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Water Research Commission



# Evaluation of a Filter Backwash Recovery Plant to Establish Guidelines for Design and Future Operation

Final report

prepared for the

WATER RESEARCH COMMISSION

by

JJ Linde

Scientific Services

Rand Water

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### EXECUTIVE SUMMARY

Spent filter backwash water is characterised by a high concentration of suspended solids. Algal cells, invertebrates, bacteria, viruses and protozoa trapped by the sand filters will be present in higher concentrations in spent filter backwash water compared to the clarified water. Recycling spent filter backwash to the head of works without treatment may upset the treatment process and/ or increase the risk of affecting the final water quality.

A filter backwash recovery plant (FWWRP) was constructed at Rand Water's Zuikerbosch Pumping Station to treat spent filter backwash to potable standard. The plant was commissioned in early 1998 and a monitoring and evaluation program was initiated in August 1998. The technology included a conventional treatment plant, consisting of pretreatment, coagulation, flocculation, high rate clarification, sand filtration and tertiary experimental treatment units such as ozone, granular activated carbon (GAC) and membrane filtration treatment. The aim of the project was to establish guidelines for the design and future operation of filter backwash recovery plants. Emphasis was placed on the removal of suspended solids, protozoa, algae and taste and odour causing compounds.

The suspended solids in the spent filter backwash were effectively reduced from 352 NTU to 0,18 NTU in the filtered water. Water with turbidity of between 5 and 10 NTU were fed onto filters and a filtered water turbidity of less than 0,3 NTU was consistently produced. The dosing of a filter aid ahead of the sand filters was therefore not required to achieve the target filtrate turbidity of 0,5 NTU. More particles were observed in the filtered water compared to the filtered water of the main process, although very similar turbidities were measured in the samples. The standard plate count of the filtered water was slightly higher than counts observed in the filtered water of the main treatment process at Rand Water. No protozoa cysts or oocysts were detected in the filtered water at the filter backwash recovery plant during the investigative period. Very high invertebrate numbers were detected in the spent filter backwash at times. Invertebrate numbers in the filter effluent exceeded the Rand Water recommended limit but complied with the maximum permissible limit. The chlorine demand of the filtered water was the same as in filtered water from the main treatment process. The average total organic carbon concentration of the filtered water was determined to be

3,7 mg/l. The levels of the inorganic species present in the filtered water were all within the SANS 0241 (2001) limits for Class O water. Algal enumeration and speciation yielded a variety of species present in the spent filter backwash at low levels. Algae were absent from nearly all of the samples, which were also reflected in the low concentrations of chlorophyll a. No taste and odour problems were experienced over the duration of the project. Spent filter backwash could therefore be treated with success to potable standard in a conventional treatment plant.

The optimum dosages of FeCl<sub>3</sub> and cationic polyelectrolyte were determined as 3,5 mg/ $\ell$  as Fe and 6 mg/ $\ell$  respectively. Anionic polyelectrolytes may perform better than the cationic polyelectrolytes but care must be exercised not to exceed the maximum permissible dosage. The recycling of sludge is advantageous to the efficiency of suspend solids removal in the clarifier. The best results were achieved when the highest density sludge was recycled. Turbidities of 5 to 10 NTU were fed onto the sand filter. Decreasing the upflow rate through the laminar plate settler could reduce the overflow turbidity to less than five NTU.

An ozone dosage of 0,36-mg O<sub>3</sub>/mg TOC increased the assimilable organic carbon (AOC) by 39%. A 5-minute empty bed contact time (EBCT) was not enough to reduce the AOC levels to that of the feed water. The TOC adsorption capacity of the granular activated carbon (GAC) was reduced within a short period (10 000 to 15 000 bed volumes) to reach a constant TOC<sub>effluent</sub>/TOC<sub>feed</sub>. The same pattern was observed for all the GAC steams. The trend was also observed for the UV absorbing substances, except that steady state conditions were reached at different UV<sub>effluent</sub>/UV<sub>feed</sub> ratios. More UV absorbing substances would be removed when ozonation precedes GAC. Chlorine demand was reduced quite significantly during GAC filtration.

Superior permeate quality was delivered by the microfiltration plant. The transmembrane pressure (TMP) developed quickly at a flux of 100 t/m².h (LMH) and a backwash interval of 30 minutes. Both acidic and caustic "cleaning in place" (CIP) procedures were required to remove the compounds responsible for fouling the membrane surface. Membrane treatment is still quite an expensive process in South Africa and high flux rates would be required for it to be competitive. The dosing of chemicals upstream of the membrane might have contributed to the blocking of the membrane. Poor control over the feedwater quality to the membrane

plant would reduce the life of the membranes and make it an even more expensive technology to use.

The increased risk of protozoa in the potable water may necessitate the use of technology capable of removing or inactivating protozoan oocysts. Recently published literature showed that protozoa could be inactivated at low dosages by both low and medium pressure ultraviolet (UV) radiation. An UV unit was tested to determine the performance limits of the technology. Standard plate counts of approximately 60 and 25 colony forming units per mt (cfu/mt) were obtained at dosages of 25 and 31 mJ.cm<sup>-2</sup> respectively. A minimum dosage of 41 mJ.cm<sup>-2</sup> would be required to produce a water quality that would comply with the production limit of 10 cfu/mt. Disinfected filtered water from the filter backwash recovery plant would be blended with filtered water from the main treatment process. Maintaining a high enough chlorine residual during the blending process would make UV disinfection a process option to minimise the risk of viable protozoa in the final water from filter backwash recovery plants.

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## Evaluation of a filter backwash recovery plant to establish guidelines for design and future operation

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## GLOSSARY

GAC - Granular Activated Carbon

PAC - Powdered Activated Carbon

SFWW - Spent Filter Wash Water

LAM - Flux f/m<sup>2</sup>.h

SPC - Standard Plate Count

MF - Membrane Filter

TMP - Trans Membrane Pressure

UV - Ultraviolet Light

U5000 - Polyamine Coagulant

CIP - Cleaning in Place

PDT - Pressure Decay Test

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#### INTRODUCTION

#### 1.1 BACKGROUND

One of the principle objectives of the water treatment and purification process is the removal of suspended solids. Suspended solids include clay and fine particular matter, colloids, organic and inorganic precipitates, algae, bacteria and viruses. The bulk of the incoming suspended solids present in the untreated water are removed during the sedimentation process in conventional water treatment. The remaining fraction of the suspended solids has to be removed by the filters. The trapped material would be released during backwashing of the filters and the spent filter backwash is normally recycled through different practices to the head of works. Spent filter backwash may contain pathogens like Giardia lamblia cysts and Cryptosporidium parvum oocysts. Recycling of backwash water would therefore increase the potential of these organisms being present in the mixed water, which would have implications for the final water quality. Giardia lamblia and Cryptosporidium parvum have caused outbreaks of disease affecting at least 450 000 people over the past three decades. The organisms cause diarrhoeal disease with symptoms lasting between one and two weeks, but with a weakened immune system (as with AIDS patients) the symptoms can be very serious and even life threatening [1]. The most severe outbreak where 403 000 people were affected occurred in 1993 at Milwaukee [2]. The cause of the outbreaks ranges from source contamination, plant breakdown and choice of treatment systems.

Rand Water reviewed it's waste water recycling practices during 1994 and considered various treatment options for spent filter backwash water. To develop treatment strategies of spent filter backwash the following implications were considered:

- Recycling of <u>untreated</u> spent filter backwash water to the raw water <u>feed at the head of the</u> works.
- Recycling of <u>untreated</u> spent filter backwash water to the <u>coagulator</u>/ <u>flocculator</u>.
- Recycle <u>treated spent</u> filter backwash water to the <u>head of the horizontal flow</u> sedimentation tanks.

- Recycle treated spent filter backwash water to the rapid gravity sand filters inlet
- Treat spent filter backwash to potable standard.

A 35 Mt/ day filtered backwash plant was constructed and commissioned during 1998 to treat the spent filter backwash from a block of filters with a total capacity of 1800 Mt/ day. A seven Mt/ day experimental plant, consisting of ozone, granular activated carbon (GAC) and membrane treatment was constructed and commissioned at the same time to investigate the application of ozone and the unit processes with special emphasis on the following:

- Removal and inactivation of Giardia lamblia cysts and Cryptosporidium parvum oocysts.
- Adsorption and oxidation of taste and odour producing compounds.

Excessive growth of algae in the raw water source was experienced during the mid-summer periods (Nov-Feb) of the early 1990's. Recycling of spent filter backwash during periods of algal blooms could increase the concentration of algal cells in the raw water. Lysis of reinocculated trapped algal cells could increase the potential of high levels of algal toxins and taste and odour causing metabolites in the finished water. To avoid this spent, filter backwash has to be released to the river for which discharge permits issued by Department of Water Affairs and Forestry (DWAF) are necessary. DWAF has issued a permit until 2003 to release spent filter backwash during periods of algal blooms. The discharge should not exceed the permit requirements in terms of suspended solids, electrical conductivity and pH. The treatment of all spent filter backwash at Rand Water would reduce the potential risks of recycling and result in a saving on raw water cost. The volume of filter backwash water is estimated to be 2% of the volume of raw water treated. Therefore if this water would be treated to potable quality in a separate system, the capacity of the plant would increase by that margin.

#### 1.2 OBJECTIVES

The objective of the project was to establish guidelines for design and future operation of filter backwash recovery plants with the emphasis on the following aspects:

- Removal of suspended matter.
- Removal of protozoa cysts and oocysts.
- Removal of taste and odour causing compounds and algae.

 Establish design, treatment requirements and operational procedures to produce water of potable standard.

Testwork was performed on-site (Zuikerbosch purification works) and samples collected were analysed at Rand Water's laboratories in Vereeniging. The intended work program included the following:

- Establish protocols to:
- Interpret data from particle size monitors.
- Evaluate and select powdered activated carbon (PAC) for the removal of taste and odour producing compounds.
- Monitor the bio-activity of granular activated carbon (GAC) preceded by ozonation.
- Determine the viability of Giardia cysts and Cryptosporidium oocysts.
- 2. Draw up programmes for effective monitoring of:
- The possible relationship between particle size and the presence of protozoa.
- Integration of present technology on algal speciation and enumeration to be applied in present project.
- 3. Plant trials would include the following:
- Optimisation of plant to remove contaminants effectively.
- Setting of ozone dosages to enhance biological growth on the GAC.
- Study the efficiency of PAC to remove taste and odours.
- Study the possible penetration of rapid gravity sand filters by PAC.
- Establish the purification efficiency of ultrafiltration.

The progress of the experimental work was hampered by mechanical and electrical failures. Electrical problems on the ozone plant limited the experimental work to only one ozone dosage. The ultrafiltration membrane was damaged by ozone and resulted in the delivery of a poor quality permeate. The ultrafiltration membrane was not replaced and the possibility of using microfiltration technology for the treatment of spent filter backwash treatment was investigated. After the commissioning of the filter wash water recovery plant and during the evaluation period no taste and odour problems were experienced at Rand Water. Therefore the removal of taste and odours through plant trials could not be determined. The PAC make-up system did not allow for the preparation of slurry without creating a health risk due to dust

levels in the room. As a result PAC carry-over to the sand filters and subsequent breakthrough could not be determined.

Rand Water was also interested in the ultraviolet disinfection as a unit process for the treatment of spent filter backwash, which could possibly provide a barrier against protozoa in the final water. A pilot plant unit was obtained and the performance limits of the unit determined. The results of the trial are included in this report. The intention was not to substitute any of the objectives but to supplement the report with relevant information (in the light of the previous statement) with regard to the treatment of spent filter backwash.

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### LITERATURE REVIEW

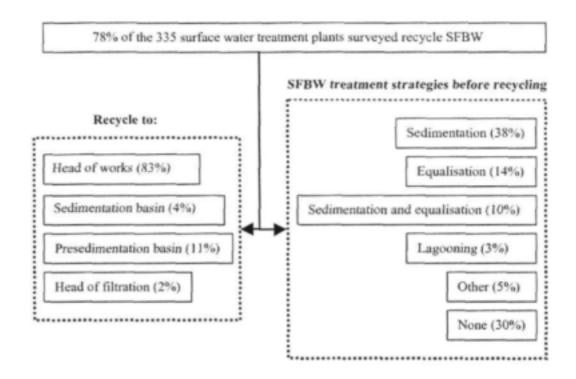
#### 2.1 RECYCLING AND TREATMENT OF SPENT FILTER BACKWASH

The quantity of suspended solids in spent filter backwash may vary between 0,1 to 1 g/t of dry solids depending on the applied solids loading to the filters [3, 4]. Spent filter backwash is currently handled as follows in South Africa at different water treatment plants [3].

- · Recycling to head of the treatment plant, as filters are backwashed.
- Collection and storage in a holding tank. Suspended solids are allowed to settle and discharged to the sludge dewatering or disposal site. As the level rises in the holding tank and overflows the supernatant is returned to the head of the treatment plant.
- Treatment in a dedicated plant comprising of coagulation and flocculation (ferricchloride and cationic polyelectrolyte), phase separation through lamellae plates, sand filtration and chlorination was commissioned at Rand Water to treat spent filter backwash to potable standard.

The filter backwash treatment philosophy of various plants in the USA and Europe is listed below.

 A survey was conducted by AWWA to determine the recycling practices of spent filter backwash water (SFBW) and the findings [5] are summarised in the flow diagram below.
 The average generation of SFBW was 2,5% and ranged between 2 and 10% of the volume of raw water treated.



- Mianus Water Treatment Plant, Greenwich, Connecticut. Sludge and filter backwash water flows to a 2 to 3 hour holding tank with supernatant recycle for 5 to 8 hours of the day [6].
- Kanawha Valley Water Treatment Plant, Charleston. Filter backwash is pumped periodically from a holding tank to the raw water line at the head of the plant. Sludge is discharged to the sewer [6].
- Coles Run Water Filtration Facility, Augusta County, USA. Backwash water from a microfiltration plant is disposed through a sub soil sand infiltration basin system. The backwash percolates into the groundwater through infiltration basins underlain with layers of sand and gravel [7].
- · Tilburg, Netherlands. UV disinfection of backwash water prior to recycling [8].
- Noordbargeres pumping station, Emmen, Netherlands. Treatment of backwash water to
  potable water with conventional treatment consisting of the following unit treatment
  processes: sand trap, coagulation and flocculation with FeCl<sub>3</sub>, phase separation with
  lamellae plates, double medium filtration (anthracite-sand) and UV-disinfection [9].
- Aalsterweg pumping station, Netherlands. Treatment of backwash water to potable water
  with conventional treatment consisting of the following unit treatment processes: a
  holding tank with one day's capacity, removal of settled sludge, coagulation and
  flocculation of the supernatant, phase separation with lamellae plates, single medium sand
  filtration and disinfection with chlorine.[10].

#### 2.2 POTENTIAL HEALTH RISKS ASSOCIATED WITH RECYCLING

A study conducted at Pittsburgh Drinking Water Treatment Plant found ten times higher Cryptosporidium oocyst concentrations in filter backwash water compared to raw water [11]. Sixty one times higher Cryptosporidium levels were found in spent filter backwash compared to the raw water in a survey of 34 water treatment plants [5]. Levels of Cryptosporidium oocysts in spent filter backwash determined at various plants in Europe and the USA ranged between 257 to 2 430 600/100 . Supernatant from settling tanks treating spent filter backwash also showed higher levels of Cryptosporidium compared to the raw water [5]. The US Environmental Protection Agency (USEPA) is required by the 1996 Safe Drinking Water Act amendments to set regulations for the recycling of spent filter backwash within the treatment process for public water systems [5]. The proposed regulations include all public water systems to:

- recycle SFBW upstream of a rapid-mix process,
- direct filtration plants to report to their respective states on its ability to treat SFBW,
- treatment systems with less than 20 filters have to perform a self assessment of their recycling practices for submission to their respective states.

The introduction of the proposed rule for SFBW illustrates the concern in the USA of concentrating contaminants through recycling practices beyond the level of a plant's multibarrier treatment capability. The effect of recycling waste streams can be illustrated through the consideration of various routes.

- Route 1 (recycling of recovered water from sludge thickening operations and untreated filter backwash).
- Route 2 (recycling of recovered water from sludge thickening operations and filter backwash treated through sedimentation).
- Route 3 (recycling of recovered water from sludge thickening operations and filter backwash treated through sedimentation and filtration).

The relative increase or decrease in protozoa numbers after repetitive cycles of recirculation compared to no recirculation is illustrated in Figure 1. A three percent recycle stream with a daily flow of 1000 Mt was used in the calculation to illustrate the effect of recycling in a conventional plant comprising of coagulation and flocculation, sedimentation, carbonation, filtration and chlorination. An assumption of one protozoa cyst or oocyst per m<sup>3</sup> of raw water

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was used. Cysts and oocyst removal efficiencies of 95% were assumed for the sedimentation and filtration processes, while 99% was assumed for the sludge thickening operation.

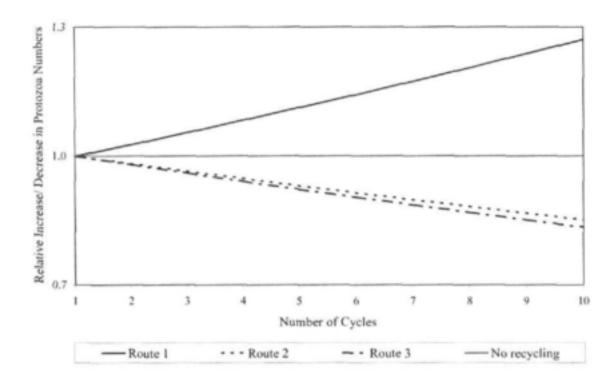


Figure 1: The relative increase or decrease in protozoa numbers after repetitive cycles through various methods of recirculation.

It is clear that the number of potential harmful organisms in the raw water could increase through the recycling of untreated filter backwash and other waste streams. Recycling filter backwash water to the raw water line could lead to increased concentrations of the protozoa in the water to be treated, or even upset the treatment process and may increase the risk of the protozoa in the potable water supply [3, 6].

In a study by Arora et al [5] to develop occurrence data for Giardia and Cryptosporidium raw and spent filter backwash samples were collected on six occasions from 25 water treatment systems. A filter backwash volume of 2% of the filter production rate was considered to determine a recycle rate that would not impact negatively on the quality of the raw water. Data from the study indicate a 1-log removal of protozoa in the unit processes prior to filtration. No treatment would be required for the spent filter backwash before recycling for a recycle rate of up to 10% when only the data of the samples that tested positive for Giardia and Cryptosporidium, are considered. The ratio of protozoa numbers in the spent filter

backwash compared to raw water was approximately 2,5 times higher when all samples were considered compared to only the positive tested samples. When all the samples were considered, the recycle rate would have to be less than 5% not to impact negatively on the quality of the raw water.

Not only will protozoa be concentrated, but also bacteria, viruses, algal cells, micro-pollutants and assimilable organic carbon that could give rise to unacceptable microbiological growth. Little literature is available that deals with these characteristics, problems and requirements for effective side stream recycle. Problems associated with recycling one or more streams range from settling of heavier material in the flocculation basin, under dosing of coagulant pumps as a result of streaming current detector control and negative impacts on clarification when recycle flow exceeds 5% of the plant flow [6]. A positive effect was observed at the Mystic facility in Connecticut by a saving in chemical costs as a result of reduced coagulant dose by streaming current detector control without adverse effects on clarification [6]. Regulatory authorities in the United Kingdom recommended the optimisation of water treatment plants with respect to coagulation, filtration and the recycling of filter backwash. The EPA to regulate protozoa in treated waters has followed a more aggressive approach, but the attempts are still hampered by poor analytical methods [14].

#### 2.3 INACTIVATION OR REMOVAL OF PROTOZOA

It has been shown in literature [12] that proper coagulation is required to achieve high removals of protozoa in water. Removals of 99,0 to 99,9% could be achieved in direct filtration with proper chemical conditioning [12, 13]. By reducing the turbidity of the final water to 0,3 NTU cyst and oocyct removal of better then 99,0% could be achieved. Disinfection is the final barrier in treatment process and consideration should be given to the likelihood of viable cycts and oocysts being present in the final water and the disinfectant employed. The effectiveness of various disinfectants can be illustrated (Table 1) by a comparison of the Ct values for Giardia cysts and Cryptosporidium oocysts (2-log inactivation at 10°C) [14].

Table 1: A comparison of the Ct values required for 2-log inactivation for four common disinfectants.

Giardia cysts (mg.min/t)	nin/t) Cryptosporidium oocysts (mg.min/t)	
1 230	14 400	
80	14 400	
15	160	
0,95	5 to10	
	1 230 80 15	

The comparison illustrates the superiority of ozone compared to the conventional disinfectants used in water treatment for protozoa inactivation. Numerous disinfection by-products are also formed by ozonation but common opinion holds that ozonation by-products are less toxic than the chlorination by-products [18]. Assimilable organic carbon is one of the by-products of ozone and should be removed with activated carbon [15, 18]. The phenomenon is also reported by other researchers [16]. The inactivation of Cryptosporidium by KMnO<sub>4</sub> and ClO<sub>2</sub> in spent filter backwash as determined by Arora et al [5] in both the presence and absence of suspended particles is summarised in Table 2.

Table 2: Efficiency of KMnO<sub>4</sub> and ClO<sub>2</sub> for the inactivation of Cryptosporidium as determined by Arora et al [5].

Disinfestant	Site	Ct (mg.min/f)	Degree of log-inactivation	
Disinfectant			With particles	Without particles
	New Jersey	2 400	0,16	0,29
VV-O	Pennsylvania	2 400	0,06	0,37
KMnO <sub>4</sub>	New York	2 400	0,40	0,14
		Average	0,21	0,27
	New Jersey	115	2,0	2,0
CIO	Pennsylvania	115	1,6	2,2
C1O <sub>2</sub>	New York	115	4,6	2,1
		Average	2,7	2,1

Ultraviolet (UV) radiation forms part of the electromagnetic spectrum and is effective against microorganisms in the wavelength range of 260 to 280 nm. The most severe damage to the nucleic acids of microorganisms occurs at 260 nm and is proportional to the UV dose and exposure time. UV alters the nitrogenous hetrocyclic components of DNA and RNA at 254 nm to form new bonds resulting in dimers that can disable microorganisms from replicating

[17]. Mouse infectivity studies indicated that a 3,9 log inactivation of *Cryptosporidium* oocyst could be achieved at a dose of 19 mJ.cm<sup>-2</sup> UV light with a medium-pressure system [18]. In vitro assays are considered as a poor indicator of infectivity and overestimate the UV dose required to achieve the required inactivation. UV doses of 41 mJ.cm<sup>-2</sup> resulting in inactivation greater than 4 log were found in the same studies. Recent studies comparing the effectiveness of low and medium-pressure lamps did not show significant differences in the inactivation of *Cryptosporidium parvum* [19]. Medium pressure UV doses of respectively 3, 6 and 9 mJ.cm<sup>-2</sup> resulted in log inactivation of 3,4 to > 4,0, doses of 11 and 20 delivered > 4,8 log inactivation. Three to 3,5 log inactivation were achieved with low-pressure UV doses (3, 6 and 9 mJ.cm<sup>-2</sup>), while 4,3 and > 4,9 log inactivation were achieved with low pressure UV doses of 16 and 33 mJ.cm<sup>-2</sup> respectively. Log inactivation of > 4,5 was achieved at a UV dose of 3 mJ.cm<sup>-2</sup> of oocysts not enmeshed within the sample matrix and added to a backwash water sample (11 NTU). The author cautions that naturally occurring oocysts would interact with particulate matter in the water and this could lead to shielding from the UV.

The latest studies in the Netherlands concentrated on the evaluation of membrane processes, particularly ultrafiltration and microfiltration for the treatment of filter backwash [10, 20]. Viruses are the smallest organisms ranging from 0,02 to 0,08 μm, followed by bacteria (0,5 to 10 μm) and protozoan cysts and oocysts (3 to 15 μm). Less than 1-log (90%) removal for the MS2 virus, better than 7-log (99,99999%) for bacteria and at least 4-log (99,99%) removal of *Giardia* cysts and *Cryptosporidium* oocysts had been achieved with microfiltration membranes under laboratory conditions [21]. The removal efficiency for particles and microorganisms not only depends on the pore size of the membrane, but also on the cake layer formed on the membrane surface. The cake layer, consisting of materials present in the feedstream, forms on the membrane surface and may provide additional removal capabilities. This phenomenon had been illustrated achieving 4-log removal for male-specific bacterial virus in an activated sludge effluent [21, 22]. No, or very little, dissolved organic carbon will be removed by microfiltration or ultrafiltration. Cross-flow microfiltration could replace conventional treatment of filter backwash in the Netherlands, as concluded by Vos *et. al.* [10] in a feasibility study.

#### 2.4 CONTROLLING ORGANIC CARBON AND MICRO-POLLUTANTS

Nutrient enrichment of raw water sources often leads to the proliferation of algae and the subsequent release of taste and odour compounds. Geosmin and 2-methyl isoborneol (2-MIB) are believed to be the major causes of customer complaints. The concentration effect of algal cells in filter backwash can increase the potential for high concentrations of organic carbon and taste and odour causing compounds being present in treated filter backwash. The formation of halogenated by-products upon chlorination of water with organic carbon is well known [23]. Precursors of THM products can be oxidised by ozone and removed by biologically activated carbon (BAC) [24]. The adsorption kinetics for MIB was found to be slow [25]. The carbon life was estimated to be less than 10 months at an EBCT of less than 13 minutes for the adsorption of MIB and geosmin. GAC life would increase to 18 months for geosmin and to less than 18 months for MIB respectively at initial concentrations of 60 and 30 ng/t and an EBCT of 20 minutes. Between 70 to 80% removal of geosmin was achieved in a GAC filter with an EBCT of 15 minutes at initial concentrations of 15,6 mg/ $\ell$ . Only 35% was removed in a conventional pilot plant (coagulation/ flocculation/ sedimentation/ filtration/ O<sub>3</sub>) in the same study [26].

Oxidation followed by filtration is generally the most effective for the control of taste and odours [16]. Ozone doses of 7 mg/ $\ell$  proved efficient for the removal of high molecular compounds as well as geosmin and MIB when followed by biological active sand filtration. It has also been shown through pilot plant studies that GAC alone could be more effective than an O<sub>3</sub>/ GAC process. The ozone by-products competed more effectively with geosmin and MIB for the GAC adsorption sites. Factors such as ozone dose, nature of the organics in the water and biological activity on the GAC would control this phenomenon.

#### 2.5 BIOLOGICAL FILTRATION

Microbiological activity in granular activated carbon GAC filters could result in the biodegradation of natural organic matter (NOM), which might increase GAC service life and thereby decrease treatment costs. Humic acids generally form the major component of NOM of which soluble fulvic acids dominate in most water supplies [4]. The biodegradability of the different components of NOM could vary drastically. A noticeable increase in the readily biodegradable fraction of the water could cause microbiological activity in GAC filters, which

would be of considerable value. As a result the intervals between regenerations, could be extended as much as fourfold compared to adsorption only [18].

Ozonation of water could alter the size and structure of NOM [27]. The readily biodegradable fraction of NOM generally increases due to a decrease in apparent molecular weight and aromatic character of the compounds that make up NOM. An increase in carboxylic acid functionality can also be expected upon ozonation. A substantial increase in the biodegradable fraction of NOM may increase the potential for bacterial regrowth in the distribution system [28]. The increased concentration of readily biodegradable fraction of NOM could be removed with biologically activated carbon [18, 29] to counter the increased regrowth potential. The extent of biodegradable organic matter (BOM) removal depends on the design specifications of the biofilter and the water quality. BOM levels in the effluent of the biofilter following ozonation often equal levels in the preozonated water [28]. Ozone dosages applied by investigators as summarised by Urfer et al [28] varied between 0,4 to 1,0 mg O<sub>3</sub>/ mg TOC.

The effect of filter media also received attention in the review on biofiltration by Urfer et al. [28]. Adsorptive (GAC) and non-adsorptive media (sand and anthracite) were investigated due to the potential cost saving in using a non-adsorptive media. Sand may provide a larger specific area available for biomass attachment due to its smaller effective size. However, the irregular surface and macroporous structure of GAC provide suitable attachment sites that offer increased protection from shear stress. GAC could also remove potential inhibitory compounds and retain slowly biodegradable compounds that can be broken down by the attached microbial activity. Determinations of attached biomass on GAC, anthracite and sand pilot plant filters indicated a three to eight fold more biomass than either sand or anthracite on the GAC [30]. A comparison of different base material GAC's indicated that lignite coalbased GAC yielded a higher biomass than wood-based or bituminous coal-based products. Carlson et al [31] investigated two GAC products for the removal of disinfection by-product (DBP) precursors. One GAC was coal-based with a high internal surface area as claimed by the manufacturer, suitable for the removal of dissolved organics. The other was a wood-based GAC primarily developed as substrate for microbial activity. Similar TOC removal patterns (16-19%), as a result of microbial activity, were displayed in the different GAC media. The coal-based carbon displayed more disinfection by-products precursor adsorptive removal capacity and considerable more water could be treated before reaching approximately the

same TOC<sub>effluent</sub> to TOC<sub>influent</sub> ratio. The coal-based carbon was therefore preferred due to its larger adsorptive capacity. The adsorptive removal mechanism would dominate when removal of high concentrations of precursors is required. Both carbons would be suitable as media when lower concentrations of DBP precursors are present and biodegradation would suffice and adsorption is not required.

The management of biomass in a biofilter during a filter cycle and the backwashing process would influence the success of biological filtration. Biological and non-biological particles are accumulated during a filtration run. The difference in detachment of the two groups of particles would determine the backwash philosophy for biofilters. Ahmad et al [32] investigated the strength of the attachment of the two groups of particles to media. The results from the study indicated that biological particles are held with greater force than the non-biological particles. A backwash strategy to remove the non-biological particles should therefore not result in excessive loss of the biofilm. Different backwash strategies were investigated through pilot plant studies by Ahmad et al [33] and a number of conclusions were supported by the results that are highlighted below.

- Air scoured backwash is not detrimental to biofiltration performance.
- Collapse pulsing backwash (air and subfluidised water flow) followed by water wash with 25% bed expansion produce water with low assimilable organic carbon (AOC) concentrations.
- Air and non-air scoured backwashes could produce effluents with similar effluent AOC concentrations.
- Air scoured backwashed filters produce higher initial TOC peaks during the ripening stage and higher initial headloss.
- Non-air scoured backwashes produced higher initial microbial counts in the effluent.
- Chlorine in the backwash water would seem to be detrimental to the quality of the effluent and biological quality in the filters.

Biomass accumulation may result in an increase in clean bed headloss between start-up and a steady state biomass condition. A water wash at 60% bed expansion is an insufficient procedure for biofilters due to the increased headloss between successive runs. The headloss increase between successive runs was insignificant with collapse pulsing followed by water at 20% bed expansion [28].

### TREATMENT TECHNOLOGY UNDER INVESTIGATION

A 35 ME/day filter backwash recovery plant was constructed to treat spent filter backwash from the sand filters with a total capacity of 1800 ME/day. The plant consisted of the following unit processes: coagulation, flocculation, solid/liquid phase separation and rapid gravity sand filtration. A brief overview is given in paragraph 0 with more detail available in Table 3.

The experimental pilot plants included ozone, granular activated carbon, microfiltration and ultraviolet disinfection. Filtered water from the filter backwash recovery plant was used as feed to the ozone, granular activated and ultraviolet pilot plants units. Clarified water after solid/liquid phase separation in the Denslator was used as feed to the microfiltration plant.

#### 3.1 FILTER BACKWASH RECOVERY PLANT

Flow to the plant is controlled by a variable speed pump that deliver spent filter backwash from a sump to a balancing tank maintaining a constant level in the tank. Gravitational flow is controlled at a rate of 1458 m<sup>3</sup>/h through the plant. The set point is entered *via* the SCADA. A minimum flow of 750 m<sup>3</sup>/h can be accommodated through the plant. A low water level (20% condition) in the spent filter backwash sump will switch off the pumps.

Taste and odours problems in the final water are controlled through the dosing of powdered activated carbon into the balancing tank. From time to time a manual discharge is required to remove the heavier settled particles from the balancing tank.

Sludge from the Denslator is returned to the first flash mixer to increase the suspended solids concentration in the spent filter backwash. Ferric chloride is dosed in the first of two identical flash mixers close to the submerged entry point at impeller height and cationic polyelectrolyte in the second flash mixer (See Table 3 for a schematic layout of the plant). The velocity gradient of the flocculator can be varied between 40 and 60 s<sup>-1</sup>.

The flocculated water enters the Denslator through five openings. The Denslator is equipped with lamellae plates to facilitate phase separation at relatively high up flow rates (13,25 m/h). Slow stirring of floc enriched suspension by means of a center drive picket fence scraper serves two functions. The vertical picket fence removes water trapped between flocs and thickens sludge mass. The horizontal scraper blades, together with the inclined floor also ensures that the sludge mass advances toward the center of thickener. A torque-limiting device protects the thickener drive. A sludge blanket monitor is installed in the thickener to control height of the sludge blanket. This information is important to control the sludge removal rate and selecting a draw-off point for recycling sludge. The sludge removal rate will be such to prevent changes in the density of the recycled sludge. A vertical distance of one meter between the bottom of the lamella plates and the top of the picket fence is maintained to prevent any carry-over of unsettled sludge in the clarified water. Sludge is discharge from the bottom of the Denslator to ensure that the sludge blanket is maintained at a constant level. Sludge is recycled from one of three points located at different heights in the denslator.

Clarified water from the Denslator feed the three declining rate sand filters. A rectangular flow-measuring weir is installed in the channel carrying water to the sand filters. Anionic polyelectrolyte as a filtration aid can be dosed at the weir if conditions require it. Water is channeled onto the filter beds by three sluice gates. A bypass weir is also installed to carry water directly into the filtered water sump. This will immediately shut down the ozone plant feed pumps. The pumps to the carbonation flume and head of works will be started to prevent any unfiltered water entering the potable water reservoir below the filters. The filtration rate is controlled by a pneumatically actuated control valve in the filtered water outlet pipe, interlocked with a level sensor situated above the filter. With water rising on the filter bed, the control valve opens gradually. If the water level decreases, the control valve will lower the filtration rate. Turbidity meters situated in the filtered water outlet pipe on each filter monitors the turbidity of the filtered water. The backwash is triggered by headloss in the filter bed, or on a timer setting.

To provide flexibility in filter operation, an operator can initiate a backwash at any time via the SCADA. With one filter in the backwash mode, the other two filters at an increased filtration rate of 10 m/h can handle the full flow. If for some reason one filter is out of service, the PLC will limit the flow through the plant so as not to exceed a filtration rate of 10 m/h. The backwash cycle consists of simultaneous air scour and water wash followed by

water rinse. The backwash sequence consisted of three steps: air scour (60 m/h), combined air and water scour (60 m/h and 12 m/h respectively) and water scour (24 m/h) only. To ensure the required filtered water quality at all times the first filtrate after backwash is rejected. The filtered water could either be pumped to the potable water sump, head of the sand filters in the main process or to head of works depending on the quality of the water.

Table 3: Schematic diagram and design details of the different unit treatment processes of the FWWRP.

	Balancing tank	Rapid mixing, flocculation and phase separation	Sand filtration
	PAC S To waste	FeCl <sub>3</sub> Cationic polyelectrolyte	Filter Aid  S
Description	PAC dosing point at the entrance of the balancing tank.  Tank has a volume of 400 m <sup>3</sup> .  Hydraulic retention time of 16.5 minutes at a flow of 1458 m <sup>3</sup> /h.	1st stage rapid mixing: Equipped with a mechanical mixer. G of 390 s <sup>-1</sup> , reactor volume of 36.5 m <sup>3</sup> , HRT of 90 seconds at design flow. FeCl <sub>3</sub> dosed in reactor. Recycled sludge (1 g/l) returned to pipeline feeding 1st flash mixer. 2st stage rapid mixing: Equipped with a mechanical mixer. G of 390 s <sup>-1</sup> , reactor volume of 36.5 m <sup>3</sup> , HRT of 90 seconds at design flow. Cationic polyelectrolyte dosed in reactor. Flocculator: Variable speed paddle-type mixer, G of 40 to 60 s <sup>-1</sup> , reactor volume of 200 m <sup>3</sup> .	Declining rate filters: Filtration rate of 7-m/h if all three filters are operational. Increase to 10-m/h with two filters operational.  Media: Sand of d <sub>10</sub> of 0.65 mm. UC < 1.5.  Filtered water: Can be directed to potable water sump if the turbidity < 0.5 NTU. Pumped to the head of the sand filters
		Denslator: Total surface area of 110 m <sup>2</sup> . Lamellae area of 55 m <sup>2</sup> divided in 5 rows of lamellae packs. Sludge recycle possible from different heights in denslator.	(main process) with a turbidity > 0.5 but < 5 (NTU). Pumped to head of works if turbidity is > 5.
Instrumentation	Ultrasonic level measurement of tank level. Flow measurement from tank.	Streaming current measurement in the 2 <sup>nd</sup> stage flash mixer, sludge blanket level meter on the denslator. Flow and turbidity measurement to filters.	Ultra sonic level measurement of head loss, turbidity, pH and particle index measurement of the filtrate.

#### 3.2 EXPERIMENTAL PILOT PLANTS

#### 3.2.1 Ozone and GAC

The ozone generator is fed with evaporated liquid oxygen at a controlled pressure. The generator has a capacity of 600 g/h, which would result in a maximum ozone dosage of 3 mg/t at a water flow of 194 m³/h. The ozone/ oxygen gas mixture is introduced into a 10 m³/h sidestream through a venture injector system. The stream then enters an absorption tank that facilitates the mass transfer of ozone into the water. The side stream is returned to the main stream ahead of an in-line static mixer. Undissolved ozone gas in the absorbtion tank is also released to the static mixer when the gas pressure in the absorption tank exceeds the set pressure. Contact time is provided in a vessel with four outlet ports at different levels representing four different contact periods (2,5; 5,0; 7,5 and 10,0 minutes respectively). The ozonated water is discharged to a sump from where it could be pumped to any of the three pressure GAC filters. The ozone-GAC flow configuration used in the study is illustrated in Figure 2 and further discussion will be restricted to the configuration used.

GAC filters 1 and 2 received ozonated water and will be referred to as BAC process 1 and 2 respectively. GAC 3 filter received filtered water from the filter backwash recovery plant and will be referred to as GAC process 1. Filters 1 and 3 were filled with a steam activated bituminous coal-based GAC<sup>1</sup> and filter 2 with an extruded steam activated peat-based GAC<sup>2</sup>. The EBCT of the filters at design flow was 5 minutes. The intervals between the backwashes could be set as required while each filter was equipped with a loss of head gauge. The GAC filters were never fed with water that exceeded a turbidity of 0,5 NTU. The maximum between backwash intervals was set at 999 hours and pressure and flow readings were recorded regularly to assess the blocking of the media.

Filtrasorb 400

Norit ROW 0,8 Supra

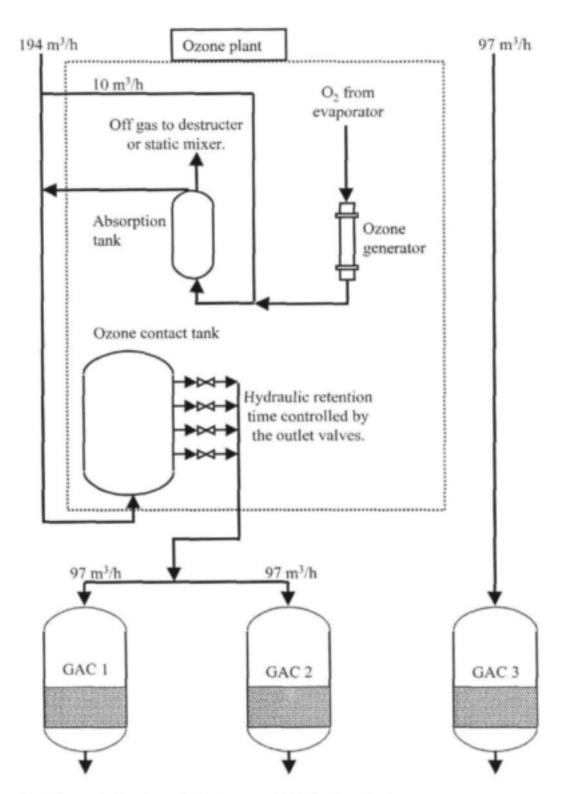


Figure 2: Schematic drawing of the ozone and GAC pilot plants.

### 3.2.2 Microfiltration

The suitability of microfiltration for the removal of suspended matter and micro-organisms was first assessed through the use of a unit with a flow rate of 200 l/h and a total membrane

surface of 4 m<sup>2</sup>. With this unit the performance limits with regard to permeate quality delivered by the membrane only could be assessed. The decline in flux over time, backwash regimes and "Cleaning in Place" (CIP) strategy could not be determined with this unit. The membrane modules had the same characteristics as the microfiltration units used in the pilot plant that are discussed in the next section.

The microfiltration pilot plant consisted of three polypropylene membrane modules with a nominal pore size of 0,2 microns. Each module had a surface area of 15 m<sup>2</sup>. It was equipped with a centrifugal pump and was run in direct filtration mode. The unit was also equipped with a programmable logic controller (PLC), feed and filtrate pressure transmitters, flow meter, conductivity meter, pH meter and temperature measurement. The pressure transmitters monitored the transmembrane pressure (TMP) and were connected to the panel view display mounted on the control panel and also to a datalogger. During filtration, water was drawn from the break tank by the feed pump and enters the modules at either end. Water flows along the outside of the hollow fibres and is forced through the walls of the fibres trapping almost all solids and particulates greater than 0,2 microns in size. The filtrate flow was maintained by the variable speed pump that is controlled to maintain the set point value and adjusted through the user interface. The majority of fouling that builds up on the membrane surface during filtration will be removed by backwashing. A decrease in TMP will be observed after each backwash. Regular backwashes were initiated by a timer with a backwash cycle duration of 2 to 3 minutes. Over time, the fouling that was not removed by backwashing will cause the TMP to increase to a point where the system requires a chemical clean. The set point for cleaning was 130 kPa TMP. During the backwash cycle highpressure air was forced through the membrane wall in the opposite direction to filtration flow. The high-pressure air expands the fibres and dislodges particles that have built up on the fibre surface. Feed water, passing along the fibres from bottom to top was used to carry the dislodged particles out of the filter module to the backwash tank. The flow of feed water is known as the backwash sweep and was set at a minimum flow of 4000 U/h per module.

Chemical cleaning was periodically required to remove fouling that was not removed by the backwash. The primary CIP regime was carried out using an acidic solution and a proprietary cleaning agent supplied by the membrane manufacturers to remove primarily inorganic deposits. The cleaning agent is a mixture of surfactants and sequestering agents. A secondary CIP regime was employed by using a caustic solution to remove inorganic impurities. The

cleaning solution was prepared using filtrate in the constant level tank which was used only once before disposal. The CIP sequence is semi-automatic, requiring the operator to initiate the cleaning cycle and add the chemicals as required. The operation of the chemical clean is then automatic until the end of the cycle when the unit must be manually reset for service.

The integrity of the membrane filters on the MF unit was determined through a pressure decay test (PDT). During the test the lumen side of the membrane is drained of liquid and pressurized with air to approximately 100 kPa, which is below the bubble point of the membrane. Once the test pressure has been reached the filtrate (lumen) side is sealed and the feed side vented to atmosphere. The drop in filtrate pressure after the second minute with time was monitored. The pressure decay will be directly related to the flow of air across the membrane and hence system integrity. A PDT result of smaller than 5kPa/min is considered acceptable. If the pressure decay is greater than 5 kPa/min the integrity of the system might have been compromised.

### 3.2.3 Pilot plant setup

The smaller micro-filtration unit was fed from three different sources of the filter backwash plant namely: spent filter backwash, clarified recovered filter backwash and filtered recovered filter backwash. The unit was not fully automated, which made 24-hour operation difficult however continuous operation was not required to assess the permeate quality performance limits of the membranes.

When the micro-filtration plant was fed from the clarified stream at the filter backwash recovery plant the 400-µm in-line strainer on the micro-filtration unit became clogged. A 500-µm screen was installed in the feed to the micro-filtration unit to remove the coarse material. The coarse material collected consisted mainly of invertebrates and to a lesser extent of debris like small stones, grass and leaves.

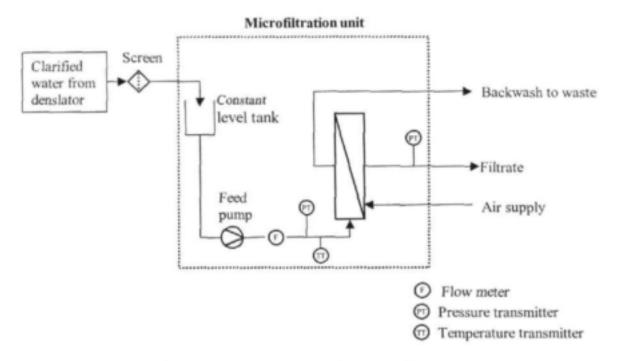


Figure 3: Schematic drawing of the micro-filtration pilot plant set up.

After commissioning the unit was run for short periods (24 to 48 hours) at different flow rates with a backwash interval of 45 minutes while the rise in TMP was observed. An acid and caustic CIP were performed and the membrane integrity was determined through the PDT-test before the start of the longer test runs. The unit was set to a flow rate of 5000 U/h with backwash intervals of 45 minutes. Changes in flow rate and backwash intervals were made as required to increase the requirements to perform CIP at acceptable frequencies.

## 3.3 ULTRAVIOLET DISINFECTION

An experimental medium pressure UV-system was offered by a supplier to evaluate.

Ultraviolet disinfection might be a viable disinfection option for the treatment of spent filter backwash to potable standard due to its ability to inactivate protozoa at low dosages.

## 3.3.1 General description of the ultraviolet test units

The ultraviolet system tested consisted of two 80-mm in-line units, each fitted with a medium pressure lamp. Each UV-unit consisted of a stainless steel capsule, which houses the UV lamp and an electrical control system. The energy output is displayed by indicator lights on the front panel of the control unit. The function of the indicator lights in a full-scale plant is

to indicate to the operator whether the UV output is still within the design dosage range. An adjustment would be required to set the energy output to 125% of the lamp's life to start off. Continuous adjustment is required, as the lamps age to maintain a power output of 125% and a reduction in power output to 100% would necessitate the replacement of the lamps. The size of the lamps allows them to be placed perpendicular to the flow in a contact chamber to achieve optimal hydrodynamic conditions as claimed by the manufacturer. As a result the difference between the minimum and maximum UV dose that the water receives would be minimal.

Three energy control levels, which consumed 350, 418 and 456 watt respectively could be selected on the first unit. That allows for adjusting the UV dosage within a small range without changing the flow rate through the unit (see Table 6). In the second unit the energy consumption was set at 350 watts. The effective UV output of the lamps at the beginning and end of their life is 50 and 40 W respectively with an input of 350 W. The flow rate and the transmission of the water determine the UV dose. Table 4 was provided by the manufacturers, indicating the dependence of UV dose on the water flow rate and the percentage transmission of the water.

Table 4: Flow rate and percentage transmission combinations to achieve an UV dose of 31 mJ.cm<sup>-2</sup> at the beginning of the lamp's life and 25 mJ.cm<sup>-2</sup> at the end of the lamp's life.

%Transmission	Flow rate (m <sup>3</sup> /h)	%Transmission	Flow rate (m <sup>3</sup> /h)	%Transmission	Flow rate (m <sup>3</sup> /h)
100	22	72	10	44	5
98	21	70	9	42	4
96	19	68	9	40	4
94	18	66	8	38	4
92	17	64	8	36	4
90	16	62	7	34	3
88	15	60	7	32	3
86	14	58	7	30	3
84	14	56	6	28	3
82	13	54	6	26	3
80	12	52	6	24	2
78	11	50	5	22	2
76	11	48	5	20	2
74	10	46	5		

## 3.3.2 Pilot plant setup

U-PVC pipes and fittings were used to construct the pilot plant to test the efficiency of UV as a disinfectant for filtered filter backwash. The two UV-units were bolted in series onto each other with the variable output unit first in line. Filtered water was pumped from a sump to the units and the flow through the unit was controlled by opening or throttling a bleed valve feeding back to the sump. A warm-up time of 15 minutes and stabilisation period of 5 minutes between changes in the energy were used in all the experiments performed. An adjustment was made at the start of the trial to set the energy output of both UV units to 125% at the lowest energy control setting. The UV energy output was maintained at 125% in both units for the duration of the trial. Feed water samples were taken at a sampling valve placed at a distance of one meter upstream of the first UV-unit. Samples after exposure to various dosages were collected at a sampling point 0,5 metres downstream the second UV unit. Samples were analysed for standard plate count, total coliforms, faecal coliforms, E coli and bacteriophages. The percentage transmission of the water before and after UV exposure was determined in the spectrophotometer at 256nm. All the samples were taken and transported in accordance with the procedures prescribed in Standard Methods.

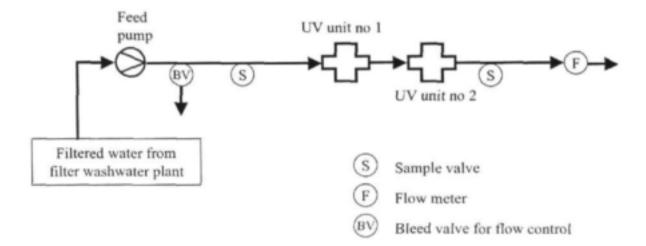


Figure 4: Schematic diagram of the ultraviolet experimental set up.

The effect of the different UV control settings, as displayed in Table 5, was tested at flow rates of 5, 15 and 25 m<sup>3</sup>/h.

Table 5: Settings used during the UV trial at different flow rates.

		UV unit no. 1				
Test	Energy control Energy control level 1 level 2		Energy control level 1	UV unit no. 2	Dosage factor	
1	ON	62653500		OFF	1.0	
2		ON		OFF	1.2	
3	STATE OF THE PARTY		ON	OFF	1.4	
4	ON			ON	1.0 + 1.0	
5		ON		ON	1.2 + 1.0	
6	SUBTRICTION.		ON	ON	1.4 + 1.0	

Considering the percentage transmission on flow rates (Table 4) a dosage factor of 1,0 would deliver a UV dose of 31 and 25 mJ.cm<sup>-2</sup> at the beginning and end of the lamp life respectively. Lamps with only a few hundred hours of operational service were used. These would therefore deliver an UV dose of 31 mJ.cm<sup>-2</sup> at the 1,0 dosage factor setting according to the percentage transmission based flow rate. The 1,2 dosage factor would deliver an UV dosage of 37 mJ.cm<sup>-2</sup> at a flow rate according to Table 4. The (1.2 + 1.0) dosage factor would deliver an UV dosage of 37 mJ.cm<sup>-2</sup> for the first unit and 31 mJ.cm<sup>-2</sup> for the second unit at a flow rate according to Table 4. The effectiveness of three UV dosage ranges were tested during the trial as indicated in Table 6. No compensation could be made for the reduction in % transmission during the different runs that resulted in small deviations in UV dosages applied during the actual trial runs.

Table 6: UV dosages aimed for during the trial.

	Flow (m <sup>3</sup> /h)	UV dosage (mJ.cm <sup>-2</sup> )				
	Plow (m /n)	Energy control level 1	Energy control level 2	Energy control level 1		
1	5	93	112	134		
2	15	31	37	43		
3	25	19	22	27		

4

## RESULTS AND DISCUSSION

## 4.1. SPENT FILTER WASH WATER QUALITY

Table 7 was compiled from data gathered over a period of 18 months. Grab samples were taken from the FWWRP and analysed on a weekly basis. The data for the determination of the suspended solids/ turbidity ratio were collected over a shorter period. Samples were collected from a point after the balancing tank and before the first flash mixer.

Table 7: Spent filter backwash quality at system 4, Zuikerbosch pumping station.

Minimum	Maximum	Average	Median
360	12800	6580	0001
2	1600	322	3
2	100	23	21
1	84	23	21
6	23020	2462	473
8,02	9,37	8,50	8,49
23,2	37,5	25,7	25,2
88,9	>1000	352	
66,6	981	336	
0,57	1,40		
0,65	3,60	2,01	1,80
78	366	145	133
3,4	20,6	5,4	4,5
	360 2 2 1 6 8,02 23,2 88,9 66,6 0,57 0,65 78	360 12800 2 1600 2 100 1 84 6 23020 8,02 9,37 23,2 37,5 88,9 >1000 66,6 981 0,57 1,40 0,65 3,60 78 366	360         12800         6580           2         1600         322           2         100         23           1         84         23           6         23020         2462           8,02         9,37         8,50           23,2         37,5         25,7           88,9         >1000         352           66,6         981         336           0,57         1,40           0,65         3,60         2,01           78         366         145

<sup>\* 4</sup> mg/t Chlorine - 15 minutes contact time

The suspended solid concentrations correlate well with determinations done by Degremont SA during the planning and design stage (Jun'96) of the filter backwash plant. At that time the average suspended solids concentration on three occasions were respectively 348, 375 and 303 mg/t. The results are attached to the appendix (Table 22, 23 and 24).

As shown in Figure 5 the ratio between suspended solids and turbidity changes continuously and therefore no constant ratio could be calculated. The water treatment chemicals used for coagulation depended to some degree on the maintenance program, equipment failures and availability of chemicals. Hydrated lime (60 to 80 mg/t as CaO) and activated sodium silicate (1 to 3 mg/t as SiO<sub>2</sub>) are the main coagulants used at Rand Water. Other coagulants used included combinations of hydrated lime and FeCl<sub>3</sub>, hydrated lime and cationic polyelectrolyte and cationic polyelectrolyte only. The nature of the suspended solids in spent filter backwash will be a function of the coagulants employed in the main treatment process. Turbidity is influenced by the properties of particles, which include size and refractive index and could offer an explanation for the wide variance and range of suspended solids to turbidity ratio in the spent filter washwater.

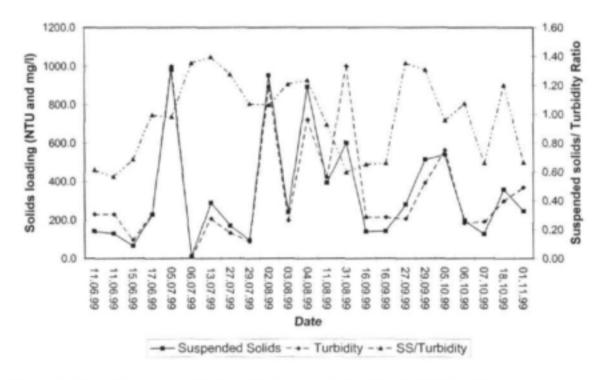


Figure 5: Graphical representation of the suspended solids/ turbidity ratio.

# 4.2 TREATMENT OF SPENT FILTER WASH WATER BY CONVENTIONAL TREATMENT

#### 4.2.1 Pretreatment

Screens are essential to prevent damage to pumps as a result of blockages due to debris (bottles, tins and plastic bags) and natural matter (leaves and filamentous algae). Screens were installed at the openings to the Denslator in the current filter backwash plant to prevent blockages of the sludge disposal pumps. The flow patterns in the Denslator are disturbed by the unequal flow through the openings to the Denslator caused by clogged and partially clogged screens. Screens should therefore be installed upstream of the plant to ensure optimum performance of the different unit treatment processes. Heavier silt and sand particles can be trapped and removed in the balancing tank.

Taste and odour problems can be treated by the addition of PAC upstream of the plant. Intermittent dosage may be required due to the seasonal occurrence of taste and odour problem. The hydraulic residence time from the entrance of the balancing tank to the entrance of the Denslator is 27,5 minutes. A longer period is required for the adsorption reaction to reach equilibrium but a high percentage of the taste and odour compound is removed at a relatively short contact time with little added benefit at longer contact periods. This phenomenon was observed for the removal of the taste and odour compound geosmin from Vaaldam water and contact times as short as 10 minutes proved cost effective. The difference in removal efficiency was also determined as a function of different water types. Testing PAC under the same conditions as Vaaldam and spent filter backwash showed similar geosmin removal efficiencies. Several authors [34, 35, 36, 37, 38 and 39] reported on the effect of water treatment chemicals on the removal of taste and odour molecules by PAC. The effect of treatment chemicals on the removal of geosmin from Vaalriver water indicated that higher dosages of the polyelectrolytes are required compared to lime activated sodium silicate process [39]. All these factors should be considered and tested in the design of PAC dosing systems to achieve the desired result. A dosage of 12 mg/t PAC would be required for the particular PAC selected for use at Rand Water to remove the taste and odour compound, geosmin, from the raw water source. A slightly higher dosage of PAC could be required at the filter backwash plant if the same activated carbon is used.

PAC selection procedures for the removal of the taste and odour compounds, geosmin and 2-MIB could be very costly. Surrogate methods are a very poor indicator of the adsorption capacity for the compounds mentioned [39] and the determination of adsorption isotherms are recommended. Carbons could then be compared on a cost-effective basis. The procedure for the determination of the adsorption isotherms and comparison of the carbons are as follow:

• Jar test methodology is used to simulate the treatment process. Untreated process water is spiked with the taste and odour compounds to levels encountered in the full-scale plant. The efficiency of different dosages (0 to 25 mg/t) of PAC is determined in the process. The recommended jar test procedure for the evaluation of powdered activated carbon for the filter backwash recovery plant is described in Table 8. The method could be changed to suit the conditions at any particular plant.

Table 8: Jar test method to determine the efficiency of taste and odour removal by PAC.

Time (min)	Mixing condition $(G = s^{-1})$	Task
0.00,,	60	Add PAC at different dosages to spiked spent filter backwash
16'30"	390	Add 1 g/t recycled sludge and 3,5 mg/t Fe as FeCl <sub>3</sub> .
18'00"	390	Add 6 mg/E cationic polyelectrolyte.
19'30"	40	
27'00"	0	Leave to settle
32'00"	0	Filter supernatant and analyse for taste and odour compounds.

• The adsorption isotherm can now be determined. The liquid phase isotherm shows the distribution of the adsorbate between the adsorbed phase (carbon) and the solution phase at equilibrium. An adsorption isotherm is a plot of the amount of adsorbate adsorbed per unit weight against the adsorbate still in the solution phase. A straight-line plot relates the amount of adsorbate in the solution phase to that in the adsorbed phase by making use of the empirical Freundlich equation [40] denoted by the expression:

$$X/M = kC^{1/n}$$

where X/M = amount of adsorbate adsorbed per unit weight carbon
C = equilibrium concentration of adsorbate in solution after adsorption
k and n are constants.

- By taking the logarithm on both sides we obtain:  $\log X/M = \log k + 1/n \log C$ . This is an equation of a straight line with a slope of 1/n and an intercept of  $\log k$ .
- The dosage to reach the required level of taste and odour compound in the final water can be used to determine the cost effectiveness in terms of R/Mt treated.

This test procedure would be impractical for smaller waterworks due to the cost involved. The Cape Metropolitan Council (CMC) uses a simpler and less expensive test procedure. The objective is to find the least expensive product that would remove 90% of the taste and odour compound. The removal efficiency of algal toxins is also taken into account in the CMC procedure.

Concerns exist that the dosing of PAC could result in PAC carry-over, penetration of filter media and ultimate breakthrough into the filtered water. It was therefore included in the extended task description with the expectation that the issue could be addressed during the evaluation. PAC penetration and breakthrough could not be studied on the full-scale plant and literature was consulted to shed some light on the issue. PAC breakthrough was noted at dosages of 60 mg/t and filtration rates of 8 to 9 m/h [41]. PAC dosage rates of 30 mg/t at a filtration rate of 4,9 m/h did not produce carbon particles in the effluent. Theoretical calculations for particles greater than 10 µm using single collector efficiency for particle capture indicate effective removal of these particles. PAC breakthrough was observed at the Franklin water treatment plant in North Carolina at dosages as low as 2 mg/t. Dosages of 1 mg/t PAC in the feedwater to the sand filter resulted in penetration of the filter to a depth of 17,8 cm as revealed by a visual inspection. The single collector efficiency calculations did not take chemical factors affecting attachment into account. The determination of the surface charge of ten PAC samples displayed a range from -16 to -28 mV with only two activated carbons displaying a surface charge higher than -20 mV. Poor control over the coagulation process would increase the potential for carry-over and penetration of the filter media.

## 4.2.2 Chemical dosing

Jar test methodology was used to assess the behaviour of different polyelectrolytes at the filter backwash recovery plant. Testwork was performed on-site with spent filter backwash, recycled sludge and partially coagulated water taken from the plant. The jar test method outlined in Table 8 was adjusted slightly for the assessment of the different polyelectrolytes. PAC dosing and the contact time in the balancing tank was not considered in the test. The recycled sludge dosing rate was calculated from actual plant flows as displayed on the SCADA system. Ferric chloride and a cationic polyamine (U5000) were used at the plant during the commissioning stage and for a period thereafter. The first objective was to determine the optimum dosage of ferric chloride and U5000. Four sets of experiments were performed and the results are graphically displayed in Figure 6. The dosage ranges and conditions of the four sets are described as follow:

- Ferric chloride dosages range of 1 to 14 mg/t as Fe and 4 mg/t U5000.
- U5000 dosage range of 0 to 15 mg/t and 3.4 mg/t Fe.
- U5000 dosage range of 0 to 15 mg/t and 8,6 mg/t Fe.
- U5000 dosage range of 0 to 15 mg/t with process water collected at the overflow of the first flash mixer. The recycled sludge addition, FeCl<sub>3</sub> dosing and the first rapid mixing step were omitted from the jar test procedure.

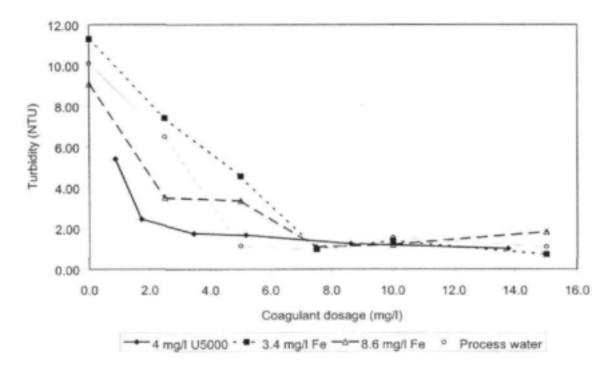


Figure 6: Optimisation of the FeCl<sub>3</sub> and U5000 dosing at the filter backwash plant.

Optimisation tests of FeCl<sub>3</sub> and U5000 chemical dosing can be summarised as follows:

 An optimum FeCl<sub>3</sub> dosage of 3,5 mg/t as Fe (10 mg/t as FeCl<sub>3</sub>) was determined. Higher FeCl<sub>3</sub> dosages resulted in a marginal improvement in clarity as determined by turbidity measurements. A visual inspection also indicated smaller flocs sizes at increased dosages

- of FeCl<sub>3</sub>. Polyelectrolyte dosing without FeCl<sub>3</sub> resulted in higher supernatant turbidities compared to the dosing of both chemicals.
- The optimum polyelectrolyte dosage was determined to be 7,5 mg/t U5000. A lower optimum dosage was obtained in the tests using process water from the first rapid mixer.

Similar tests were performed with FeCl<sub>3</sub> and a polyDADMAC/ polyamine blend (LP526). The same trend was observed for the FeCl<sub>3</sub> with the best performance between 1,7 and 3,4 mg/t as Fe. The optimum dosage of Zetafloc LP526 was determined to be 4 mg/t. The performance of a polyDADMAC (LP226), polyDADMAC/polyamine blend (LP526) and a polyamine were also compared with each other. The polyDADMAC (LP226) and polyDADMAC/polyamine blend (LP526) products illustrated similar suspended solids removal capacity with the polyDADMAC (LP226) performing slightly better.

The efficiency of each of the following anionic polyelectrolytes was also tested: Zetafloc 3103, Zetafloc 747 and AP161 (Figure 8).

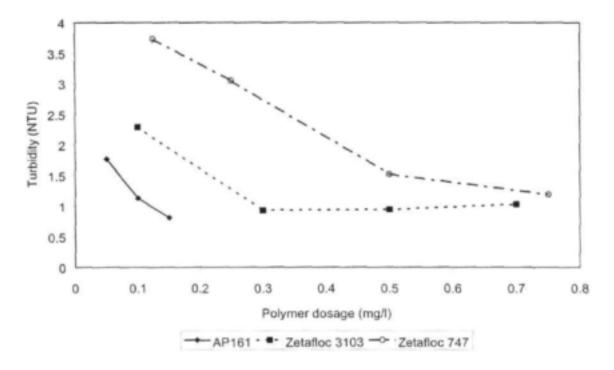


Figure 7: A comparison of the performance of the three anionic polyelectrolytes tested.

The products were then tested with process water at the optimum dosage. The averaged results are summarised in Table 9. The settling of the flocculated material with the anionic

polyelectrolytes was rapid. The settling period was reduced to one minute in order to emphasise the difference in efficiency between the products more clearly.

Table 9: Effeciency of the anionic polyelectrolytes in removing suspended solids from process water.

	Zetafloc LP526	Zetafloc 3103	Zetafloc 747	AP161
Dosage (mg/f)	6	0.5	0.5	0.1
Turbidity (NTU)	12,62	2,91	1,86	1,60

The effectiveness of AP161 was tested on plant scale. AP161 is a high molecular weight anionic polyelectrolyte available in liquid (1% w/w) or solid form. Promising results were obtained at dosages of 0,1 to 0,2 mg/t during the laboratory tests. The results from a plant trial are graphically displayed in Figure 8. Care was exercised in controlling the dosage so as to not exceed a dosage higher than 0,2 mg/t due to the possibility of filter blinding. The dosing pumps at the filter backwash plant were not able to dose the required quantity of the diluted polyelectrolyte (1% m/v) due to the high viscosity of the product. The product was dosed with a peristaltic pump run manually. The spent filter washwater flow rate remained constant at 1458 m<sup>3</sup>/h.

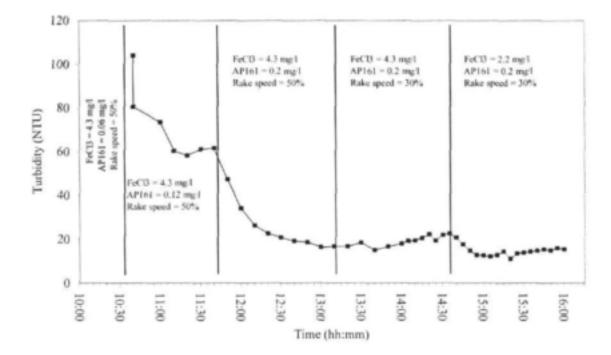


Figure 8: Performance of an anionic polyelectrolyte as a function of turbidity of the clarified water.

The following observations and conclusions could be drawn from the results displayed in Figure 8:

- An increase in polyelectrolyte dosage from 0,06 mg/t to 0,2 mg/t showed a drastic decrease in turbidity of the clarified water from the Denslator.
- The turbidity was between 10 and 20 mg/t at a polyelectrolyte dosage of 0,2 mg/t. A
  better performance was expected based on the test results.
- A polyelectrolyte dosage of 0,35 mg/t was required to reduce the turbidity of the clarified water to below 5 NTU as determined in further trials.
- A dosage of 2,2 mg/t Fe as FeCl<sub>3</sub> proved to reduce the turbidity even further as displayed in the last two periods in Figure 8.

The cationic polyelectrolyte was changed at some point from U5000 to Zetafloc LP526 due to supplier problems. Significant floc carry-over was experienced during this time and turbidities of up to 70 NTU in the water from the Denslator were measured occasionally. This happened without warning and lasted for relatively short periods, which made test work very difficult. This problem was resolved by switching the sludge recycle withdrawal point to the bottom of the Denslator. Recycled sludge should therefore be returned from the lowest point in the Denslator. Recycling sludge from points higher up in the Denslator results in floc with a lower settling velocity. This leads to carry over through the lamellae plates and higher turbidity in the clarified water to the sand filters.

Provision was made to dose a filter aid if the required filter effluent quality could not be achieved. The turbidity of the filter effluent was consistently below 0,5 NTU thus negating the need to dose a filter aid.

## 4.2.3 Phase separation

The clarification efficiency of the Denslator over a twenty-week period is displayed graphically in Figure 9. The data was compiled from manual turbidity readings. It would seem that an overflow turbidity of 5 NTU could be achieved. Laboratory measurements indicated slightly higher turbidities (Table 10) compared to the data collected by the operator.

Table 10: Summary of the laboratory measurement of clarified water turbidity.

	Minimum	Maximum	Average	Median
Turbidity (NTU)	2,34	21,5	7,86	6,52

The minimum turbidity value obtained in the laboratory measurement agrees well with the data collected by the operators. Turbidity data obtained during the microfiltration trial did indicate turbidity readings closer to 10 NTU. The periods during which the turbidity data were obtained did not necessarily overlap. The following could therefore be concluded from the data obtained:

- Clarified water with a turbidity ranging from 5 to 10 NTU could be obtained from the Denslator with the present configuration.
- Turbidity spikes as high as 20 NTU may occur. A turbidity of 2,5 NTU may however be achieved by careful operation.

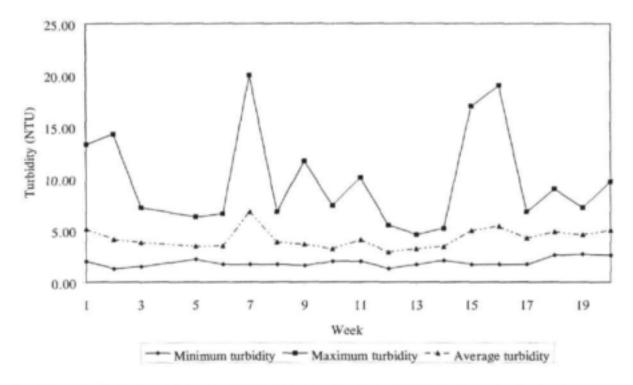


Figure 9: Performance of the Denslator measured by the turbidity of the clarified water.

The effectiveness of the Denslator can be increased by decreasing the rise rate through the lamellae plates. The flow set point was changed to 1100 m<sup>3</sup>/h to still ensure continuous operation of the filter backwash plant. The rise rate through the lamellae plates decreased

from proportionally 26,5 m/h to 20 m/h. Overflow turbidities were consistently less than 5 NTU at the reduced loading.

The sludge production rate was determined over a seven-week period and was found to be 9,2 tons dry material per day while the average quantity of water treated for the period was 27,5 Mt/day. This mass of sludge corresponds to the removal of 357 mg/t of suspended solids. Sludge from the filter backwash recovery plant is blended with water treatment sludge pumped to the disposal site. The settling rate of the filter backwash sludge was higher than the lime-silica sludge produced in the main process at Rand Water. Sludge from the filter wash water recovery plant conditioned with different concentrations of polyacrylamide, showed settling rates that was higher than that of lime-silica sludge produced during the treatment process. Blending of these sludges would therefore not impact negatively on sludge thickening operations at the sludge disposal site.

#### 4.2.4 Filtration

A sample point was provided on the effluent pipe from each sand filter. A sample could also be taken from the filtered water sump or the sample point on the feedline to the ozone pilot plant. The data records would display five different sample points with results from each sample point. The results of these sample points are summarised in Table 11. The filtered water was analysed for total organic carbon (TOC) and dissolved organic carbon (DOC) content. The TOC-analyses proved to be more accurate and DOC was therefore not used in this report. The TOC concentrations of the filtered water that passed through a 0,45 µm filter were compared with unfiltered samples. The same result was obtained for the filtered and unfiltered samples. The TOC results could therefore be used in the calculation of the specific UV absorbance (SUVA).

Table 11: A summary of the filtered water quality obtained at the filter backwash water recovery plant for the parameters shown below.

Parameters	Minimum	Maximum	Average	Median
Standard Plate Count (cfu/mt)	13	20200	890	163
Total Coliforms (TC/ 100mt)	0	320	10	1
Faecal Coliforms (FC/10 mt)	0	100	3	0
Confirmed Eschericia coli (EC/100 mt)				
Coliphages (pfu/ 10mf) Adams method	0	17	1	0
Sulphate reducing Clostridia/100 mt				
Invertebrate: Diptera (organisms/m <sup>5</sup> )	0	14	3	2
Invertebrate: Other organisms (organisms/m <sup>1</sup> )	0	1281	74	22
рН	7,46	8,45	8,14	8,16
Conductivity (mS/m)	25,0	31,1	27,6	27,5
Turbidity (NTU)	0,10	2,3	0,18	0,18
Chlorine demand (mg/t)	0,2	4,0	1,2	1,0
Total alkalinity (mg/f as CaCO <sub>3</sub> )	67	101	84	84
TOC (mg/f as C)	2,9	5,9	4,2	3,8
UV absorbance @ 254 nm (5 cm path length)	0,127	0,457	0,313	0,312
Specific UV absorbance (UV abs/ unit length/ TOC)	1,0	2,0	1,4	1,3
Chlorophyll a (µg/l)	0,81	0,10	0,46	0,50

No limits for invertebrates appear in the SABS 241 (1999) guideline for potable water supplies and the Rand Water standards were used as a guideline. The average and median invertebrate counts (excluding diptera) exceeded the Rand Water recommended limit of 20 organisms per one m³ of water but is still within the maximum permissible limit of 100 organisms/m³. Diptera numbers exceeded the recommended limit of one organism per m³ but was within the maximum permissible limit of 4 organisms/m³. Since it is possible that the internal water quality production standard is unrealistically stringent, it may be necessary to review the standard. If, however the standard is found to be realistic, filter efficiency should be improved. Throughout the evaluation period the invertebrate removal efficiency through the plant was high. The invertebrate numbers in the spent filter backwash was at times exceptionally high and the sand filters had to cope with numbers as high as 60 000 organisms per m³ of water. Preliminary investigations did indicate that the dosing of filter aids could reduce invertebrate numbers in the filter effluent.

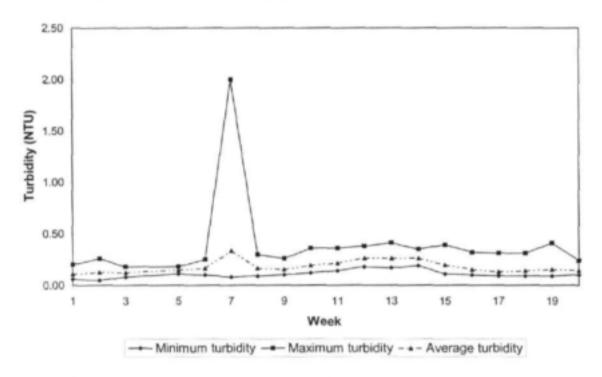


Figure 10: Minimum, average and maximum turbidity profile over a period of twenty weeks.

Data and trends presented in Table 11 and Figure 10 indicate that filtered water turbidities of less than 0,3 NTU could be delivered by the filter backwash plant. By applying a turbidity standard of 0,3 NTU the USEPA attempted to regulate the presence of protozoa in the filtered water. Performance regulations have also been stipulated for different unit treatment processes.

Particles counts of filtered water at different filter houses (Table 26 to 29) at Rand Water indicated lower counts for filter house 3 (Vereeniging) and 4 (Zuikerbosch) compared to the filter backwash water recovery plant. Higher particle counts were observed at filter house 2 (Vereeniging) compared to that at the filter backwash plant. Particle count data from the different installations are summarised in Table 12.

Table 12: Averaged particle count data extracted from Table 26 to 29.

	FWWRP	FH 2	FH 3	FH 4
Average particle count/ mℓ > 1 μm	937	1864	677	479
% Fraction particles between 1 and 2 µm	61	52	58	47
Average turbidity (NTU)	0,25	0,24	0,25	0,24

Examination of the data (Table 12, Table 26 to 29) revealed the following:

- A wide range of particle counts was detected in the filtered water from the different filter houses.
- Significant differences were observed between the particle count data of the FWWRP and FH3 with very similar turbidity data. Turbidity data could therefore not be correlated with particle counts.
- · The best quality water in terms of particle counts and turbidity was obtained at FH 4.
- Particle counting is a very sensitive technique to assess water quality and should be used more often to optimise plants. Particle counting should be able to give an early warning of substandard plant performance.
- Particle counting would not reveal the presence of protozoa in filtered water.

Although provision was made in the current design for the dosing of a filter aid, the need did not arise as no problems were experienced in delivering filtered water with a turbidity of less than 0,5 NTU that was the performance criteria for the plant.

The microbiological quality of the filtered water at the FWWRP was also compared to that of filter house 4 at Zuikerbosch pumping station. Standard plate counts (SPC) were determined on samples obtained at the two locations and the comparison is graphically displayed in Figure 11. The highest percentage of standard plate counts of the filtered water at the FWWRP occurs in the 100 to 149 cfu/mt range. For FH 4 most of the results occur in the 0 to 49 cfu/mt range. It could therefore be concluded that the filtered water at the FWWRP is of

slightly poorer quality compared to the filtered water at filter house 4. A significant percentage of the SPC of the filtered water at the FWWRP occurs in the 1200 to 19999 cfu/mt range. These counts do not form part of the pattern of the rest of the graph and could be ascribed to poor filter operation. The FWWRP was periodically out of commission for extended periods and filters were commissioned without following a filter disinfection procedure during start up of the plant. This could contribute to poor sporadic microbiological quality. The other microbiological parameters that were determined during the evaluation of the FWWRP were present in very low numbers. It could therefore be concluded that the filtered water at the FWWRP is not of a significantly poorer quality than the other filtered water at Rand Water, although higher standard plate counts were observed. The filtered water was tested on a weekly basis for the presence of protozoan cysts and oocysts. None of these were detected during the duration of the project.

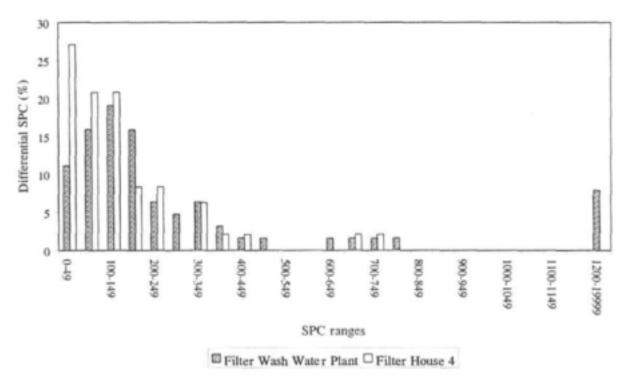


Figure 11: Comparison of the standard place count of the filtered water from the Filter Back Wash Water Recovery plant and the main treatment process at Rand Water.

The following are recommended with regard to the design of sand filters for future FWWRP's:

 The current design of 7,5 m/h filtration rate and effective sand grain size of 0,65 mm were able to deliver a turbidity of 0,3 NTU.  An initial filter-to-waste period after commissioning a filter would be essential to minimise the risk of a breakthrough of protozoan cysts and oocysts during the ripening period of the filter.

### 4.3 EXPERIMENTAL UNIT TREATMENT PROCESSES

#### 4.3.1 Ozone and GAC

The effectiveness of ozone as a disinfectant is well illustrated in Table 1. Filtered water from the filter backwash recovery plant was used as feed to the ozone plant. Dosages of 1,5 mg/t ozone and a contact time of ten minutes reduced the standard plate count to an average and median value of 5 and 1 cfu/mt respectively. No total or faecal coliforms were detected in the ozonated water. A reduction in chlorine demand of approximately 20% was also observed. The average and median chlorine demand was reduced from 1,.18 to 1,.02 and from 1,.00 to 0,.78 mg/t respectively. The primary objective of the ozonation was to transform the dissolved organic carbon to a more biodegradable form to enhance microbiological growth in the GAC/ BAC filters and therefore the removal of the contributing compounds by bioassimilation. The benefits of microbiological activity on the GAC/ BAC filters were highlighted in the literature review. Assimilable organic carbon gives an indication of the quantity and the bio-availability of biodegradable organic carbon present in the water. The effect of ozonation (0,36 mg O<sub>3</sub>/ mg TOC) upon AOC levels is illustrated in Table 13. The percentage change was calculated from the relative change in the bio-availability of assimilable compounds based on the AOC determination

Table 13: Increase in AOC levels after ozonation (0,36 mg O<sub>3</sub>/ mg TOC).

	Minimum	Maximum	Average	Median
Increase in AOC (%)	15	100	39	31

The chlorine demand was reduced significantly by the different activated carbon processes preceded by ozonation. The average and median values are summarised in the Table 14.

Table 14: Chlorine demand (mg/t) of the effluent after the different activated carbon process.

	BAC process 1	BAC process 2	GAC process 1
Average (mg/t)	0,57	0,66	0,74
Median (mg/t)	0,47	0,55	0,55

Only one ozone dosage/ TOC ratio was tested during the project as a result of inconsistent performance of the mechanical and electrical equipment. A dosage of 0,36 mg O<sub>3</sub>/mg TOC (1,5 mg/t O<sub>3</sub>) was applied at a hydraulic residence time of 10 minutes. The removal of TOC and UV absorbing substances were monitored during the trial. The removal of TOC as a function of bed volume is graphically displayed in Figure 12. The removal capacity of TOC in all three activated carbon processes declined rapidly as a function of the number of the accumulated bed volumes passing through the activated carbon beds.

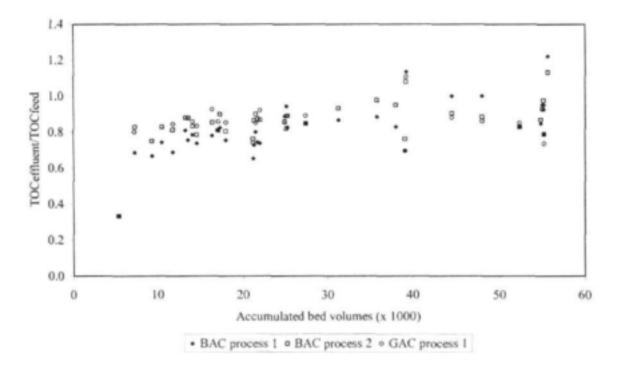


Figure 12: The removal of TOC by the different activated carbon processes as a function of accumulating bed volume.

The removal of UV absorbing substances is a function the volume of water that was treated. This relationship is graphically displayed in Figure 13. The adsorption capacity of the three activated carbons for UV absorbing substances declined as rapidly as for TOC. Significant differences existed between the activated carbons once steady state conditions were reached. BAC process 1 removed more UV absorbing substances than either BAC process 2 and GAC process 1. BAC process 2 in turn removed more UV absorbing substances than GAC process 1. The removal of AOC was also monitored for BAC process 1 and 2. Only a limited number of AOC analyses could be performed and less attention was paid to GAC process 1 in terms of AOC removal. However even the small number of samples submitted for AOC removal showed that no AOC was removed by the GAC process 1. This can be attributed to the fact that the absence of O<sub>3</sub> did not convert a fraction of the TOC to more assimilable compounds. The percentage removal of AOC in BAC process 1 and 2 is summarised in Table 15. Removal percentages higher than 100% and less than 0% were omitted from the calculations in Table 15.

Table 15: AOC removal in BAC process 1 and 2

	Minimum	Maximum	Average	Median
% AOC removal (BAC process 1)	9	56	28	28
% AOC removal (BAC process 2)	0	100	46	54

The higher AOC removals in BAC process 2 might be due to higher microbial activity in the GAC filter. The GAC used in BAC process 2 is an extruded product and marketed as a medium that would promote microbial activity in the media.

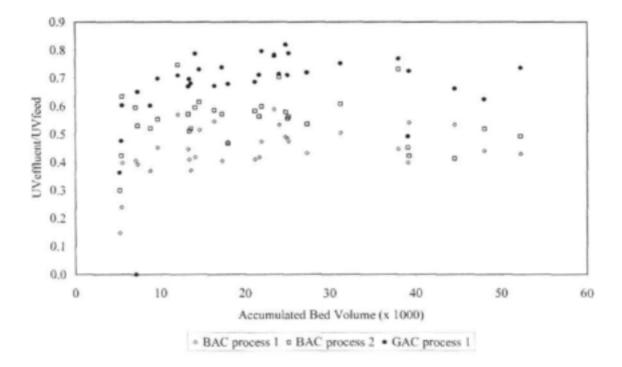


Figure 13: The removal of UV absorbing substances for the different activated carbon as a function of accumulated bed volume.

The average and median total trihalomethane (TTHM) concentrations in the feedwater were 17,9 and 15,0 µg/t respectively. A great deal of scatter was observed in the data points and Table 16 was compiled after data points with percentage removals higher and lower than 100 and 0 % respectively were omitted from the data set. Although the results might indicate enhanced TTHM removal in the ozonated streams, care should be exercised interpreting the results due to the scattering of the data.

Table 16: Average and median % TTHM removal in the different activated carbon processes.

	BAC process 1	BAC process 2	GAC process 1
Average % TTHM removed (μg/t)	44	36	12
Median % TTHM removed (μg/t)	42	33	8

#### 4.3.2 Membrane treatment

The results for the mini-pilot plant MF-trial are summarised in Table 17. The permeate quality remained the same in all the trials irrespective of the feed water. A CIP was performed between the different feedwater trials.

Table 17: Summary of the results obtained with the mini pilot plant MF unit.

Feed Source	Feed Turbidity (NTU)	Permeate Turbidity (NTU)	Silt Density Index
Untreated filter backwash	320	0,13	3,57 (12,8°C)
Clarified filter backwash	16,1	0,15	2,53 (13,9°C)
Filtered filter backwash	0,34	0,18	3,47 (12,1°C)

Standard plate counts of less than 20 ctu per mt were detected in the permeate. No change in flux was observed when the sand filtered water or Denslator (clarified) water was treated in the membrane plant. The Denslator feed may be considered as a seed source for microfiltration, but should be tested on a long term basis on a properly sized microfiltration pilot plant. The effect of water treatment polymer carry-over should be carefully considered as irreversible fouling may occur. The test results indicated that low fluxes, frequent backwashes and chemical cleaning might result if the untreated filter backwash is considered for capillary microfiltration.

The Denslator overflow turbidities during the time of the MF trial are summarised in the Table 18. Both acidic and caustic CIPs were performed at the start of the trial. A high flux rate (110 LMH) was selected with a backwash interval of 45 minutes. The TMP increased from 45 kPa to 120 kPa within two days. The run was then terminated and acid and caustic CIPs were performed together with a pressure decay test. The flux was reduced to 100 LMH with a backwash interval of 30 minutes. After this the TMP increased from 40 kPa to 130 kPa within 5 days (Figure 14). The run was once again terminated and a caustic CIP and pressure decay test performed. The TMP increased to 130 kPa within two days. The membranes fouled up in half the time of the previous run, which indicated the need for the acidic CIP. Standard plate counts of less than 10 cfu/mt and SDI results of approximately three were determined in the permeate. The trial was terminated due to the unsatisfactory results obtained.

Table 18: Feedwater turbidity variation during the MF trial.

	Minimum	Maximum	Average	Median
Turbidity (NTU)	7,10	12,60	9,78	9,00

TMP, Resistance and Flow Rate, February 2001 Backwash Interval = 30 minutes

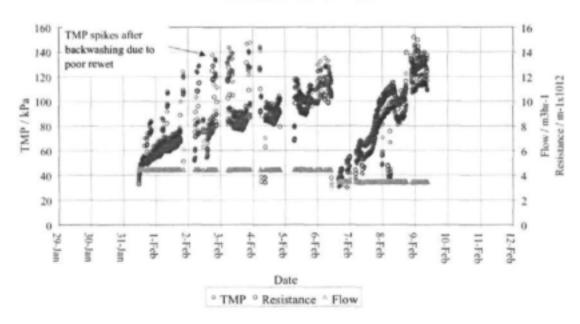


Figure 14: Results from the microfiltration trial indicating the rise in resistance and TMP at a flux of 100 and 78 LMH.

## 4.3.3 Ultra violet disinfection

Tables with the detailed results from the UV test work are attached to the appendix. More attention will be given in this section on the effect of UV radiation on microbiological quality, since UV radiation at the doses tested was very effective against indicator groups such as total coliforms, faecal coliforms and bacteriophages. SPC numbers were not reduced to be within the SANS 0241 guideline of 100 cfu/mt for 95% of the time and at all doses tested and more detail is presented in the results and discussion below.

Run 1 to 3 (Figure 15) for the 25 m³/h flow contained relatively large SPC numbers in the untreated water. A comparison of the SPC numbers for run 1 to 3 at the different UV dosages showed comparable numbers except for the 17 mJ.cm² dosage of run 1. Contamination may have occurred in this particular sample although standard microbiological sampling practices were followed to eliminate the possibility. A much lower SPC after radiation of all the UV dosages was detected in the feed water of run 4 with similar SPC numbers (38 to 58 cfu/mt). The SPC was reduced to be within the SANS 0241 guideline for 95% compliance to 100 cfu/mt water only in run 4 with a low SPC number in the feedwater. SPCs between 100 and 130 were observed for average UV dosages between 17 and 24 mJ.cm². UV radiation in the range of 17 to 24 mJ.cm² was very effective in reducing the total coliform, faecal coliform, E coli and bacteriophage counts in nearly all the runs, to zero. A maximum and median total coliform count of 3 and 1 per 100 mt respectively were observed for run 1.

A slight reduction in SPC was observed with increasing UV dose by comparing the median SPC numbers with each other in Figure 16. The SPC was reduced to approximately 100 cfu/mt with the two UV units in series.

The standard plate count was reduced to less than 100 cfu/mt for all the dosages (29 to 41 mJ.cm<sup>-2</sup>) in all the runs. The results are graphically displayed in Figure 17. The best results were achieved during run 1 with SPCs approaching 20 cfu per 100 mt and less. The worst results were obtained during run 4 with SPCs of approximately 70 cfu/mt for UV doses of 29 to 41 mJ.cm<sup>-2</sup>. The median and average total coliform, faecal coliform, E coli and bacteriophage counts were all zero after treatment at the dosages tested as given in Figure 17.

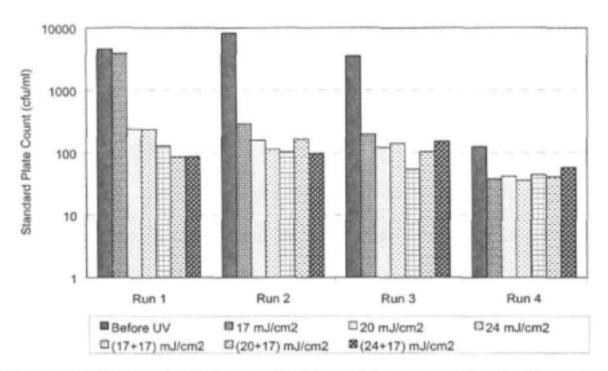


Figure 15: Effect of UV radiation on SPC of the sand filtered water from the filter backwash recovery plant. Flow rates of approximately 25m<sup>3</sup>/h were used.

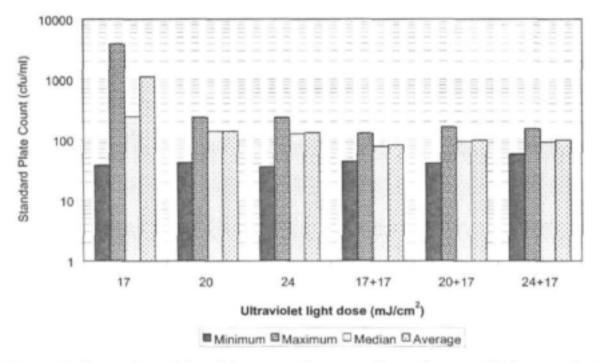


Figure 16: Comparison of the minimum, maximum, median and average SPC count after UV radiation at doses obtained with a flow rate of approximately 25 m<sup>3</sup>/h flow.

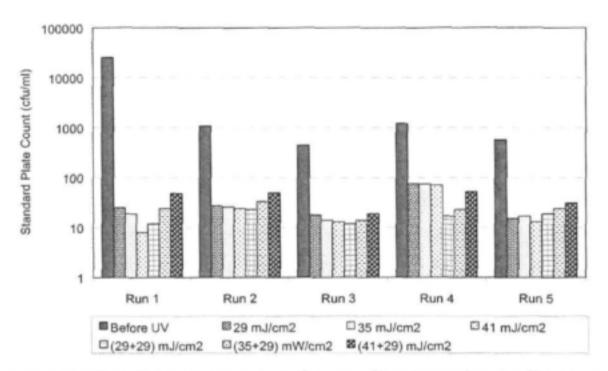


Figure 17: Effect of UV radiation on SPC of the sand filtered water from the filter backwash recovery plant. Flow rates of approximately 15 m<sup>3</sup>/h were used.

A comparison of the median SPCs in Figure 18 at different UV doses indicates a reduction with increasing UV dose for the single UV dose experiments (29 to 41 mJ.cm<sup>-2</sup>). The opposite was observed for the double UV dose ([29+29], [35+29] and [41+29] mJ.cm<sup>-2</sup>). More test work will be required to confirm the pattern and to explain the phenomenon. This trend was not observed for the lower dosages as illustrated in Figure 16.

The SPC has reduced to 10 cfu/mt and less at UV doses of 93 to 130 mJ.cm<sup>-2</sup>, with the best results achieved during runs 2 and 3. The median total coliform, faecal coliform, E coli and bacteriophage counts were all zero after treatment at the dosages tested as shown in Figure 19.

The SPCs increase with increasing UV dose in the double UV dose experiments ([93+93], [111+93] and [130+93] mJ/cm<sup>-2</sup>; Figure 20) as was observed for the results of experiments as illustrated in Figure 18.

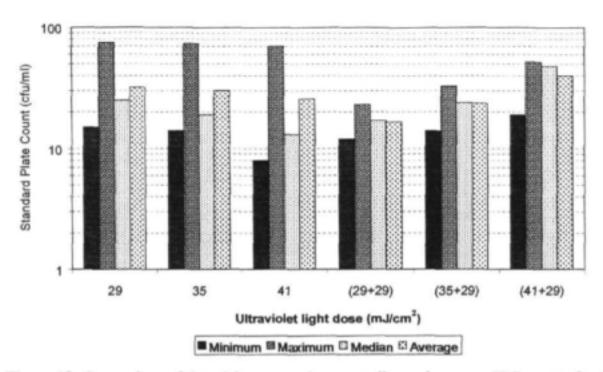


Figure 18: Comparison of the minimum, maximum, median and average SPC count after UV radiation at doses obtained with a flow rate of approximately 15 m<sup>3</sup>/h flow.

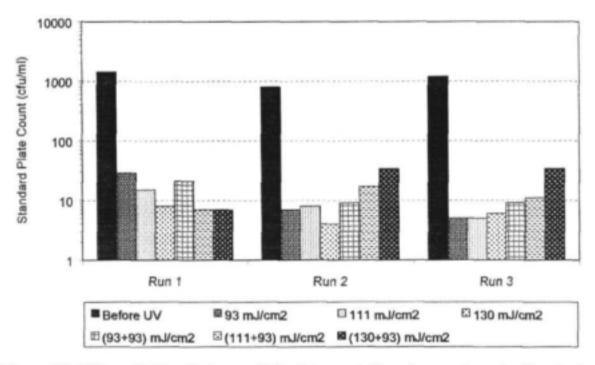


Figure 19: Effect of UV radiation on SPC of the sand filtered water from the filter backwash recovery plant.

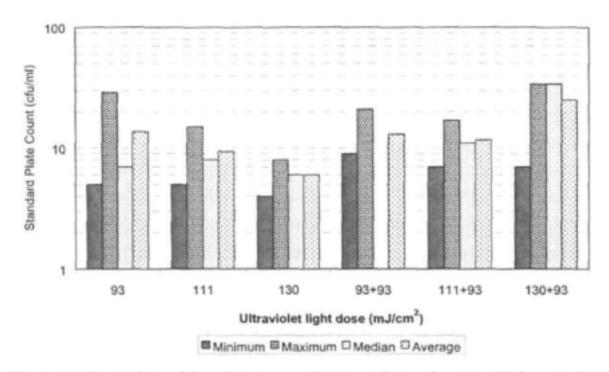


Figure 20: Comparison of the minimum, maximum, median and average SPC count after UV radiation at doses obtained with a flow rate of approximately 15 m<sup>3</sup>/h flow.

The SPC of the treated water was plotted against the UV dose from the results of the single UV dose experiments. The minimum UV dose required to disinfect recovered spent filter backwash to comply with the SANS 0241 (2001) guideline could be determined. Maximum contaminant production limits as practiced at Rand Water is more stringent than the SABS guideline, see Table 19 for the maximum microbiological contaminant limits.

Table 19: Maximum contaminant production limit goals at Rand Water.

Standard plate	Total coliforms	Faecal coliforms	E. coli (EC/100 mt)	Bacteriophages	
count (cfu/mt)	(TC/100 mf)	(FC/ 100 mf)	E. con (EC/100 mt)	(pfu/10 mf)	
10	0	0	0	0	

Disinfected potable filter backwash would be blended with chlorinated filtered water from the main treatment process. Standard plate counts in the filter backwash could then be reduced even further during the blending process, which would allow for higher maximum microbiological contaminant levels for the UV disinfected filter backwash.

A UV dose of 25 mJ.cm<sup>-2</sup> is guaranteed at the end of the lamps life. A reduction in UV dose from 31 to 25 mJ.cm<sup>-2</sup> could result in an approximate STP increase from 25 to 60 cfu/mt.

The treated filter backwash (35Mt/day) would be blended with chlorinated water from system 4 with a total capacity of 1800 Mt/day. UV radiation is very effective against the other microbiological determinants listed in Table 19 no problem is foreseen to reach the Rand Water production standards, even at dosages as low as 20 mJ.m<sup>-2</sup>.

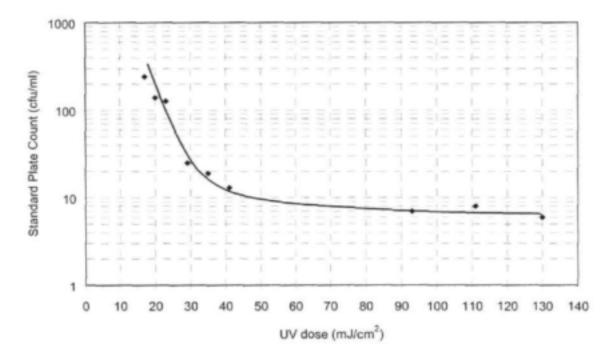


Figure 21: The effectiveness of various UV radiation doses in relation to the median SPC of the treated water.

The primary interest in UV radiation as a disinfectant is for the inactivation of protozoan cysts and oocysts. Dosages lower than the design range suggested by the manufacturers (25 to 31 mJ.cm<sup>-2</sup>) are reported in literature to be effective against *Giardia* and *Cryptosporidium* [18, 19]. A small increase in chlorine dosage would be needed to maintain the microbiological quality after blending to comply with the production limit at Rand Water. The chlorine demand of the filtered water at the filter backwash recovery plant was determined to be the same as filtered water from the main treatment process at Rand Water.

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## COST ESTIMATES

## 5.1 Ultra violet disinfection

A transmission (T10) of 80% was assumed and a dosage of 25mJ.cm<sup>-2</sup> to provide the required level of disinfection. The estimated capital and operating costs for a 50 Mt/day plant are outlined below.

## Capital costs

(Recommended spares accompanying the equipment, air freight/training and commissioning, freight clearing, transport, installation and all other local costs to complete turnkey installation)

Total turnkey costs (Euro 132 000)	R 973 200
Operating costs	
The total energy requirement for the lamps is 34 kWh.	
Annual electricity costs (R0,12/kWh)	R 35 741
Annual lamp replacement costs (Euro 4600 @ R 7,10)	R 32 660
Other maintenance materials (Euro 2400 @ R 7,10)	R17 040
Total annual operating costs excluding labour	R 85 441

The operating costs of a 50 Mt/day plant would be 0,468 c/kl.

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## CONCLUSIONS

A conventional treatment plant consisting of coagulation, phase separation and rapid gravity sand filtration was evaluated for the treatment of spent filter wash water. An experimental plant, comprising ozone, granular activated carbon, microfiltration and ultraviolet disinfection for the further treatment of the recovered filter wash water was evaluated as part of the project.

# 6.1 TREATMENT OF SPENT FILTER BACKWASH BY CONVENTIONAL TREATMENT

The conclusions drawn from the investigation can be outlined as follow:

 The effectiveness of the unit treatment processes is illustrated in Table 20, which summarises the reduction in the median values for standard plate count, turbidity, chlorine demand and total organic carbon content.

Table 20: Water quality from different treatment stages expressed as the median values of four water quality parameters.

Parameter	Treatment Process			
	Coagulation	Clarification	Sand filtration	
Standard plate count (cfu/mf)	1600	307	163	
Turbidity (NTU)	352	6,52	0,18	
Chlorine demand (mg/l)*	1,8	1,7	1,0	
Total organic carbon (mg/f)	4.5	4,5	3,8	

<sup>\*4</sup> mg/t chlorine - 15 minutes contact time

Filtered water turbidities of less than 0,3 NTU were achieved constantly at the Filter Back
Wash Water Recovery plant. The dosing of a filter aid was not necessary to achieve the
maximum permissible water turbidity of 0,5 NTU. Higher particle counts were observed
for filtered water at the Filter Wash Water Recovery plant in comparison with filtered
water from the main treatment process with similar turbidities. This emphasises the
sensitivity of particle counting as a tool to optimise plant performance. The risk of

protozoa in the finished water can be minimised by operating a plant under optimal conditions. Some of the advantages and disadvantages of turbidity and particle counting techniques are summarised in Table 21.

Table 21: Advantages and disadvantage of turbidity meters and particle counters.

Turbidity	Particle counting
Turbidimetric measurement	Count and size particles
<ul> <li>Qualitative measurement</li> </ul>	Quantitative measurement
<ul> <li>No clogging problems</li> </ul>	<ul> <li>Prone to clog with particles &gt; 800um</li> </ul>
<ul> <li>Can handle high test frequency</li> </ul>	Limited to low volume water processes
<ul> <li>Can handle high spikes in turbidity</li> </ul>	<ul> <li>Cannot handle high spikes in particles</li> </ul>
<ul> <li>Standard calibration for turbidimeters</li> </ul>	Proper setup, calibration and maintenance
	is critical
<ul> <li>Data interpretation is simple.</li> </ul>	Data interpretation more complex.

- The microbiological quality of the filtered water was comparable with filtered water from the main treatment process as measured by standard plate counts. Slightly higher SPCs were observed for the filter backwash.
- No protozoa were detected in the filtered water during the duration of the project.
- The chlorine demand of the recovered filter water was 1,2 mg/t with a median value of 1,0 mg/t, which is similar to that of filtered water. If water from the two sources were mixed the chlorine demand of the blended water would remain unchanged.
- The total organic carbon is within the SANS 0241 guideline for Class O water with a low SUVA value being consistent with water of low organic loading.
- Invertebrate numbers (excluding diptera) exceeded the Rand Water recommended limit of 20 per m³ but were within the maximum permissible Rand Water production limit of 100 per m³. Numbers as high as 60 000 organisms/m³ were occasionally determined in the feed water to the plant.
- Diptera numbers exceeded the recommended limit of one organism per m³ but was within the maximum permissible limit of four organisms per m³.
- Most of the filtered water samples did not contain any algal cells. No taste and odour problems were experienced over the duration of the study. PAC dosing should remove taste and odour compounds effectively.

- All the chemical parameters were within the SANS guideline of Class O water. Spent filter wash water can successfully be treated to potable standard.
- The optimum chemical dosing was 3,5 and 6 mg/t Fe as FeCl<sub>3</sub> as Fe and cationic
  polyelectrolyte respectively. High density underflow sludge should be recycled to provide
  optimum particle interaction for the most effective coagulation. The dosing of a filter aid
  was not required to produce filtered water of 0,5 NTU turbidity or less.
- A lower rise rate through the lamellae plates could produce Denslator overflow turbidities
  of less than five NTU. The cost implications should be weight against the benefits.

## 6.2 EXPERIMENTAL TREATMENT UNITS

## 6.2.1 Ozone/ GAC filtration

- AOC concentrations are increased by 39% after a 0.36 mg O<sub>3</sub>/mg TOC dosage. Five minutes EBCT in the activated carbon filter was not enough to reduce the AOC to feed water levels.
- No difference in TOC removal was observed between the different GAC processes.
   Steady state conditions were reached within 10 000 to 15 000 bed volumes.
- The BAC processes showed improved removal of UV absorbing substances. Steady state conditions (TOC<sub>effluent</sub>/TOC<sub>feed</sub> = constant ratio) were reached within 10 000 to 15 000 bed volumes.
- · A significant reduction in chlorine demand was observed after GAC filtration.
- If taste and odour producing compounds are present in the water, GAC should be effective in removing it.

## 6.2.2 Microfiltration

 A rapid increase in TMP at higher fluxes resulted in significantly reduced CIP intervals, which renders microfiltration unsuitable for the treatment of clarified (with chemical addition) spent filter wash water.

## 6.2.3 Ultraviolet disinfection

 UV disinfection at doses of 25 mJ/cm<sup>-2</sup> is a viable option for the treatment of spent filter backwash when the inactivation of protozoa is of primary concern. A post disinfection, chlorine, step would be required as well. 7

## RECOMMODATIONS FOR FUTURE STUDIES

 The suspended material trapped on filter beds particles that escaped the sedimentation system. Little is known about mechanisms of the recoagulation and flocculation of destabilised particles and flocculated material.

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## TECHNOLOGY TRANSFER

 A presentation was delivered at WISA Technical Divisions on the 26<sup>th</sup> of October 2000 on the following topic: The treatment of filter backwash water. 9

## APPENDICES

Table 22: Suspended solid concentration of the spent filter backwash collected from the sump at different heights after four consecutive backwashes.

Sampling level	Suspended solids concentration (mg/l) in spent backwash sump.				
	1	2	3	4	
High	770	786	378	139	
Medium	338	319	206	234	
Low	276	369	192	166	
Average	348				

Table 23: Suspended solids concentration of the spent filter backwash taken from the discharge of the backwash basin pumps (set 1).

Sample no.	Time from start-up of backwash pump (min)	Volume sampled (f)	Suspended solid concentration in sample (mg/t)	Mass of solids in sample (mg)
I	0.5	0,5	3380	1690
2	1,0	0,5	1640	820
3	1,5	0,5	1220	610
4	2	0,5	485	243
5	3	1,0	380	380
6	4	1,0	325	325
7	6	2,0	300	600
8	10	4,0	305	1220
9	15	5,0	255	1275
10	25	10	220	2200
Total		25		9363
Average			375	

Table 24: Suspended solids concentration of the spent filter backwash taken from the discharge of the backwash basin pumps (set 2).

Sample no.	Time from startup of backwash pump (min)	Volume sampled (f)	Suspended solid concentration in sample (mg/f)	Mass of solids in sample (mg)
1	1	1,0	420	420
2	2	1,0	300	300
3	3	1,0	265	265
4	4	1,0	440	440
5	5	1,0	315	315
6	6	1,0	290	290
7	8	2,0	310	620
8	10	2,0	295	590
9	15	2,0	295	590
10	25	5,0	270	1350
Total		17		6065
Average			303	

Table 25: A summary of the calculated sludge production for a seven-week period.

Week	Average Water Treated (Mt/day)	Turbidity [NTU]	Total Sludge Produced [tons]
1	29,8	8,1	4,185
2	32,3	8,0	4,885
3	27,3	8,7	19,380
4	25,0	7,4	17,700
5	25,2	6,0	6,315
6	19,5	7,3	8,560
7	33,3	12,2	3,300
Average	27,5	8,3	9,2
Total	192,4		64,325

Table 26: Cumulative particle counts of the filtered water at the filter backwash plant.

	Cumu	Cumulative Particle Count per ml larger than channel size (µm) indicated below												Turbidity			
	1	2	3	4	5	6	7	8	10	12	15	20	25	35	50	100	(NTU)
Minimum	473	235	142	75	39	27	21	17	10	5	2	0	0	0	0	0	0,21
Maximum	2150	833	493	250	111	65	48	39	27	15	9	4	1	1	1	0	0,32
Average	937	362	222	120	61	42	32	26	16	8	4	2	1	0	0	0	0,25
10th Percentile	541	235	143	77	40	27	22	18	10	5	2	1	0	0	0	0	0,22
50th Percentile	778	268	160	91	53	40	30	22	14	7	3	2	1	0	0	0	0,26
90 <sup>th</sup> Percentile	1411	550	339	179	90	61	46	37	24	14	7	3	1	1	0	0	0,28

Table 27: Cumulative particle counts of the filtered water at filter house 4, Zuikerbosch.

	Cumul	Cumulative Particle Count per ml larger than channel size (µm) indicated below												Turbidity			
	1	2		4	5	6	7	8	10	12	15	20	25	35	50	100	(NTU)
Minimum	172	81	50	30	18SB;		10	8	6	4	2	1	0	0	0	0	0,12
Maximum	1402	754	478	283	161	111	84	64	36	22	9	3	2	1	0	0	0,38
Average	479	230	143	83	45	31	24	20	13	9	4	2	1	0	0	0	0,24
10th Percentile	181	85	53	33	19	14	11	9	6	4	2	1	0	0	0	0	0,15
50th Percentile	402	183	112	63	34	23	18	15	10	8	3	1	1	0	0	0	0,22
90th Percentile	722	334	207	121	66	44	36	30	20	15	5	2	2	1	0	0	0,33

Table 28: Cumulative particle counts of the filtered water at filter house 2, Vereeniging.

	Cumulative Particle Count per ml larger than channel size (µm) indicated below												Turbidity				
	1	2	3	4	5	6	7	8	10	12	15	20	25	35	50	100	(NTU)
Minimum	582	261	175	115	60	40	28	21	12	9	5	3	2	1	0	0	0,27
Maximum	3439	1820	1194	690	381	280	222	177	107	70	33	15	7	3	1	0	0,77
Average	1864	985	656	400	221	149;		83	48	31	15	7	4	2	0	0	0,52
10th Percentile	627	279	185	116	68	47	37	29	18	12	6	4	3	1	0	0	0,31
50th Percentile	1846	1134	779	474	241	154	109	80	41	26	12	6	3	2	0	0	0,55
90th Percentile	3298	1739	1136	668	370	270	206	161	95	63	33	13	6	2	1	0	0,70

Table 29: Cumulative particle counts of the filtered water at filter house 3, Vereeniging.

	Cumui	Cumulative Particle Count per ml larger than channel size (µm) indicated below													Turbidity		
	1	2	3	4	5	6	7	8	10	12	15	20	25	35	50	100	(NTU)
Minimum	449	166	87	44	20	12	9	7	4	3	1	0	0	0	0	0	0,21
Maximum	1000	508	343	210	112	77	60	48	30	19	10	5	4	2	1	1	0,33
Average	677	278	161	92	49	33	25	20	14	9	5	3	2	1	0	0	0,25
10 <sup>th</sup> Percentile	489	200	113	55	26	1612x3	435Y	10	7	4	2	1	0	0	0	0	0,22
50 <sup>th</sup> Percentile	593	255	143	82	43	29	23	19	12	8	4	3	2	1	0	0	0,24
90th Percentile	890	373	227	137	79	57	44	37	26	16	8	4	3	1	0	0	0,31

Table 30: Microbiological quality of water exposed to various UV doses at a flow rate of 24,7 m<sup>3</sup>/h(Run 1)

UV dose	Standard Plate	Total Coliforms	FaecalColiforms	E coli/100 m &	Coliphages/	%Transmission
mJ.m <sup>-2</sup>	Count (cfu/mℓ)	100 mℓ	100 mℓ		10 m€	
0	4595	37	1	1	0	82,93
17	3900	1	0	0	0	82,89
20	237	0	0	0	0	83,29
23	235	0	0	0	0	83,50
17+17	128	0	0	0	0	82,76
20+17	85	0	0	0	0	83,20
23+17	87	0	0	0	0	82,72

Table 31: Microbiological quality of water exposed to various UV doses at a flow rate of 227 m<sup>3</sup>/h(Run 2)

UV dose	Standard Plate	Total Coliforms	FaecalColiforms	E coli/100 m U	Coliphages/	%Transmission
mJ.m <sup>-2</sup>	Count (cfu/m l)	100 mℓ	100 m t		10 mℓ	
0	8350	1	1	1	0	76,24
15	290	0	0	0	0	81,92
18	157	0	0	0	0	77,21
21	113	0	0	0	0	81,47
15+15	102	0	0	0	0	78,56
18+15	164	0	0	0	0	79,03
21+15	97	0	0	0	0	77,77
					1	

Table 32: Microbiological quality of water exposed to various UV doses at a flow rate of  $23.6 \text{ m}^3/h(\text{Run }3)$ 

UV dose mJ.m <sup>-2</sup>			FaecalColiforms 100 mℓ	E coli/100 mt	Coliphages/	%Transmission
m.tm	Count (cfu/mℓ)	100 mc	100 mc		10 m c	
0	3550	2	0	0	4	82,53
17	195	3	0	0	0	83,25
21	120	0	0	0	0	84,14
24	140	0	0	0	0	83,39
17+17	54	0	0	0	0	82,87
21+17	103	0	0	0	0	82,41
24+17	152	0	0	0	0	82,00

Table 33: Microbiological quality of water exposed to various UV doses at a flow rate of  $25.5 \text{ m}^3/h(\text{Run 4})$ 

UV dose mJ.m <sup>-2</sup>			FaecalColiforms	E coli/100 mt		%Transmission
mJ.m	Count (cfu/mf)	100 mC	3m 001		10 mC	
0	125	11	0	0	4	87,58
19	38	0	0	0	0	87,86
22	42	0	0	0	0	87,10
26	36	0	0	0	0	86,26
19+19	44	0	0	0	0	86,54
22+19	41	0	0	0	0	86,42
26+19	58	0	0	0	0	86,78

Table 34: Microbiological quality of water after exposed to various UV dosages at a flow rate of approximately  $25 \text{ m}^3/\text{h}$ .

UV dose mJ.m <sup>-2</sup>	Standard Plat Count (cfu/mt)	Total Coliforms 100 mt	FaecalColiforms 100 m C	E coli/100 mt	Coliphages/
Average UV d	lose of 17 mJ.cm <sup>-2</sup>				
Minimum	38	0	0	0	0
Maximum	3900	3	0	0	0
Median	243	1	0	0	0
Average	1106	1	0	0	0
Average UV d	lose of 20 mJ.cm				
Minimum	42	0	Ø	ø	o
Marinum	237	9	ø	O .	ø
Median	239	.0	0	0	o
Average	139	0	0	ø	0
Average UV d	lose of 24 mJ.cm <sup>-2</sup>				
Minimum	36	0	0	0	0
Maximum	235	0	0	0	0
Median	127	0	0	0	0
Average	131	0	0	0	0
Average UV a	lose of (17+17) mJ.cr	$n^{-2}$			
Minimum	44	0	0	0	0
Maximum	128	0	0	0	0
Median	78	0	0	0	0
Average	82	0	0	0	0
Average UV a	lose of (20+17) m.J.ci	$n^{-2}$			
Minimum	41	0	0	0	0
Maximum	164	0	0	0	0
Median	94	0	0	0	0
Average	98	0	0	0	0
Average UV a	lose of (24+17) mJ.ci	n'-2			
Minimum	58	0	0	0	0
Maximum	152	0	0	0	0
Median	92	0	0	0	0
Average	99	0	0	0	0

Table 35: Microbiological quality of water exposed to various UV doses at a flow rate of 15,0 m<sup>3</sup>/h

UV dose mJ.m <sup>-2</sup>	Standard Plate Count (cfu/m1)		FaecalColiforms 100 mt	E coli/100 mt	Coliphages/	%Transmission
0 (sample 1)	23200	5	3	1	0	06.40
0 (sample 2)	27600	10	4	4	0	85,49
100	25	0	0	0	0	86,54
120	19	0	0	0	0	86,70
140	8	0	0	0	0	86,78
200	12	0	0	0	0	85,09
220	24	0	0	0	0	83,44
240	48	0	0	0	0	84,61

Table 36: Microbiological quality of water exposed to various UV doses at a flow rate of 15,5 m<sup>3</sup>/h

		FaecalColiforms 100 mt	E coli/100 mt	Coliphages/	%Transmission
1110	11	4	2	0	00.13
1040	11	4	1	0	88,12
27	0	0	0	0	88,59
26	0	0	0	0	87,98
24	0	0	0	0	88,31
23	0	0	0	0	87,70
33	0	0	0	0	88,49
50	0	0	0	0	88,65
	Count (cfu/mf) 1110 1040 27 26 24 23 33	Count (cfu/mt) 100 mt  1110 11  1040 11  27 0  26 0  24 0  23 0  33 0	Count (cfu/mt) 100 mt 100 mt  1110 11 4  1040 11 4  27 0 0  26 0 0  24 0 0  23 0 0  33 0 0	Count (cfu/mf)         100 mf         100 mf           1110         11         4         2           1040         11         4         1           27         0         0         0           26         0         0         0           24         0         0         0           23         0         0         0           33         0         0         0	Count (cfu/mt)         100 mt         100 mt         10 mt           1110         11         4         2         0           1040         11         4         1         0           27         0         0         0         0           26         0         0         0         0           24         0         0         0         0           23         0         0         0         0           33         0         0         0         0

Table 37: Microbiological quality of water exposed to various UV doses at a flow rate of  $15,3 \text{ m}^3/\text{h}$ 

UV dose mJ.m <sup>-2</sup>	Standard Plate Count (cfu/m1)		FaecalColiforms 100 mt	E coli/100 m(	Coliphages/	%Transmission
0 (sample 1)	440	5	0	0	0	05.07
0 (sample 2)	446	16	1	1	0	85,87
100	18	0	0	0	0	86,16
120	14	0	0	0	0	86,22
140	13	0	0	0	0	86,32
200	12	0	0	0	0	86,20
220	14	0	0	0	0	86,22
240	19	0	0	0	0	86,00

Table 38: Microbiological quality of water exposed to various UV doses at a flow rate of  $15.0 \text{ m}^3\text{/h}$ 

UV dose mJ.m <sup>-2</sup>	Standard Plate Count (cfu/mf)		FaecalColiforms 100 mt	E coli/100 mt	Coliphages/	%Transmission
0	1210	19	0	0	0	84,63
100	75	0	0	0	0	84,37
120	74	0	0	0	0	84,33
140	70	0	0	0	0	84,06
200	17	0	0	0	0	85,11
220	23	0	0	0	0	84,98
240	52	0	0	0	0	84,88

Table 39: Microbiological quality of water exposed to various UV doses at a flow rate of  $15.5 \text{ m}^3/\text{h}$ 

UV dose	Standard Plate	Total Coliforms	FaecalColiforms	E coli/100 mt	Coliphages/	%Transmission
mJ.m <sup>-2</sup>	Count (cfu/ml)	100 m€	100 mt		10 mC	
0 (sample 1)	180	3	0	0	0	84,63
0 (sample 2)	960	4	0	0	0	84,37
100	15	0	0	0	0	84,33
120	17	0	0	0	0	84,06
140	13	0	0	0	0	85,11
200	19	0	0	0	0	84,98
220	24	0	0	0	0	84,88
240	31	0	0	0	2	

Table 40: Microbiological quality of water after exposed to various UV dosages at a flow rate of approximately  $15 \text{ m}^3/\text{h}$ .

Statistical	Standard Plate	Total Coliforms	FaecalColiforms	E coli/100 mt	Coliphages/10 mf
parameter	Count (cfu/m()	100 m E	100 mt		
Average UV	dose of 29 mJ.cm <sup>-2</sup>				
Minimum	15	0	0	0	0
Maximum	75	0	0	0	0
Median	25	0	0	0	0
Average	32	0	0	0	0
Average UV	dose of 35 mJ.cm <sup>-2</sup>				
Minimum	14	0	0	0	0
Maximum	74	0	0	0	0
Median	19	0	0	0	0
Average	30	0	0	0	0
Average UV	dose of 41 mJ.cm <sup>-2</sup>				
Minimum	8	0	0	0	0
Maximum	70	0	0	0	0
Median	13	0	0	0	0
Average	26	0	0	0	0
Average UV	dose of (29+29) mJ.cn	r <sup>2</sup>			
Minimum	12	0	0	0	0
Maximum	23	0	0	0	0
Median	17	0	0	0	0
Average	17	0	0	0	0
Average UV	dose of (35+29) m.J.cn	1-2			
Minimum	14	0	0	0	0
Maximum	33	0	0	0	0
Median	24	0	0	0	0
Average	24	0	0	0	0
Average UV	dose of (41+29) mJ.cn	y-2		1	
Minimum	19	0	0	0	0
Maximum	52	0	0	0	2
Median	48	0	0	0	0
Average	40	0	0	0	0

Table 41: Microbiological quality of water exposed to various UV doses at a flow rate of  $4.32 \text{ m}^3/\text{h}$ 

UV dose mJ.m <sup>-2</sup>	Standard Plate Count (cfu/m1)		FaecalColiforms 100 mt	E coli/100 mű	Coliphages/	%Transmission
0 (sample 1)	1625	7	7	7	2	86,6
0 (sample 2)	1230	14	14	14	5	80,0
100	29	0	0	0	0	87,4
120	15	0	0	0	57	87,8
140	8	0	0	0	0	87,9
200	21	0	0	0	1	88,1
220	7	0	0	0	0	87,8
240	7	0	0	0	0	87,7

Table 42: Microbiological quality of water exposed to various UV doses at a flow rate of 5,14 m<sup>3</sup>/h

UV dose			FaecalColiforms	E coli/100 mt		%Transmission
mJ.m <sup>-2</sup>	Count (cfu/ml)	100 m (	100 m t		10 m t	
0 (sample 1)	928	12	4	4	0	84,9
0 (sample 2)	660	4	6	6	0	104,5
100	7	0	0	0	0	85,8
120	8	10	5	5	0	86,3
140	4	0	0	0	0	85,9
200	9	0	0	0	0	84,3
220	17	0	0	0	0	88,9
240	34	0	0	0	1	87,3
						1

Table 43: Microbiological quality of water exposed to various UV doses at a flow rate of  $5.14 \text{ m}^3/\text{h}$ 

UV dose	Standard Plate	Total Coliforms	FaecalColiforms	E coli/100 mt	Coliphages/	%Transmission
mJ.m <sup>-2</sup>	Count (cfu/m()	100 mC	3m 001		3m 01	
0 (sample 1)	1280	10	4	4	0	87.7
0 (sample 2)	1110	13	10	10	0	87.7
100	5	0	0	0	0	88.1
120	5	0	0	0	0	88.1
140	6	0	0	0	0	88.6
200	9	0	0	0	0	87.8
220	11	0	0	0	0	83.2
240	34	0	0	0	0	87.5

Table 44: Microbiological quality of water after exposed to various UV dosages at a flow rate of approximately 5 m<sup>3</sup>/h.

Statistical	Standard Plate	Total Coliforms	Faecal Coliforms	E coli/100 mC	Coliphages/10 m(
parameter	Count (cfu/ml)	100 m t	100 m t	L con roo me	
Average UV	dose of 93 mJ.cm <sup>-2</sup>				
Minimum	5	0	0	0	0
Maximum	29	0	0	0	0
Median	7	0	0	0	0
Average	14	0	0	0	0
Average UV	dose of 111 mJ.cm <sup>-2</sup>				
Minimum	5	0	0	0	0
Maximum	15	10	5	5	57
Median	8	0	0	0	0
Average	9	3	2	2	19
Average UV	dose of 130 mJ.cm <sup>-2</sup>				
Minimum	4	0	0	0	0
Maximum	8	0	0	0	0
Median	6	0	0	0	0
Average	6	0	0	0	0
Average UV	dose of (93+93) mJ.cn	2	-		
Minimum	9	0	0	0	0
Maximum	21	0	0	0	1
Median	9	0	0	0	0
Average	13	0	0	0	0
Average UV	dose of (111+93) m.J.c	m <sup>-2</sup>	-		
Minimum	7	0	0	0	0
Maximum	17	0	0	0	0
Median	11	0	0	0	0
Average	12	0	0	0	0
Average UV	dose of (130+93) mJ.c	m <sup>-2</sup>			
Minimum	7	0	0	0	0
Maximum	34	0	0	0	1
Median	34	0	0	0	0
Average	25	0	0	0	0

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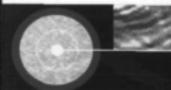
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