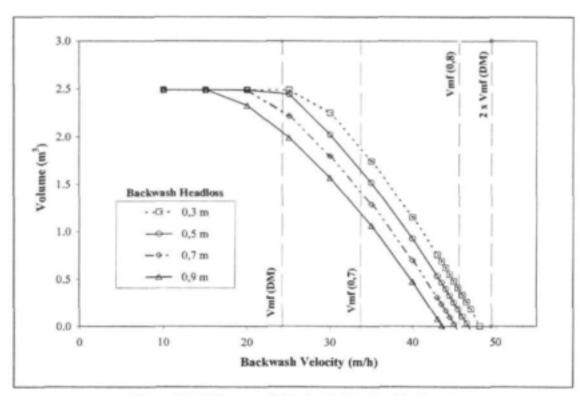
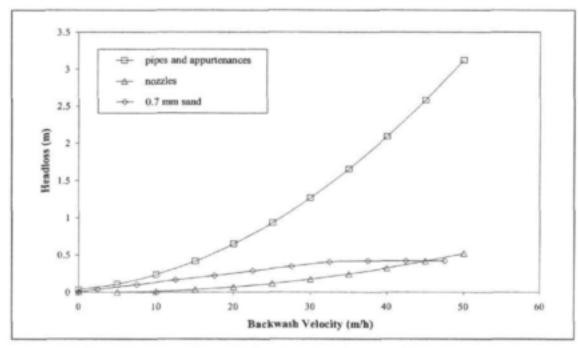Figure 5.1: Volume available for fluidised bed backwash

Figure 5.2   Backwash headloss through various filter components

Most of the headloss during backwashing occurs in the pipes and appurtenances and in particular in the backwash pipe itself. The backwash rates can therefore be increased by

either increasing the available head or decreasing the headlosses in the backwash pipe (by increasing the pipe diamater).

Figure 5.3 shows the effect of varying various design parameters on the rates of backwash. The media headlosses during backwash are a small fraction of the total so the mass of media used can be increased to reduce the risk of breakthrough without negatively impacting the backwash rates. Increasing the backwash reservoir height increases the backwash rates and backwash volume. The effect of increasing the diameter of the last section of the effluent pipe from 77 mm to 98 mm is also shown in **Figure 5.3**. This results in an increase in the initial backwash rates and a steeper decrease with time. The velocity at which backwash terminates is also nearly twice as high which would negatively impact the re-stratification of a dual media filter, but presumably not be a problem in a monomedia filter.
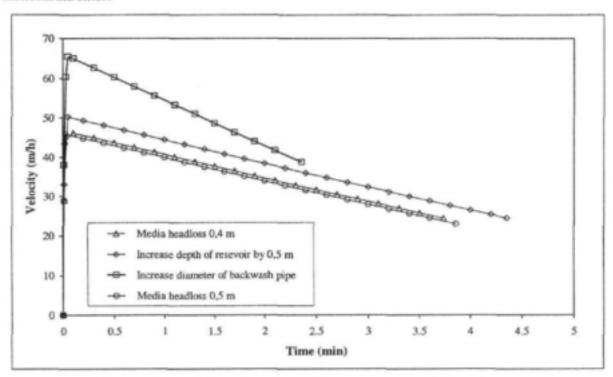

Figure 5.3.   Effect of varying design parameters on backwash rates

A potentially serious disadvantage, is that the wash rates towards the end of backwash are too low to affect solids removal and some portion of the wash water is essentially wasted. Increasing the minimum backwash rate would, however, require an increase in available head which implies a larger and more expensive filter.

5.2.4. Effect of design parameters on production rate and water recovery

Water recovery is defined as the percentage of the total volume of water filtered which goes out to service as opposed to the volume used to backwash the filter. The latter has to be treated and/or discharged and the amount of effluent produced has to be taken into account when designing a plant. Water recovery is a function of the filter run volume and the backwash volume as illustrated in Figure 5.4
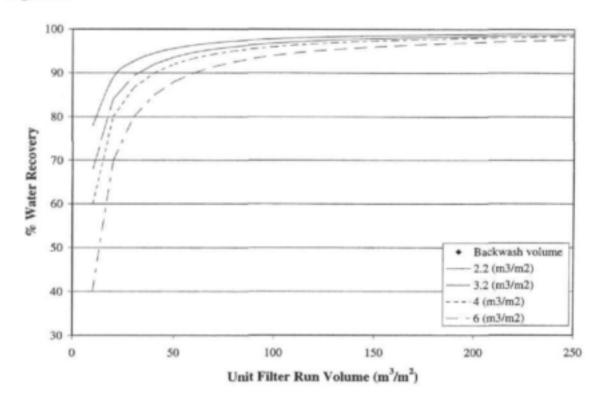

Figure 5.4 Water recovery as a function of Unit Filter Run Volume

For a 3.2 m^3/m^2 backwash tank (the minimum recommended for monomedia filters based on this study) a filter run volume of at least 65 m^3/m^2 is required to achieve a water recovery of 95 % or more. This can be compared to the filter run volumes for various conditions presented in Section 4.3. For all media designs, filter run volumes of around 100 m^3/m^2 or greater were typical for raw water turbidities < 15 NTU, providing water recoveries in excess of 96 %, with out excessively long run times in most cases. This would tend to justify the choice of 1.5 m as the terminal headloss under conditions where there is a low risk of premature breakthrough.

At higher influent turbidities, the rate of headloss development increases and the filter volume at which breakthrough occurs decreases. If the option to reduce the terminal headloss is considered then its impact on the water recovery and rate of effluent production should be taken into account. In the example given in Section 4.3, breakthrough in the 0.7 mm filter occurred at around 50 m^3/m^2. Lowering the terminal headloss to prevent breakthrough without modifying the filter bed would

reduce the water recovery to 93.6 %, that is 7 m^3 of backwash effluent would be produced per 100 m^3 of finished water. This may be acceptable to deal with short periods of high influent turbidity

5.2.5. Options for auxiliary backwash

Using occasional auxiliary backwash to supplement routine autonomous backwash may slow down the deterioration of the filter bed due to mudballing although it may not be able to breakdown existing mudballs (Martin, 1998). Air scour can be introduced in the system if an isolating valve is installed on the pipe connecting the backwash tank to the filter under drain. This arrangement will exclude simultaneous air scour and backwash because the backwash tank would be isolated during air scour. However, simultaneous air/water backwash may be possible by using water from a finished water storage reservoir.

Another option which may deserve further investigation is the use of surface or subsurface wash. During a manually initiated backwash, the raw water could be diverted into a system of surface or subsurface nozzles. Typical surface wash rates would be 7.2 to 9.6m/h (Kawamura, 2000). Due to the higher headloss at the nozzles, pumping may necessary.

Any auxiliary backwash procedures will be carried out by skilled personnel on a fixed schedule eg. once a week or once a month. The frequency of auxiliary backwash required to arrest deterioration of the filter bed needs further investigation. The cost of such a strategy would have to be compared to other cleaning options eg. chemical cleaning or more frequent media replacement.

6. Operation of the autonomous valveless gravity filter

The design and specification of the AVGF for application to rural water treatment depend on a number of factors with the main objectives being to ensure that the filters produce sustainable filtrate quality and quality with minimal operator intervention, cost effectively.

The functional description and operation of the AVGF is adequately treated in Chapter 3, under apparatus. Chapter 6 deals with the operating experience of the filter and the operating guidelines that arose from that experience.

6.1. Operating experience

6.1.1. Power requirements

The AVGF was operated from the process evaluation facility, where there was an available head of at least 10metres on the raw water delivery side. This driving force was sufficient for movement of the raw water to the inlet of the filter raw water reservoir. A hydraulically driven dosing pump (Dositron) was used for coagulant addition without electrical power requirements. Later in the project, an electrically driven metering pump was used for coagulant dosing due to some mechanical problems experienced with the dositron pump, In general no electrical power may be required if the placement of the filter ensures sufficient head at the raw water inlet and filtered water delivery.

6.1.2. Chemical Pretreatment

The filter was operated in the direct filtration mode. Operating the filter without coagulant, yielded poor filtrate quality. The use of organic coagulants resulted in irreversible accumulation of solids in the filter in the form of mudballs resulting in poor final water turbidities and short run times within three months of continuous operation. Alum and ferric chloride produced a more sustainable water quality and reasonable filtration run times. Initially, doses were kept as low as possible to effect charge neutralization and keep floc size as small as possible for direct filtration. Dose fluctuations using the dositron dosing pump varied between 25 and 30%. This variation did not significantly affect final water quality when inorganic coagulants were used. Later in the project, alum doses were adjusted to maintain filter run rimes around 24 hour.

6.1.3. Filtration cycle

The filtration rate was between 6 and 7 m/h. Without coagulant addition, the filter run time for a 0.8mm sand was approximately 20 hours for raw water turbidities between 1 and 10 NTU. During this period, filtrate turbidities less than 1NTU was rarely achieved (see Figure 4.5 in chapter 4). After coagulant addition (alum) the filter run time increased to between 20 and 60 hours. However, the increase in run time was mainly due to a reduction in filtration rate caused by the inclusion of the hydraulically driven coagulant dosing pump   As expected, an increase in raw water turbidity decreased filter run times, while lower raw water turbidities increased filter run times. The filtered water turbidity was typically below 1NTU by the time the back wash tank was full and the water began to flow to service. During steady state operation of the AVGF, with alum addition, final water turbidities <1NTU were consistently obtained for raw water turbidities ranging between 1.5 and 10NTU with an alum dose between 5 and 8 mg/l. During this time, final water turbidity was mostly affected by disruption of the dosing system. For dual media, similar final turbidities were achieved at 6 to 7 m/h filtration rate. However, filter run times ranged between 14 and 44 hours depending on the coagulant dose.

6.1.4. Backwash cycle

 When the maximum headloss is reached, the filter backwash is initiated automatically. The duration of the backwash cycle is 3.75minutes. The declining backwash rate peaks at a maximum of 46m/h, approximately 20 seconds after the onset of the backwash cycle and drops to a terminal velocity of about 22m/h at the end of the cycle (see Figure 4.2 in chapter 4). A visual inspection immediately after a backwash, through a site glass installed on the filter, indicated the presence of sludge on the surface of the media. As the backwash is initiated by the terminal headloss, fixed by the design, the operator has no control in setting the backwash frequency. However, the operator can manually initiate a backwash by opening a valve on a pipe connecting the backwash tank to the venturi. The length of time it takes to initiate manual backwash depends on how far the filter is from reaching terminal headloss. A visual indication that the filter is close to terminal headloss, is an increase in the water level in the effluent tank and the steady stream of bubbles from the venturi. Large air bubbles are periodically forced out from the backwash pipe due to the increasing water level in the pipe throughout the filter operation. This does not indicate imminent backwash.

6.1.5. Filter media

Both the 0.8mm sand and the dual media produced adequate filtrate turbidities for raw water turbidities less than 10NTU. However, the backwash rates were not sufficient to fluidise the 0.8mm media. A large amount of accumulated sludge was found in the media after 7 months of operation. Mudballing extended all the way down to the filter nozzles.

The dual media filter was operated for approximately a year before problems with short and erratic run times, unrelated to dosing problems, were experienced. Inspection of the filter during the one year of operation, indicated a continuous accumulation of mudballs in the anthracite layer. Interestingly, mudballing did not extend beyond the sand/anthracite interface. This was the significant difference in mudball distribution in the monomedia and the dual media.

6.2. Operational problems

6.2.1. Variable raw water quality

One of the main challenges for the maintenance a consistent final water quality is coping with changing raw water turbidities. Overdosing with alum is more forgiving (with respect to filtered turbidity) than underdosing but in the long term, excess solids in the filter generated by high doses of alum, result in shorter run times and poor water recoveries. As expected with any direct filtration process, the AVGF performs poorly when treating water with high raw water turbidities. During times of high rainfall and consequent turbulence and mixing at the abstraction point, the filter failed and had to be put off line. Results indicated that raw water turbidity greater than 15NTU was not suitable for treatment by the AVGF.

6.2.2. Coagulant type

The use of polymeric organic coagulants resulted in rapid and irreversible solids accumulation in the filter media. The available backwash volume and rate proved ineffective in removing the solids which had rapidly formed into large mudballs at various sections at the bottom of the filter. Inorganic coagulants, particularly alum produced favourable results.

6.2.3. Coagulant dose control

For optimum operation of the filter, coagulant dose required frequent monitoring, expecially during times of high variability in raw water turbidity.

6.2.4. Backwash water recovery

No attempts were made to recover the backwash water which was discharged directly into the sewer.

6.3. Operating guidelines:

- The dosing system should be regularly checked and cleaned to remove precipitates, grit etc.,
- Operators need to trained to adjust the coagulant dose when the filtrate turbidity falls outside certain limits. If the plant is operating in direct filtration mode and the deviation from the required limits is not too great then incremental adjustment of the dose with monitoring of the filtrate quality is the most convenient and accurate method of dose optimization. If there is a substantial delay before a dose change is reflected in the filtrate quality, then jar tests should be conducted.
- The filtrate quality should be checked at least once daily and the operator should ensure that the filter backwashes once a day. He or she can easily check whether the filter has backwashed since their previous visit by sprinkling sand on the weir of the effluent tank. The sand will be washed away when the filter backwashes.
- If sedimentation is used, the plant should be operated to maintain a filter influent turbidity of < 15 NTU
- If iron and manganese removal is required, the operator should check daily that Fe and Mn are completely oxidized before being fed to the filter.
- Adjustments to the terminal headloss should only be made by skilled personnel. Local operators may be trained to conduct manual backwashes if the filters do not backwash at least once a day and should advise their supervisors when this occurs.
- The backwash rates and/or duration should be checked every few months. Low rates and prolonged backwash durations indicate potential blockages or other hydraulic problems.

7. Conclusions

The aim of this research was to assess the suitability of autonomous valveless gravity filters (AVGF's) for use in rural water treatment schemes. It was hoped that the robust operation and automatic backwashing of this type of filter could offset or eliminate problems experienced with operator reliability. Various aspects of the design and operation of AVGF's were investigated and the following conclusions drawn:

7.1. Suitability of the AVGF for surface water treatment in rural areas

From the various experiments conducted on pilot scale filters and more experiments on the full scale AVGF plant, it was concluded that the AVGF system may be used in rural applications. However, the sustainable water quality and production rate is dependent on a number of operational issues that need to be understood and be given consideration. The findings are presented under these and other operational issues that have been specifically tested and evaluated.

7.1.1. Coagulant dosing

- In general coagulant is required to achieve acceptable filtered water turbidities. This means that the filter performance will only be as reliable as the dosing system.
- During the project the reliability of the dosing equipment was found to be poor when not attended to regularly.
- The hydraulically driven dosing pump tested during the project showed a variability of 25-30% in coagulant dose at constant raw water flow through the dosing pump. The variability was acceptable for dosing alum or ferric which have relatively broad operating ranges with respect to dose and pH.
- Local operators need to be trained to do jar tests and to adjust the coagulant dose when required.

7.1.2. Backwash efficiency

- Water-only backwash was insufficient to prevent the formation and accumulation of mudballs. However, for raw water turbidities of < 5 NTU, partially mudballed filters were able to operate for extended periods of time (approximately one year in this study) without operator intervention (except coagulant dosing) and without any detectable decline in filter performance.

7.1.3. Filter hydraulics

- No valve adjustment is required to maintain a constant filtration rate. This is a significant advantage with respect to plant monitoring requirements.

- Backwash velocities are controlled by the head in the backwash tank and the headlosses in the pipes, nozzles and media. Most of the backwash headloss (70 %) is in the last section of the backwash effluent pipe.

- The maximum backwash rate is determined by the size (height) of the backwash tank while the rate of decline is determined by the friction losses. Increasing the height of the backwash tank increases the maximum rate and duration of backwash but preserves the rate of decline of the velocity.

- Increasing the diameter of the backwash pipe increases the maximum backwash velocity but results in steeper decline in rate and shorter duration of backwash.

- The media headlosses make a relatively minor contribution to the total backwash headlosses.

7.1.4. Media selection

o It is imperative that the media selection is compatible with the available backwash rates for a given filter.

o Dual media filters had the advantage of longer run times with less risk of breakthrough at higher influent turbidities than mono-media sand filters.

o At low turbidities anthracite was more susceptible to mudballing than sand, however the mudballing was entirely restricted to the anthracite layer in dual-media filters whereas the mudballs in the mono-media filters gradually moved down towards the filter floor. Therefore it may be easier to manage mudballs in dual-media filters.

o Increasing the fraction of sand in dual media filters delayed turbidity breakthrough and shortened the run time.

o The smaller and lighter media particles are, the more prone they are to mudballing. Coarse deep bed filters may be a more robust alternative to dual media filters. However, the larger filter compartment and higher backwash velocities required would increase the cost of the design and coarser media could be more susceptible to breakthrough.

o The maximum bed depth is restricted by the height of the filter compartment, the maximum bed expansion at the coldest temperature expected, and the effect of clogging on the initial expansion of the filter bed.

7.1.5. Terminal headloss

The manufacturers of AVGF's claim that setting the terminal headloss at 1,5 m is adequate to prevent both breakthrough and mudballing. However, in this investigation:

- Backwashing at a terminal headloss of 1.5m did not prevent turbidity breakthrough at raw water turbidities greater than 16 NTU whilst using mono-media sand (0.7mm), or dual-media (DM#1).

- A terminal headloss of 1.5m also sometimes resulted in run times of up to 60 hours when operated under conditions of low turbidity and low coagulant dose. The literature indicates that run times in excess of 36 hours can result in deterioration of the filter media even when auxiliary backwash is used (Monk and Willis, 1987).

7.1.6. Raw water turbidity

- No turbidity breakthrough was observed in any of the filter or media combinations for a raw turbidity of <10 NTU.

- The dual media filter (DM#2) reached terminal headloss without breakthrough for raw water turbidities up to 26 NTU. The 0.5mm sand operated with similar performance under raw water conditions up to 40 NTU.

7.1.7. Backwash volumes

- The actual backwash volume of the AVGF evaluated in this study is 2,2 m^3/m^2 which is substantially less than the minimum recommended in the literature

- The effective backwash volume, defined to be the volume required to reduce the backwash turbidity to 10 NTU, was found to vary with solids loading and floc conditions. The effective volumes measured ranged from 2,3 to 3,1 m^3/m^2 for monomedia filters and 2,8 to 3,6 m^3/m^2 for dual media filters.

- The effective volumes for dual media filters are approximately 0,5 m^3/m^2 greater than those for mono-media filters although approximately the same amount of solids is removed from the filter. This appears to be due to lower detachment rates from anthracite grains as opposed to the denser sand grains.

7.1.8. Mudballing

- For water only backwash of both mono-media and dual media filters, the top layer of the fluidized bed consists entirely of relatively fine individual grains with floc deposits attached. The shear at the surface of the bed appears to be inadequate to detach these deposits and this layer may play a role in subsequent mudball formation.

- The evidence suggests that mudballing initiates near the surface of the bed and the mudballs sink into the filter as they grow larger. Anthracite mudballs collect sand as they sink and stop when they reach the dual media interface. Mono-media mudballs continue towards the filter floor and could end up clogging the nozzles.

- Skimming the surface of both the sand and anthracite to remove fines did not prevent mudball formation.

- Flow mal-distribution and deadzones promoted mudballing in certain regions.

- It is not clear to what extent intermixing of media played a role in the mudballing of the dual media. Some of the intermixing appeared to be a consequence rather than a cause of mudballing.

7.1.9. Power requirements

In general no electrical power may be required if the placement of the filter ensures sufficient head at the raw water inlet and filtered water delivery.

7.2. Operating guidelines

The efficient operation of the AVGF for the cost effective delivery of potable water depends on careful consideration of the raw water characteristics, suitable head for delivery of raw water to the filter and treated water storage and distribution. The filter media specification will also impact greatly on final water production and sustainable water quality. Also very important is the compatibility of the selected filter media with the available backwash rates and volume.

Operating experience indicates that AVGF requires operator supervision on a daily basis for efficient operation of the process. While the operator may not be expected to spend the whole day on site attending to the plant, he will need to ensure that chemical dosing and regular backwashing of the filter is taking place and that water quality and supply expectations are met through an appropriate monitoring programme.

7.3. Design modifications for improved reliability and cost effectiveness

After analyzing the components and dimensions of the AVGF and monitoring the filtration process under controlled conditions on a pilot plant scale and fullscale AVGF, some modification to the design and other considerations for design were suggested. Implementation of these suggestions will be influenced by cost and other factors like raw water variability, infrastructure, readily available technically competent operators. Although these are presented in some detail in chapter 5, the main conclusions are restated below:

- It is possible to provide facilities for manual auxiliary backwash and air scour.
- Additional tappings on the backwash pipe for adjustments to the terminal headloss.
- Dual media provides more protection against premature turbidity breakthrough at high raw water turbidities (> 15NTU) than monomedia filters.
- The volume of backwash water required is $3.1m^3/m^2$ for monomedia filters and $3.6m^3/m^2$ for dual media (anthracite/sand) filters.
- Filter nozzle density should be atleast 50 nozzles/m^2.
- The backwash tank should be covered.

8. Recommendations

The application of the AVGF for the supply of drinking water in the rural areas, considering the lack of resources and infrastructure and the level of readily available technical expertise, is a challenging prospect. Experience has shown that the AVGF cannot operate efficiently without operator intervention.

Recommendations arising from the technical investigations and operating experience of the AVGF plant have been dealt with in detail in chapter 6. In chapter 8, the more practical recommendations are presented.

- In situations where the inflow turbidity is greater than 15NTU and a large variations in raw water turbidity is anticipated, e.g., a river abstractions, a roughing filter or clarifier should be installed upstream of the AVGF. This would ensure a smoother solids loading profile, in addition to keeping the solids low enough for the filter to handle.

- Greater emphasis should be placed on ensuring that the material of construction of the filter should be corrosion resistant. This is most critical to the internals of the filter, where rust and scale deposits contaminate the final water and may also block the nozzles. Although mild steel may be preferred for its strength and cost, suitable coating of the insides with a corrosion resistant material should be considered.

- Due the inherent inadequacy of the 'water only' backwashing system, mudball formation and associated operating problems, will require more frequent access to the filter media. Filter media handling during loading of fresh media, inspection and removal of spent media can be significantly improved by increasing the size of the manholes. A thicker and more durable rubber gasket should be used to provide a better seal without damaging the nuts and bolts through excessive tightening.

- To increase the efficiency of backwashing and thus reduce mudballing, , the use of air scour is recommended. A valve has to be connected on the pipe that connects the backwash water tank to the underdrain compartment to prevent the air from bypassing the media and escaping through backwash tank into the atmosphere.

- Filling of the filter bed with water before loading the media, will reduce air entrainment and consequent filter binding. When using anthracite as a filter media, it is imperative that the anthracite is allowed to soak in water for atleast 24 hours before backwashing to prevent media loss. A further advantage of filling the bed with water first, is that the filter can also brought into service in a relatively shorter time.

- The inclusion of a chute for the removal of wet filter sand from the filter, will make the filter more ergonomically acceptable.

- Nozzle damage during sand inspection for mudballs or sand removal is a distinct possibility and should be avoided. When sand levels reach a few centimeters above the nozzle caps, extreme care should be exercised and the use of spades should be avoided.

- For safety reasons, the vertical fixed ladder mounted on the side of the filter for access to the top of the filter, should have hooped guards to prevent accidents due to slipping and falling off.

- Because the filter operation is not as labour intensive as manually backwashed rapid filters, they are recommended in situations where there may be a shortage of appropriately skilled labour.

References

1.   Allen, H S; "The Motion of a Sphere in a Viscous Fluid", *Philosophical Magazine J. Sci,* 5th Series, 50, (1900)

2.   Amburgey, J E; *Ph.D. Thesis,* Georgia Institute of Technology, Atlanta, (2002)

3.   Amirtharajah, A; "Design of Granular-Media Filter Units" Chapter 28 in *Water Treatment Plant Design for the Practicing Engineer,* R L Sanks, editor, Butterworth-Heinemann, Stoneham, MA , pp 675 – 737, (1978a).

4.   Amirtharajah, A; "Optimum Backwashing of Sand Filters", *J. Env. Eng. Div., ASCE,* 104(EE5), pp 917 -932 (1978b)

5.   Amirtharajah, A; "Optimum Backwashing of filters with Air Scour: A Review", *Water Science and Technology,* 27(10), pp.195 – 211, (1993)

6.   ASTM; "Concrete and Mineral Aggregates", *1985 Annual Book of ASTM Standards,* 04.02, American Society for Testing and Materials, Philadelphia, (1985)

7.   AWWA Committee Report; "The Status of Direct Filtration", *Journal of the American Water Work Association,* 72(7), p. 405, (1980)

8.   Baumann, E R; "Granular-Media Deep-Bed Filtration" Chapter 12 in *Water Treatment Plant Design for the Practicing Engineer,* R L Sanks, editor, Butterworth-Heinemann, Stoneham, MA (1978)

9.   Ceronio, A D and Haarhoff, J; *Die Evaluasie van Suid-Afrikaanse Filtermedia vir Diepbedfiltrasie,* Water Research Commission Report No. 472/1/94 (1994)

10. Cleasby, J L; "Filtration", Chapter 8 in *Water Quality and Treatment: A Handbook of Community Water Supplies* 4th. ed. American Water Works Association, pp 455 – 560, (1990)

11. Cleasby, J L, Arboleda, A, Burns, D E, Prendiville, P W and Savage, E S; "Backwashing of Granular Filters (AWWA Filtration Subcommittee Report)", *Journal of the Ameican Water Works Association,* 69(2) p. 115, (1977)

12. Cleasby, J L and Baumann, E R; "Selection of Sand Filtration Rates", *J. AWWA,* 54(5), p 579, (1962)

13. Cleaby, J L, Hilmoe, D J, Dimitracopoulos, C J and Diaz-Bossio, L M; *Effective Filtration Methods for Small Water Supplies,* USEPA Cooperative Agreement CR808837-01-0, NTIS No. PB84-187-905, Environmental Protection Agency, (1984)

14. Gregory, R and Zabel, T F; "Sedimentation and Flotation" Chapter 7 in *Water Quality and Treatment: A Handbook of Community Water Supplies* 4th. ed. American Water Works Association, pp 455 – 560, (1990)

15. Haarhoff, J; "Filtration", Chapter 12 in *Water Purification Works Design,* F A Van Duuren, editor, Water Research Commission, Pretoria, pp. 180 – 205, (1997)

16. Haarhoff, J and Malan, W M; "The Disturbance of Sand Filter Supporting Layers by Air Scouring", *Water SA.* 9(2), pp 41 – 48, (1983)

17. Ives, K J; "Testing of Filter Media", *J. Water SRT-Aqua,* 39(3), pp 144-151, (1990)

18. James M. Montegomery Consulting Engineers, Inc; *Water Treatment Principles and Design,* John Wiley and Sons, Inc, New York, (1995)

19. Kawamura, S; "Design and Operation of High-Rate Filters – Part 1", *J. AWWA,*67 (10), pp 535 – 544, (1975a)

20. Kawamura, S; "Design and Operation of High-Rate Filters – Part 2", *J. AWWA,*67 (11), pp 653 – 662, (1975b)

21. Kawamura, S; "Integrated Design and Operation of Water Treatment Facilities" 2nd ed., John Wiley and Sons, Inc., New York, (2000)

22. Larkin, J; *Personal Communication,* 2000.

23. Leva, M; *Fluidization,* McGraw-Hill Book Co., Inc., New York, (1959)

24. Martin, S R; "High Rate Granular Media Filtration", Chapter 8 in *Water Treatment Plant Design,* 3rd ed., American Water Works Association, pp 153 –192, (1998)

25. Massey, B S; *Mechanics of Fluids,* 5th edition, Van Nostrand Reinhold Co. Ltd., UK, p 453, (1983).

26. Monk, R D G and Willis, J F; "Designing Water Treatment Facilities", *J. AWWA,* 79 (2), pp 45 – 57, (1987)

27. Monk, R D G, Hall, T and Hussain, M; "Real World Design: Appropriate Technology for Developing Nations", *Journal of the American Water Works Association,* 76(6), pp. 68 – 74, (1984)

28. Robeck, G G, Dostal, K A and Woodward, R L; "Studies of Modification in Water Filtration", *J. AWWA,* 56(2), p 198 (1964)

29. SABS; *SM 829 – Sieve Analysis, Fines Content and Dust Content of Aggregates,* South African Bureau of Standards, Pretoria, (1976)

30. Schulze, C R and Okun, D A; *Surface Water Treatment for Communities in Developing Countries,* JohnWiley and Sons, Inc., (1984)

31. SPSS Inc; *SPSS for Windows,* Chicago, (2003)

32. Voortman, W J and Reddy, CD; *Package Water Treatment Plant Selection,* Water Research Commission Report No. 450/1/97 (1997)

33. Wen, C Y and Yu Y H; "Mechanics of Fluidization", *Chemical Engineering Progress Symposium Series 62,* 62, pp 100 – 111, (1966)

34. Walker, P; *Personal Communication,* (2000).

Appendix 1: Modelling the AVGF Backwash Hydraulics

The backwash rates in the AVGF are determined by the headlosses in the pipes, appurtenances, nozzles and filter media. This section describes the experimental determination of the headloss characteristics of the empty filter and the filter nozzles and the subsequent development of the backwash hydraulics the AVGF.

A1.1. Headloss in the pipes and appurtenances

The headlosses in the pipes and appurtenances were measured by initiating manual backwash in the filter after both the nozzles and the media had been removed and recording the rate at which the level of backwash tank dropped. The tank level was measured using the sight glass (**Section 3.2.1.2**) marked off in intervals of 5 cm. The time at which the tank level reached each marking was recorded using a 100-lap stop watch.

Once the siphon in the backwash pipe was established, the total head available for backwash, H, was equal to the difference in height between the water levels in the backwash and backwash effluent tanks. This is an unsteady state problem and therefore, at any given instant the available head equaled the sum of the friction losses and the head required to accelerate the fluid. The headloss due to an accelerating flow in a pipe is given by (Massey, 1983)

$$h - k\frac{u^2}{2g} = \frac{l}{g}\frac{\partial u}{\partial t}$$ .... (A1.1)

h	Total head loss (m)
k	Friction loss coefficient
g	Acceleration constant = 9,81 m/s^2
u	Velocity (m/s)
l	Length of pipe (m)

Since the AVGF consisted of a number of pipes with different diameters, it is more convenient to express **Equation (A1.1)** as

$$h - k'Q^2 = \frac{1}{g}\sum_i \frac{l_i}{A_i}\frac{\partial Q}{\partial t}$$ .... (A1.2)

Q	Volumetric flowrate (m³/s)
k'	Friction loss coefficient (s²/m⁵)
l_i	Length of pipe i (m)
A_i	Cross sectional area of pipe i (m²)

$\sum_i \dfrac{l_i}{A_i}$ was calculated to be 1920,4 m/m². It was found that the headloss of acceleration was neglible after the maximum flowrate had been obtained therefore, for the purpose of calculating the headloss characteristic, **Equation (A1.2)** reduced to

$$H = k'Q^2 \qquad \text{..............................} \qquad (A1.3)$$

H	Distance in height between level in backwash tank and crest above overflow weir (m)

Q was calculated from the rate of change of the level in the backwash tank. The tank level was marked from 0 when it was full to 2.2 m when it was empty so the volumetric flowrate was given by

$$Q = tank\ area \times \frac{\partial L}{\partial t} \qquad \text{..............................} \qquad (A1.4)$$

L	Marked tank level (m)

The measured time intervals between level markings were rather short (3 – 4 s) therefore small timing errors were amplified in the calculation of the numerical derivative, $\dfrac{\Delta L}{\Delta t}$. As a result, it was necessary to smooth the raw data before calculating the derivative. This was achieved by using Microsoft Excel to fit a polynomial expression to the tank level vs time data and taking the derivative of the polynomial. The order of the polynomial determines the properties of the calculated derivative. The choice of a binomial fit was rationalised as follows

$$H = const - L$$

$$\therefore \quad \frac{\partial L}{\partial t} = -\frac{\partial H}{\partial t} \quad \text{from \textbf{Equation (A1.3)} and \textbf{(A1.4)}}$$

$$H \propto \left(\frac{\partial H}{\partial t}\right)^2 \qquad \text{..............................} \qquad (A1.5)$$

It can be shown that **Equation A1.5** is satisfied if L is a binomial function of t, but not if it is a higher order polynomial. The difference in height between $L = 0$ and the water level above the

backwash overflow weir was 3601 m. Therefore :

$$H = 3,601 - L \qquad \text{.............................} \qquad (A1.6)$$

If $\quad L = at^2 + bt + c \qquad \text{.............................} \qquad (A1.7)$

then $\quad Q = Area \times (2at + b)$

a,b,c $\qquad$ Regression coefficients

k' in **Equation (A1.3)** is found by determining the regression constants a,b,c then using these to calculate Q, and further regressing H against Q^2.
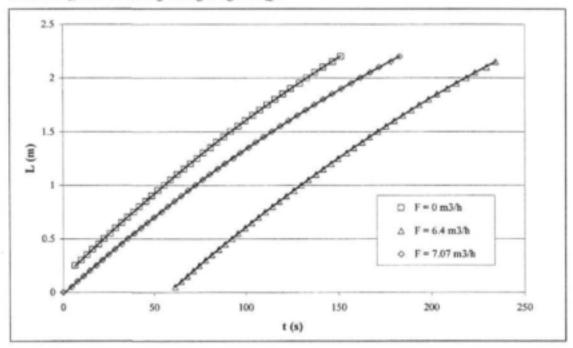

Figure A1.1   Backwash tank level as a function of backwash time

Under normal circumstances, the coagulated water feed remains on during backwashing with the feed water being diverted to the backwash pipe with the wash water. The headloss associated with the change of momentum of the bulk flow due to the entrance of the feed was found to be neglible due to the large diameter of the filter compartment. However, the increase in flow in the backwash pipe results in significant increase in the total friction loss.
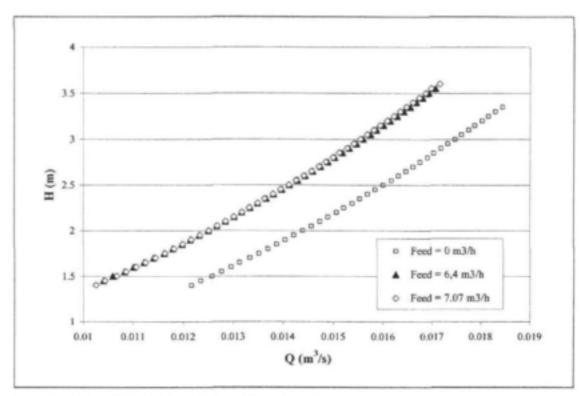
Figure A1.2   Empty filter headloss characteristic

In order to quantify the effect of the feed flow on the total headloss, the backwash profile was measured at three different feed rates: 0, 6.4 and 7.07 m³/h. **Figure A1.1** shows the tank level vs time data for manual backwash with the three feed flowrates. **Figure A1.2** shows the corresponding H vs Q relationships. It was assumed that the total headloss in the system can be modelled as a quadratic equation relating H, Q and F.

$$H = k'_1 Q^2 + k'_2 (Q + F)^2$$ .... (A1.8)

Q	Flowrate from backwash tank, m³/s
F	Feed flow, m³/s
k'_1	Friction loss coefficient for pipes and appurtenances up to feed inlet, s²/m⁵
k'_2	Friction loss coefficient for pipes and appurtenances after feed inlet, s²/m⁵

A1.2   Headloss through the nozzles

The headloss characteristics of the old AVGF nozzles, disc type strainers and new C2 type nozzles were determined using the apparatus shown in **Figure A1.3**. Each nozzle was screwed into the plate and the headloss between the two manometer ports determined as a function of flowrate through the

nozzle. The corresponding relationship between headloss and backwash velocity was established for the AVGF based on the number of nozzles installed in the filter.
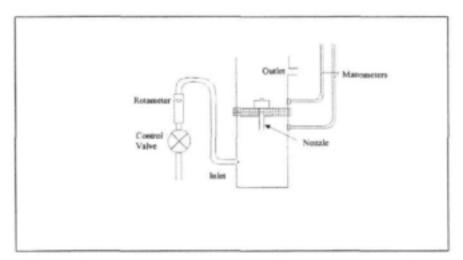

Figure A1.3   Nozzle headloss apparatus

A1.3   Prediction of AGF backwash rates

Using headloss measurements for the empty filter, nozzles and media, an unsteady-state, numerical model of the AGF was developed in Microsoft Excel. The algorithm used is presented in **Table A1.1**

Table A1.1   Model algorithms for calculating AVGF backwash rate vs time

$t = 0$	$Q = 0$	$V = \dfrac{Q}{A \times 3600}$
	$L = 0$	$H = 3{,}601 - L$
	$h = \text{eqn (3.8)} + \text{eqn (3.9 or 3.10)} + \Delta h_{max}$	
	$\dfrac{\partial Q}{\partial t} = g\left(\dfrac{H - h}{\sum \dfrac{l}{A}}\right)$	
	$Q_i = Q_{i-1} + \left(\dfrac{\partial Q}{\partial t}\right)_i \Delta t$	$V_i = \dfrac{Q_i}{A \times 3600}$

$t = t_i$ $i > 0$	$$Q_i = Q_{i-1} + \left(\frac{\partial Q}{\partial t}\right)_i \Delta t \qquad V_i = \frac{Q_i}{A \times 3600}$$ $$L_i = L_{i-1} + \left(\frac{Q_i + Q_{i-1}}{2A}\right)\Delta t \qquad H_i = 3{,}601 - L_i$$ $$h_i = \text{Equation (A1.8)} + \text{Equation (A1.9 or A1.10)}$$ $$+ \begin{cases} \Delta h_{man} & \text{if } V > Vmf \\ \text{eqn (2.?)} & \text{if } V < Vmf \end{cases}$$ $$\left(\frac{\partial Q}{\partial t}\right)_i = g\left(\frac{H_i - h_i}{\sum \frac{l}{A}}\right)$$

Δt	=	time increment = 0.5 s
Q	=	flowrate from backwash tank, m³/s
F	=	raw water feed, m³/s
V	=	backwash velocity, m/h
A	=	filter area, m²
L	=	backwash tank level, m
H	=	available head, m
h	=	total friction losses, m
$\dfrac{\partial Q}{\partial t}$	=	acceleration of the fluid, m³/s²
$\sum \dfrac{l}{A}$	= 1920.4 m/m²	

The model results are presented in **Section 4.2.1.**

Appendix 2:   Consistency of Coagulant Dosing

For rural or peri-urban water supply, there is often limited power supply and as it had been established that the AVGF required coagulant addition to maintain acceptable filtered water turbidity, the performance of the dosing system became important. Should the dosing system fail, the efficiency of the filtration process would be in jeopardy.

The *Dosatron* is a positive displacement pump where a piston is driven up and down by the hydraulic action of the fluid passing through the pump. The dosing rate is theoretically ratioed to the raw water feed flow. The consistency of the *Dosatron* operating in-line with the AVGF was monitored by regularly performing "drop tests" to establish the rate of coagulant addition. A *Magflow* flowmeter was installed in line with the dosing pump in order to regulate the flow to the pump and hence be able to alter the dose of coagulant. **Figure A2.1** shows the measured coagulant dose obtained for variations in the flow of water through the pump. There was a noticeable variation of the measured dose at different flowrates. In attempting to dose between 4 and 6 mg/l as alum, dosages of up to 8 mg/l and as low as 2 mg/l were recorded.
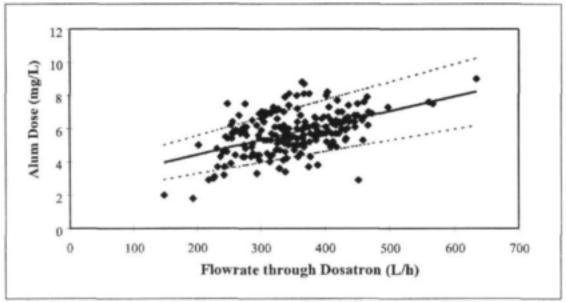

Figure A2.1   The relationship between the alum dose and flow through the *Dosatron* (whilst dosing from a 10,6 g/L Alum solution)

The fluctuation in measured dose was found to be between 25 and 30%. Where alum and metal coagulants are used for conventional treatment there is a large range of dose that can be used before

re-stabilisation occurs and filtration performance becomes affected, provided the water is well buffered and the pH remains above 6.5 to prevent dissolution of Aluminium. Where polymeric coagulants are used the doses are significantly lower and the range of dose required for effective control is much smaller. In such cases a variation of 25 to 30% in dose, where the required dose may only be 1,5 mg/l represents a significant enough variation to affect filtration performance. It is therefore recommended that the *Dosatron* pump only be used for this application when using alum or other metal coagulant and where fluctuations in dose can be tolerated.

During the filter operation the setting and maintenance of the pump was low as is typical of how such equipment might be operated in remote areas. The reported performance of the dosing pump therefore is believed to be a reasonable indication of what might be expected from the equipment in the field.

Operation of "pulsing" type chemical feeders has in the past been highlighted as a concern in rural or peri-urban applications due to the lack of subsequent backmixing for effective distribution in the bulk of the fluid (Voortman and Reddy, 1997). In this case, the results show that the performance with in-line dosing of this nature has been adequate. With final disinfection however, the production of water with adequate chlorine residual, where there is limited reservoir volume, may be more of a concern.

After a period of 18 to 24 months, without regular attention, the *Dosatron* dosing pump had to be replaced. The pump had over a period of time accumulated solids within the mechanism which resulted in extensive wear on the internal parts to the point that they were not worth replacing. During routine operation no procedure for flushing or maintenance of the pump had been performed. It is therefore recommended that regular flushing or maintenance be performed to prolong the life of the pump. This would have to be done by a suitably qualified person (probably the supplier) during which time a standby dosing pump would need to be used. During the design of such systems standby equipment (either in-line or on-the-shelf) must be provided.

The *Dosatron* pump was replaced with an electrically powered positive-displacement dosing pump. In the operation that followed the dose appeared to be more consistent, although after some time, clogging of the dosing line appeared to become a problem. As mentioned previously the reliability of the filtration process to produce and acceptable quality filtered water relies not only on the filter but also on the dosing equipment. Regular drop tests and checks on the dosing equipment are required for sustained operation.

Appendix 3: Statistical Analysis of Trends in Pilot Plant Performance

A3.1 Stepwise Multiple Linear Regression

Model selection techniques utilising multivariate linear regression provide a relatively simple way of determining which of a number of possible parameters are significantly correlated with filter performance and whether the correlations are positive or negative. The long-term filtration performance data was analysed using the stepwise multiple linear regression analysis tool in the statistical analysis package SPSS for Windows (SPSS Inc, 2001).

A3.2 Variables

The two performance parameters considered were filtrate turbidity and filter run volume. An increase in filtrate turbidity and/or decrease in filter run volume would be considered to be deterioration in filter performance. The inputs to the model selection process are summarised in **Table A3.1**.

Table A3.1 Variables considered in the stepwise regression model selection process

Dependent variable	Independent variables
Filtrate turbidity	Cumulative volume filtered Volume filtered Dose Influent (coagulated water) turbidity Filtration rate
Filter run volume	Cumulative volume filtered Dose Average filtration rate Viscosity (based on average of temperature measurements)

Note that pH was not considered, as there was some doubt about the accuracy of the meter calibration and some of the readings. Since the model selection procedure was based on examining linear relationships between variables, its use was valid only if the variables are actually linearly related or if the variations involved are sufficiently small that a linear model can be assumed with little error. This was clearly not true for filter ripening where filtrate turbidity first decreased and

then approached a constant value as the run progressed. Therefore, the filtrate turbidity data was split into four ranges of volume filtered (since the last backwash) based on the filter ripening curve. Each range was then analysed separately.

A3.3 Results

The results of the regression analysis and model selection are presented below. Note that cumulative volume filtered refers to the total volume of water filtered since the last air-scour backwash and volume filtered refers to filtrate produced since the last ordinary backwash.

The regression coefficients are the slopes of the linear dependence of the dependent variable on the independent variables. A positive coefficient indicates that the dependent variable increases as the independent variable increases and a negative coefficient indicates the opposite. The magnitude of the coefficient indicates the strength of the dependence. The standardised coefficients are scaled to allow direct comparison of the relative effects of parameters with different sets of units and orders of magnitude (e.g. viscosity and filtration rate). The significance is the probability that the apparent correlation between the independent and the dependent variable could be the result of random variations in the variables. (0 = 0 % probability and 1 = 100 % probability). Parameters were only included in the model if the significance was < 0,05 (95 % confidence that the apparent relationship is real.)

A3.3.1 Filtrate Turbidity

Table A3.2   Multivariate linear regression results for DM#3 filtrate turbidity

Range of volume filtered	Variables	Regression Coefficient	Standardised Regression Coefficient	Significance
0–3,2 m³/m²	Constant	0.768		0.000
$R^2 =$ 0.604	Influent turbidity (NTU)	0.222	0.626	0.002
	volume filtered (m3/m2)	- 0.18189 821	-0.565	0.004
3,2 – 19 m³/m²	Constant	-1.011		0.025
$R^2 =$	influent turbidity	0.105	0.632	0.000

0.457				
	volume filtered	-0.01162389	-0.401	0.000
	cumulative volume filtered	-3.0285E-05	-0.282	0.008
	filtration rate	6.974335691	0.281	0.002
19 – 32 m³/m² $R^2 = 0,336$	Constant	0.499		0.000
	influent turbidity	0.05417	0.528	0.000
	volume filtered	-9.05E-03	-3.39E-01	0.013
> 32 m³/m² $R^2 = 0,61$	Constant	-0.117		0.394
	influent turbidity	0.06656	0.596	0.000
	cumulative volume	-3.0568E-05	-0.49	0.000
	filtration rate	1.733844986	0.163	0.011

Table A3.3. Multivariate linear regression results for 0,7 mm sand filtrate turbidity

Range of volume filtered	Variables	Regression Coefficient	Standardised Regression Coefficient	Significance
0-3,2 m³/m²	constant	0.636		0.000
R² = 0,748	volume filtered (m3/m2)	-0.2463009	-0.65	0.000 .
	influent turbidity	0.311	0.557	0.000
3,2 – 19 m³/m²	constant	-0.881		0.006
R² = 0,505	influent turbidity	0.109	0.743	0.000
	cumulative volume filtered (m3/m2)	-4.335E-05	-0.483	0.000
	filtration rate (m/h)	0.199	0.32	0.000
	volume filtered (m3/m2)	-0.007477	-0.301	0.000
19 – 32 m³/m²	Constant	-0.959		0.004
R² = 0,729	influent turbidity	0.07488	0.914	0.000
	filtration rate (m/h)	2.23E-01	4.20E-01	0.000
	volume filtered (m3/m2)	-8.39E-03	-3.80E-01	0.000
	cumulative volume filtered (m3/m2)	-2.023E-05	-3.44E-01	0.005

> 32 m³/m²	Constant	-0.394		0.013
R² = 0,472	cumulative volume filtered (m3/m2)	-3.707E-06	-0.548	0.000
	influent turbidity	0.05888	0.491	0.000

Based on the standardised regression coefficients, influent turbidity was usually the most important factor controlling filtrate turbidity. This was because the dose was not adjusted as the influent turbidity changed. Dose was not selected as a significant factor because its variation was small. Filtrate turbidity improved as each filter run progressed and tended to decline with increases in flowrate. Where cumulative volume was selected as a significant factor, it always showed a negative correlation with filtrate turbidity i.e. an improving trend. This may have resulted from improvements in operating technique and/or the fact that up to a point, dirty media provides better removal efficiencies than clean media.

A3.3.2 Filter run volume

Table A3.4 Multivariate linear regression results for 0,7 mm sand filter run volume

Variables		Regression Coefficient	Standardised Regression Coefficient	Significance
Constant		-187.72641		0.002
Average	filtration rate	49.312	0.68	0.000
Average	influent turbidity	4.20544655	0.312	0.014

R² = 0,68

For the 0,7 mm sand filter the most important factor controlling the filter run volume appeared to be the average filtration rate. Filter run volume increased with both filtration rate and influent turbidity. Higher filtration rates may have led to greater penetration of floc and hence lower headloss development. Increases in influent turbidity without any increase in dose could have led to weaker floc which also penetrated more deeply into the bed. The accumulation of mudballs in the filter bed (represented in the model by cumulative volume filtered) did not appear to have a significant impact.

Table A3.5 Multivariate linear regression results for DM#4 filter run volume

Model	Variables	Regression Coefficient	Standardised Regression Coefficient	Significance
Viscosity allowed as an independent variable $R^2 = 0,891$	Constant	-556.132		0.000
	viscosity	4.40E+08	0.629	0.000
	filtation rate	47.24955 351	0.545	0.000
Viscosity excluded as an independent variable $R^2 = 0,752$	constant	-145.6		0.012
	cumulative volume filtered	7.44E-03	0.556	0.000
	filtration rate	43.29114 677	0.495	0.000

For the dual media filter, the stepwise regression analysis suggested the volume filtered to reach terminal headloss was a primarily a function of average filtration rate and temperature (viscosity).In this case, influent turbidity did not appear to be a significant factor in determining the run time. A possible explanation is that the deeper penetration of floc in the dual media filter means that the effects of changes influent turbidity and floc strength are spread more evenly through the filter depth than in the monomedia filter therefore have less impact on the headloss development.

The regression model suggested that the filter run volume increased with increasing viscosity. There is, however, no obvious physical explanation for this phenomenon. An alternate explanation is that the viscosity on average increased over time as did the cumulative volume filtered. Possibly, the numerical values for viscosity fit the linear regression model better than those for cumulative volume filtered and consequently the apparent dependency on viscosity actually indicates the effect of changes in the bed structure.

When viscosity was disallowed as a possible independent variable, cumulative volume was selected in its place. Run volume may have increased as a result of wash out of fines and increased channelling and wormhole formation as a result of mudballing of the anthracite layer. It is important to note, however, that these changes did not negatively impact either of the two performance parameters evaluated (filtrate turbidity and run volume).

A3.4   Conclusions

The overall conclusion drawn from the statistical analysis is that mudballing did not result in a decline in performance in the period over which the two laboratory filters were monitored. This may have been because the conditions of low influent turbidity and relatively short run times (< 24 hours) did not particularly challenge the filters so that lost of floc storage capacity and probable increase in channelling and worm hole formation did not pose a problem and would not necessarily have been the case if there had been a large surge in influent turbidity.

Other related WRC reports available:

Inhibition of biofilm regrowth in potable water systems

MNB Momba · S Ndaliso · MA Binda · N Makala

In South Africa, as elsewhere, the degradation of the bacteriological quality of water in distribution systems is currently one of the main problems facing suppliers of potable water. The major water quality objectives in water supply consist of the removal of microorganisms which cause waterborne diseases and the prevention of re-contamination of drinking water by theses organisms during distribution.

The need to provide adequate water supplies to communities is a major objective of the South Africa Government's Reconstruction and Development Programme. To ensure bacteriologically safe drinking water, drinking water producers try to avoid bacterial growth in a drinking water distribution system. This could be possible by identifying appropriate disinfection processes and pipe materials used for the distribution of potable water.

This project pursued the use of ozone or chlorine as the primary disinfectant followed by monochloramine or hydrogen peroxide as secondary disinfectant and assessing their ability to prevent the deterioration of the microbial quality of ozonated and chlorinated water in distribution systems as long as residual monochloramine or residual hydrogen peroxide persists.

It further evaluated the influence of chloramination and hydrogen peroxide disinfection on the inhibition of biofilm regrowth in ozonated water systems as well as in chlorinated water systems using a laboratory-scale system

Report Number: 1023/1/02   ISBN: 1 86845 925 X