RAND AFRIKAANS UNIVERSITY

WATER RESEARCH GROUP

Report to the

WATER RESEARCH COMMISSION

on

OPTIMIZATION OF COMBINED FLOTATION AND FILTRATION AT A LARGE WATER TREATMENT PLANT

by

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Summary

The Rietvlei water treatment plant has recently (1988) been upgraded and redesigned to accommodate dissolved air flotation and filtration (DAFF) in the same process unit. The reason for this drastic change was the eutrophication of the raw water which made conventional sedimentation before filtration impossible due to the high algal content of the water. Rietvlei is the first treatment plant in South Africa that makes permanent use of DAFF at full-scale.

After operating for a number of years it was possible to identify areas where the operation of this process could possibly be improved. Three major areas were investigated namely :

- **Flocculation requirements**
- **Rapid gravity filtration**
- **Recovery of filter backwash water**

The objectives of the investigation regarding **flocculation requirements** were :

- to determine the utility of the Bratby reactor for testing eutrophic water,

- to determine the influence of the primary coagulant, secondary coagulant and dosage selection in terms of the empirical aggregation and breakup constants, K_A and K_B, in order to find an optimal operational point for flocculation.

The Argaman-Kaufman flocculation model with the Bratby reactor, provides a bench scale test method for determining the empirical aggregation and breakup constants, K_A and K_B, for water with a synthetic kaolin suspension at any velocity gradient.

The experimental method for the Bratby reactor was adapted to suit the conditions for algae-laden waters and a series of bench scale tests were conducted for a range of secondary coagulant types.

Full-scale flocculation tests were also conducted to compare the bench-scale predicted aggregation and breakup constants with that obtained in the baffled flocculation channel.

The results indicated that :

☞ flocculation conditions in the baffled flocculation channel can be satisfactory predicted with the use of the Bratby reactor and the Argaman/Kaufman flocculation model,

☞ flocculation can be significantly improved with the aid of a nonionic polyelectrolyte.

The effectiveness of **rapid gravity filters** in the removing of suspended matter as a final clarifying process after flocculation and sedimentation has undergone noteworthy improvement over the last few years. This improvement is due to both a deepening of the knowledge and understanding of filtration mechanisms and technical improvement such as multimedia filtration.

Rietvlei water treatment plant has ten rapid gravity deep-bed filters. The filters are special in the sense that they are combined with dissolved air flotation. The two processes are combined in the same tank and are called the DAFF (Dissolved Air Flotation and Filtration) process. The original treatment plant had four settling tanks and six filters which were converted into ten DAFF units during 1988. The filters for the DAFF units consisted of homogeneous layers of sand (0,70 mm effective size) and 880 mm depth. The sand is supported on thin layers of gravel and grit to prevent the sand from leaking into the pipe lateral underfloor system. Each filter has an effective area of 22,5 m^2. The theoretical rate of filtration then equals 177,8 m/day at full production.

The Rietvlei water treatment plant initially was experiencing filter runs as short as 14 hours while in full production (40 Mℓ/d). As an experiment the top 150 mm of sand of one of the deep-bed sand filters was replaced with fine coal and the latter was later replaced with anthracite. This eventually led to the replacement of the top layer of seven of the other nine filters with anthracite over the period from June 1990 to December 1992. Two filters are still operated with sand only.

Practical experience at the plant indicated that the filter run lengths were indeed improved. During this investigation three questions were addressed :

- How much was the production improved ?
- Was the benefit of longer run lengths due to multimedia filtration economical in terms of production (m/run) on this DAFF plant ?
- Were the multimedia filters at this DAFF plant adequate in terms of quality and breakthrough ?

The quality parameters used to evaluate the final water for the dual-media filters were :

- turbidity
- Fe
- UV adsorption at 254 nm

These parameters and the head loss were monitored for time spans of initially 36 hours and later 48 hours. At first two filters (one single- and one dual-media) were monitored in parallel. This created operating problems and failures on the measuring apparatus and many data had to be disregarded. It was then decided to monitor one filter at a time.

The result of this monitoring process can be summarized as follows :

- ☞ Actual performance data has proved that production increases of 39 % average can be achieved at Rietvlei water treatment plant with the use of dual-media filtration. The percentage increase is further related to the production in the sense that high production rates (m/run) results in a higher increase and vice versa.
- ☞ The use of dual-media filtration at Rietvlei water treatment plant has no adverse affect on the quality of the final effluent.
- ☞ Dual-media filters in use at Rietvlei water treatment plant can safely be operated with head loss being the indicator for backwashing.
- ☞ It is recommended that filter run lengths should not exceed 22 hours.

At Rietvlei, the option to **recirculate the filter backwash water** was made available during the design and construction of the upgrading of the plant. Two settling tanks with a sludge retention tank and recirculation pumps were installed for the purpose of recirculation of filter backwash water. The recirculation option can still be bypassed and the filter backwash water is then released into the Hennops River which runs into Verwoerdburg Lake and eventually into Hartbeespoort Dam.

The problem with the recovery of filter backwash water on a DAFF plant with highly eutrophic raw water is twofold :

- filter backwash water comprises of both suspended material from the float layer on the surface of the filters (due to dissolved air flotation) and the material trapped in the filter bed (due to rapid filtration). Due to the difference in density between suspended material (from the float layer) and suspended matter (from filter), the filter backwash water settles very slowly,
- some of the air bubbles from the flotation process is trapped in the float layer which, if not completely removed, further impedes settling of the backwash water.

The purpose of this section of the study was :

- to increase the recirculation capacity of the backwash recovery system through chemical dosing of the filter backwash water,
- to select a coagulant for this purpose,
- to determine the optimum coagulant dosage and
- to verify the improvement in settling characteristics caused by the coagulant addition.

A chemical dosing unit was installed in the filter gallery just upstream of the outlet to the backwash water settling tanks. The effect of various coagulants were tested by taking samples from the settlings tanks at different heights and times. These were then analyzed for turbidity and settling curves were produced showing the percentage turbidity removed with time for each type of coagulant.

The results can summarized as follows :

- Chemical dosing can reduce settling times, resulting in a reduction in minimum wash cycle lengths.
- Chemical dosing of DAFF filter backwash water have little benefit for settling times longer than 50 minutes with average settling depths of 1,35 m.
- A combination of inorganic and organic coagulants (typically Ferrifloc 1820) gives the best settling results after 25 minutes of settling.
- It is recommended that the filter backwash water at Rietvlei be chemically dosed with 34 mg/ℓ $FeCl_3$ and recycled on a permanent basis.

Opsomming

Die Rietvlei-watersuiweringsaanleg is onlangs opgegradeer en herontwerp om gekombineerde opgelostelug flottasie en filtrasie (OLFF) te akkommodeer. Die rede vir hierdie drastiese verandering was eutrofikasie van die dam en die gevolglike verswakking in rouwaterkwaliteit. Die hoë algkonsentrasies het die konvensionele besinking voor filtrasie feitlik heeltemal oneffektief gemaak. Rietvlei is die eerste aanleg in Suid-Afrika wat op 'n permanente volskaalse basis van die OLFF proses gebruik maak.

Nadat die aanleg vir 'n aantal jare in bedryf was kon sekere moontlike verbeterings ten opsigte van die optimale bedryf ondersoek word. Drie gebiede is ondersoek naamlik :

- Flokkulasietoestande
- Snelsandfiltrasie
- Herwinning van die filterwaswater

Die doel van die ondersoek met betrekking tot die flokkulasietoestande was :

- om te bepaal of die Bratby reaktor geskik sal wees vir gebruik tydens die toets van eutrofe water,
- om die invloed van die primêre koagulant, sekondêre koagulant en dosis keuse op die emperiese versamel- en opbreek konstantes, K_A en K_B, te bepaal.

Die Argaman-Kaufman flokkulasie model tesame met die Bratby reaktor maak dit moontlik om die emperiese versamel- en opbreek konstantes, K_A en K_B, deur middel van bankskaaltoetse, vir sintetiese water met 'n kaolien oplossing, te bepaal teen verskillende snelheidsgradiënte.

Die eksperimentele prosedure vir die Bratby reaktor met kaolien oplossings is aangepas vir eutrofe water en bankskaaltoetse is uitgevoer vir 'n verskeidenheid sekondêre koagulant tipes.

Volskaalse flokkulasietoetse is ook gedoen om die voorspelling van die versamel- en opbreek konstantes, K_A en K_B, verkry met behulp van die Bratby reaktor, te kontroleer.

Die resultate het getoon dat :

- ☞ die Bratby reaktor en die Argaman-Kaufman model, flokkulasietoestande in 'n propvloei reaktor, bevredigend modelleer,
- ☞ flokkulasie aansienlik verbeter kan word deur die byvoeging van 'n nie-ioniese elektroliet.

Die effektiwiteit van snelsandfiltrasie in die proses om van gesuspendeerde materiaal ontslae te raak na flokkulasie en besinking het groot verbetering ondergaan gedurende die afgelope paar jaar. Die verbetering is hoofsaaklik te danke aan die

verbreding van kennis rakende filtrasiemeganimes en tegniese verbeterings soos multimedia filtrasie.

Rietvlei watersuiweringsaanleg het tien diepbedsandfilters. Die filters is spesiaal in die sin dat opgelostelug flottasie in dieselfde reaktor plaasvind. Hierdie proses word OLFF (Opgelostelug Flottasie en Filtrasie) genoem. Die oorspronklike aanleg het vier besinktenks en ses filters gehad wat gedurende 1988 omskep is na tien OLFF eenhede. Die filters bestaan uit homogene sandlae (0,7 mm effektiewe grootte) met 'n diepte van 880 mm. Die sand word ondersteun deur dun lae gruis en grint om lekkasie deur die valsvloersisteem te verhoed. Die effektiewe area van elke filter is 22,5 m^2 met 'n teoretiese filtrasietempo van 177,8 m/dag teen volproduksie.

Die Rietvlei watersuiweringsaanleg het probleme begin ondervind met kort filterlopies, so kort as 14 uur teen vol produksie (40 Mℓ/d). In hierdie tyd is een van die filters se boonste 150 mm sand verwyder en vervang met aanvanklik fyn steenkool en later antrasiet. Sewe van die ander nege filters het later ook 'n laag antrasiet gekry met twee filters wat as net sand filters gehou is.

Praktiese ervaring het aangedui dat die filter looptye wel verleng het. Hierdie deel van die ondersoek het drie vrae aangespreek :

- Met hoeveel het die produksie toegeneem ?
- Het die langer looptye as gevolg van multimedia filtrasie wel die produksie (m/lopie) van die OLFF aanleg verhoog ?
- Is die multimedia filters by die OLFF aanleg voldoende in terme van kwaliteit en deurbreek.

Die kwaliteitsparamaters wat gebruik is om die finale water se gehalte te evalueer was :

- troebelheid
- Fe
- UV adsorpsie by 254 nm

Hierdie parameters is tesame met die hoogteverlies gemonitor vir tydperke van aanvanklik 36 en later 48 uur. Twee filters (een enkelmedia en een multimedia) is aanvanklik gelyktydig gemonitor, maar probleme is met die meettoerusting ondervind wat tot gevolg gehad het dat heelwat data verontagsaam moes word. Daar is besluit om slegs een filter op 'n slag te monitor.

Die resultate van die moniteringsproses was soos volg :

☞ Werklike produksiedata het getoon dat die produksie met 39 % verhoog is as gevolg van multimedia filtrasie. Daar is verder aangetoon dat die toename in produksie tydens hoë filtrasietempo's (m/lopie) wel hoër is as tydens lae filtrasietempo's.

☞ Multimedia filters by Rietvlei het geen nadelige invloed op die finale waterkwaliteit getoon nie.

☞ Hoogteverlies kan met veiligheid gebruik word as indikator vir die was van filters.

☞ Daar word aanbeveel dat filterlooptye beperk word tot 22 uur.

Tydens die opgradering van die aanleg is daar voorsiening gemaak vir die herwinning van die filterwaswater. Twee besiktenks met 'n slykretensietenk en hersirkulasiepompe is geïnstalleer vir die doel. Die opsie om die herwinningstelsel te systap is ook gelaat, sodat water direk in die Hennopsrivier vrygelaat kan word. Die water vloei dan deur die Verwordburgmeer en uiteindelik in die Hartebeespoortdam.

Die probleem met waswaterherwinning by 'n OLFF aanleg is tweeledig :

- die waswater bestaan uit gesuspendeerde partikels wat in die skuim laag as gevolg van flottasie gevang is en dit wat in die filters deur filtrasie gevang is. Die verskil in digtheid van hierdie partikels het tot gevolg dat die partikels baie stadig besink.

- van die lugborrels afkomstig van die flottasie proses word vasgevang tussen die partikels en verhoed verder effektiewe besinking.

Die doel van hierdie deel van die ondersoek was :

- om die hersirkulasiekapasiteit van die waswatersisteem te vergroot deur chemiese dosering,

- om 'n geskikte koagulant vir die doel te selekteer,

- om die optimum koagulant dosis te bepaal,

- om te bevestig of 'n koagulant wel die besinkingseienskappe van die waswater verbeter.

'n Chemiese doseereenheid is in die filtergallery net stroomop van die uitlaat na die waswaterbesinktenks geïnstalleer. Die effek van verskillende koagulante is bepaal deur monsters van die waswater te onttrek op verskillende vlakke en tye in die besinktenks. Hierdie monsters is ontleed vir troebelheid en besinkingskrommes is opgestel wat die persentasie troebelheid verwyder met tyd vir elke koagulant aangedui het.

Die resultate kan soos volg opgesom word :

- ☞ Chemiese dosering kan die besinktye verkort met 'n gevolglike verkorting in die was siklus tye.
- ☞ Chemiese dosering van waswater het slegs 'n geringe effek indien die besinktyd langer as 50 minute is vir 'n 1,35 m diep tenk.
- ☞ A kombinasie van anorganiese en organiese koagulante (tipies Ferrifloc 1820) het die beste resultate gelewer na 25 minute besinktyd.
- ☞ Daar word aanbeveel dat die waswater by Rietvlei watersuiweringsaanleg chemies doseer word met 34 mg/ℓ $FeCl_3$ op 'n permanente basis.

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Chapter 1 : Rietvlei dam and water treatment plant

1.1 Rietvlei Dam

1.1.1 Location and catchment area

Location

Rietvlei Dam is situated on the Hennops River, at 25,87°S and 28,26°E. The dam has been a source of water supply to the City of Pretoria since 1933. The Kempton Park sewage works discharges its treated effluent into the upper Hennops River some 25 km from Rietvlei Dam . This discharge is considered to make up at least 39 % and can be as much as 85 % of the annual inflow into the impoundment (City Council of Pretoria, 1983).

Catchment area

Rietvlei Dam has a catchment area of 492 km^2 which includes the urban complex of Kempton Park and the Van Riebeeck Nature reserve adjacent to the dam. Average precipitation is 675 mm per annum with an average nett evaporation of 968 mm per annum from the impoundment. The mean annual runoff is about 12 million m^3 per annum including the discharge from the Kempton Park sewage works (City Council of Pretoria, 1983). The Hennops River is the main draining channel in the catchment, but is joined by smaller non-perennial tributaries. The river rises in a vlei area a few kilometres east of Kempton Park and passes through a swamp before receiving the discharge of the Kempton Park sewage treatment plant. From Kempton Park it passes through an extensive vlei system and the Marais Dam in the Van Riebeeck Nature Reserve, before flowing into Rietvlei Dam. Figure 1.1 shows the Rietvlei Dam catchment area and non-perennial streams.

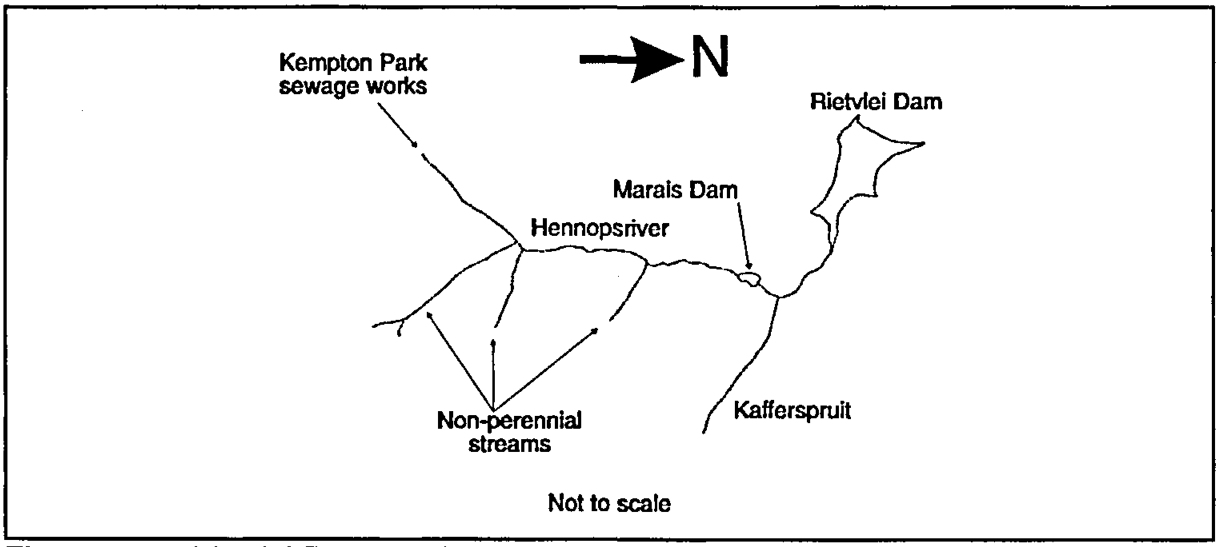

Figure 1.1 : Rietvlei Dam catchment area.

1.1.2 Raw water quality

The waters of this impoundment are nitrogen growth-limiting and as a result of nitrogen fixation, nuisance blooms of algae develop and when present, increase the cost of production of potable water (Ashton, 1976).

Eutrophication

The word eutrophy in general refers to nutrient-rich. The general concepts of oligotrophy and eutrophy are distinguished on the basis of phytoplanktonic populations. Oligotrophic lakes contain less phytoplankton and eutrophic lakes contain more phytoplankton. The term eutrophication is synonymous with excessive growth rates of biota in lakes; higher than the rates that would have prevailed without any disturbances in the form of nutrient enrichment. Increased productivity in a lake can be detected by an increase of carbon assimilated by algae and larger plants per given area. The most important nutrient factors causing the shift from a lesser to a more productive state are phosphorous and nitrogen. Typical plant organic matter of aquatic algae contains phosphorous, nitrogen and carbon in approximately the following ratios :

1 P : 7 N : 40 C per 100 dry algae weight or
1 P : 7 N : 40 C per 500 wet algae weight.

If one of the three elements is limiting and all other elements are present in excess of physical needs, phosphorous can produce 500 times its weight in living algae, nitrogen 71 (500 ÷ 7) times and carbon 12 (500 ÷ 40) times. Phosphorous and secondly nitrogen are generally the first elements to impose limitation to increased productivity of a system (Wetzel, 1983).

The progression of a lake from oligotrophic (low organic and nutrient content) through mesotrophic (moderate organic and nutrient content) through eutrophic to hypereutrophic state is a natural process but is accelerated by human activities. Most lakes are phosphorous limiting i.e. enough nitrogen and carbon are available and once the phosphorous level reaches a certain limit productivity will increase.

A general classification of the state of a lake based on phosphorous and nitrogen content of a lake (Table 1.1) was given by Wetzel (1983) after studying over 200 water bodies.

Table 1.1 : General trophic classification of lakes and reservoirs in relation to phosphorous and nitrogen (after Wetzel, 1983).

Parameter	Oligotrophic	Mesotrophic	Eutrophic	Hypereutrophic
Total phosphorous (mg/m^3)				
Mean	8.0	26.7	84.4	-
Range	3.0-17.7	10.9-95.6	16-386	750-1200
N	21	19	71	2
Total nitrogen (mg/m^3)				
Mean	661	753	1875	-
Range	307-1630	361-1387	393-6100	-
N	11	8	37	-
Chlorophyll a (mg/m^3) of phytoplankton				
Mean	1.7	4.7	14.3	-
Range	0.3-4.5	3-11	3-78	100-150
N	22	16	70	2
Chlorophyll a peaks (mg/m^3)				
Mean	4.2	16.1	42.6	-
Range	1.3-10.6	4.9-49.5	9.5-275	-
N	16	12	46	-
Secchi Transparency Depth (m)				
Mean	9.9	4.2	2.45	-
Range	5.4-28.3	1.5-8.1	0.8-7.0	0.4-0.5
N	13	20	70	2

The phosphate (Figure 1.2) and nitrate (Figure 1.3) frequency distributions for the period from October 1988 to October 1993 were analyzed for the raw water in Rietvlei Dam. The phosphate distribution shows that the phosphorous content has a mean value of 2,15 mg/ℓ and a 90 % confidence range of between 1,26 and 3,84 mg/ℓ. The nitrate distribution graph shows that the nitrogen content has a mean value of 0,80 mg/ℓ and 90 % confidence intervals between 0,11 and 2,86 mg/ℓ.

The ratio of nitrate as N to phosphate as P content for Rietvlei is 0,8 N : 2,17 P or 0,4 N : 1 P. From this and the graphs it is clear that Rietvlei Dam can be classified as hypereutrophic in terms of phosphorus and that the dam is nitrogen limiting, thus having great potential for blooms of blue-green algae during hot conditions.

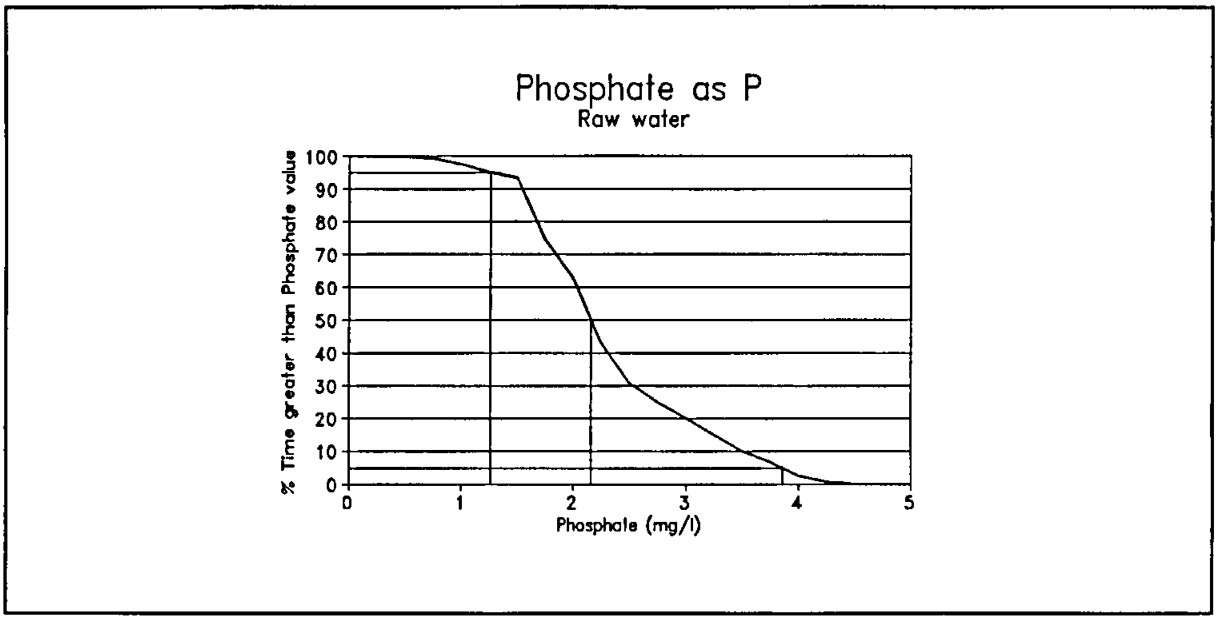

Figure 1.2 : Phosphate frequency distribution for Rietvlei Dam

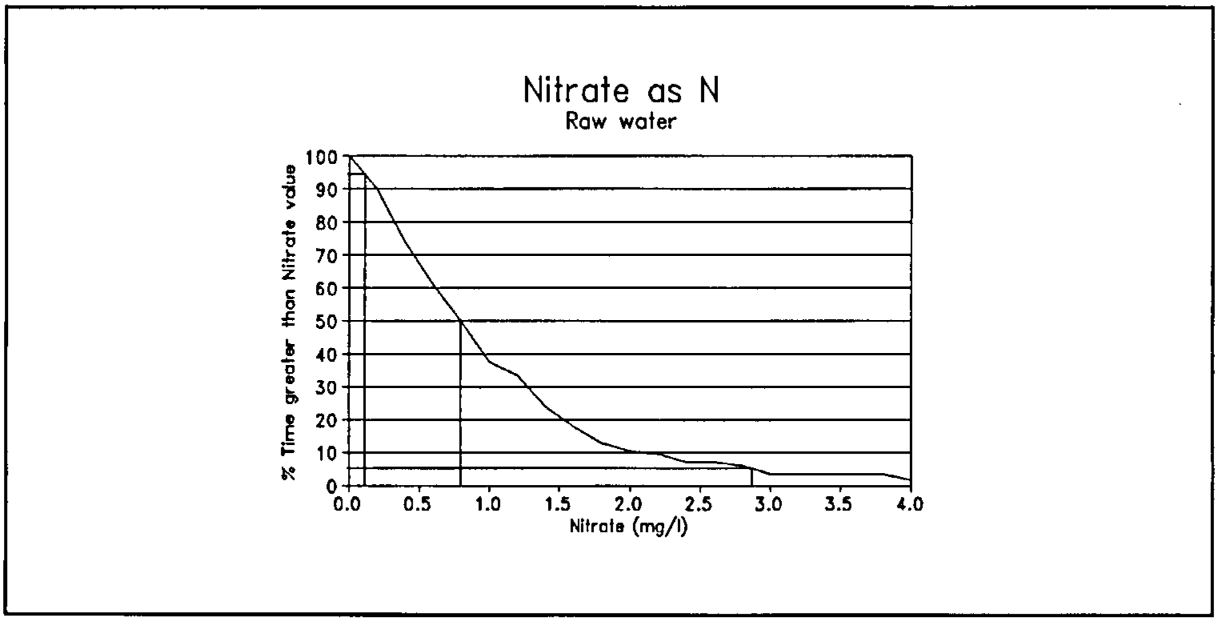

Figure 1.3 : Nitrate frequency distribution plot for Rietvlei Dam

1.2 Rietvlei Water Treatment Plant

1.2.1 History

The City Council of Pretoria started to utilize Rietvlei Dam as a source of drinking water in 1934. The original treatment plant only had sedimentation, filtration and chlorination facilities which operated for almost 50 years before being upgraded in 1988. This upgrading was mainly due to the eutrophication of the impoundment and the concomitant algal production which caused grave operational problems with the original plant set-up.

The redesigned plant was commissioned in June 1988 and has been operated at full design capacity (supply level permitting) since.

1.2.2 Plant configuration after June 1988

The configuration of the current process chain at the Rietvlei Water Purification Plant is explained with reference to Figure 1.4 .

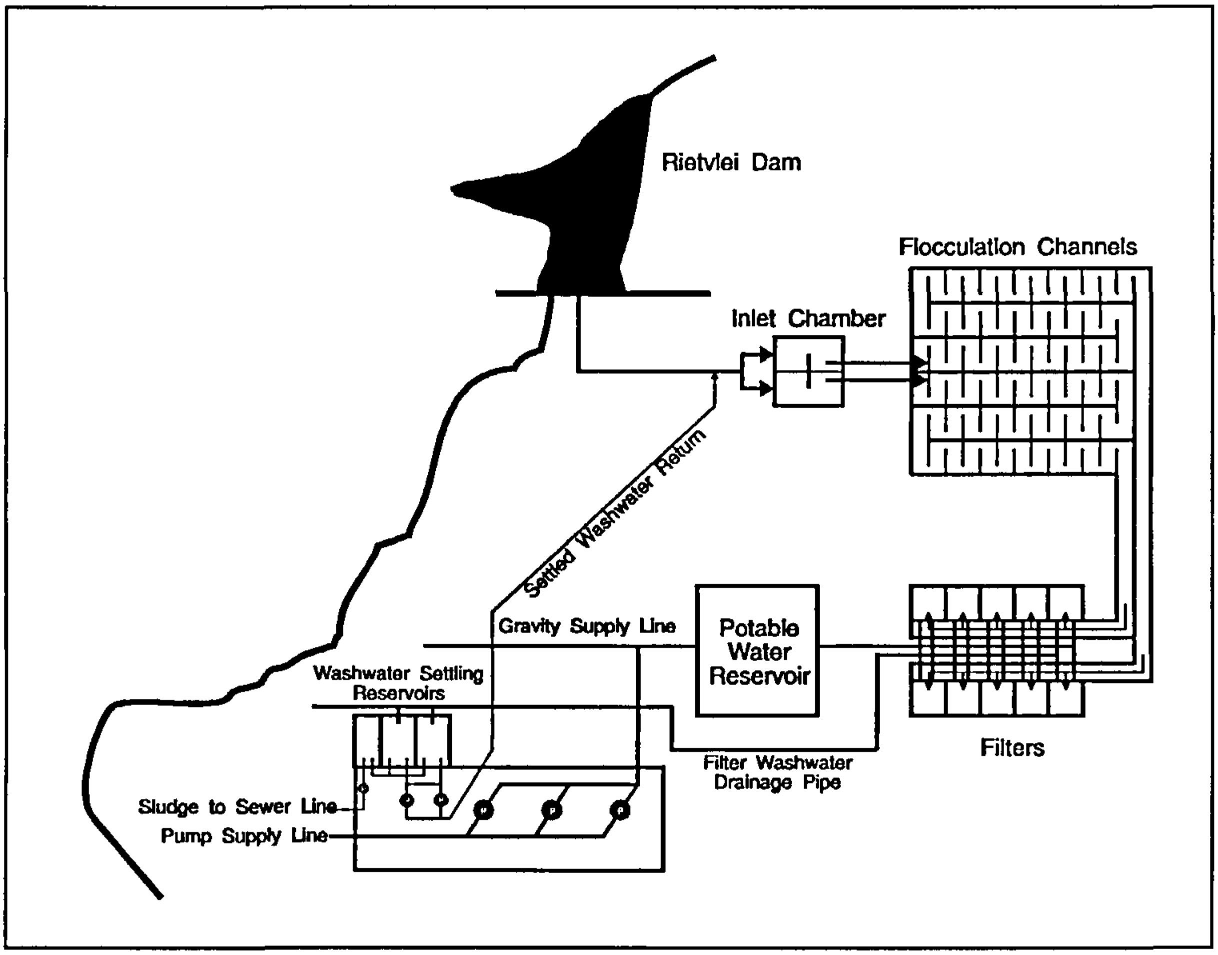

Figure 1.4 : Schematical overview of the Rietvlei Water Treatment Plant.

Water is extracted from the dam by means of the inlet tower (not shown) on the upstream side of the dam wall. This tower has three levels at which water can be extracted. From the intake tower water gravitates to the inlet chamber where chemical dosing takes place after the stream is divided. The divided streams flow through the around-the-end flocculation channels and into ten filtration units where combined filtration/flotation takes place. Chlorine gas is added to the filtered water which is stored in a reservoir before distribution.

The drainage water produced when backwashing filters can be settled in two settling tanks adjacent to the pumphouse. The settled water can also be returned to the upstream end of the inlet chamber. When no recovery of filter backwash water takes place the drainwater gravitates back to the Hennops River.

1.2.3 Final water quality

Water quality had been monitored on a constant basis by the operating personnel and the laboratory personnel of the City Council of Pretoria. The water quality data for the first five years of operation (October 1988 to September 1993) has been extensively analyzed (Haarhoff & Van Beek, 1995). Only four parameters are briefly discussed here.

The turbidity of the final water (after chlorination and mixing with a small percentage of fountain water) was below 1 NTU for 87 % of the time, with an average value of below 0,5 NTU (Figure 1.5). The maximum permissible limit of 5 NTU (DWAF, 1993) was never exceeded.

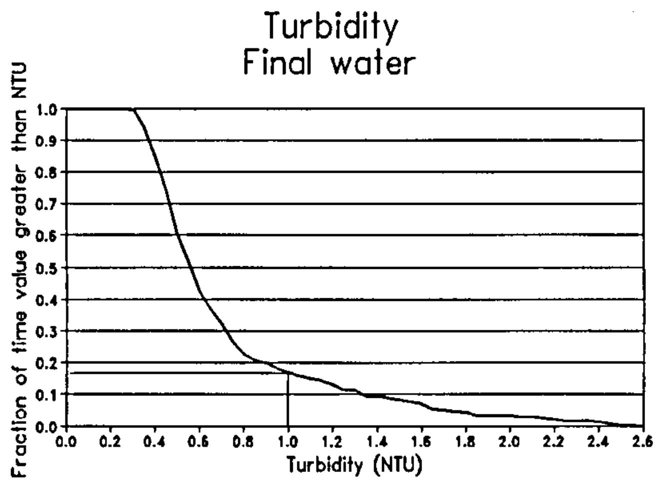

Figure 1.5 : Turbidity frequency distribution of final water for the period 1988-1993.

The colour of the final water was never higher than 30 Hazen units with the average colour around 15 Hazen units.

Figure 1.6 shows the frequency distribution for Chlorophyll a as measured by Waterlab Research (Pty) Ltd for the hydrological years 1989 and 1991. The Chlorophyll a values are higher than 5 µg/ℓ for 10 % of the time with an average value of 1,88 µg/ℓ. This is very high and again indicates the high level of nutrients in the raw water.

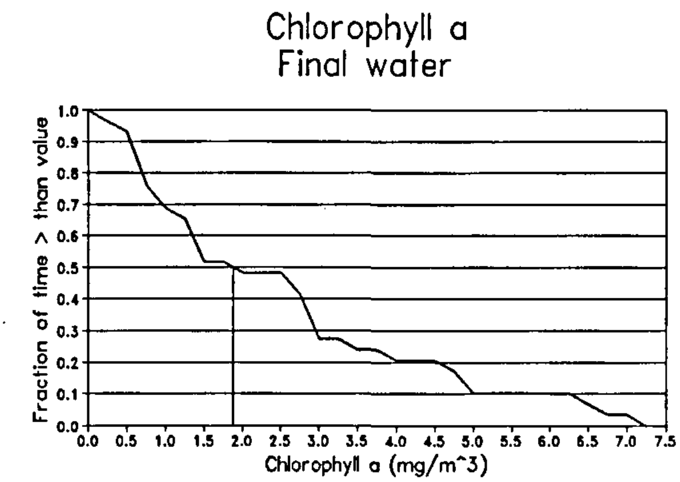

Figure 1.6 : Chlorophyll a frequency distribution in the final water for the period 1989-1991.

The concentrations for the trihalometanes were measured for the 1991 hydrological year and was never above 70 µg/ℓ. This value did not show any change since 1989 despite the fact that the general water quality has deteriorated (Haarhoff, 1993).

Chapter 2 : Flocculation requirements for the DAFF process

2.1 Problem statement

The Rietvlei water treatment plant makes use of a combined flotation filtration process called DAFF (Dissolved Air Flotation and Filtration). DAFF, and DAF (Dissolved Air Flotation) are relatively new processes and is rapidly gaining popularity in South Africa and elsewhere for treatment of eutrophic waters.

DAFF flocculation requirements are poorly defined and often contradictory. Bratby *et al* (1977) showed that tests based on the Argaman & Kaufman flocculation model and executed in the Bratby reactor, provide a promising way for the rational determination of design parameters, for continuous kaolin flocculation from batch test data.

The objectives of this part of the investigation are :

- to determine the utility of the Bratby reactor for testing eutrophic water,
- to determine the influence of the primary coagulant, secondary coagulant and dosage selection in terms of the empirical aggregation and breakup constants, K_A and K_B, in order to find an optimum operational point for flocculation.

2.2 Literature review

2.2.1 Theoretical development of flocculation model

Perikinetic flocculation can cause a change in the number of colloidal particles but has only a minor influence on particles larger than $\pm$ 1 μm. The rate of change of number of particles (Montgomery,1985) in suspension for Brownian flocculation based on the general model for aggregation, assuming no particle break-up, and Von Smoluchowski's collision frequency function is:

$$\frac{dn_T}{dt} = -\frac{4}{3}\alpha\frac{kT}{\mu}n_T^2 \tag{2.1}$$

where :

α	=	collision efficiency factor (ratio of successful collisions to total collisions)
k	=	Bolzmann's constant
T	=	absolute temperature (K)
μ	=	dynamic viscosity (Pa.s)
n_T	=	total particle concentration

Orthokinetic flocculation is used to accelerate the particle aggregation for colloidal and coarse particles by inducing a shear force in the water due to some sort of mixing. The number of particle collisions depends on drag velocities of the particles and thus of the velocity gradient between adjacent layers of liquid (Bratby *et al*, 1977). The difference in velocity across layers of liquid is called the instantaneous velocity gradient G. It should be obvious that this value will differ throughout the profile of the flocculation system.

The aggregation rate of particles due to laminar shear,neglecting break-up, and incorporating Von Smoluchowski's collision frequency function and the instantaneous velocity gradient is :

$$\frac{dn_T}{dt} = -\frac{4}{\pi}\frac{du}{dZ}\alpha\phi n_T \tag{2.2}$$

where :

du/dZ	=	velocity gradient
α	=	collision efficiency factor
ϕ	=	volume fraction of the dispersed phase (volume particles / volume solution)
n_T	=	total particle concentration

For equation (2.2) an uniform velocity gradient throughout the reactor was assumed. This is seldom the case in practice and it is found that the velocity gradient varies under turbulent flow conditions.

The root mean square velocity gradient,G was proposed by Camp & Stein (1943) which calculated the work per unit time put into a unit volume,

$$G = \sqrt{\frac{W}{Vt\mu}} = \sqrt{\frac{P}{V\mu}} \tag{2.3}$$

where :

G	=	the root mean square velocity gradient (s^{-1})
W	=	work energy ($m^2.kg.s^{-2}$)
P	=	work per unit time, power input ($m^2.kg.s^{-3}$)
V	=	volume (m^3)
t	=	retention time (s)
μ	=	dynamic viscosity ($m^{-1}.kg.s^{-1}$)

For water at 20 $^\circ$ C the dynamic viscosity is 0.001005 Nsm^{-2}, the density is 1000 kgm^{-3} and the total loss in energy $=\rho gh$, equation (2.3) then becomes

$$G = 3123\sqrt{\frac{h}{t}} \tag{2.4}$$

where :

h	=	head loss (m)
t	=	retention time (s)

Thus for turbulent flow the rate of particle aggregation becomes proportional to G :

$$\frac{dn_T}{dt} = -KG\phi n_T \qquad (2.5)$$

where :

K = empirical aggregation constant

Argaman and Kaufman (1970) proposed a diffusion model for orthokinetic flocculation. This model is based on the hypothesis that particles suspended in a turbulent regime experience a random motion resembling gas molecules. A simplified bimodal floc size distribution comprising primary particles and large flocs was assumed for analytical purposes. This bimodal distribution was experimentally verified by Argaman and Kaufman by analyzing floc size measurements (Bratby et al, 1977). They further recognised the important fact that the change in particle concentration during flocculation is governed by two opposing processes namely, aggregation and breakup of flocs. The breakup mechanism assumed by Argaman and Kaufman as depicted in equation (2.6) is described by the rate of formation of primary particles by shearing from the floc surface.

$$\left[\frac{dn_1}{dt}\right]_{breakup} = \frac{BR_F^2 n_F \overline{u^2}}{R_1^2} \qquad (2.6)$$

where :

B = breakup constant
R_F = radius of floc
R_1 = radius of particle
n_F = number concentration of flocs
$\overline{u^2}$ = mean square velocity fluctuation (related to G)

Argaman & Kaufman showed by applying their bimodal floc size distribution model to the case of a single completely mixed continuous flow tank reactor that the number concentration of the primary particles can be described by :

$$\frac{dn_1}{dt} = -K_A n_1 G + K_B n_0 G^2 \qquad (2.7)$$

where

K_A = floc aggregation constant
K_B = floc breakup constant
n_0 = number concentration of primary particles at time = 0
n_1 = number concentration of primary particles at time = 1
G = root mean square velocity gradient

Integration and rearranging of equation (2.7) yields :

$$\frac{n_o}{n_1} = \left[\frac{K_B}{K_A}G + \left(1 - \frac{K_B}{K_A}G\right)e^{-K_A G T}\right]^{-1} \tag{2.8}$$

The values of K_A and K_B for a given type of water can be determined empirically in laboratory or pilot scale tests.

The duration of the flocculation is a function of the flow Q and the cross sectional area of the flocculator. For perfect plug flow conditions the duration will be equal to the average distance that the water has travelled divided by the average velocity of the water. In practice the determination of the retention time is not as simple because perfect plug flow is never achieved. There will always be a degree of backmixing in the flocculator and the retention time is best determined by tracer studies.

In many cases designers refer to the dimensionless Gt product, this is simply the product of the root mean square velocity gradient (s^{-1}) and the retention time (s). This number is an indication of the total number of particle collisions. It must be kept in mind that the value for G has an upper limit above which floc breakup will occur.

2.2.2 Recommended flocculation and breakup constants

Some values for flocculation parameters reported in the literature are given below.

Camp (1955) analyzed several existing water treatment plants for flocculation and found that G values from 20 s^{-1} to 70 s^{-1} gave satisfactory flocculation performance. The detention times in these basins were multiplied with the G values and the Gt values ranged from $2*10^4$ to $2*10^5$ (AWWA, 1971).

Typical values for flocculation time before settling and filtration varies from 10 to 40 minutes or even longer. It has been reported by Edzwald *et al* (1992) that flocculation times of 8 and 16 minutes respectively performed equivalent at a dissolved air flotation plant.

Equation (2.7) describes the rate of formation of primary particles during flocculation. The values for K_A and K_B in this equation are unique for each type of reactor and have to be determined experimentally. Table 2.1 below gives some values reported for certain flocculation conditions.

Table 2.1 : Reported kinetic parameters (Montgomery,1985)

System	K_A	K_B	Reference
Kaolin-alum	$4{,}5 \times 10^{-5}$	$1{,}0 \times 10^{-7}$	Argaman (1970)
Kaolin-alum	$2{,}5 \times 10^{-4}$	$4{,}5 \times 10^{-7}$	Bratby (1977)
Natural particles-alum	$1{,}8 \times 10^{-5}$	$0{,}8 \times 10^{-7}$	Argaman (1971)
Alum-phosphate precipitate	$2{,}8 \times 10^{-4}$	$3{,}4 \times 10^{-7}$	Montgomery (1985)
Alum-phosphate plus polymer	$2{,}7 \times 10^{-4}$	$1{,}0 \times 10^{-7}$	Odegaard (1979)
Lime-phosphate, pH 11	$5{,}6 \times 10^{-4}$	$2{,}4 \times 10^{-7}$	Montgomery (1985)

2.2.3 Hydraulic properties of flocculators

Two types of flocculators are commonly found in water purification, namely :
- mechanical and
- hydraulic flocculators

Mechanical flocculators normally consist of some type of mechanical agitator situated in a simple tank. The intensity of the mixing is adjustable but high operating and maintenance costs together with short circuiting of flow are the major disadvantages.

In hydraulic flocculators the energy of the flowing water is used for flocculation. The retention time of all the suspended particles is more or less the same (very little short circuiting exist) in these type of flocculators and they are therefore also referred to as plug flow reactors (AWWA, 1971). Hydraulic flocculators have many advantages which are summarized below :
- low capital cost
- simple to build and use
- no mechanical equipment and therefore low maintenance and operating costs

The major disadvantages are (Montgomerey, 1985):
- mixing intensity cannot be adjusted for a constant flow rate
- mixing intensity is a function of the flow rate and therefore changes with change in flow rate
- high head losses can occur

- the channels need to be cleaned, which can be quite labour intensive.

Rietvlei Water Treatment Plant like most other plants in South Africa makes use of hydraulic flocculators with baffles causing horizontal flow direction changes, i.e. around-the-end-flocculators. The discussion that follows will therefore deal only with hydraulic flocculators.

Perfect plug flow conditions exist when all the particles in suspension move through the flocculator at the same velocity so that no overtaking (short circuiting) takes place. Tracer studies are useful to determine whether plug flow conditions exist or not.

By adding a tracer eg. salt, to the inlet of the flocculation channel and measuring the electric conductivity at time intervals at the outlet, a trace curve can be drawn of conductivity versus time. Perfect plug flow conditions exist when the curve at the outlet has exactly the same shape as the block that was added at the inlet in the form of the tracer. All flocculators will have some deviation from perfect plug flow conditions (Levenspiel, 1972). Two indexes namely the Morril Index and the Short Circuiting Index can be used to determine how closely a flocculator approaches plug flow conditions.

Before these indexes can be applied to a tracer study, the data should be normalized regarding time and concentration in order to make different flocculators comparable. If the initial concentration of the tracer is not known the normalized concentration can be determined by (2.9) (Levenspiel,1972) :

$$C_{normalized} = \frac{C\Delta t}{\Sigma(C\Delta t)} \tag{2.9}$$

where :

C	=	measured concentration
Δt	=	time interval

The hydraulic retention time can be calculated with (2.10) :

$$\bar{t} = \frac{\Sigma(C*t)}{\Sigma C} \tag{2.10}$$

The normalized time can be calculated by dividing the actual measured time, t by the hydraulic retention time $\bar{t}$.

$$t_{normalized} = \frac{t}{\bar{t}} \tag{2.11}$$

From the frequency plot of the normalized data, the Morril Index can be determined : where :

$$\text{Morril Index} = \frac{t_{90}}{t_{10}} \qquad (2.12)$$

t_{10}	=	time at which 10 % of the tracer has passed the outlet.
t_{90}	=	time at which 90 % of the tracer has passed the outlet.

A value of one for the Morril Index will imply perfect plug flow conditions. The Short circuiting Index is expressed as :

$$\text{Short Circuiting Index} = \frac{t_g - t_p}{t_p} \qquad (2.13)$$

where :

t_g	=	true retention time.
t_p	=	time of peak concentration

Plug flow conditions are perfect when this value equals zero.

2.2.4 Flocculation tests with the Bratby reactor

The advantages of laboratory batch tests over pilot scale or full-scale tests are obvious. Laboratory batch tests involve simple inexpensive equipment and shorter testing times compared to pilot scale or full-scale tests. The disadvantage however lies in the uncertainty of scale-up effects.

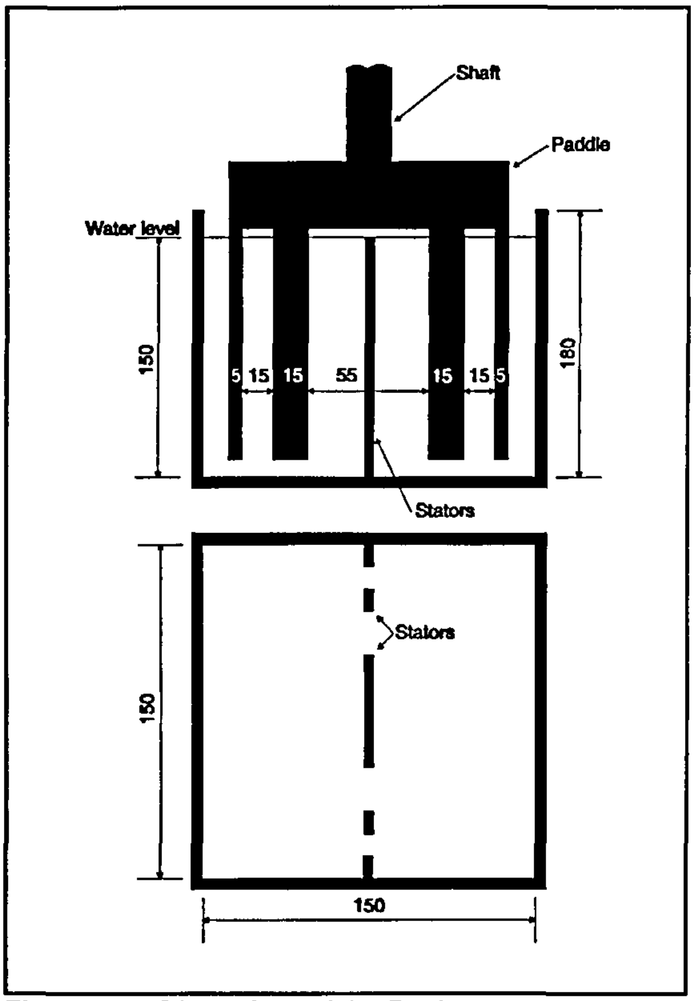

Figure 2.1 : Dimensions of the Bratby reactor

Argaman and Kaufman (1968, 1970) found good correlation between laboratory-scale, continuous type, flocculation tests when compared with a full scale treatment plant. Bratby *et al* (1971) compared laboratory-scale, continuous and batch type flocculation tests. The results indicated that batch data can be applied to full-scale design for kaolin flocculation. The dimensions of the batch reactor used by Bratby is shown in figure Figure 2.1. This reactor has a capacity of 3 ℓ and is fitted with a constant speed motor with pulley arrangements (not shown in Figure 2.1). The pulley value for the root mean square velocity gradient, G, for the reactor was experimentally determined and the relationship is shown in Figure 2.2. Bratby (1981) suggests that this batch reactor can be applied to most turbulent agitation devices for corresponding G values and retention times. The apparatus was fitted with a withdrawal port which made it possible to withdraw various samples from the same test (single G-value) at different flocculation times.

A single test can thus produce a set of N_o/N values with corresponding flocculation times. From such a data set the values for K_A and K_B can be determined using equation (2.8). Seasonal variation in raw water quality does however influence the values of K_a and K_b.

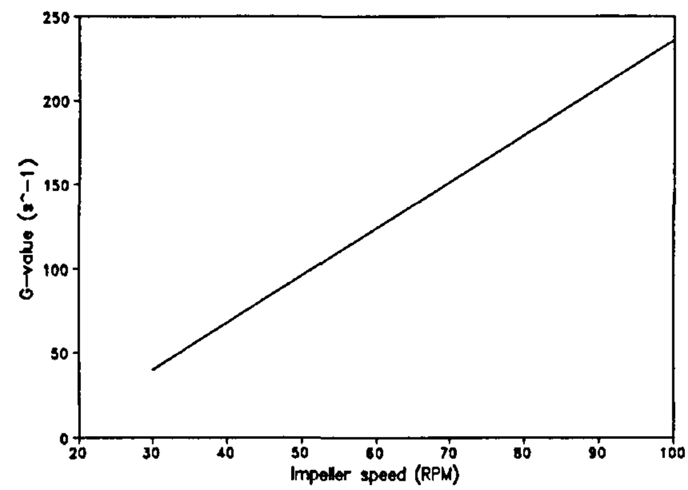

Figure 2.2 : G-values for a range of impeller speeds for the Bratby reactor

2.3 Experimental objectives and methods

2.3.1 Flocculation intensity and duration at Rietvlei Water Treatment Plant

Tracer studies were conducted to determine the G-values and retention time for various flow rates. Salt (NaCl) was dissolved in a large bin and discharged at the inlet of one of the flocculation channels. The electric conductivity was measured with a hand held conductivity meter at 30 second intervals at positions 39,8 m (B) and 128,1 m (D) from the inlet Figure 2.3. The plant was set at operating rates of 11,08 ; 24,86 ; 29,80 ; 33,80 ; and 39,29 Mℓ/d and a tracer study was done at each flow rate. These rates are equivalent to 5,54 ; 12,43 ; 14,90 ; 16,94 and 19,64 Mℓ/d in a single channel.

Previous to each tracer study the water levels were measured at the inlet, A and points B and D. These levels were reduced to a height relative to the bottom of the inlet and the head loss from the inlet to each monitoring position could be determined for every flow rate.

From the normalized graphs the hydraulic retention time was determined and the G-values could be calculated according to Equation (2.4). These values are shown in Table 2.2.

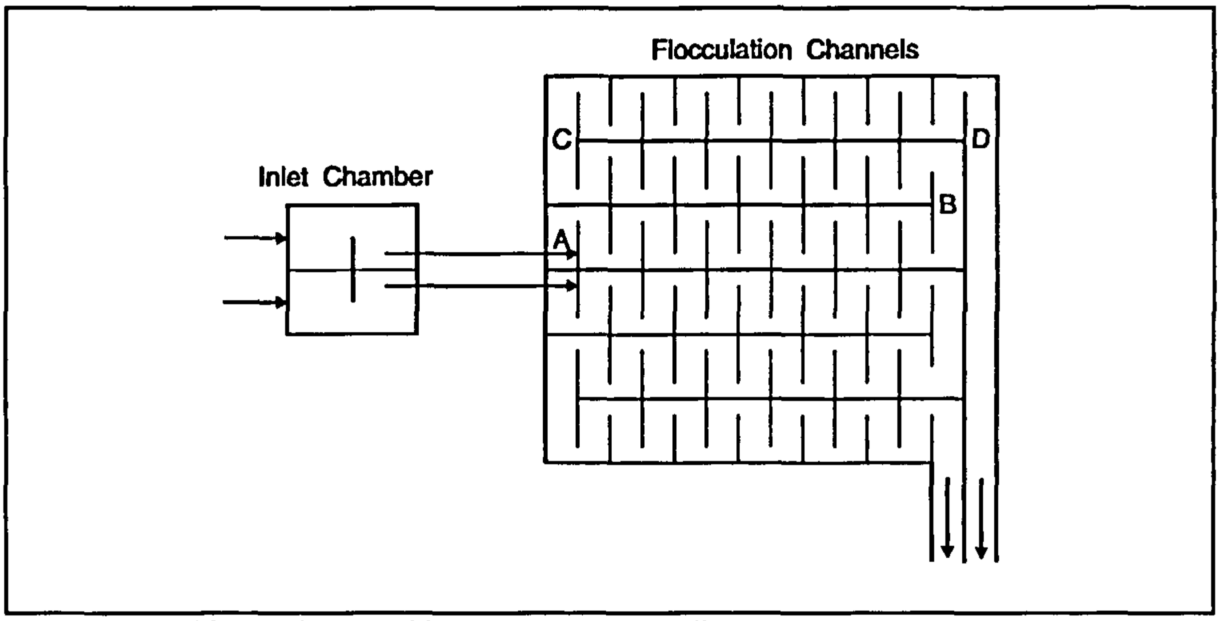

Figure 2.3 : Measuring positions for tracer studies.

Table 2.2 : Measured G-values

Flow for one channel (Mℓ/d)	Time at B (s)	Head loss at B (m)	Velocity gradient at B (s^{-1})	Time at D (s)	Head loss at D (m)	Velocity gradient at D (s^{-1})
5,54	520	0,018	18,4	1580	0,062	19,6
12,43	335	0,048	37,4	998	0,179	41,8
14,90	291	0,075	50,1	830	0,246	53,8
16,94	272	0,078	52,9	770	0,286	60,2
19,64	258	0,088	57,7	713	0,326	66,8

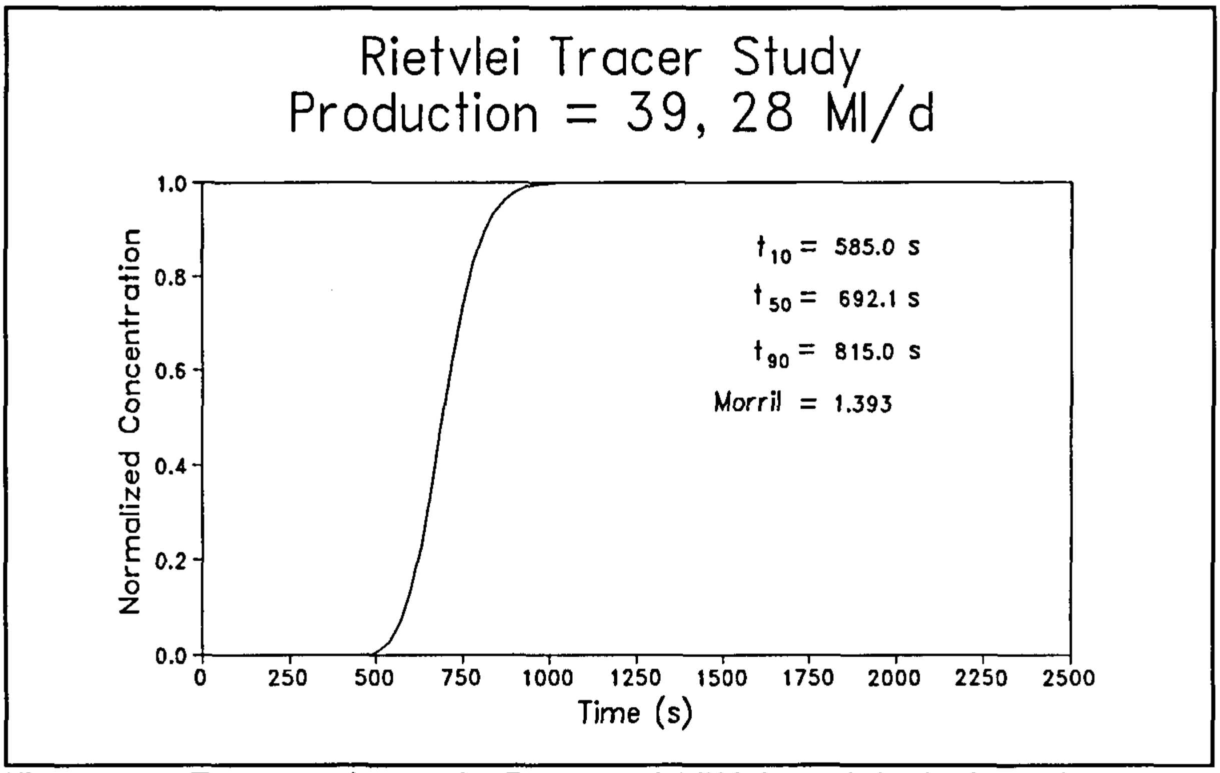

Figure 2.4 : Tracer study at point D for 39,28 Ml/d through both channels.

The normalized concentration frequency distribution for a flow rate of 39,28 Mℓ/d as measured at point D is shown in Figure 2.4 and that for 11,08 Mℓ/d in Figure 2.5.The Morril Index calculated for the higher flow rate is 1,393 and that for the lower flow rate 1,434. This indicate that rather good plug flow conditions exist and that these conditions prevail at full production as well as at low flow conditions.

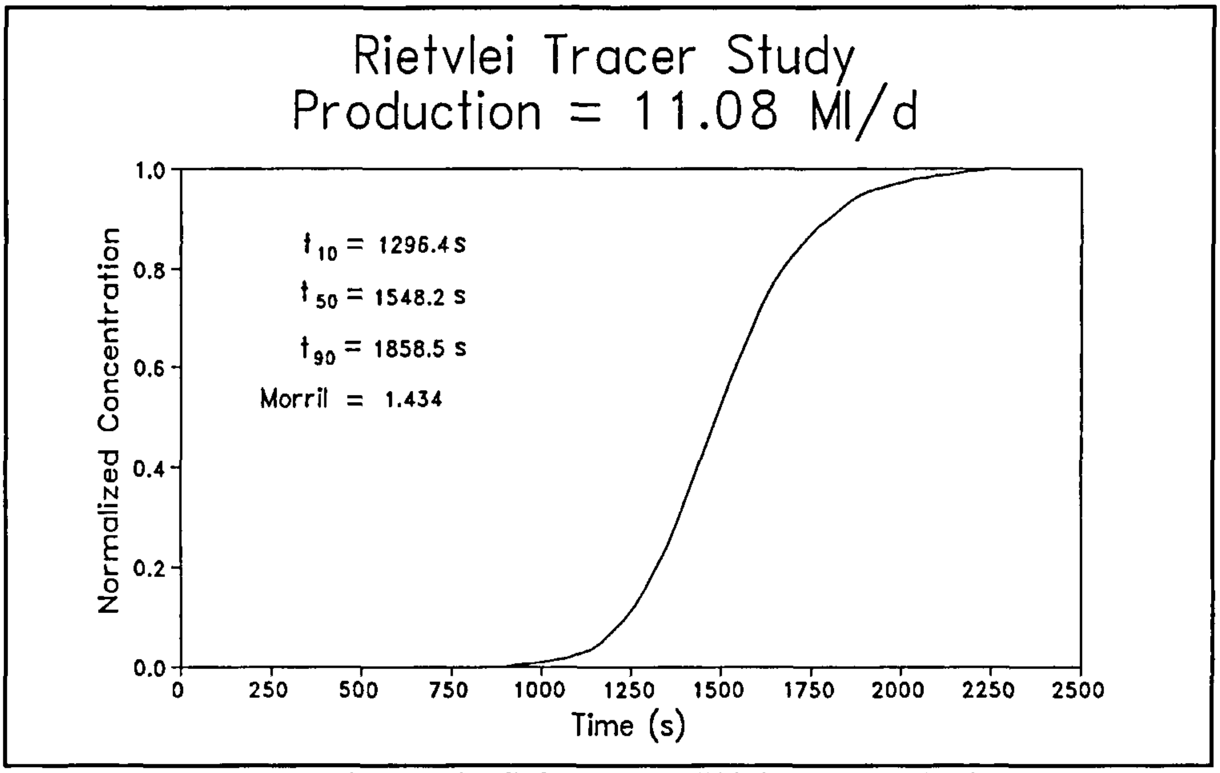

Figure 2.5 : Tracer study at point D for 11,08 Ml/d through both channels.

2.3.2 Adaption of the Bratby test procedure for eutrophic water

An 800 mℓ and 5 ℓ sample (for use in the Bratby reactor) were taken at the inlet (position A) to the flocculation channel and two 800 mℓ samples were taken at positions B and D in the flocculation channel. The three 800 mℓ samples were left to settle for 60 minutes whereafter turbidity readings were obtained after careful withdrawal of 50 mℓ from the water surface with a 90° bent pipette. The turbidity readings for samples B and D were expressed relative to the turbidity of the sample collected at the inlet, A. N_0/N_1 values were thus obtained with N_0 the turbidity of the sample taken at the inlet and N_1 the turbidity of the sample taken at position B or D. Higher values for N_0/N_1 represents more turbidity removed and thus better settling which could be related to better flocculation or performance.

The 5 ℓ sample was added to the Bratby reactor and the motor was set to the correct speed for obtaining the same G-value as that for the flocculation channel. Two 800 mℓ samples were withdrawn after mixing times equal to the hydraulic retention times from the inlet of the channel to the positions B and D for the corresponding G-value. These samples were left for 60 minutes and turbidity readings were obtained after careful withdrawal with a 90° bent pipette.

The turbidity readings were again expressed in terms of the initial turbidity, thus resulting in performance parameters N_0/N_1 for each flocculation time. Substitution of these N_0/N_1 values, the corresponding G-values and retention times in equation (2.8) and solving with the help of a micro-computer resulted in values for K_A and K_B for each comparison test for the channel and Bratby reactor.

2.3.3 Comparison of Bratby reactor and channel performance

With the obvious benefits of bench-scale testing opposed to full-scale testing regarding cost and time, the performance of the Bratby reactor was compared to that of the flocculation channel.

All tests were conducted with raw water from the inlet structure after the addition of the primary coagulant, $FeCl_3$. The values shown for $FeCl_3$ thus represents the optimum dosage for the specific prevailing water conditions.

Table 2.3 shows the results for three comparison tests conducted at different G-values and with different coagulant dosages.

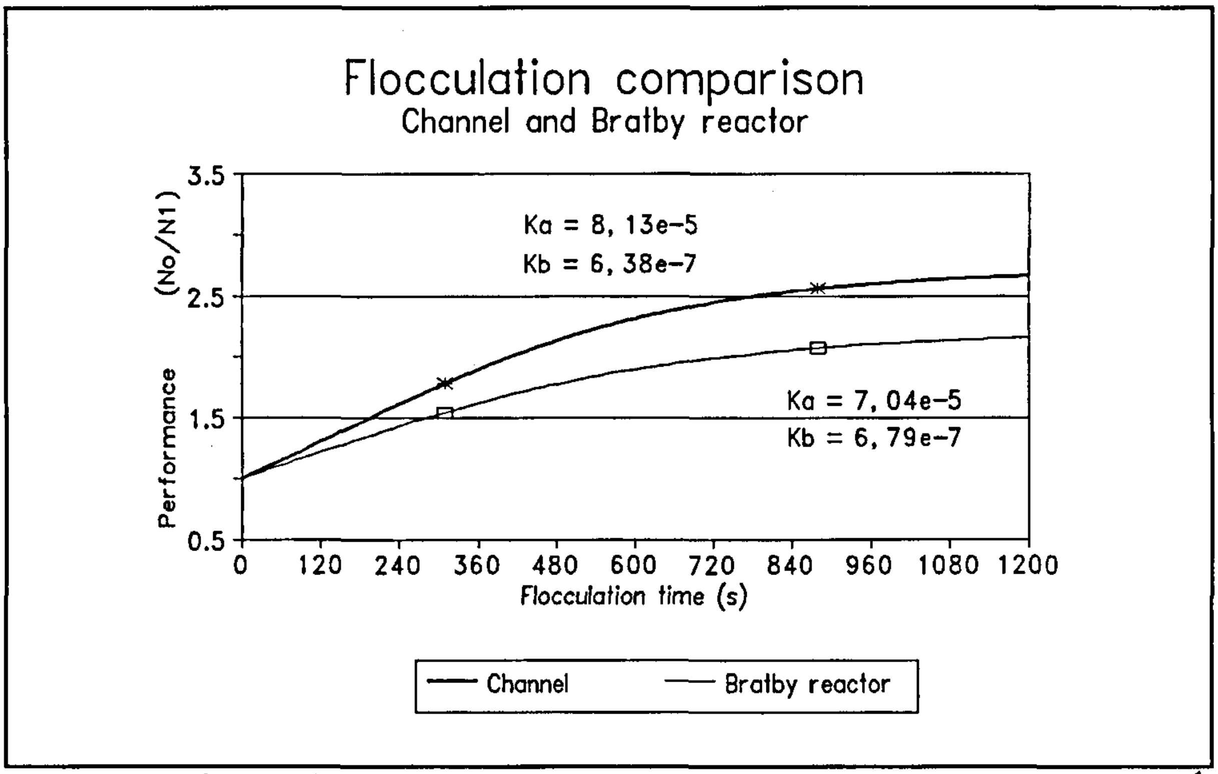

Figure 2.6 : Comparison of Bratby reactor and channel performance, $G = 47\ s^{-1}$, $FeCl_3$: 40 mg/ℓ.

The Argaman & Kaufman flocculation parameters K_A and K_B were used to draw graphs of the performance parameter N_0/N_1 against flocculation time with the data points from Table 2.3 superimposed on these. From Figure 2.6, Figure 2.7 and Figure 2.8 it can be seen that the Bratby flocculator constantly underpredicts

Table 2.3 : Performance comparison between channel and Bratby reactor.

Channel			Bratby reactor		
Retention Time (m:s)	Turbidity after settling (NTU)	Performance parameter (N_0/N_t)	Retention Time (m:s)	Turbidity after settling (NTU)	Performance parameter (N_0/N_t)
$G = 47\ s^{-1}$, $FeCl_3$ = 40 mg/ℓ					
00:00	7,49		00:00	7,49	
05:10	4,21	1,78	05:10	4,85	1,54
14:40	2,93	2,56	14:40	3,62	2,07
$G = 52\ s^{-1}$, $FeCl_3$ = 40 mg/ℓ					
00:00	7,25		00:00	7,25	
04:50	3,25	2,23	04:50	4,10	1,77
13:50	2,43	2,98	13:50	3,07	2,35
$G = 55\ s^{-1}$, $FeCl_3$ = 40 mg/ℓ , 0,7 mg/ℓ nonionic polymer					
00:00	5,31		00:00	5,69	
04:25	2,79	1,90	04:25	3,37	1,69
12:50	1,74	3,05	12:50	2,12	2,68

(approximately 30 %)the performance compared to the actual performance measured in the channel. The actual K_A and K_B values calculated for every test are shown on the graphs.

The third comparison test (Figure 2.8) was carried out at a much later stage (two months later) and it was decided to make use of a nonionic polymer as flocculant aid to the dosed 40 mg/ℓ $FeCl_3$. The results show an overall improvement of the performance and still the same trend in underprediction (approximately 15 %) of performance from the Bratby reactor compared to the actual channel performance.

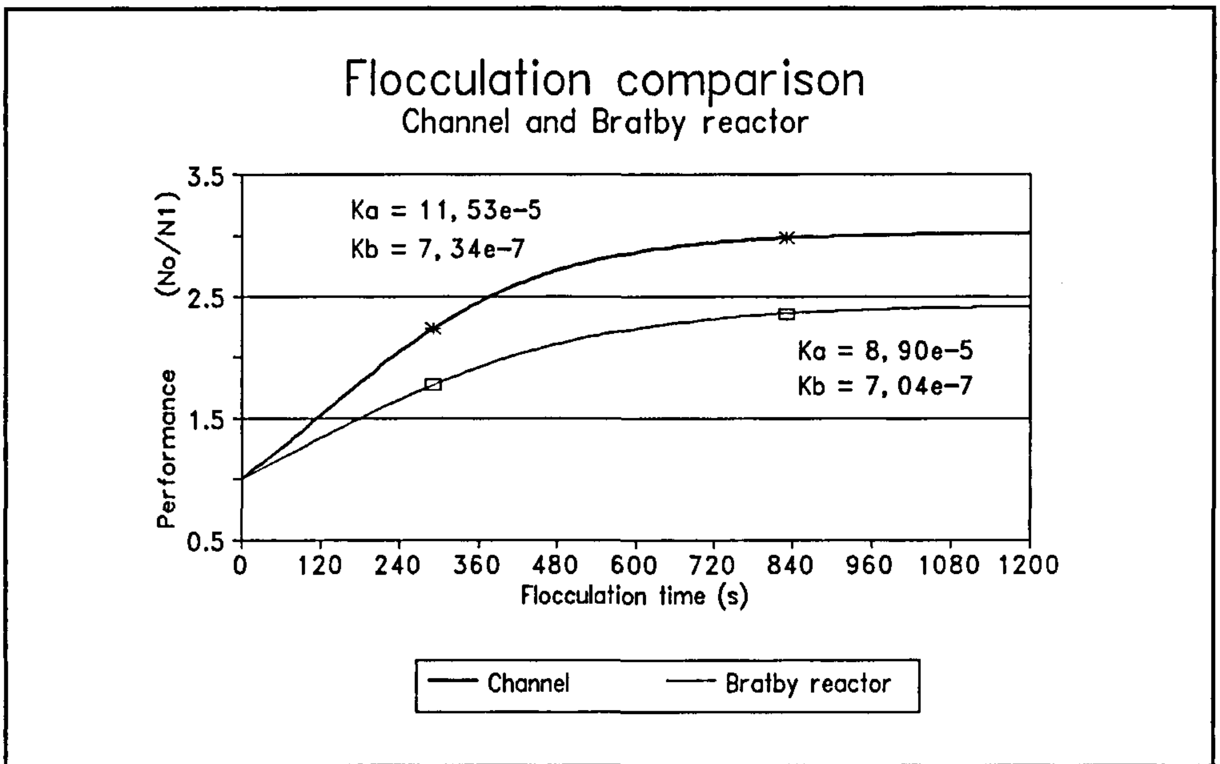

Figure 2.7 : Comparison of Bratby reactor and channel performance, G = 52 s^{-1}, $FeCl_3$: 40 mg/ℓ.

2.3.4 Coagulant aid and optimization

Polymers used

Since the aim of a coagulant aid is to act as a secondary coagulant with the purpose of improving flocculation, three **polymers** were selected for investigation. A local polymer manufacturer supplied three types of polymers, one anionic, one cationic and one nonionic. These polymers were selected on their reputation as secondary coagulants for treatment of eutrophic water types. The three polymers are listed in Table 2.4.

Polymer selection and optimization

The Bratby reactor was used as a bench-scale apparatus for the selection and optimization of polymer dosages.

The first test was a simple comparison of flocculation performance. The flocculation performance of water with 20 mg/ℓ $FeCl_3$ and no coagulant aid and water with 20 mg/ℓ $FeCl_3$ and 0,2 mg/ℓ coagulant aid was compared in the Bratby reactor. Rapid mixing was applied for 90 seconds at the maximum impeller speed, which corresponds to a G-value of 150 s^{-1}. After rapid mixing the impeller speed was

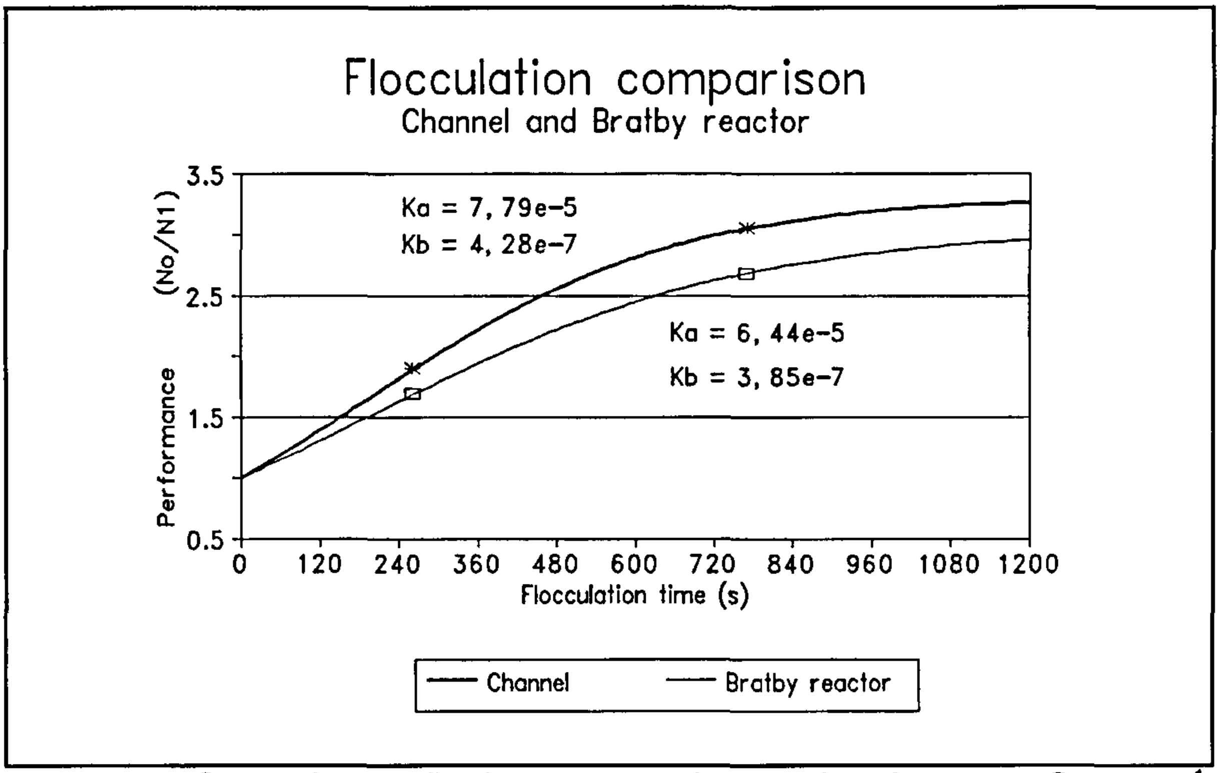

Figure 2.8 : Comparison of Bratby reactor and channel performance, G = 55 s^{-1}, $FeCl_3$: 40 mg/ℓ, nonionic polymer : 0,7 mg/ℓ.

Table 2.4 : Polymers used as secondary coagulants

Name	Ionic charge
4166	Anionic (-)
4157	Nonionic (0)
7125	Cationic (+)

brought down to a G-value of 62 s^{-1}, which corresponds to the G-value measured in the channel at full production. Five tests were done in total, two with no coagulant aid and one each with 0,2 mg/ℓ of the three types of polymers. Four 600 mℓ samples were withdrawn from the Bratby reactor after rapid mixing, 180s, 300s and 720s respectively for each test. The samples were allowed to settle for 60 minutes whereafter turbidity measurements were made after careful sample withdrawal. The results for the five tests are tabulated below.

From the N_0/N_1 values obtained for every sample of each test the flocculation parameters K_A and K_B could be calculated by making use of equation (2.8) . The corresponding G-value used was 62 s^{-1}. These parameters on their own are rather

Table 2.5 : Results from comparison of flocculation performance with and without coagulation aid. (Coagulant used : 20 mg/ℓ $FeCl_3$)

Type of flocculant	Retention Time (min)	Turbidity (NTU)	Performance Parameter (N_0/N_1)	Aggregation constant (K_A*10^{-5})	Breakup constant (K_B*10^{-7})
None	00:00	2,53		5,81	6,18
	03:00	2,09	1,21		
	05:00	1,81	1,40		
	12:00	1,65	1,53		
None	00:00	2,44		5,81	6,18
	03:00	2,19	1,11		
	05:00	2,09	1,17		
	12:00	1,71	1,43		
4166 (-)	00:00	2,42		6,99	6,48
	03:00	1,77	1,37		
	05:00	1,72	1,41		
	12:00	1,41	1,72		
7125 (+)	00:00	2,36		7,17	7,31
	03:00	2,00	1,18		
	05:00	1,62	1,46		
	12:00	1,52	1,55		
4157 (0)	00:00	2,47		6,63	6,19
	03:00	2,00	1,24		
	05:00	1,67	1,48		
	12:00	1,47	1,68		

difficult to evaluate and the values were substituted into equation (2.8) and graphs were plotted for various N_0/N_1 values corresponding with various retention times for each test. These graphs are shown in Figure 2.9.

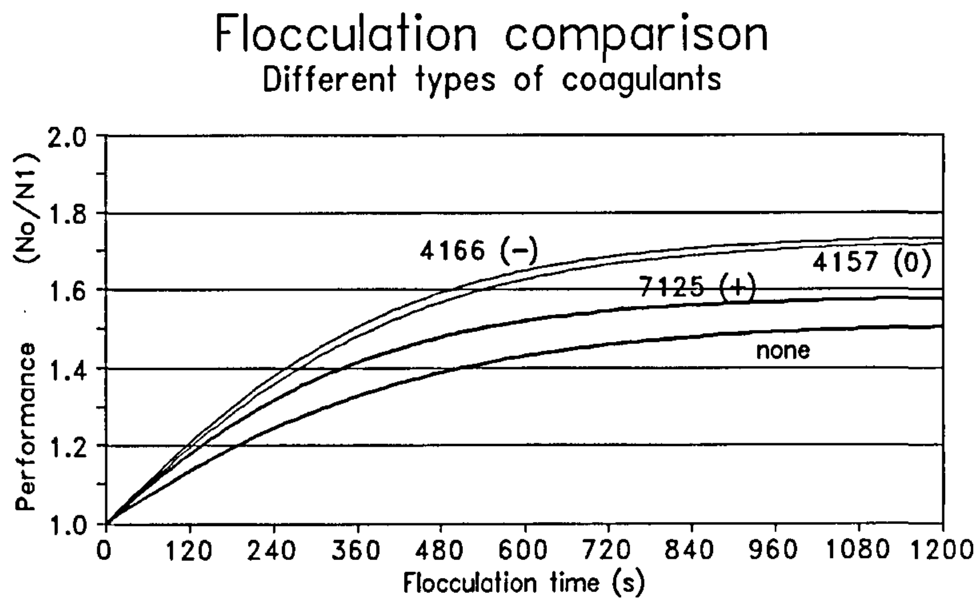

Figure 2.9 : Comparison of flocculation performance with and without different coagulant aids.

These graphs were still very difficult to evaluate since two graphs (4166 and 4157) were very close together. It was decided to plot the improvement of performance for each kind of polymer. The improvement of performance (N_0/N_1) was taken as the difference between the performance curve for each polymer and the performance curve without polymer dosing.

The performances of the polymers were thus expressed relative to the performance without polymer dosing. Figure 2.10 shows the graph with the improvement of performance for each polymer.

From Figure 2.10 it was obvious that the cationic polymer did not perform as well as the nonionic and anionic polymers. The nonionic polymer was selected for dosage optimization as it was more readily available and also had the least restrictions in terms of maximum dosage for use with drinking water.

Polymer dosages of 0,00 ; 0,25 ; 0,45 and 0,70 mg/ℓ were tested with 25 mg/ℓ $FeCl_3$ on a bench-scale with the Bratby reactor. The results of the four tests are shown in Table 2.6 below with the performance graphs depicted in Figure 2.11.

The improvement of performance for every dosage were again plotted and from Figure 2.12 it can be seen that there is significant improvement of performance between a dosage of 0,25 mg/ℓ and 0,7 mg/ℓ of nonionic polymer as coagulant aid to 25 mg/ℓ $FeCl_3$ as primary flocculant.

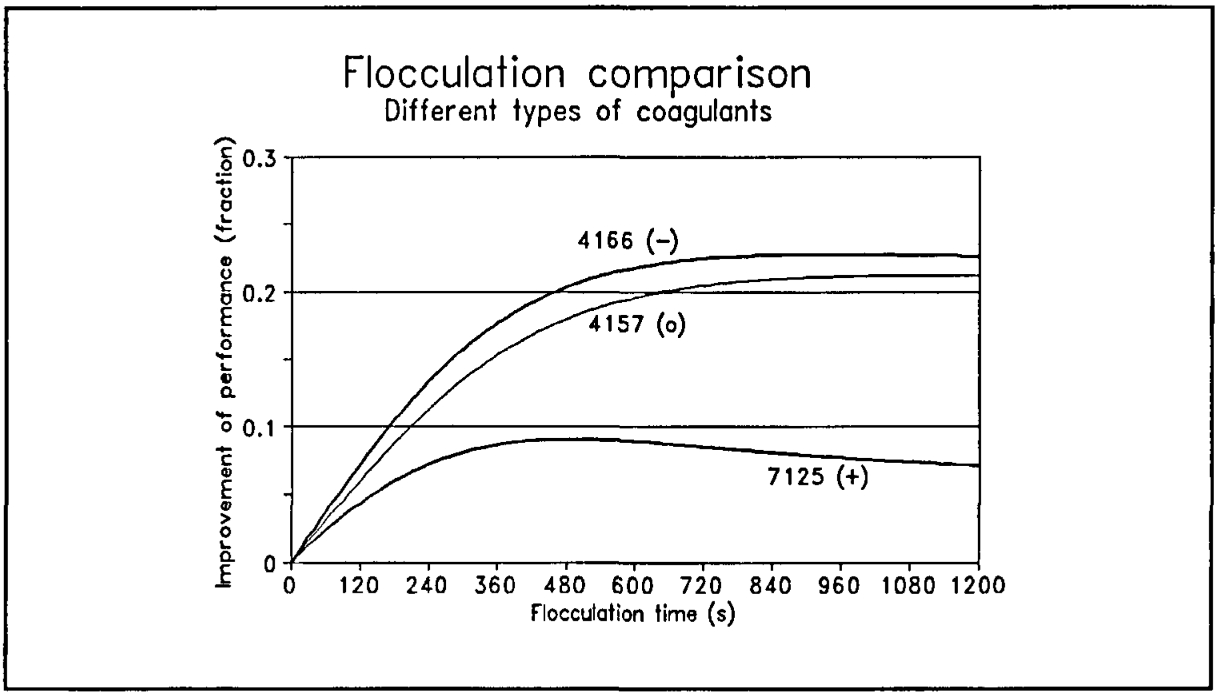

Figure 2.10 : Improvement of performance with coagulant aids.

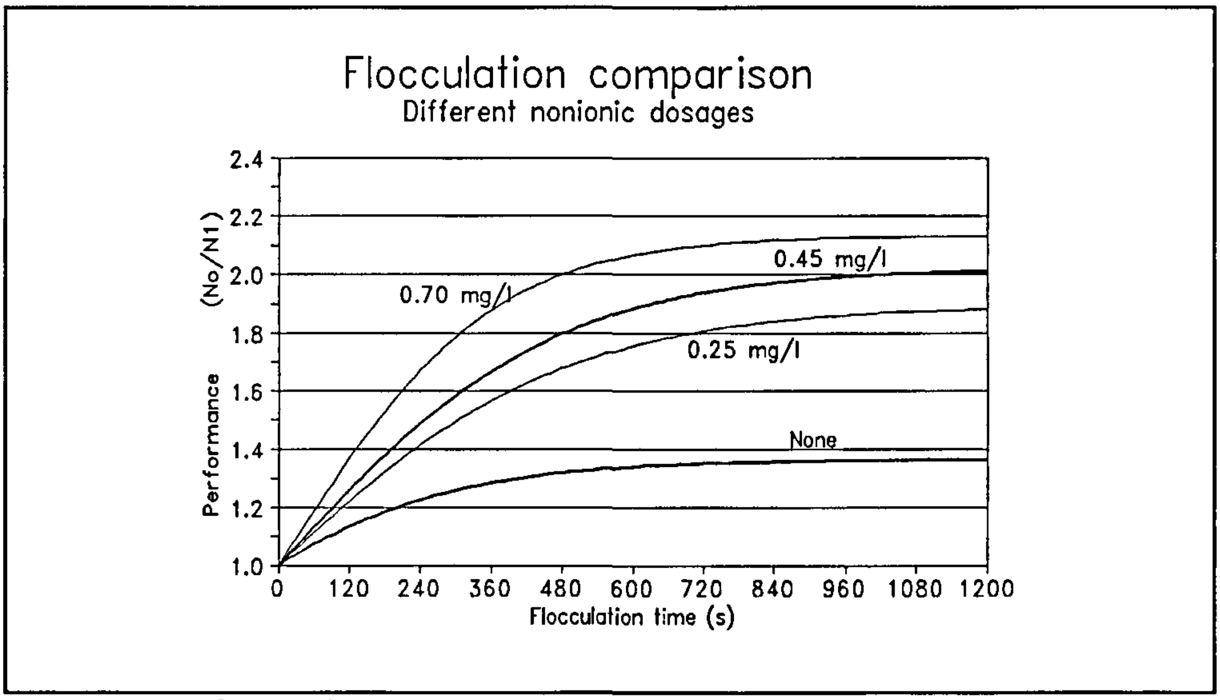

Figure 2.11 : Comparison of different nonionic dosages.

Table 2.6 : Results of performance tests with different dosages of nonionic polymer

Type of flocculant	Retention Time (min)	Turbidity (NTU)	Performance Parameter (N_0/N_t)	Aggregation constant (K_A*10^{-5})	Breakup constant (K_B*10^{-7})
None	00:00	3,03		7,91	9,35
	03:00	2,51	1,21		
	05:00	2,31	1,31		
	12:00	2,17	1,39		
4157 (0) 0,25mg/ℓ	00:00	3,98		9,50	7,18
	03:00	2,91	1,37		
	05:00	2,48	1,61		
	12:00	2,03	1,96		
4157 (0) 0,45mg/ℓ	00:00	3,64		7,04	5,16
	03:00	2,76	1,32		
	05:00	2,41	1,51		
	12:00	1,99	1,83		
4157 (0) 0,70mg/ℓ	00:00	3,69		6,52	5,55
	03:00	2,40	1,54		
	05:00	2,05	1,80		
	12:00	1,73	2,13		

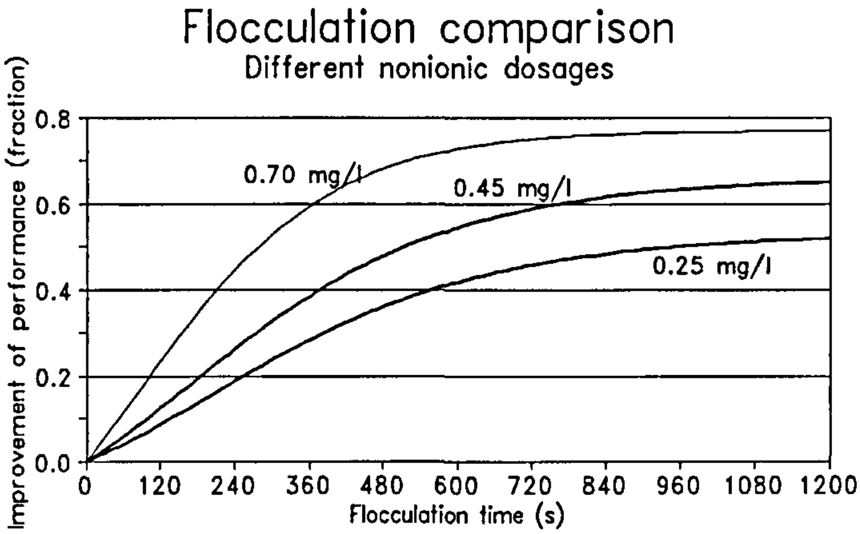

Figure 2.12 : Improvement of performance for different nonionic dosages.

2.3.5 Full-scale trial run

The final step of this part of the investigation was to compare the performance of full-scale polymer dosing as secondary coagulant to the predicted values of the Bratby reactor. The full-scale tests were done over a period of 3 days with polymer dosages of 0,25 ; 0,40 and 0,70 mg/ℓ. The G-values varied between 47 and 55 s^{-1}.

On the first day the plant was run under normal conditions with only 40 mg/ℓ $FeCl_3$ as primary coagulant. A set of three tests were done to determine the flocculation performance parameters. The results of these tests are shown in Table 2.7.

On the second and third days of full-scale testing 40 mg/ℓ $FeCl_3$ with 0,25 ; 0,40 and 0,70 mg/ℓ nonionic polymer respectively was dosed at the inlet chamber. Tests were done during every dosage condition and the results are shown in Table 2.8. From the results in Table 2.7 and Table 2.8 K_A and K_B values could be determined for two situations namely :

- 40 mg/ℓ $FeCl_3$ without coagulant aids, $K_A = 9,62*10^{-5}$, $K_B = 8,46*10^{-7}$
- 40 mg/ℓ $FeCl_3$ with coagulant aids, $K_A = 7,98*10^{-5}$, $K_B = 4,76*10^{-7}$

Table 2.7 : Results of performance tests done on flocculation channel with no coagulant aids.

Type of flocculant	Retention Time (min)	Turbidity (NTU)	Performance parameter (N_0/N_1)	Aggregation constant (K_A*10^{-5})	Breakup constant (K_B*10^{-7})
None	00:00	6,63		9,62	8,46
	04:45	4,16	1,59		
	13:30	3,45	1,92		
None	00:00	7,68			
	05:10	4,13	1,86		
	14:40	3,28	2,34		
None	00:00	7,49			
	05:10	4,21	1,78		
	14:40	2,93	2,56		

Table 2.8 : Results of full-scale performance tests with polymer dosing

Type of flocculant	Retention Time (min)	Turbidity (NTU)	Performance parameter (N_0/N_1)	Aggregation constant (K_A*10^{-5})	Breakup constant (K_B*10^{-7})
4157 (0) 0,25 mg/ℓ	00:00	5,98		7,98	4,76
	04:40	2,92	2,04		
	13:20	2,04	2,93		
4157 (0) 0,40 mg/ℓ	00:00	5,65			
	04:40	3,30	1,71		
	13:20	2,03	2,78		
4157 (0) 0,70 mg/ℓ	00:00	5,31			
	04:25	2,79	1,90		
	12:50	1,74	3,05		

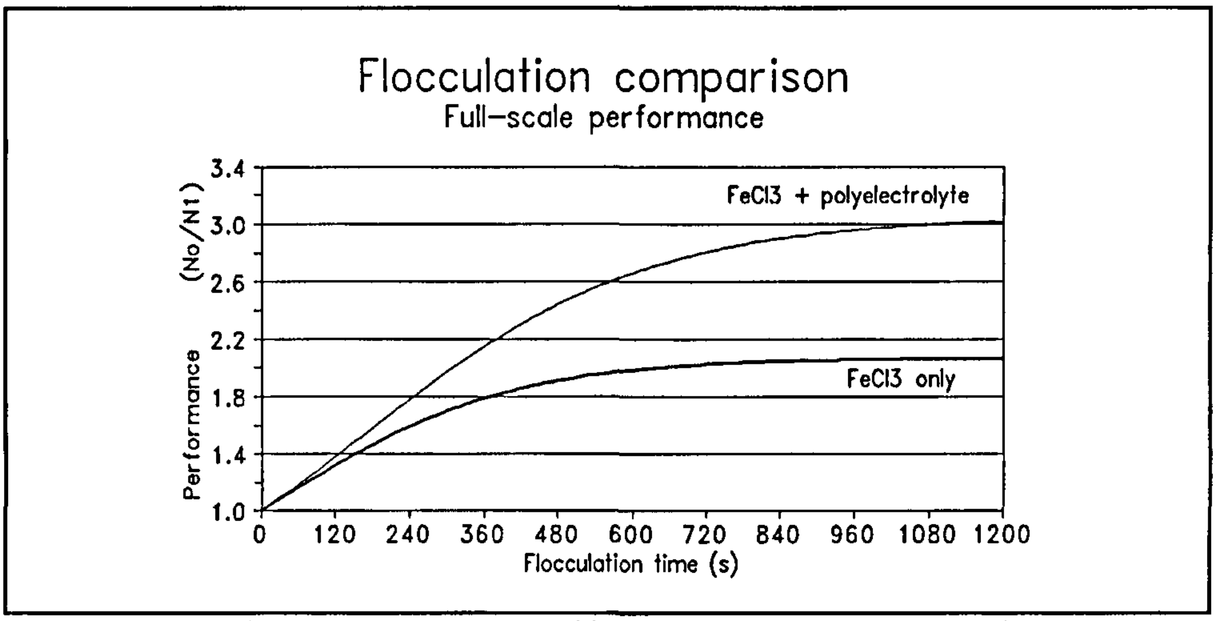

Figure 2.13 : Full-scale performance with polymer (0,7 mg/ℓ) and without polymer dosing.

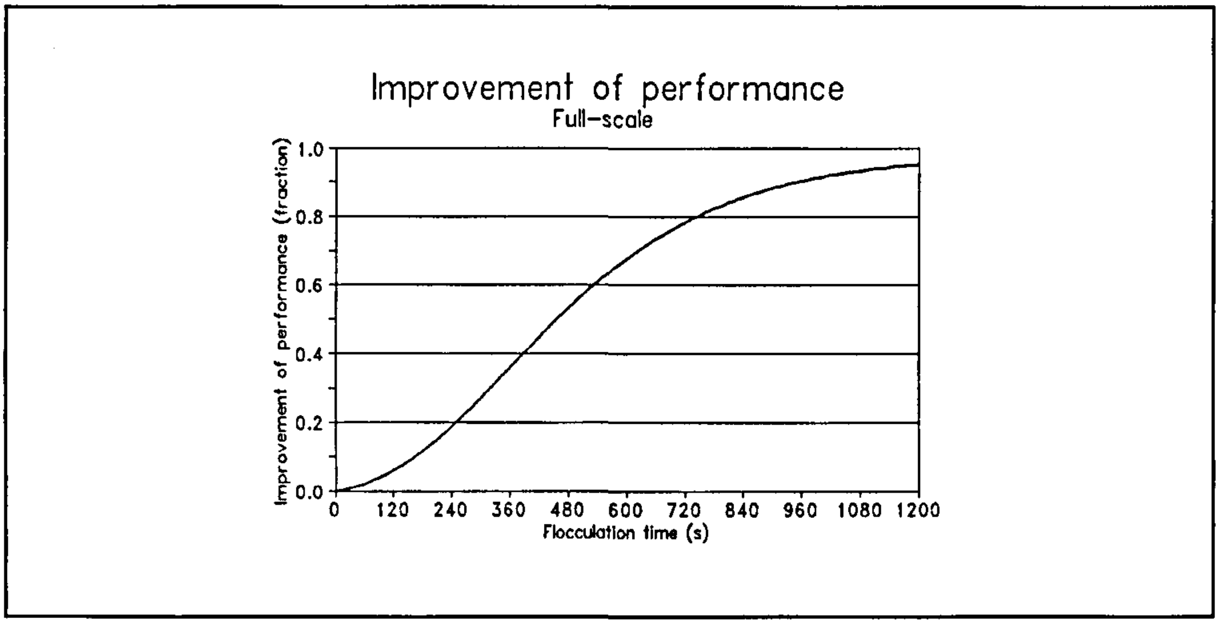

Figure 2.14 : Improvement of full-scale performance with (0,7 mg/ℓ) polymer dosing

The values on their own are difficult to interpret and a graph depicting the performance with and without polymer addition is shown in Figure 2.13. The increase in performance with the aid of polymers relative to no polymer dosing is depicted in Figure 2.14

On the final day of full-scale testing a set of bench-scale tests were conducted with the Bratby reactor. Three tests were done, two with 40 mg/ℓ $FeCl_3$ and no polymer dosing and one with 40 mg/ℓ $FeCl_3$ and 0,7 mg/ℓ nonionic polymer. The results are shown in Table 2.9 and the performance and improvement in performance graphs are shown in Figure 2.16 and Figure 2.17.

Table 2.9 : Results of Bratby reactor tests.

Type of flocculant	Retention Time (min)	Turbidity (NTU)	Performance parameter (N_0/N_1)	Aggregation constant (K_A*10^{-5})	Breakup constant (K_B*10^{-7})
None	00:00	7,25		7,44	6,00
	04:50	4,10	1,77		
	13:10	3,07	2,63		
None	00:00	7,49			
	05:10	4,85	1,54		
	14:40	3,62	2,07		
4125 (0) 0,7 mg/ℓ	00:00	5,69		6,44	3,85
	04:40	3,37	1,69		
	13:20	2,12	2,68		

The improvement of performance for the channel and that predicted by Bratby reactor is plotted on the same graph in Figure 2.15 and again indicates that the Bratby reactor underestimates the performance of the channel.

The Bratby reactor shows a decline in the rate of improvement of performance after 11 minutes while a similar trend is observed for the full scale test after only 9,5 minutes.

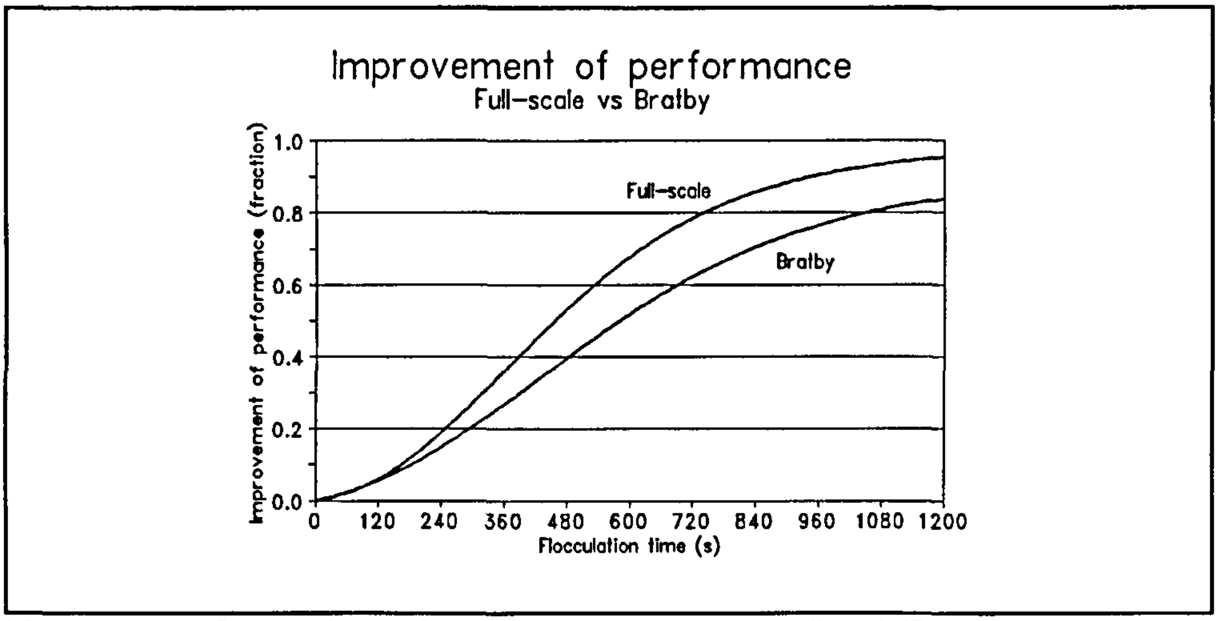

Figure 2.15 : Comparison of actual (channel) and predicted (Bratby) improvement of performance.

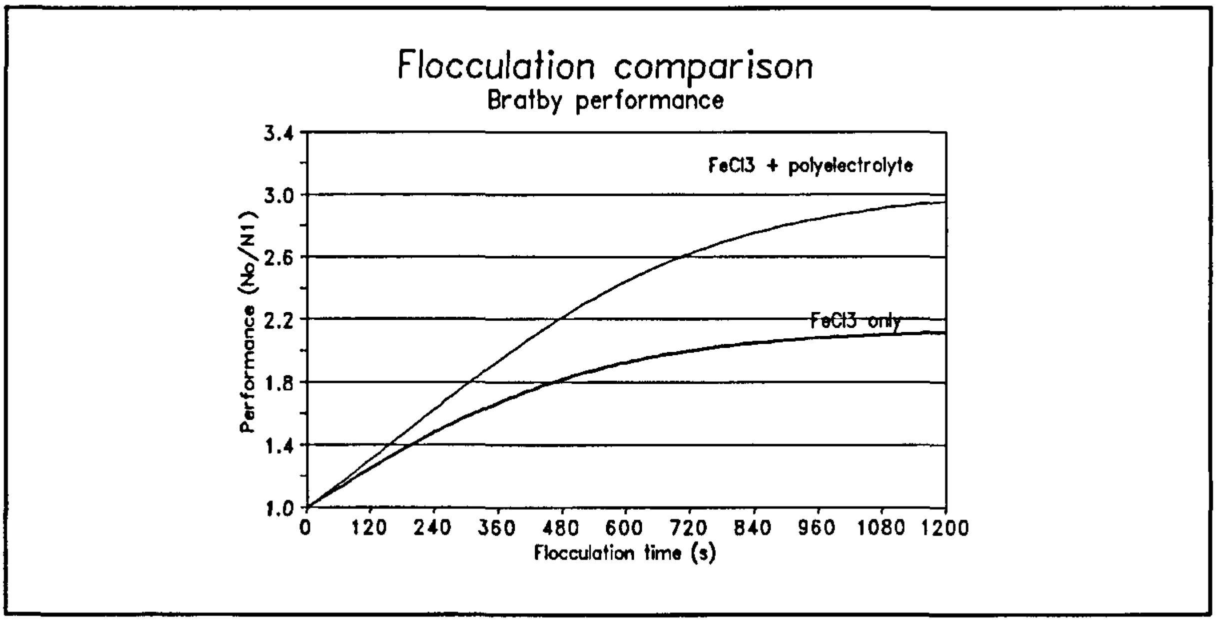

Figure 2.16 : Bench-scale (Bratby) performance with and without polymer dosing.

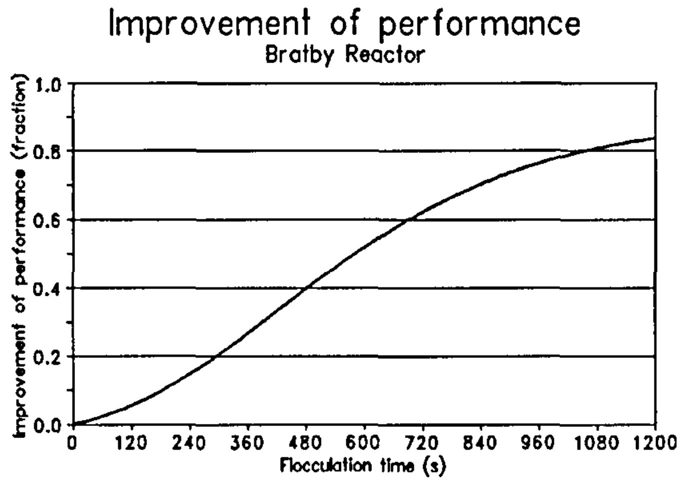

Figure 2.17 : Improvement of bench-scale performance with 0,7 mg/ℓ nonionic polymer dosing

2.4 Results and discussion

Utility of Bratby reactor for testing eutrophic water

Great savings in time and costs can be made by using this bench-scale reactor for flocculation performance tests for eutrophic waters. The test underpredicts the actual performance of an around-the-end flocculation channel by between 15 and 30 %. It can therefore be used as a conservative predictor for full-scale performance. The greater differences seem to appear during very low raw water turbidity conditions.

Polymers as coagulant aid to $FeCl_3$

It was found that either nonionic or anionic polymers can be used to improve the flocculation performance of eutrophic water types. This improvement was between 20 and 75 % for conditions of low turbidity raw water and was proved to be as high as 90 % during high turbidity raw water conditions. The full-scale implementation of secondary coagulant dosing worked well and a dosage of 0,7 mg/ℓ nonionic polymer is suggested as the optimum dosage for treatment of Rietvlei Dam water under the prevailing conditions.

Kinetic parameter comparison

The kinetic parameters, K_A and K_B for eutrophic waters as derived in this study, were compared to reported values for kaolin and natural waters. Due to the complexity of the Argaman/Kaufman model it is difficult to visualize the effect of difference between these values. However, when these values are substituted in the Argaman/Kaufman model for the same G-value and N_0/N_1 versus time is plotted, the effect can be seen clearly. Figure 2.18 shows such a simulation for kaolin, natural and algae laden waters. From this graph it is clear that according to the Argaman/Kaufman model, algae laden water from Rietvlei Dam performs much worse than water with natural or kaolin suspensions in terms of the coagulation and aggregation of particles.

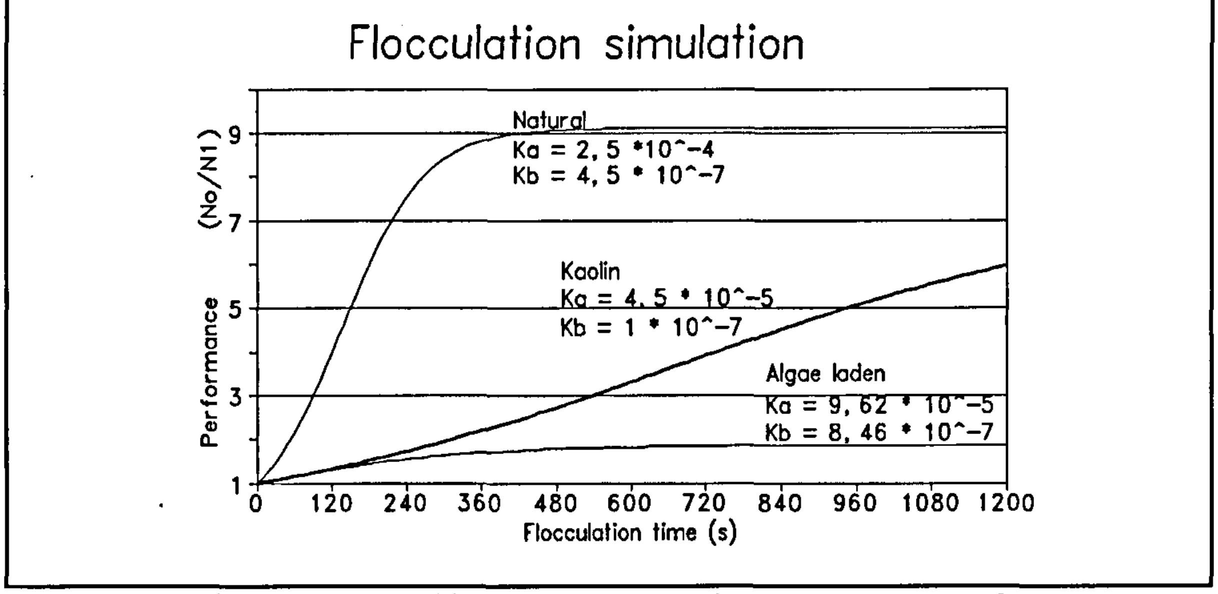

Figure 2.18 : Comparison of kinetic parameters for different types of waters.

Effect of polymer addition on kinetic parameters.

For the Argaman/Kaufman model it is generally true to state that an increase in K_A (agglomeration constant) would result in an increase in the performance parameter, N_0/N_1 and that an increase in K_B (breakup constant) would result in a decrease in the performance parameter, N_0/N_1.

The addition of a polymer as coagulant aid did not significantly influence the agglomeration constant for algae laden water. During the full-scale test, the value of K_a decreased slightly from $9{,}62*10^{-5}$ to $7{,}98*10^{-5}$. The breakup constant however showed a significant decrease as the original value for $K_b = 8{,}46*10^{-7}$ was nearly halved to $K_b = 4{,}76*10^{-7}$ with the addition of a nonionic polymer. This proves that the benefit of adding a polymer as coagulant aid is primarily due to less breakup during flocculation.

2.5 Summary

- The Bratby reactor can be used as a conservative predictor for full-scale flocculation performance with eutrophic water.

- Nonionic polymer addition as secondary coagulant, slightly reduces the agglomeration constant, K_A but greatly reduces the breakup constant K_B thus resulting in better flocculation performance.

- The kinetic parameters K_A and K_B are much lower for eutrophic water than for any other water type and can therefore be expected to perform worse during flocculation.

Chapter 3 : Rapid gravity filtration

3.1 Problem statement

The effectiveness of rapid gravity filters in the removing of suspended matter as a final clarifying process after flocculation and sedimentation has undergone noteworthy improvement over the last few years. This improvement is due to both a deepening of the knowledge and understanding of filtration mechanisms and technical improvement such as multimedia filtration.

Rietvlei water treatment plant has ten rapid gravity deep-bed filters. The filters are special in the sense that they are combined with dissolved air flotation. The two processes are combined in the same tank and are called the DAFF (Dissolved Air Flotation and Filtration) process. The original treatment plant had four settling tanks and six filters which were converted into ten DAFF units during 1988. The filters for the DAFF units consisted of homogeneous layers of sand (0,70 mm effective size) and 880 mm depth. The sand is supported on thin layers of gravel and grit to prevent the sand from leaking into the pipe lateral underfloor system. Each filter has an effective area of 22,5 m^2. The theoretical rate of filtration then equals 177,8 m/day at full production.

The Rietvlei water treatment plant initially was experiencing filter runs as short as 14 hours while in full production (40 Mℓ/d). As an experiment the top 150 mm of sand of one of the deep-bed sand filters was replaced with fine coal and the latter was later replaced with anthracite. This eventually led to the replacement of the top layer of seven of the other nine filters with anthracite over the period from June 1990 to December 1992. Two filters are still operated with sand only.

Practical experience at the plant indicated that the filter run lengths were indeed improved. During this investigation three questions were adressed :

- How much was the production improved ?
- Was the benefit of longer run lengths due to multimedia filtration economical in terms of production (m/run) on this DAFF plant ?
- Were the multimedia filters at this DAFF plant adequate in terms of quality and breakthrough ?

3.2 Experimental objectives and methods.

3.2.1 Calculation of average weekly filter production

For the period 14 November 1990 (week 46) to 6 December 1992 three filters 1,2 and 4 were operating on sand only. Filter number 9 consisted of dual-media, coal and sand, and six filters 3,5,6,7,8,10 operated on dual-media anthracite and sand. The daily production sheets kept by the operators give a summary at the end of each week of the average daily plant production in $M\ell/d$. The number of times each filter was washed during that week is also indicated. Assuming that the filters always operate at a constant filtration rate depending on the average plant production, the production of an individual filter can then be determined with the following sample calculation :

Weekly average daily plant production	=	39,7	$M\ell/d$
Total number of filter washes	=	119	no/week
Number of filter washes for Filter 10	=	9	no/week
Surface area of filter	=	22,5	m^2
Number of filters	=	10	no

$$\frac{\text{avg produc/day} * 7}{\text{total no filters}} * \frac{1000}{\text{area/filter} * \text{no washes/filter}} = 137{,}2\,\text{m/run} \tag{3.1}$$

The average run length for filter number 10 then is :

$$\frac{\text{m/run}}{(\text{production/day})/\text{total filter area}} * 24 = 18{,}66\,\text{hours} \tag{3.2}$$

The production could then be determined for each filter as an average weekly value for the monitoring period mentioned.

3.2.2 Final effluent quality monitoring

The head loss through any filter is monitored electronically as a pressure differential across the filter media. As the resistance increase the downstream valve is gradually opened to allow the flow rate to remain constant. With the valve fully open and further increase in the resistance through the clogged media. The water level above that specific filter will rise and a signal light connected to a height probe will indicate to the operator that the filter should be washed. This system proved to be adequate in terms of the final water turbidity. Figure 3.1 shows that the turbidity of the final water was less than 1.52 NTU for 90 % of the time when only single media filters were in operation. This implies that the maximum head loss level was low enough to ensure no turbidity leakage before the filter is washed on the grounds of head loss. This operation system had to be checked for the new dual-media filters.

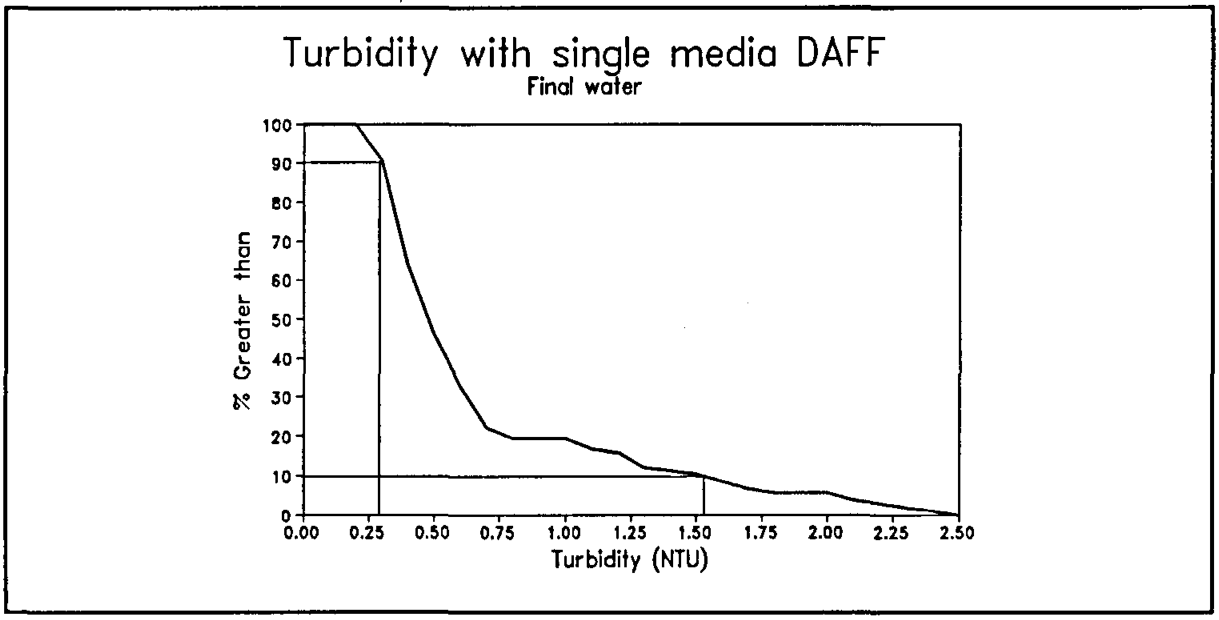

Figure 3.1 : Final water turbidity frequency plot (for the period October 1988 to June 1990 with turbidity measurements taken every 8 hours)

The quality parameters used to evaluate the final water for the dual-media filters were :

- turbidity
- Fe
- UV adsorption at 254 nm

These parameters and the head loss were monitored for time spans of initially 36 hours and later 48 hours. At first two filters (one single- and one dual-media) were monitored in parallel. This created operating problems and failures on the measuring apparatus and many data had to be disregarded. It was then decided to monitor one filter at a time. The measuring techniques will be discussed in detail below.

i) Pressure measurement

The head loss across the filter was measured using two Vegabar P10RMP pressure transducers with an output signal of 4 to 20 mA. The output signal was converted to a 0 to 5 V signal which could be read into a micro computer. The calibration of the transducers was done with a piezometer as shown in Figure 3.2 . The one pressure transducer (PT1) was connected downstream of the inlet valve to the filter on the inlet pipe and the other (PT2) was connected upstream of the outlet valve on the outlet pipe of the filter. Figure 3.3 shows these locations. The head loss could then be determined by adding the difference in elevation of the two probes to the height recorded by PT1 and then subtracting the value recorded by PT2. The maximum theoretical head loss as discussed in 3.2.2 will only be recorded if the outlet valve downstream of PT2 is fully open.

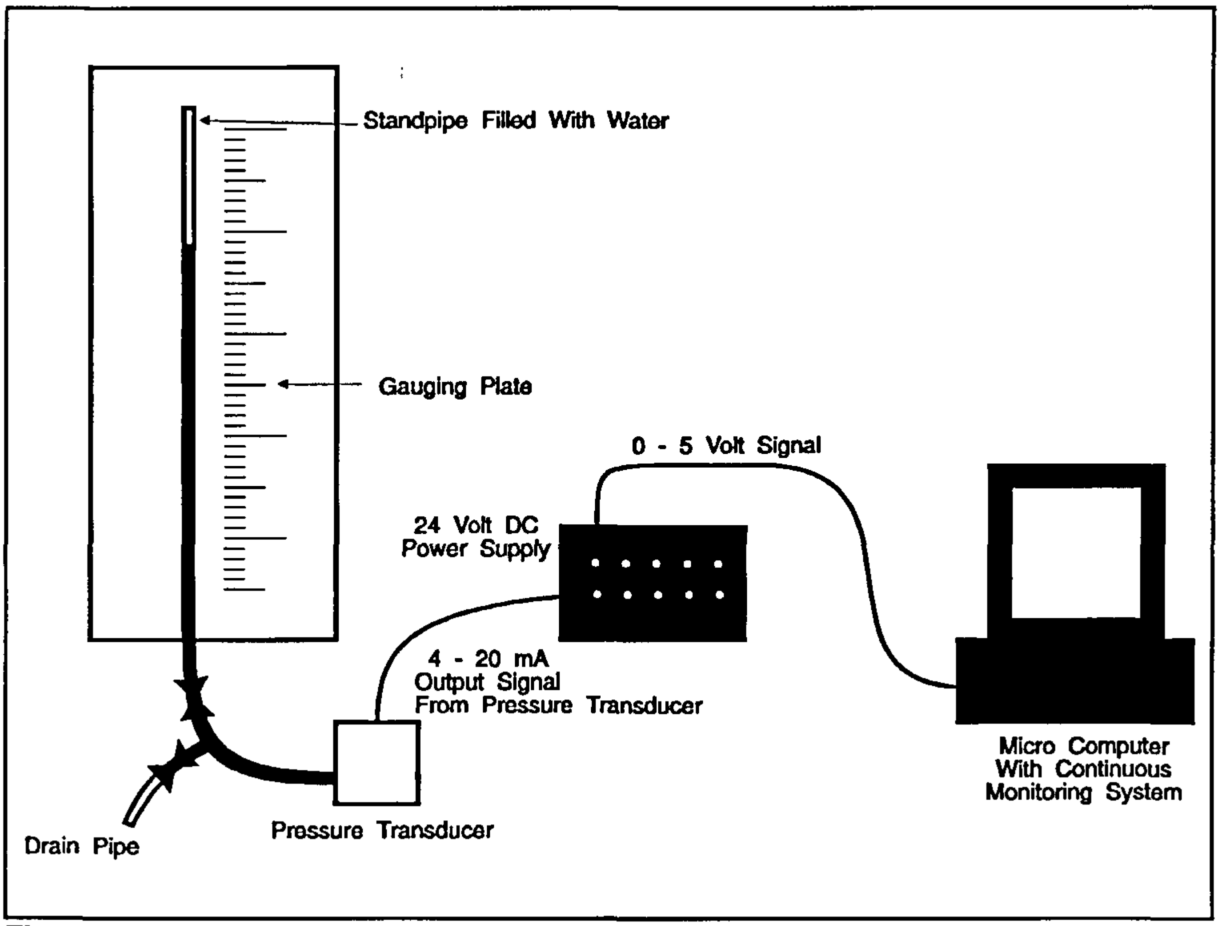

Figure 3.2 : Pressure transducer calibration set-up

Measurements were taken at 10 minute intervals and the sensitivity of the transducer set-up was ± 5 mm.

ii) Turbidity measurements

Two Hach 1720C in-line low range turbidity measuring instruments were installed (one on each side of the filter gallery). The sampling pipe of each instrument could be placed in the outlet chamber of either one of two filters (enabling final water turbidity measurements from either single or dual-media filters at any time). The instruments have a digital display as well as a 0-1 V output.

The output signal was recorded in volts with the use of a micro computer. Figure 3.4 shows a schematical lay-out of the set-up. The sensitivity of this set-up was better than ± 0,01 NTU. Measurements were recorded at an interval of 10 minutes.

iii) Sampling for Fe and organic matter

Although turbidity is an accepted measure for indicating the amount of insoluble particles in water it does not account for soluble matter. The high level of organic

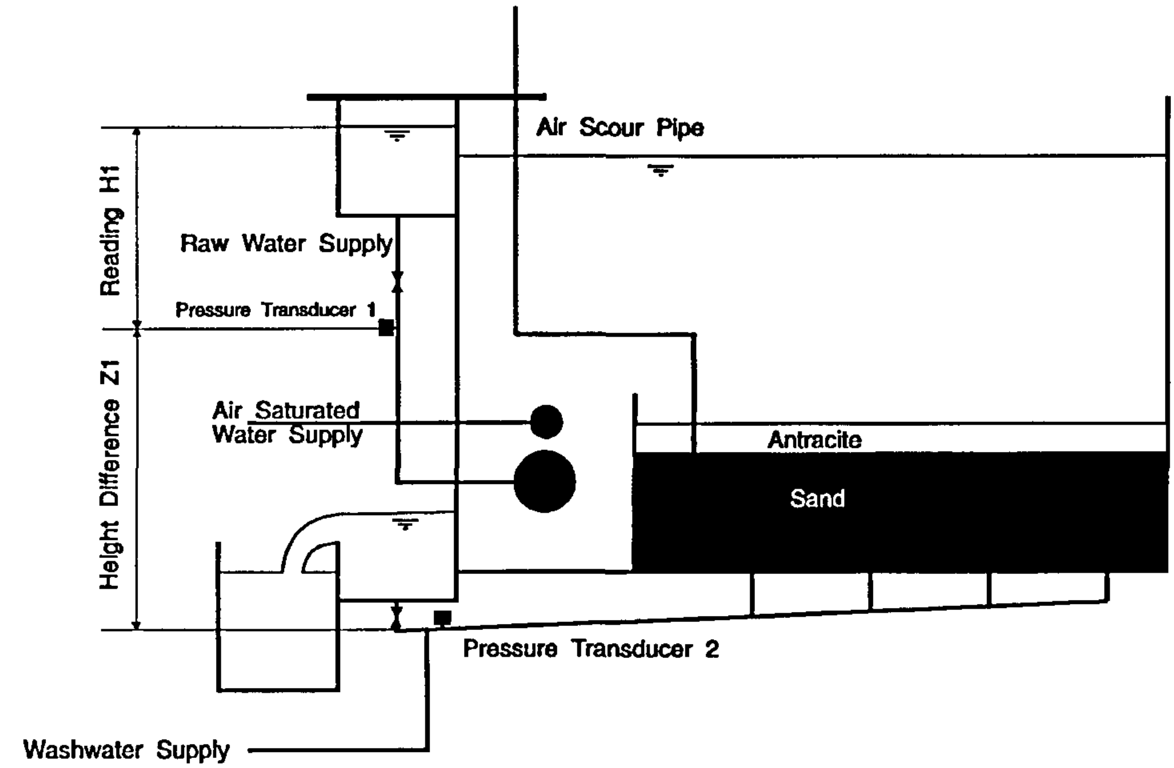

Figure 3.3 : Positions of pressure transducers

matter in the raw water increases the danger of Fe inhibition (or Fe non-precipitation) which is added as primary coagulant. It was deemed important to check for the breakthrough of complexed Fe. Samples were taken with an Isco 2900 Autosampler at the outlet chambers for single and dual-media filters every 90 minutes and later every 120 minutes for the duration of turbidity and head loss measuring. The samples were sent to Waterlab Research (Pty) Ltd for determination of the total iron present.

Fractions of these samples were also analyzed in an Ultraspec II UV/visible spectrophotometer for UV adsorption at the 254 nm range.

Monitoring of all the above parameters was difficult and some data sets were obtained where one or more of the four parameters had to be omitted. Figure 3.5 shows a typical graph of all the parameters that were monitored for a filter over a timespan of 48 hours.

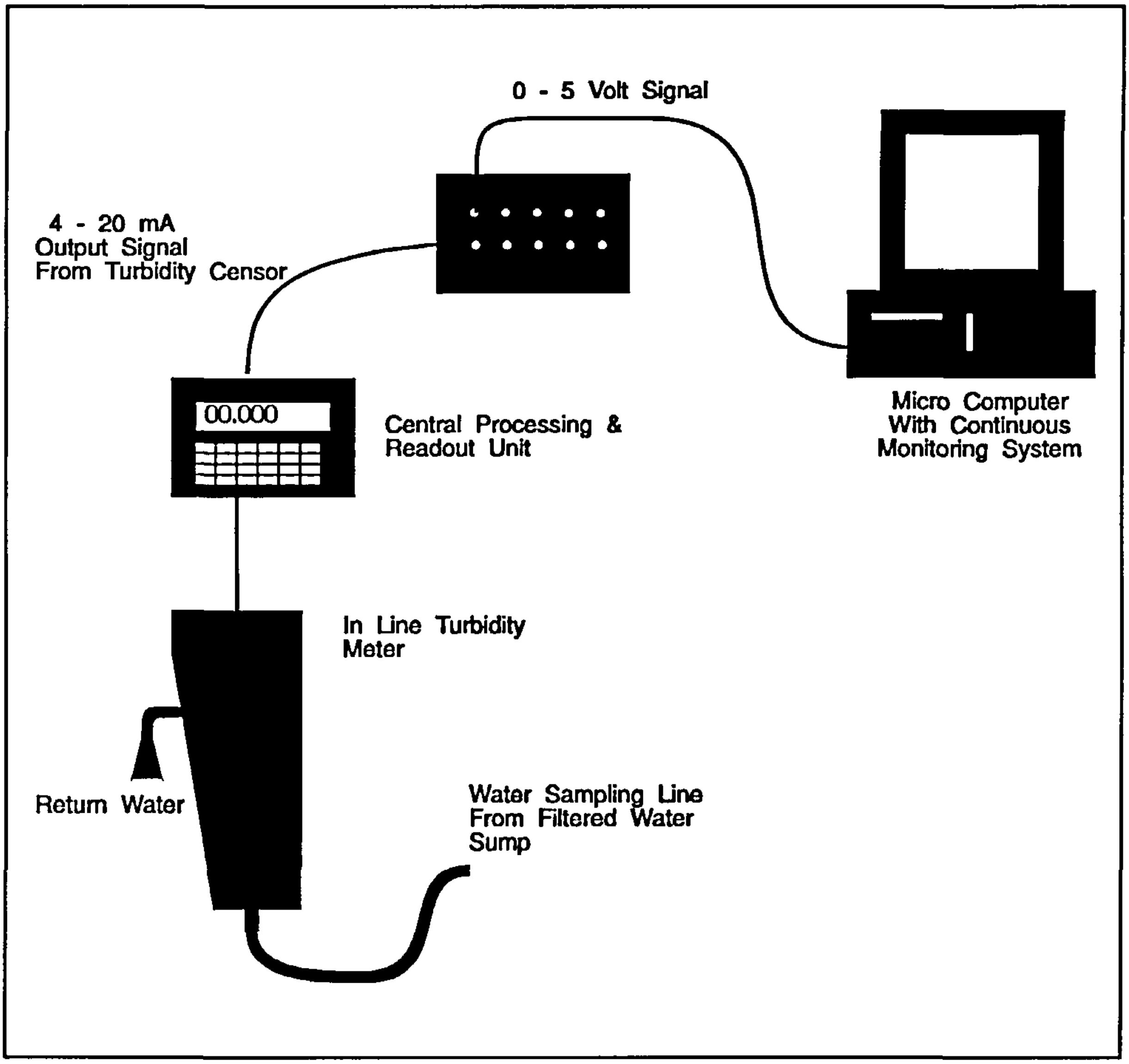

Figure 3.4 : In-line turbidity measurement set-up

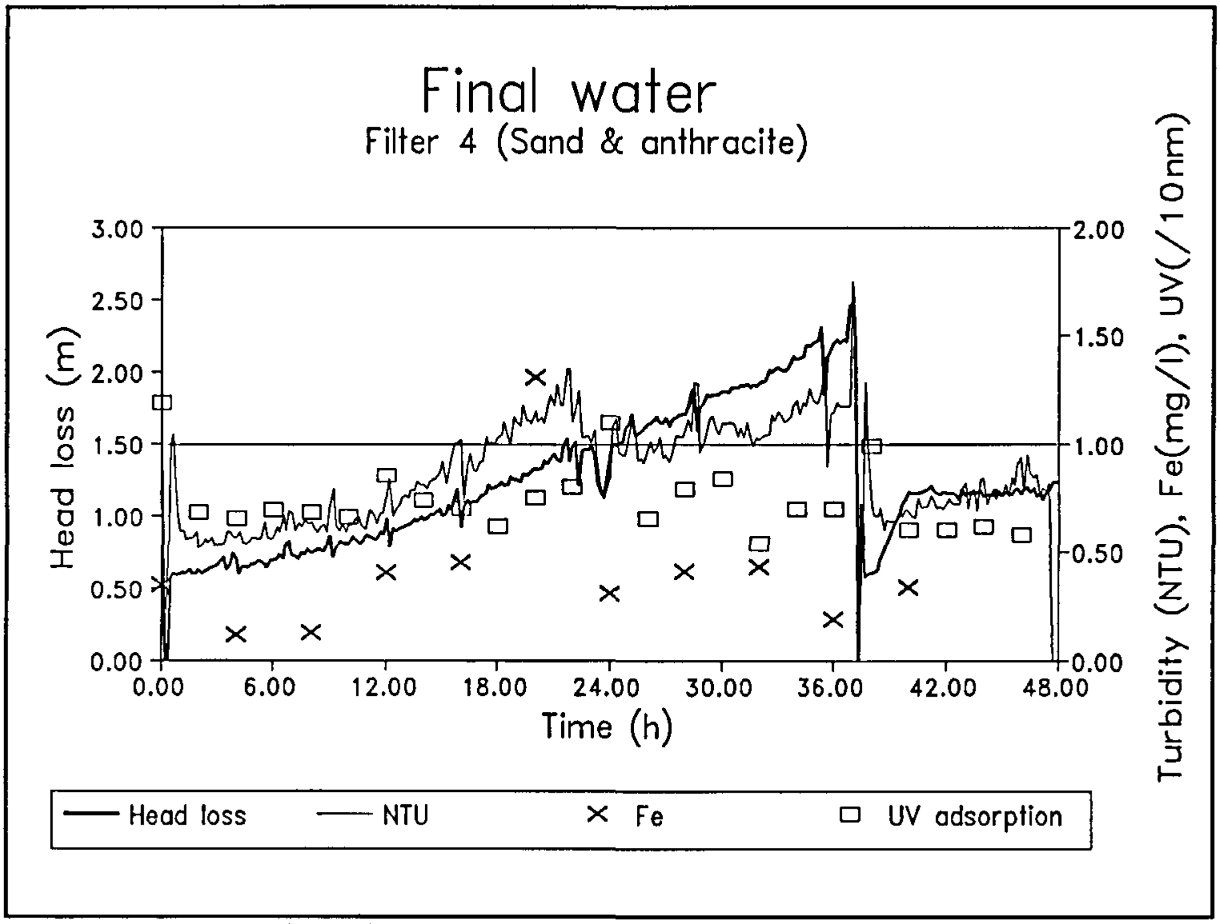

Figure 3.5 : Typical results from monitoring a filter.

3.3 Results and discussion

3.3.1 Single- vs Dual-media filter production

The results for the three single (sand) and six dual-media (anthracite and sand) filters were averaged separately on a weekly basis. These weekly averages were averaged for the whole monitoring period. The average production for the sand and anthracite filters were 121 m/run against 87 m/run for the sand filters. An improvement of 39 % ! Baylis *et al* (1971) reported an improvement of up to 40 % for single filters with coarser material. The higher production achieved with dual-media filters can be attributed to better distribution of the solids load in the filter (Baumann, 1982)

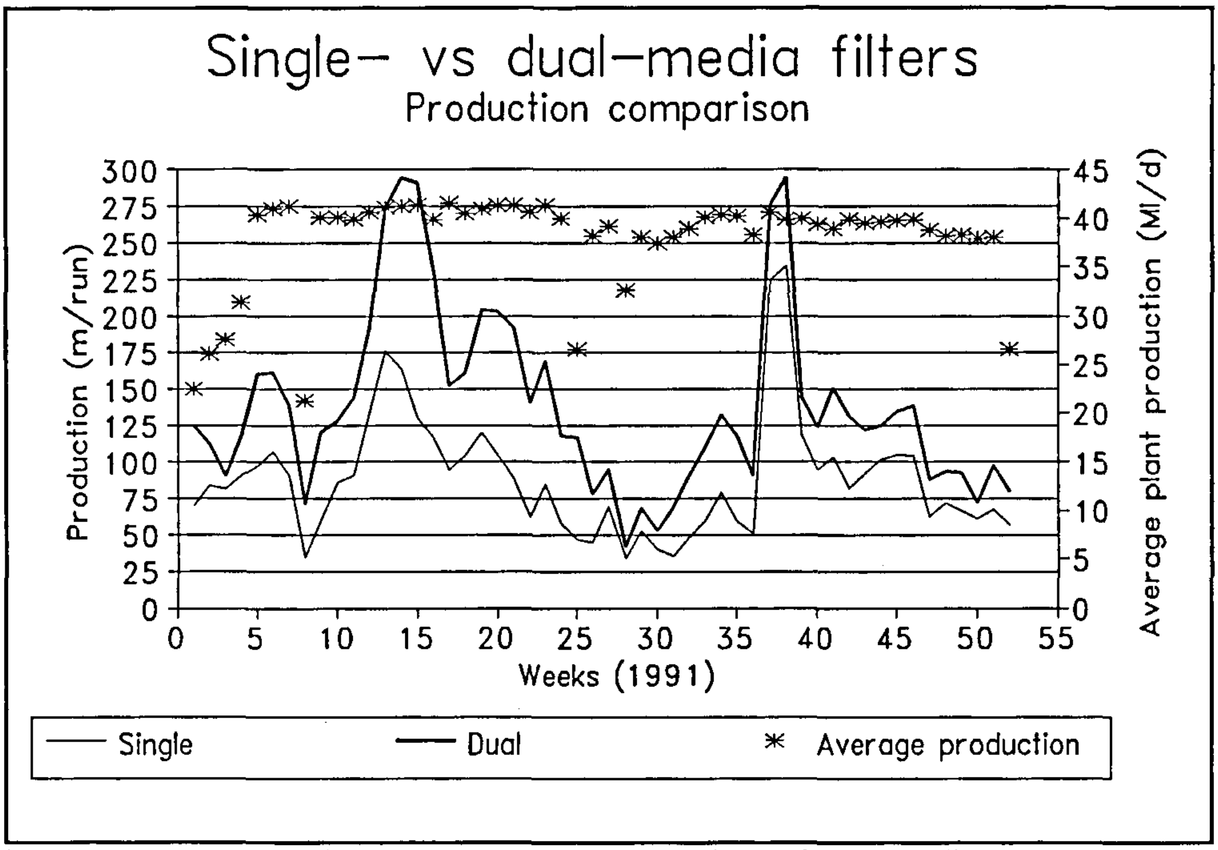

Figure 3.6 : *1991 production comparison for single- and dual-media filters at Rietvlei.*

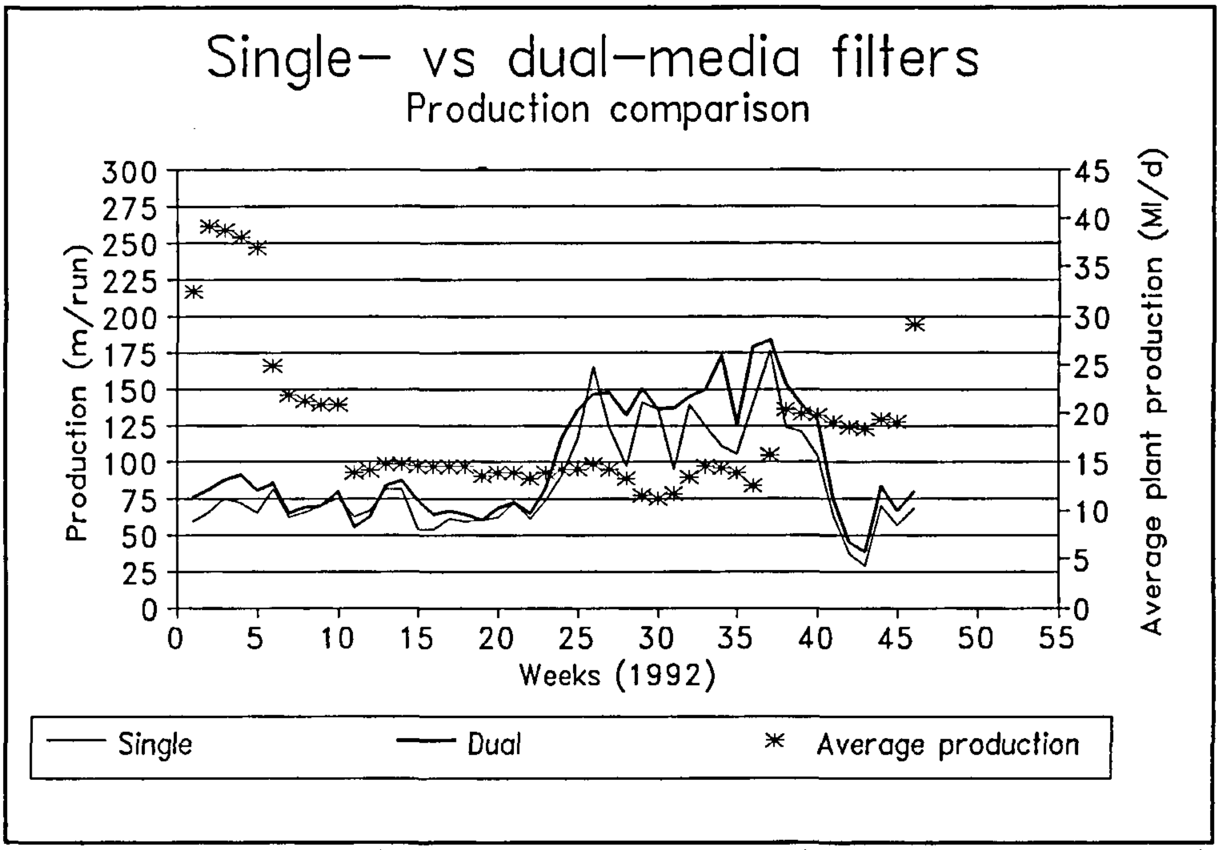

Figure 3.7 : 1992 Production comparison for single- and dual-media filters at Rietvlei.

Figure 3.6 and Figure 3.7 shows the weekly average production rates for single- and dual-media filters at Rietvlei for 1991 and 1992 respectively. The average plant production for this period was 29,4 Mℓ/d which can be ascribed to the drought period from week 11 to week 37 for 1992.

In order to investigate the effect of dual-media filtration compared to single-media filtration under differing production rates the percentage increase in performance was determined for certain bands of plant production rates. The production rates used were those of the single media filters only. The results are shown in Table 3,1 and it can be seen that the increase in performance is related to the plant performance. At high production rates (above 100 m/run) the increase in production was 53 % on weighted average and 32 % at low production rates (below 100 m/run).

Table 3.1 : The increase in performance as a result of dual-media filtration for various rates of plant production.

Plant production (Single media filters) (m/run)	Improvement (%)	Weeks (no)
< 40	22	5
40 - 59	38	17
60 - 79	23	31
80 - 99	45	23
100 - 119	57	15
120 - 139	46	8
140 - 159	27	2
> 160	63	7

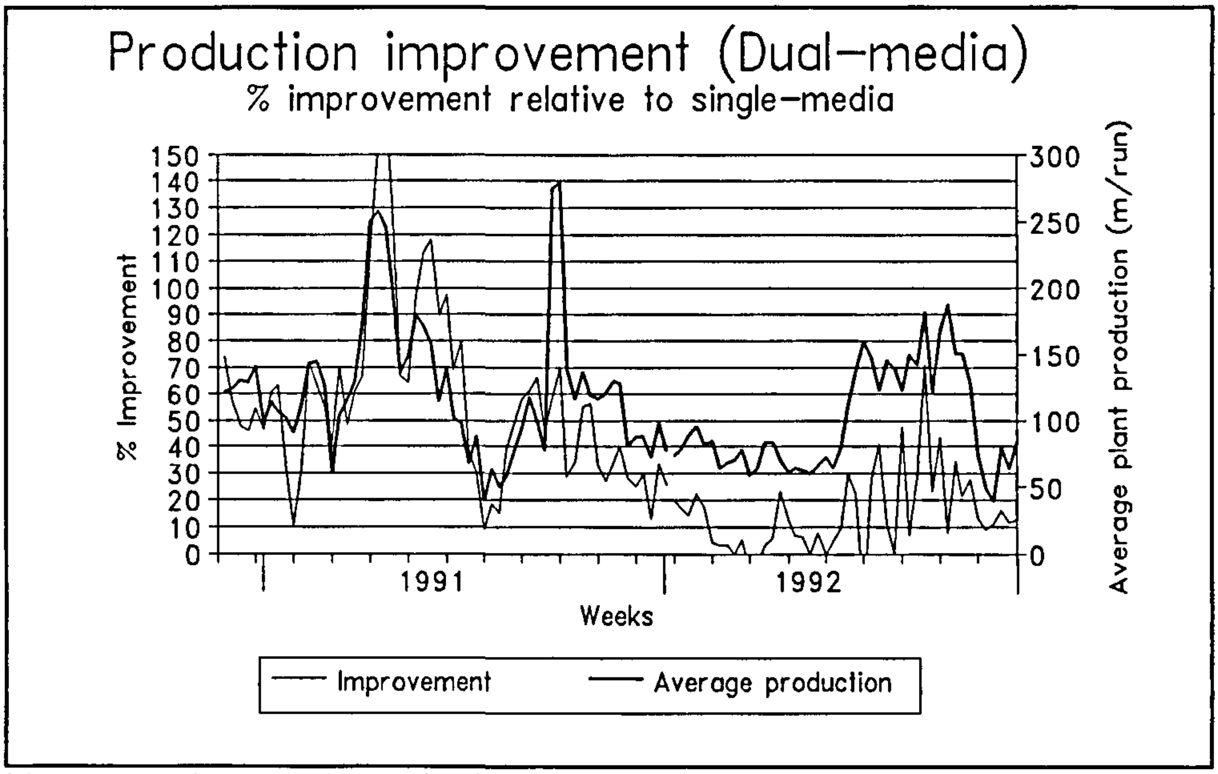

Figure 3.8 : Increase in production due to dual-media filters.

This trend can be verified from Figure 3.8. It can be seen that the increase in production of dual-media filters relative to the production of single-media filters tend to increase with an increase in average plant production and decrease with a decrease in average plant production and that no specific seasonal fluctuations exist.

It can further be seen that with the exception of weeks 11,12,26 and 27 in 1992, dual-media filters constantly outperformed single-media filters.

3.3.2 Final effluent quality

Dual-media filters performed better than single-media filters in terms of production. The question should be asked whether they are as effective in terms of quality of the final water.

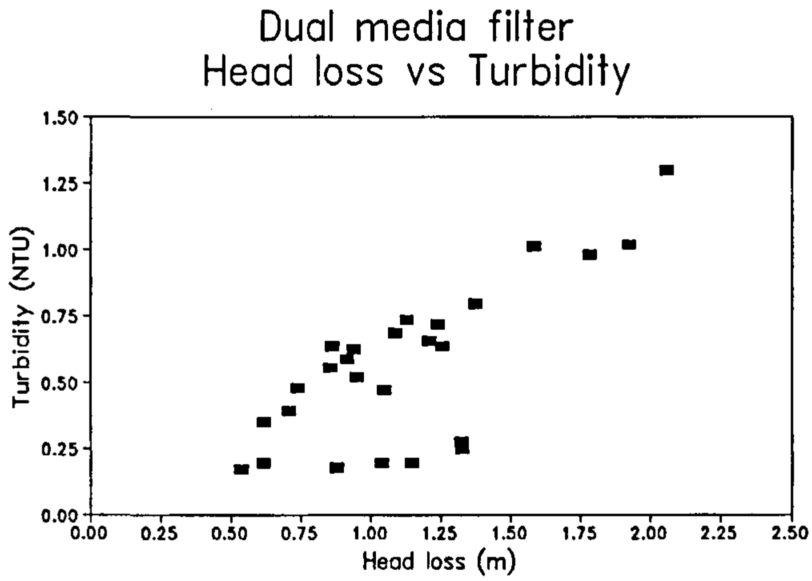

Figure 3.9 : Turbidity analyses for dual-media filters

The turbidity, Fe and UV adsorption for all the filter runs were analyzed and plotted separately against the head loss for single- and dual-media filters. The data values were very scattered as can be seen from Figure 3.9 and Figure 3.10. The same data were used to plot frequency analyses for turbidity and Fe with the values for single- and dual-media filters plotted on the same graph. These frequency distributions (Figure 3.11 and Figure 3.12) show that dual-media filters did better than 1 NTU 88 % of the time, while single-media filters did better than 1 NTU 82 % of the time. Dual-media filters performed 20 % better than single-media filters in terms of the recommended Fe value of 0,1 mg/ℓ. The two types of filters performed equally well in terms of the tolerable value of 0,3 mg/ℓ Fe.

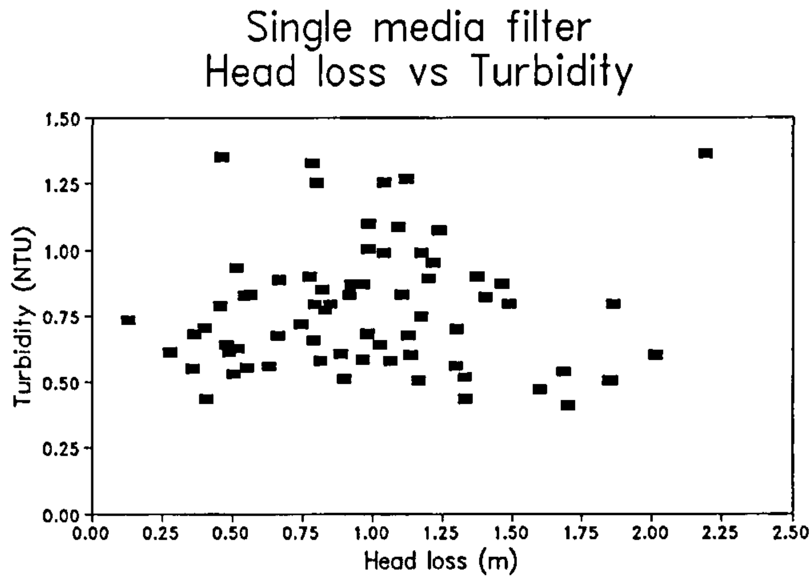

Figure 3.10 : Turbidity analyses for single-media filters

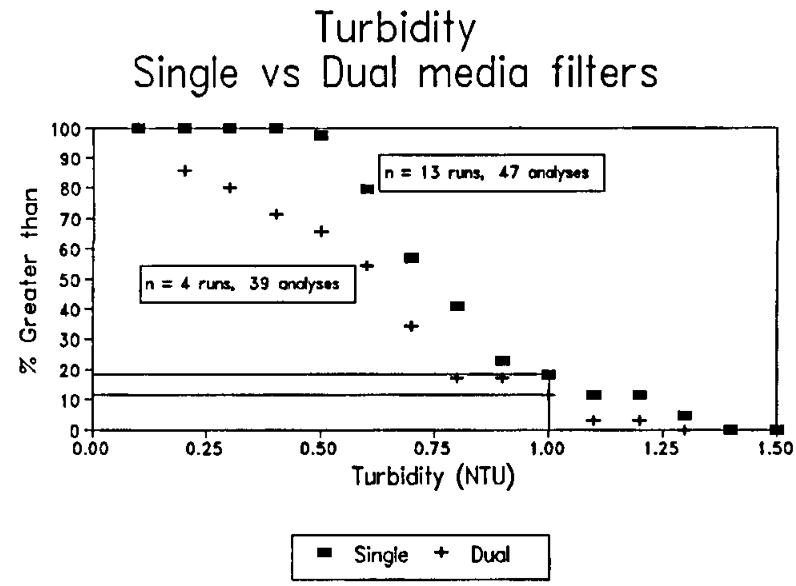

Figure 3.11 : Turbidity frequency analyses

Analyses of four of the monitored dual media runs indicate that the head loss development rate through the filter is linear. The turbidity development rate plot also proved to be linear to some extent. The slope of the head loss development rate plot is steeper than that for turbidity. Extrapolation of the data plot in Figure 3.13 indicates that the maximum head loss of 1,90 m will be reached after approximately 40 hours, whereas the turbidity will only break through the value of 1 NTU after 60 hours.

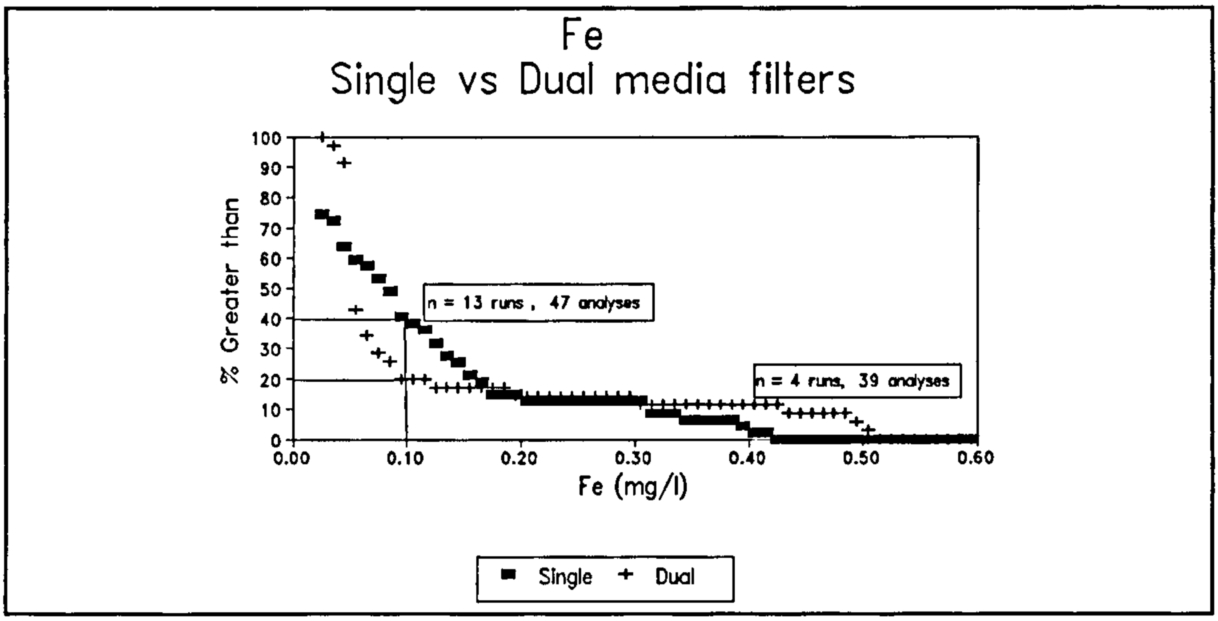

Figure 3.12 : Fe frequency analyses

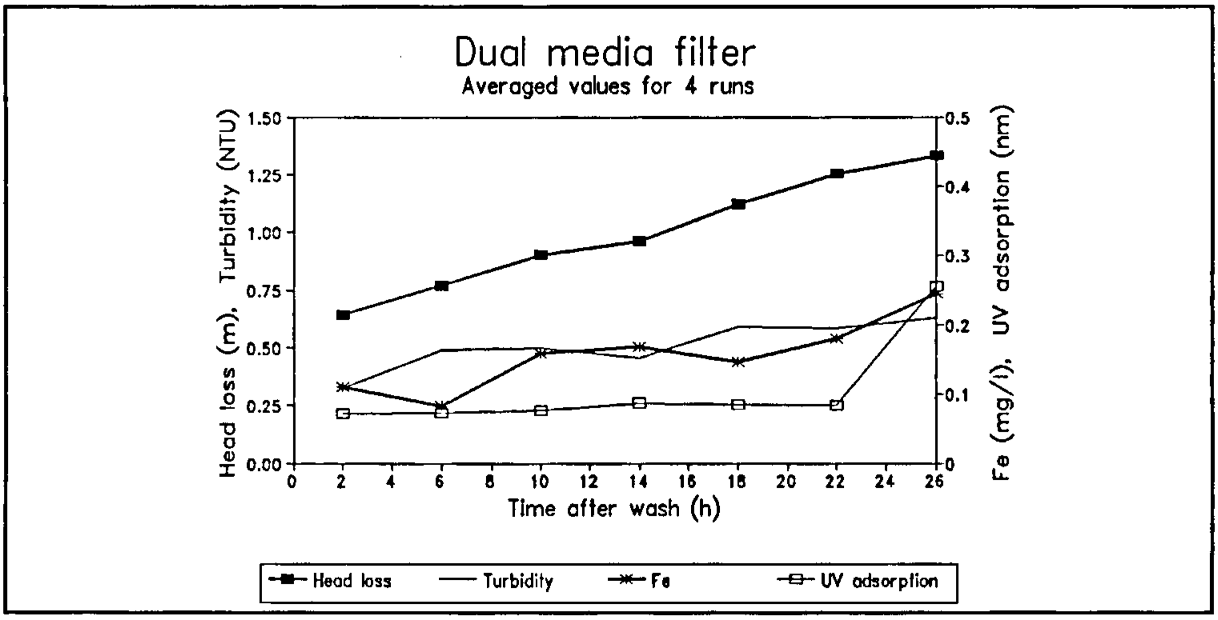

Figure 3.13 : Head loss and turbidity in dual-media filters

The values for Fe and UV adsorption are also shown in Figure 3.13. The values for Fe are rather high considering the fact that 0,1 mg/ℓ is the recommended maximum value for domestic use although values up to 0,3 mg/ℓ are well tolerated. If a value of 0,3 mg/ℓ is accepted as the upper value,extrapolation of Figure 3.13 indicates this maximum will be reached after 37 hours. The values for UV adsorption has no upper limit and serve only as an indication of an increase in organic material. The slope of these values rises sharply after 22 hours which indicates that dissolved organic matter are breaking through.

From the above it is clear that the dual media filters can safely be operated with head loss being the indicator for backwashing of a filter but that filter run lengths should not exceed about 22 hours.

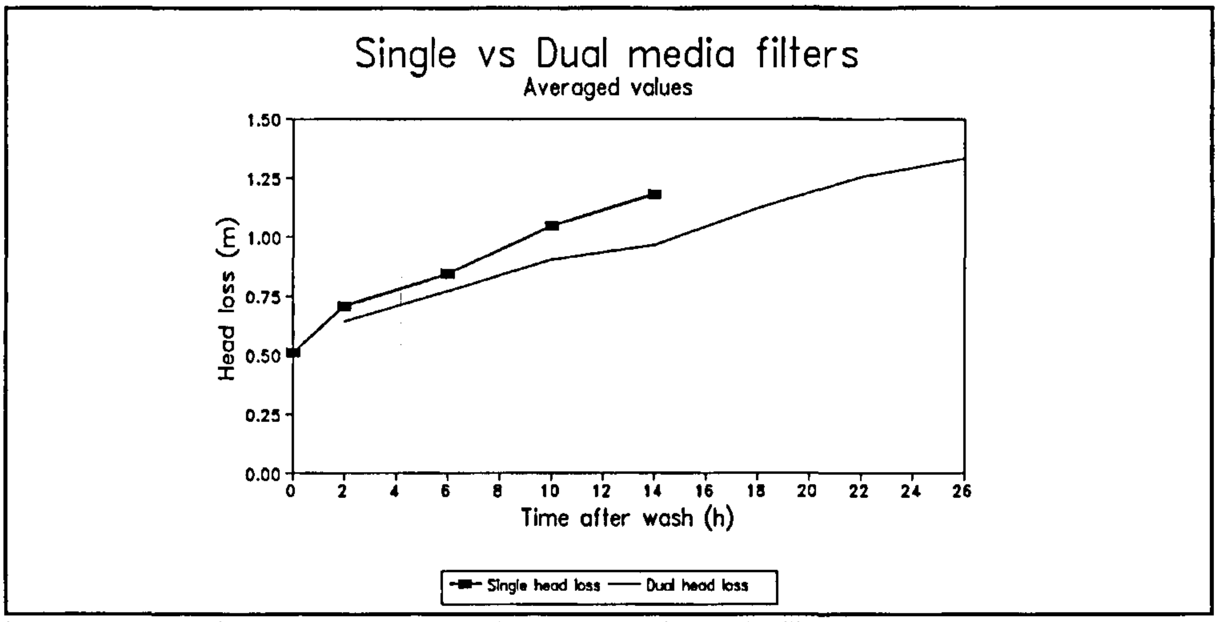

Figure 3.14 : Comparison of single- and dual-media filters in terms of head loss.

3.4 Summary

- Actual performance data has proved that production increases of 39 % average can be achieved at Rietvlei water treatment plant with the use of dual-media filtration. The percentage increase is further related to the production in the sense that high production rates (m/run) results in a higher increase and vice versa.
- The use of dual-media filtration at Rietvlei water treatment plant has no adverse affect on the quality of the final effluent.
- Dual-media filters in use at Rietvlei water treatment plant can safely be operated with head loss being the indicator for backwashing.
- It is recommended that filter run lengths should not exceed 22 hours.

Chapter 4 : Recovery of filter backwash water

4.1 Problem statement

Every water treatment plant should be operated in such a way that water losses due to filter backwashing are minimised. This minimisation of losses can normally be achieved with a combination of three methods namely :

- using less water for every wash,
- lengthening the filter runs, thus reducing the number of washes per filter
- and recirculation of the filter backwash water.

The recirculation of filter backwash water does not only save water but also reduces the waste stream from the treatment plant.

At Rietvlei, the option to recirculate the filter backwash water was made available during the design and construction of the upgrading of the plant. Two settling tanks with a sludge retention tank and recirculation pumps were installed for the purpose of recirculation of filter backwash water. The recirculation option can still be bypassed and the filter backwash water is then released into the Hennops River which runs into Verwoerdburg Lake and eventually into Hartbeespoort Dam.

The problem with the recovery of filter backwash water on a DAFF plant with highly eutrophic raw water is twofold :

- filter backwash water comprises of both suspended material from the float layer on the surface of the filters (due to dissolved air flotation) and the material trapped in the filter bed (due to rapid filtration). Due to the difference in density between suspended material (from the float layer) and suspended matter (from filter), the filter backwash water settles very slowly,

- some of the air bubbles from the flotation process is trapped in the float layer which, if not completely removed, further impedes settling of the backwash water.

The purpose of this study was :

- to increase the recirculation capacity of the backwash recovery system through chemical dosing of the filter backwash water,

- to select a coagulant for this purpose,

- to determine the optimum coagulant dosage and

- to verify the improvement in settling characteristics caused by the coagulant addition.

4.2 Literature review

4.2.1 Types of settling

Separation of suspended material under gravity from aqueous solution is one of the earliest and most widely used processes in water and wastewater treatment. Numerous researchers have contributed to the classification of the different settling characteristics of aqueous suspensions. Coe and Clavenger (1916) were the pioneers and their classification of the possible types of settling was refined into four general groups by Camp (1946) and Fitch (1956).

The groups were categorized based on :

- the concentration of the particles and
- the ability of the particles to interact

The theory of sedimentation often helps to understand the performance of a particular sedimentation tank for a given quality of turbid water. The four types of general groups of sedimentation are summarized below. Although four general types of sedimentation exists, only two broad type of particles exist namely discrete and flocculent.

Type I settling involves the free settling of discrete particles without flocculation. An example of this type of settling is the sedimentation of sand particles in grit chambers.

Type II settling is the settling of flocculent particles. The sedimentation of particles that have been subjected to chemical addition is a typical example.

Type III settling occurs when high concentrations of particles (flocculent or discrete) with very small interparticle distances, hinder the settling of neighbouring particle and all particle settle as a zone. Examples of this type of settling is seen during sludge thickening in the upper layer of the sludge blanket.

Type IV settling is the settling of high concentrations of particles (flocculent or discrete) that have no more inter particle distances between neighbouring particles. These particles settle only due to compression of the compaction mass. The bottom layer of a sludge thickener is a typical example.

4.2.2 Settling of flocculent particles

Flocculation during sedimentation takes place due to differences in the settling velocities of particles where faster settling particles overtake the slower particles and coalesce with them, or where a velocity gradient within the liquid causes collisions of faster moving particles with slower particles in adjacent stream paths.

The benefits of flocculation for sedimentation are :

- faster settling velocities because of increased diameter of the settling particle
- collecting of small particles (which would otherwise not have settled) by overtaking them with large flocs.

Attempts by Camp and Stein (1943) to quantify the frequency of particle collisions leading to flocculation, have not proved as useful for process unit design, as the experimental results from the quiescent settling column technique.

The technique involves a settling column with diameter large enough (300 mm) to have no side-wall effects and height at least equal to the proposed sedimentation tank, with withdrawal ports at various intervals.

A suspension is poured into the top and gently stirred to obtain a uniform distribution of particles. A sample is withdrawn immediately and thereafter from all the ports at different time intervals. The suspended solids concentration is determined and the percent removal calculated for every sample.

The percent removal is plotted on a graph as number versus time and depth of collection for each sample. Curves of equal percent removal, R_{10}, R_{20}, R_{30} etc, are then interpolated between these plotted points (Figure 4.1).

The overall percent removal, R_T, can be determined for different times. For example the overall percent removal after T minutes of settling time is :

$$R_T = R_{30} + \frac{H_{40}}{H}(R_{40}-R_{30}) + \frac{H_{50}}{H}(R_{50}-R_{40}) + \ldots \qquad (4.4)$$

where :

R_T	=	overall percent removal after time, T
R_{30}	=	substitute the numerical value of the percentage represented
H_{40}	=	height from water surface to 40 % removal line
H	=	total depth of water

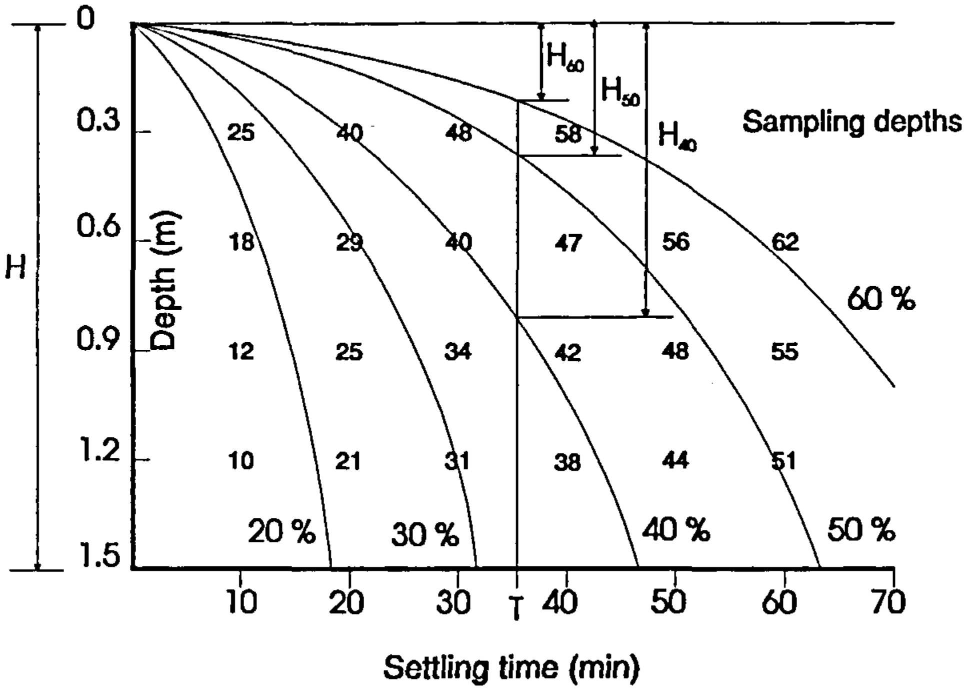

Figure 4.1 : Typical settling test results showing lines of equal percentage removal of suspended solids (after Reynolds,1982).

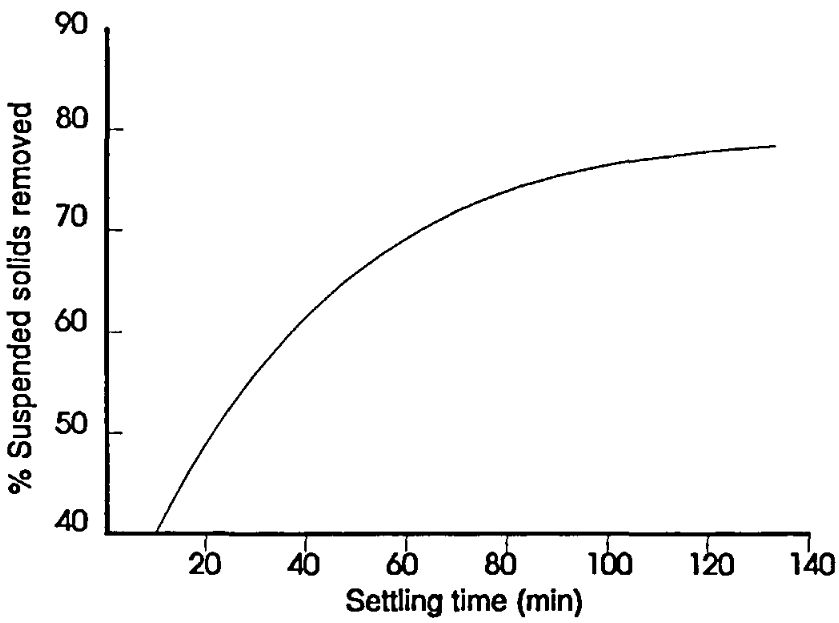

Figure 4.2 : Percent removal versus detention time, (after Reynolds, 1982).

A graph can be drawn of the percent removal versus detention time (Figure 4.2). The settling time for a required percent removal can be determined and with T known the overflow rate can be determined from :

$$v_s = \frac{H}{t_c} R_T \qquad (4.4)$$

where :

R_T = fraction removed

t_c = settling time

4.3 Experimental objectives and methods

4.3.1 Type of settling

It was suspected that the washwater would settle according to Type II settling. A sample of the washwater was captured, mixed thoroughly and poured into a 2ℓ glass cylinder and photographed every 5 minutes. The photographs in Figure 4.3 below clearly show that the particles flocculate, increase in size and settle out forming a sludge layer at the bottom of the cylinder.

Figure 4.3 : 0, 5, 10, 15, 20, 25 and 40 minutes of settling time for a typical washwater sample without any chemical dosing.

The settling was clearly of Type II and the experimental procedures below are based on the tests required to evaluate the settling of flocculent suspensions.

4.3.2 Chemicals

As discussed earlier, the problem with the settling of the filter washwater is mainly the time available for settling. It was suspected that chemical dosing might help in reducing the settling time and a visual test was made to test this suspicion.

A filter washwater sample was poured into four 5ℓ glass beakers. The water in each beaker was stirred to obtain a homogeneous distribution of suspended solids and 0,5 mg/ℓ Profloc Z11, 50 mg/ℓ Ferrifloc 1820 and 35 mg/ℓ $FeCl_3$ was added to each of three beakers respectively. Photographs were taken at time T = 0,1,2,3,4,5,6,7,8,9,10,20,30,40,50 and 120 minutes. The photographs for times T = 0,5,10 and 120 minutes are shown in Figure 4.4 and it can be seen that settling can be accelerated with the addition of a chemical flocculant.

It will be appreciated that a multitude of coagulants exist on the market. It was therefore decided to investigate the effects on settling time of the filter washwater by dosage of one inorganic coagulant, one organic coagulant and a combination of organic and inorganic coagulants.

Inorganic coagulant

$FeCl_3$ is used as primary coagulant in the treatment of raw water at Rietvlei Water Treatment plant and was therefore the obvious choice as inorganic coagulant.

Organic coagulant

The polymer tested was Profloc Z-11 and is a cationic polymer available in powder form.

Combination of inorganic and organic coagulants

In order to eliminate the double dosing requirements and avoid make-up difficulties a chemical which is a combination of an iron salt and a synthetic polymer was used. This product was patented under the Ferrifloc range of products and the specific chemical used was Ferrifloc 1820 (FF1820). FF 1820 comprises of 90 % $FeCl_3$ and 10 % polyamine blend.

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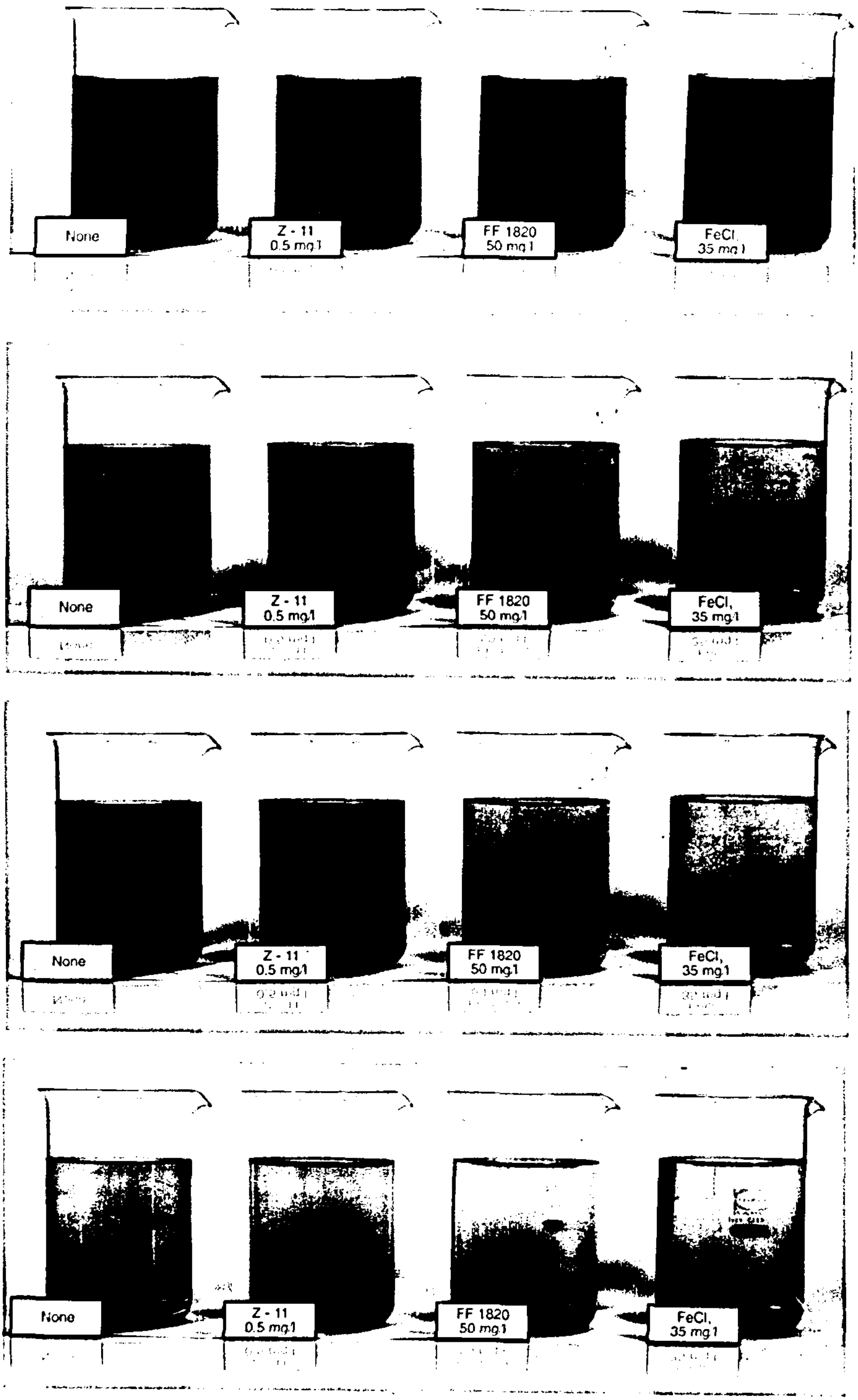
None
Z - 11
0.5 mg/1
FF 1820
50 mg/1
FeCl,
35 mg/1
None
Z - 11
0.5 mg/1
FF 1820
50 mg/1
FeCl,
35 mg/1
None
Z - 11
0.5 mg/1
FF 1820
50 mg/1
FeCl,
35 mg/1
None
Z - 11
0.5 mg/1
FF 1820
50 mg/1
FeCl,
35 mg/1

Figure 4.4 : Settling of filter washwater time T = 0,5,10 and 50 minutes from top to bottom.

4.3.3 Dosage unit of filter backwash water

A chemical dosing tank with five small valves as dosing volume control was installed in the filter gallery. This tank was fitted with a pneumatic actuated valve (upstream of the dosage control valves) which was connected to the PLC to allow automatic dosing during the drain stages within the wash cycle.

The PLC could be programmed to open and close this pneumatic valve at certain stages after the filter drain valve was opened.

The dosing tank was installed on the upstream side of the drain pipe leading to the filter washwater tanks and on the downstream side of the drainage collection channel inside the filter gallery.

Figure 4.5 below shows a schematical layout of filter washwater recovery system and the position of the dosing tank.

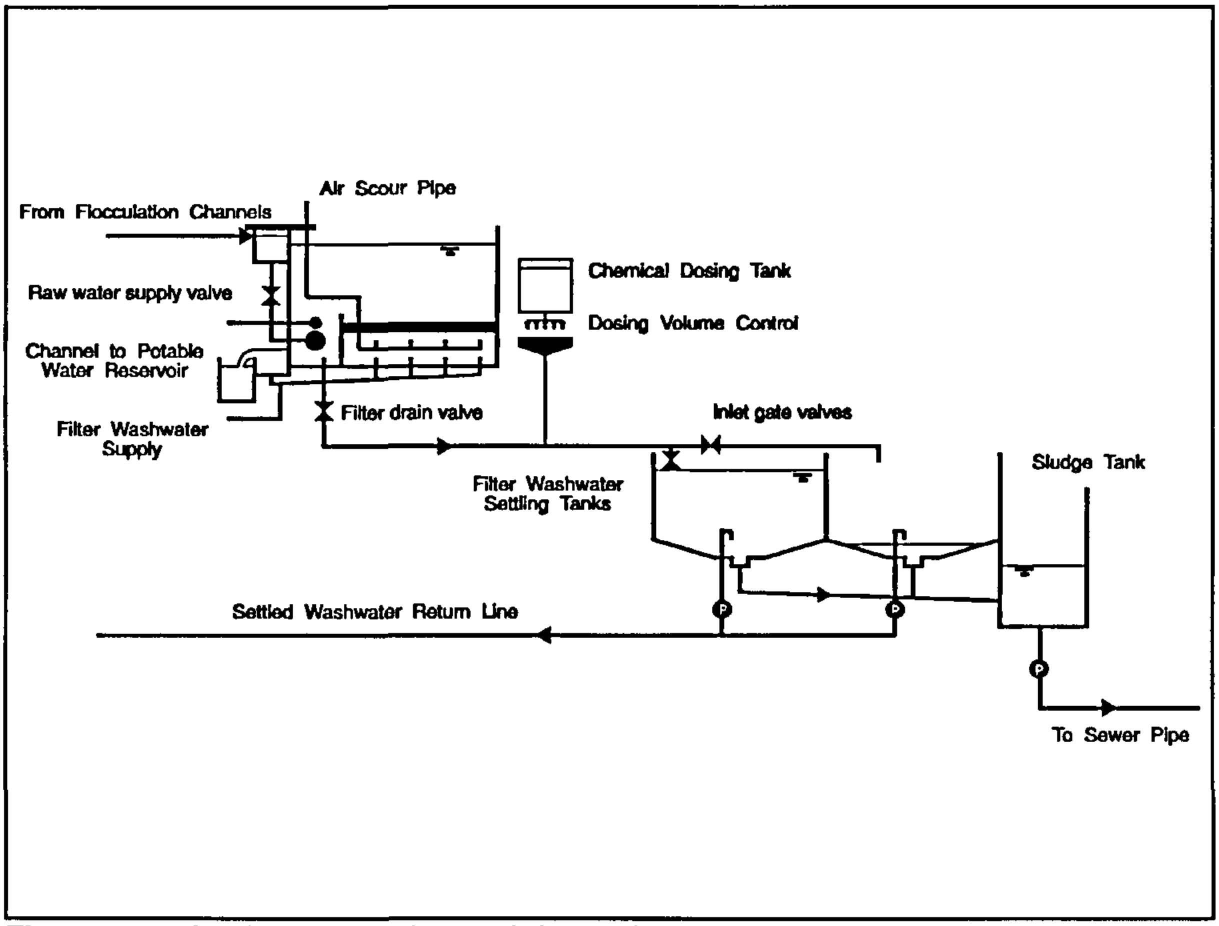

Figure 4.5 : Position of dosing tank in washwater recovery system.

4.3.4 Chemical dosing

The physical dosing apparatus was described in 4.3.3 above and made provision for a main valve with five smaller valves to enable the operator to dose volumes of 20%, 40%, 60%, 80% and 100% of full flow through the main valve. The main valve was connected to the PLC in order to dose at various times after each opening of a filter's drain valve. The idea was to dose only when water with high concentrations of suspended material passed the dosing point. These time and duration had to be determined and it was seen by visual inspection that three distinct plugs of dirty water passed the dosing point during each wash. On each occasion directly after the draining of water from the filter during the wash cycle.

The times and durations were eventually determined by studying the flow hydrographs for a number of filters at the dosing point. The maximum combined value of hydrographs of all the separate hydrographs are shown in Figure 4.6 below.

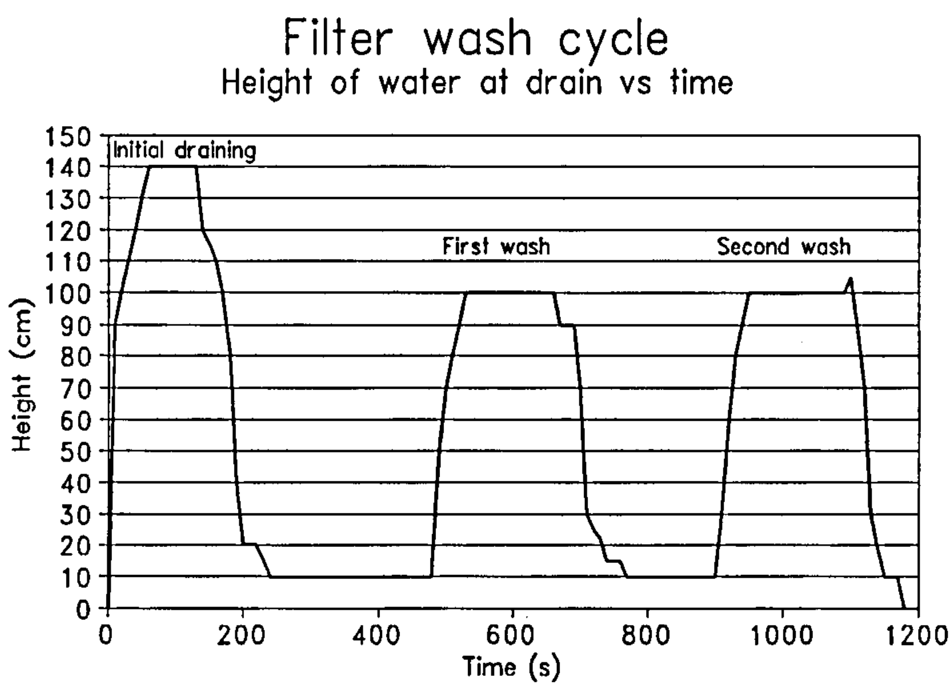

Figure 4.6 : Filter drain hydrographs at the dosing point

To determine at what time and at what dose, after the initial drain of the filter the dosing should commence, the suspended solids was determined for a set of samples taken 10 s apart during each drain within the wash cycle. Figure 4.7 below shows the results graphically. It can be seen that the maximum suspended solids load occurs during the drain after the first wash. This was visually verified, the water discharged after the first wash was markedly dirtier than the other two drains.
The pneumatic valve was set to open and close at certain times after the filter drain

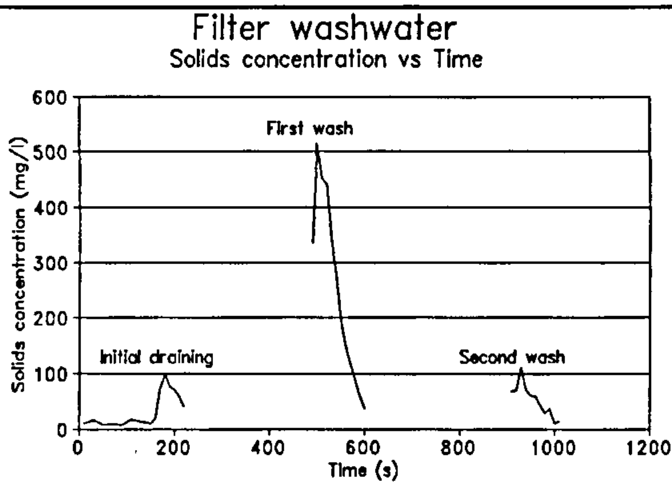

Figure 4.7 : Suspended solids distribution during wash cycle.

valve signal was received by the PLC. These times are shown in Table 4.1 below. These dosing durations resulted in approximately equal volumetric dosages for filters with different distances from the dosing point.

The dosing volume was determined by measuring the volume of the chemical dosing tank before and after each wash and measuring the height in the appropriate settling tank after each wash. The dosage concentration was determined by calculating the concentration of the make-up volume in the dosing tank multiplied by the dosage volume and divided by the volume of water treated.

Table 4.1 : Valve settings for pneumatic valve at chemical dosing tank

Pneumatic valve settings in the PLC	Status
20 s after initial draining	Open
200 s after initial draining	Close
20 s after first wash	Open
200 s after first wash	Close
20 s after second wash	Open
200 s after second wash	Close

A typical calculation is as follows :

Coagulant make-up :

15ℓ $FeCl_3$ (0.625 kg/ℓ) added to 150 ℓ water	=	0.625 *15/150 kg/ℓ
	=	62 500 mg/ℓ $FeCl_3$
Chemical dosing volume before wash	=	165 ℓ
Chemical dosing volume after wash	=	140 ℓ
Volume coagulant used	=	25 ℓ
Washwater level in settling tank	=	1,3 m
Washwater volume	=	72 684 m^3
Concentration of $FeCl_3$ per wash	=	62500 * 25 / 72 684
	=	21,5 mg/ℓ

4.3.5 Data acquisitioning procedure

Every type of chemical discussed above was extensively tested at different dosage levels and an optimum (if any) dosage was determined. These optimum dosages were determined in terms of the subnatant turbidity at different levels and times in the settling tank. The sampling and experimental procedure is explained below.

After noting the washwater dosing level the filter was drained and washed. Immediately after the final drain was completed, samples were taken at 2 minute intervals at levels 0,0 m, 0,4 m, 0,8 m and 1,2 m from the bottom of the settling tank. An autosampler was used for this purpose and Figure 4.8 below shows the sampling set-up.

These samples were analyzed for turbidity and the percentage NTU removed was calculated as a fraction of the turbidity taken of the first sample directly after the last drain at the bottom of the tank.

The percentage turbidity removed for each sampling point was plotted on a grid with the time of sampling at the horizontal axis and the height of sample on the vertical axis. This grid also showed the final water level in the settling tank as the uppermost horizontal line. Lines of equal percentage turbidity removed were then interpolated resulting in a graph as shown in Figure 4.9 below.

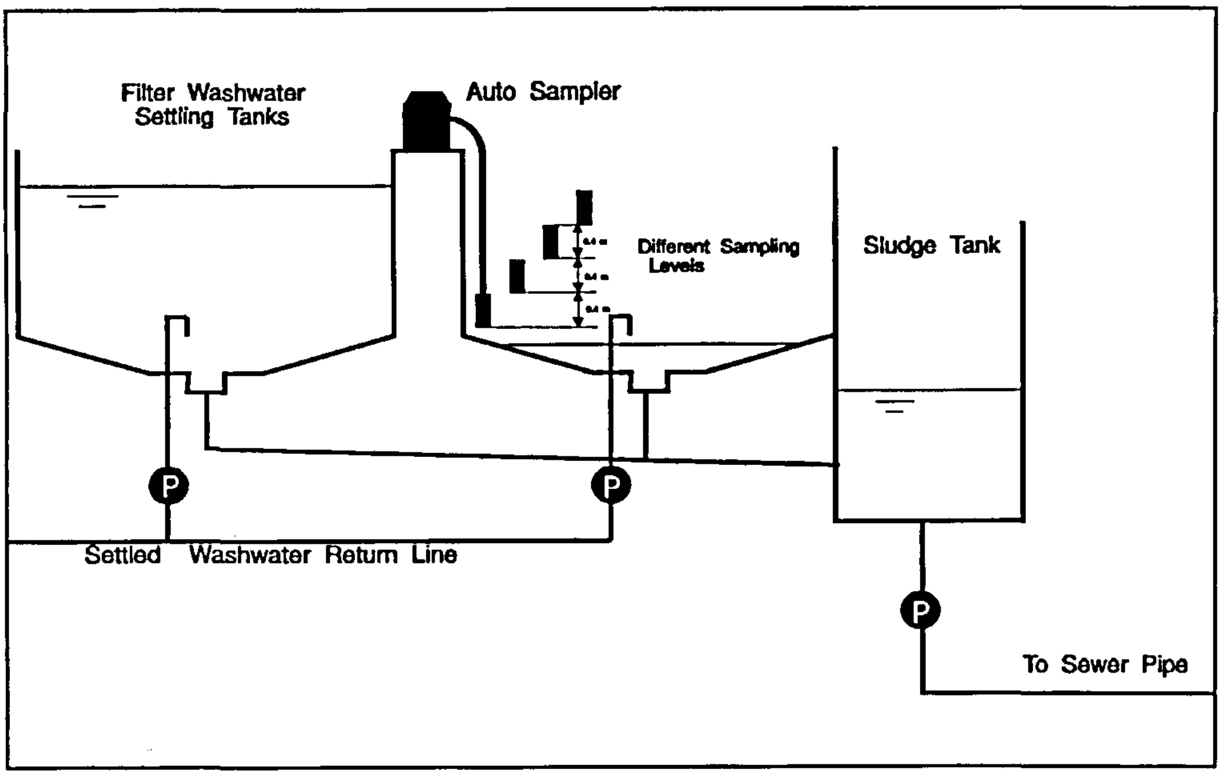

Figure 4.8 : Filter washwater sampling set-up

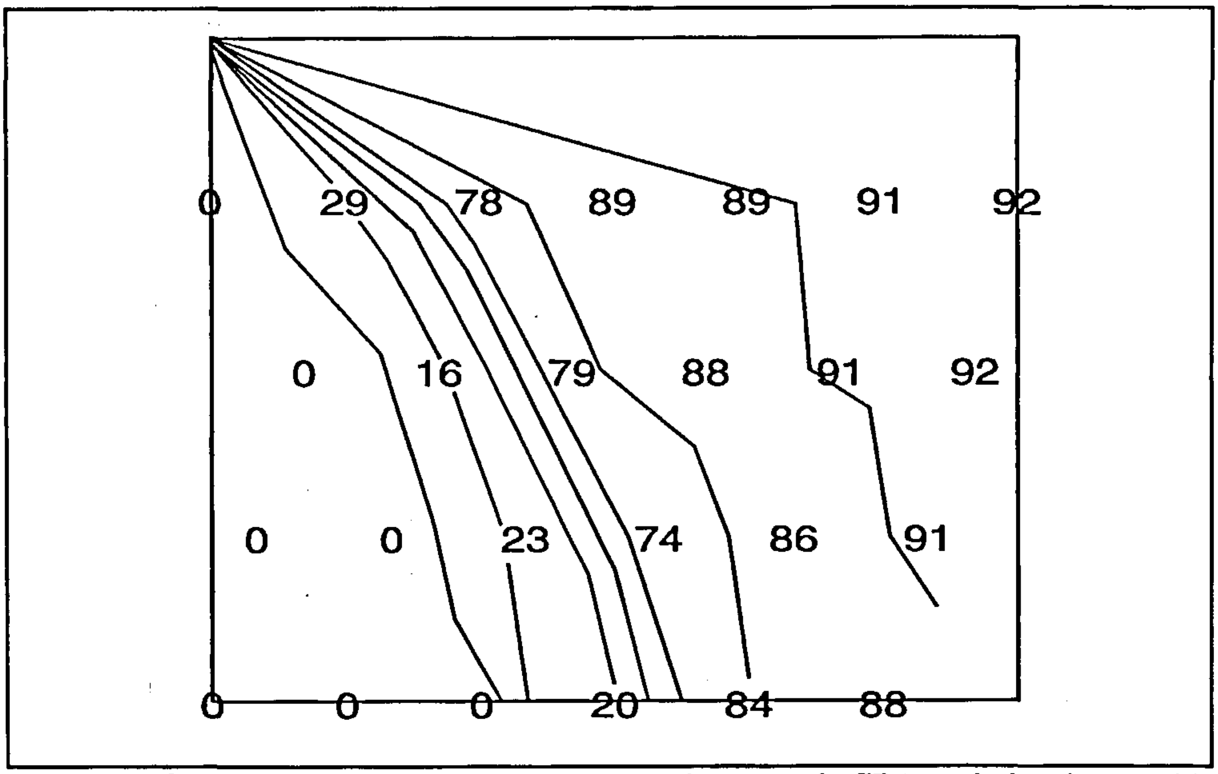

Figure 4.9 : Settling grid with equal percentage removal contours for FF 1820 dosing. (5,21 mg/ℓ)

To determine the total percentage removed R_T at times 15,20,30,40 and 50 minutes, heights from the bottom of the tank to every specific percentage removed line measured for the times mentioned. The value R_T could then be calculated according to Equation (4.4). From these values settling curves could be drawn for each different type of chemical used. Figure 4.10, Figure 4.11, and Figure 4.12 respectively show the settling curves for various $FeCl_3$, FF1820 and Z11 dosages.

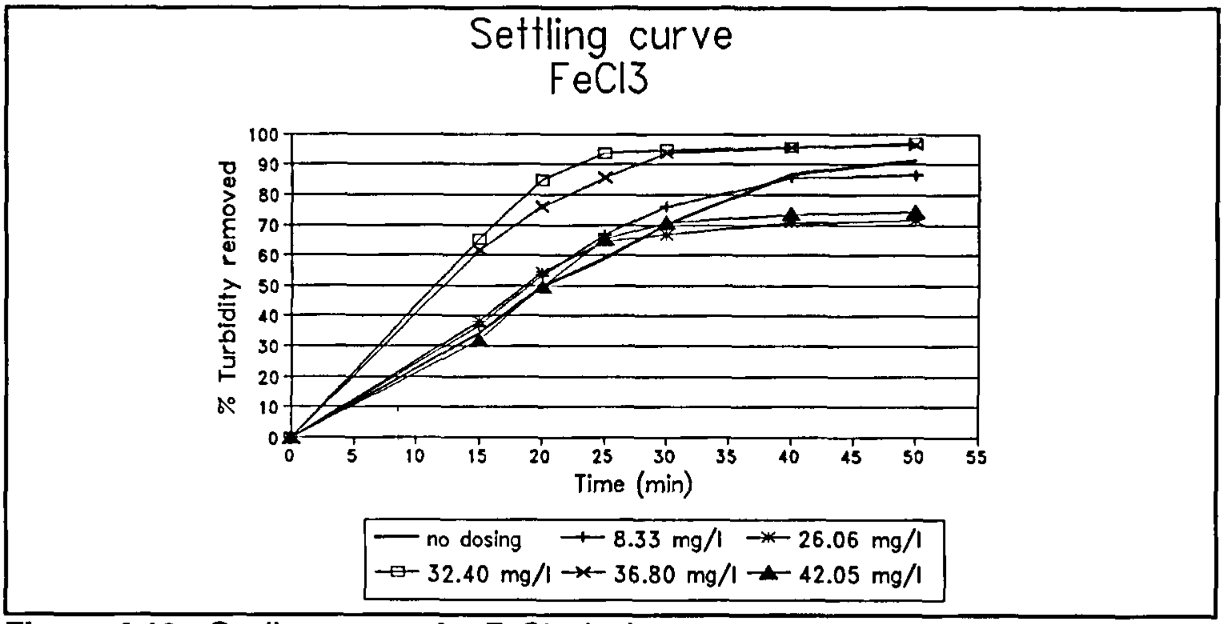

Figure 4.10 : Settling curves for $FeCl_3$ dosing

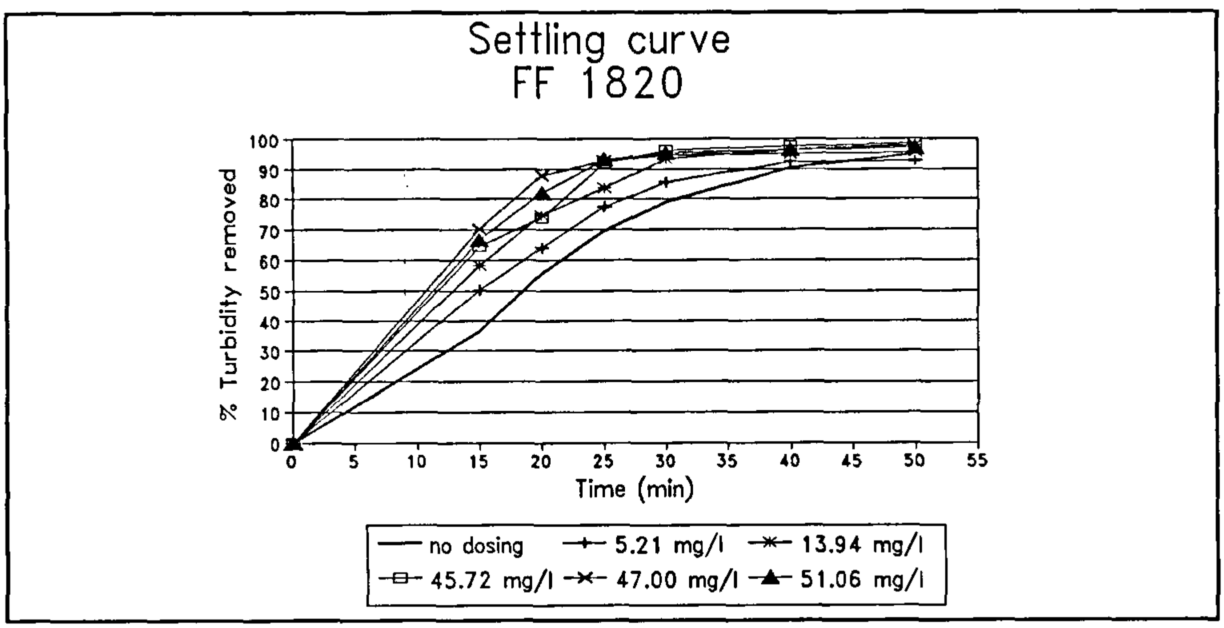

Figure 4.11 : Settling curves for FF 1820 dosing.

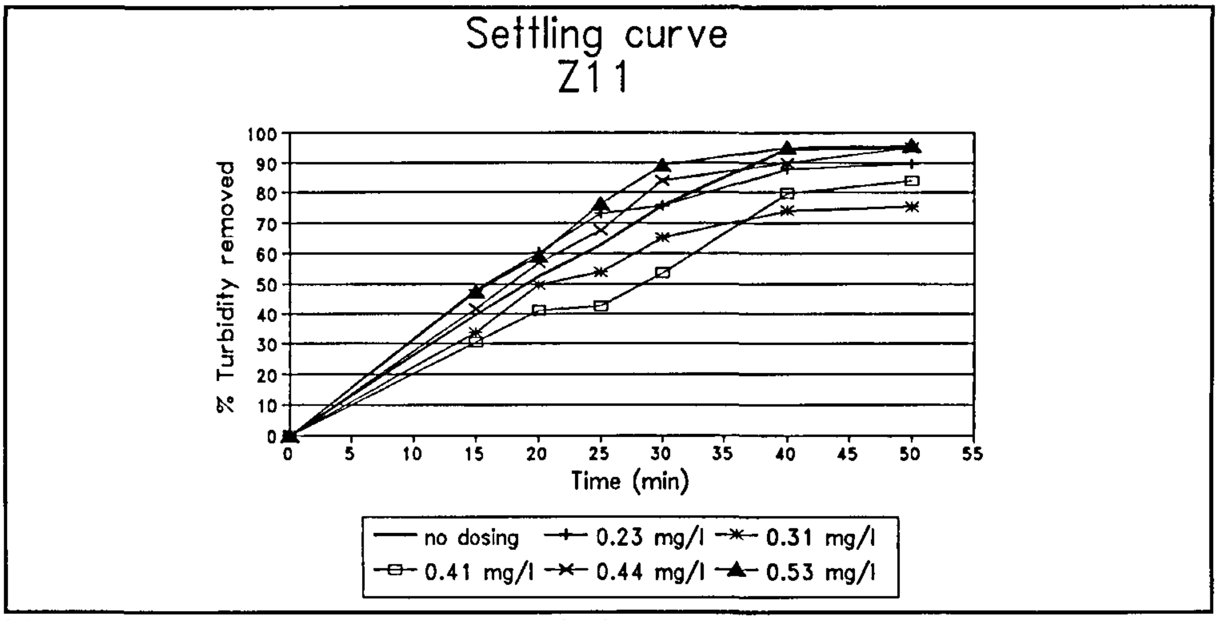

Figure 4.12 : Settling curves for Z11 dosing.

The percentage turbidity removed for every dosage of a specific chemical can be read from the settling curves. Figure 4.10, Figure 4.11, and Figure 4.12 show that the optimum dosing for a settling time of 50 minutes is 32 mg/ℓ $FeCl_3$, 50 mg/ℓ FF 1820 and 0.5 mg/ℓ Z11. From Figure 4.12 it can also be seen that the dosing with Z11 was not successful and that no dosing resulted in a higher percentage removal than any Z11 dosing. FF 1820 dosing proved to perform quite well with only the dosage of 5,21 not performing better than no dosing. To achieve a high percentage of removal at an early stage, say after 25 minutes settling, rather high dosages of up to 50 mg/ℓ would be needed. $FeCl_3$ also showed an increase in percentage removal over no dosing, but it is clear that over or under dosing can give less removal than with no dosing. The optimum dosing for $FeCl_3$ lies between 32 and 36 mg/ℓ.

4.4 Results and discussion

Comparing the line of no dosing from Figure 4.10, Figure 4.11 and Figure 4.12 with the optimum dosing lines clearly shows that the settling characteristics of the filter washwater can be improved by chemical dosing after 25 minutes of settling but, the improvement becomes smaller with the increase of settling time with virtually no improvement after 50 minutes of settling.It is important to note that the percentage removed with dosing of 32 mg/ℓ $FeCl_3$ (after 50 minutes settling) and that without dosing is 97 % and 91 % respectively. The system actually performs well without any dosing !

Raw water turbidity limit

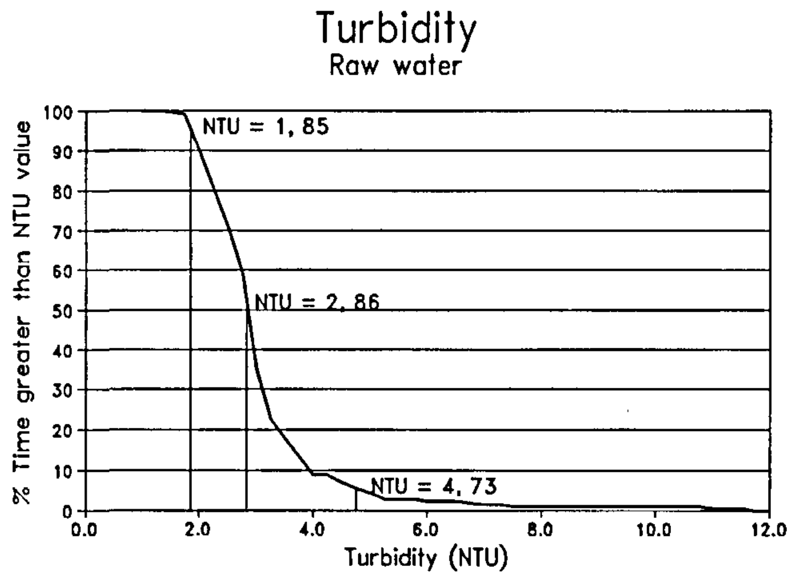

Figure 4.13 : Raw water turbidity distribution.

The question that should be asked is " What is a tolerable limit of percentage turbidity removed in the washwater recovery process ?". For the plant operating at 40 Mℓ/day the inflow at the inlet chamber is 463 ℓ/s. The average raw water turbidity in Figure 4.13 is 2,86 NTU. A raw water turbidity of up to 10 NTU can easily be handled by the DAFF process at Rietvlei water treatment plant and if this is set as an upper limit for the raw water turbidity, the turbidity of the returning settled washwater (x) can be calculated .

It takes 20 minutes to empty a volume of 80 kℓ i.e. 66,7 ℓ/s. This flow is added to the inflow at the inlet chamber for a total flow of 530 ℓ/s. The turbidity for the settled washwater, x, should then be less than :

(463 * 2,86) + (66,7 * x) = 530 * 10
x ≤ 60 NTU to avoid turbidity values exceeding the raw water limit of 10 NTU.

The measured washwater turbidity varies between 186 and 428 NTU with a mean of 308 NTU and a standard deviation of 65 NTU. For the worst case of NTU = 428, with 90 % removal without dosing after 50 minutes settling time, a turbidity of only 43 NTU remains. If settled washwater with a turbidity of 31 NTU (10 % of mean value of 308 NTU) is returned, the raw water entering the plant will have a turbidity of y, where :

(463 * 2,86) + (66,7 * 31) = 530 * y
y = 6,4 NTU

Filter run length

The average production for the period October 1988 to October 1993 was 108 m/run (Figure 4.14) with 5 % of all runs shorter than 54 m/run and 5 % of all runs longer than 195 m/run. When the production is converted to washes/day the average number of washes are 13,66 with the upper and lower 90 % limits 27,32 and 4,22 respectively. (Figure 4.15)

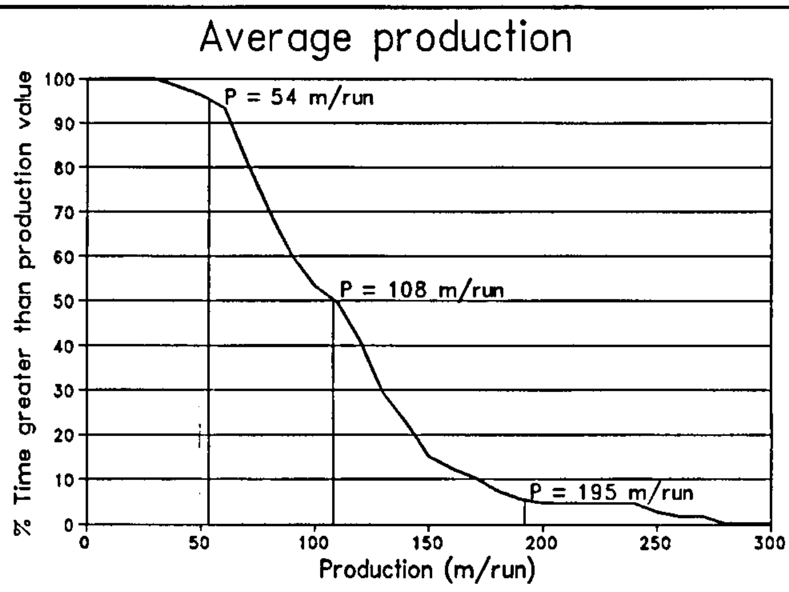

Figure 4.14 : Production distribution.

With a settling time of 49 (say 50) minutes 28,8 washes per day can be handled should flawless operating conditions prevail. This implies that the filter washwater can be returned for more than 90 % of the time. It should be kept in mind that the run

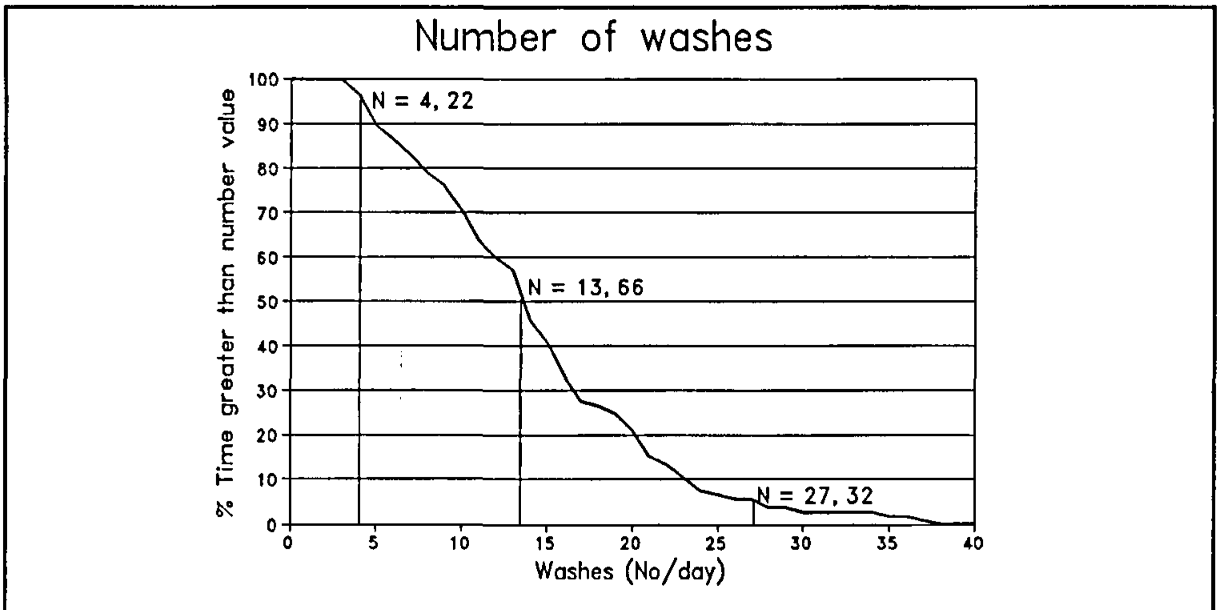

Figure 4.15 : Distribution of washes per day.

length of the filters should not exceed 22 hours and that washing should be spread evenly over the 24 hour period, to avoid demands of more than one filter wash at a time.

4.5 Summary

- Chemical dosing can reduce settling times, resulting in a reduction in minimum wash cycle lengths.
- Chemical dosing of DAFF filter backwash water have little benefit for settling times longer than 50 minutes with average settling depths of 1,35 m.
- A combination of inorganic and organic coagulants (typically Ferrifloc 1820) gives the best settling results after 25 minutes of settling.
- It is recommended that the filter backwash water at Rietvlei be chemically dosed with 34 mg/ℓ $FeCl_3$ and recycled on a permanent basis.

Chapter 5 : Conclusions and recommendations

5.1 Conclusions

- The Bratby, bench scale reactor can be used as a conservative predictor for full-scale flocculation performance with eutrophic water.

- Nonionic polymer addition as secondary coagulant, slightly reduces the agglomeration constant, K_A but greatly reduces the breakup constant K_B thus resulting in better flocculation performance.

- The kinetic parameters K_A and K_B are much lower for eutrophic water than for any other water type and can therefore be expected to perform worse during flocculation.

- Actual performance data has proved that production increases of 39 % average can be achieved at Rietvlei water treatment plant with the use of dual-media filtration. The percentage increase is further related to the production in the sense that high production rates (m/run) results in a higher increase and vice versa.

- The use of dual-media filtration at Rietvlei water treatment plant has no adverse affect on the quality of the final effluent.

- Dual-media filters in use at Rietvlei water treatment plant can safely be operated with head loss being the indicator for backwashing.

- Chemical dosing can reduce settling times, resulting in a reduction in minimum wash cycle lengths.

- Chemical dosing of DAFF filter backwash water have little benefit for settling times longer than 50 minutes with average settling depths of 1,35 m.

- A combination of inorganic and organic coagulants (typically Ferrifloc 1820) gives the best settling results after 25 minutes of settling.

5.2 Recommendations

- Short filter runs caused by low turbidity, highly eutrophic water can successfully be eliminated by the addition of dissolved air flotation, to the treatment train.

- Dual media filtration (coarse anthracite on top of conventional filter sand) can improve the filtrate production/run without compromising quality, but is less effective for short filter runs.

- Backwash water from DAFF units can be successfully de-aerated and settled in batch settling tanks.

- The use of the Bratby reactor with the Argaman-Kaufmann model for the prediction of the kinetic parameters K_A and K_B shows promise but much further research is required in order to verify and calibrate the use thereof.

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