Scenario Studies into Advanced Particle Removal in the Physical-Chemical Pre-treatment of Wastewater

Arjen Frans van Nieuwenhuijzen

Scenario Studies into Advanced Particle Removal in the Physical-Chemical Pre-treatment of Wastewater

Proefschrift

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Prof.ir. J.H.J.M. van der Graaf
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Rector Magnificus	Voorzitter
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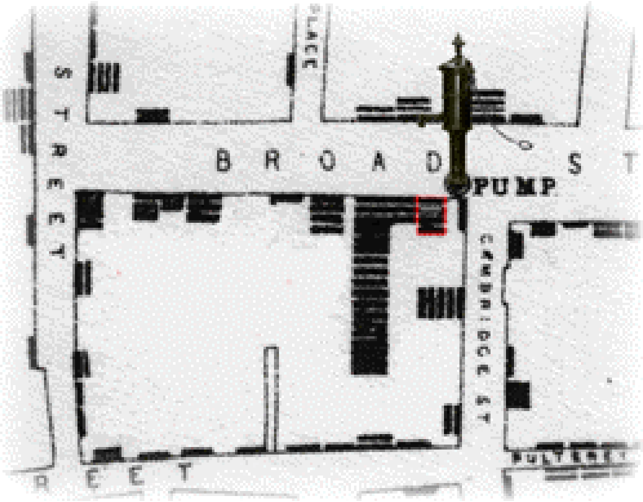

"Further inquiry, however, showed me that there was no other circumstance or agent common to the circumscribed locality in which this sudden increase of cholera occurred, and not extending beyond it, except the water of the [Broad Street] pump.

….

I had an interview with the Board of Guardians of St. James's parish, on the evening of Thursday, 7th September [1854], and represented the above circumstances to them. In consequence of what I said, the handle of the pump was removed on the following day."

John Snow M.D., On the Mode of Communication of Cholera,
second (extended) edition
London: John Churchill, New Burlington Street, England, 1855

To Marscha and Youri

PREFACE

The research project leading to this dissertation aimed at the identification of more sustainable wastewater treatment scenarios based on physical-chemical pre-treatment. The project was initiated and funded by STOWA, the Dutch Foundation for Applied Research on Water Management and was conducted by a research consortium of the Department of Sanitary Engineering at the Delft University of Technology and the Department of Environmental Engineering of the Wageningen University. In close co-operation over a period from September 1996 until June 2001, knowledge was exchanged and research topics were divided according to special fields of expertise.

Research project

The overall research objective concerns identifying the possibilities of physical and physical-chemical pre-treatment techniques as a basis for the development of more sustainable wastewater treatment scenarios. Furthermore, this project points to necessary long-term research to achieve development and implementation of these alternative wastewater treatment methods in the Netherlands [STOWA (1998)].

In the course of the research co-operation, the main objective was split into two parts. These sub-objectives were answered individually by one of the researchers. The role of particles in wastewater systems was of particular interest within the main context of the research. Knowledge was gained about the contribution of particles regarding the pollution of the incoming wastewater influent due to adsorbed or incorporated pollution fractions on or in particulate material. Furthermore, the effect of particle removal in the pre-treatment on the performances of total wastewater treatment systems and the related environmental criteria and costs were explored. The specific research question that the dissertation presented here answers is:

"What is the specific role of particulate matter characteristics in wastewater influent and of advanced particle removal techniques in creating more sustainable treatment scenarios based on physical-chemical pre-treatment?"

The dissertation "Physical-chemical pretreatment as an option for more sustainable municipal wastewater treatment plants ", published by ir. Adriaan R. Mels parallel to the dissertation outlined above, deals with the specific sub-objective:

"Exploration of the utilisation of organic polymers combined with either settling or dissolved air flotation and studying the effect of pre-treatment with polymers on the removal of nutrients in a biological post treatment step "

Since each of the executive researchers from both universities finalised their research with a Ph.D.-thesis within the scope of the same overall research context, some of the work in this thesis will be comparable to parts published by Mels [Mels (2001)]. Research results obtained in close co-operation are handled as common ranges of thoughts and individually gained outcomes are used by both researchers, are cross-referenced and appended with short notes added.

The organisation and time schedule of the project are pictured in the diagram below. The chapters of this thesis, referred to below in parentheses are listed per research topic. In cases in which Mels is mentioned the research topic is described in the earlier mentioned thesis of Mels.

period	**in co-operation**		**Delft University of Technology**		**Wageningen University**	
Sept.'96 - Sept.'97	Development of DEMAS	Identification and Evaluation of Physical-chemical Wastewater Treatment Scenario's (Chapter 1, 2 & 3; Mels)				
Sept.'97 - Sept.'98	Implementation of DEMAS	Evaluation study (Chapter 2 & 3, Mels)				
		Experimental Research into the Application of Organic Polymers (Chapter 5, Mels)				
Sept.'98 - Sept.'99	Development of DEMAS 2		Experimental Research Coarse Media Filtration (Chapter 6)	Experimental Research into Direct Membrane Filtration (Chapter 7)	Experimental Research into Flotation of Raw Wastewater (Thesis Mels)	Experimental Research into Aerobic and Denitrifying A-step
Sept.'99 - Sept.'00	Development of DEMAS$^+$		Experimental Research into Wastewater Fractionation (Chapter 4)		Experimental Research into Dosing Strategy (Thesis Mels)	Experimental research into biological post-treatment technique (Thesis Mels)
		Final Evaluation Study (Chapter 8, Mels)				
Sept.'00 - Sept.'01	Implementation of DEMAS$^+$		Reuse Applications (Chapter 7)			
			Thesis		Thesis	

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First, I like to thank my promoter Jaap van der Graaf, for giving me the opportunity to conduct this research. I thank him for his inspiring way of doing research and finding new challenges, his guidance and co-operation during the past five years and his personal and friendly companionship. I like to thank my co-promoter Jaap de Koning who I consulted many times as vademecum, for his wise support and collegiality. My special acknowledgements are going out to my second promoter Wim Rulkens and Bram Klapwijk from who I learned different investigative approaches and research niceties. I like to express my gratitude to Adriaan Mels my close co-researcher, without whom this thesis would never had become as it is today. I like to thank him for our perfect co-operation, his views, shared expertise, and his friendship. The special and close co-operation within our research project between Delft University and Wageningen University has a unique status, which should be continued in the future.

I like to thank my good friend and colleague Jeroen Langeveld, for his active involvement in this thesis and his critical suggestions and our discussions.

For over seven years, I studied and worked with lots of fun and enthusiasm at the Department of Sanitary Engineering and I am grateful having so many excellent and friendly colleagues. I especially like to mention the staff of the Laboratory of Sanitary Engineering for their valuable technical and analytical support. I like to thank my Masters students Ingrid, Herman and Sebastien for their work and efforts.

The overall research project was initiated and funded by STOWA the Dutch Umbrella Organisation for Applied Research on Water Management. From this organisation, I like to thank Pieter Stamperius, the STOWA eminence, who shared with me his many years of experience in wastewater treatment and teached me with his famous red pen how to write a perfect Dutch report. I also like to thank Paul Roeleveld for his shared knowledge and his enthusiastic way of initiating and organising STOWA research. I am grateful having been able to work with a very specialised and skilful STOWA supervision committee; all members of the committee Andy Schellen, Jan Slange, Egbert van 't Oever, George van der Geest, Helle van der Roest and Kees de Korte I highly appreciate for their expertise and inspiration.

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Moreover, last but absolutely not at least I want to thank my beloved Marscha and Youri for providing me every day with so much joy and happiness. Without their support and inspiration, this thesis would never have been written.

Arjen van Nieuwenhuijzen
Amsterdam, January 2002

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CHAPTER 1 INTRODUCTION AND SCOPE

SUMMARY

This introductory chapter deals with the present situation of sanitary systems in the Netherlands. To describe the scope of the research, the topics of sustainable development regarding to wastewater management and physical-chemical pre-treatment of wastewater are discussed.

Since potable water was supplied from the year 1853, and sewer systems were constructed in the early 1900, surface waters were increasingly polluted by increasing human and industrial discharges. In 1960, surface waters were so polluted that several rivers and local lakes suffered from a lack of oxygen and nearly were dead. With the enforcement of the Pollution of Surface Waters Act in 1970 a massive building programme of wastewater treatment plants started. Water management initially directed towards the removal of oxygen consuming substances and later advanced nutrient removal was introduced to fight eutrophication of surface water. In 1998, 409 municipal wastewater treatment plants with a design capacity of nearly twenty-five million population equivalents were in operation treating more than 96% of the nation-wide produced wastewater.

Although the existing system for sewerage and wastewater treatment operates properly, policy makers, technologists and scientists in the Netherlands are considering whether, in view of sustainable development, new concepts should be adopted for future developments. In this context, a research project was conducted with the objective to identify the possibilities of physical and physical-chemical pre-treatment techniques as a basis for the development of more sustainable wastewater treatment scenarios. Within this investigation, the role of particles in wastewater systems is of particular interest. This dissertation will answer a more specific research question about which impact particulate matter characteristics of wastewater and advanced particle removal techniques have on developing more sustainable wastewater treatment scenarios.

1.1 Introduction

This introductory chapter deals with the present situation of sanitary systems in the Netherlands. To describe the scope of the research, the topics of sustainable development regarding to wastewater management and physical-chemical pre-treatment of wastewater are discussed. In the last sections of this introduction, the research objectives of this dissertation are defined and the dissertation chapters outlined.

1.2 The state of the art of municipal wastewater treatment in the Netherlands

History and present situation

In the Netherlands, potable water was supplied from the year 1853, starting in towns and cities. An increase in the quantity of wastewater and therefore also in surface water pollution resulted. Wastewater from households and industrial activities was discharged directly into surface water until around 1900. During those times one of the major functions of the Dutch canals was the receipt of wastewater. In densely populated areas, this caused severe problems for public health, specifically because water from the canals was used for consumption and for personal hygiene. To protect the public health, sewer systems were constructed to convey wastewater from urban areas into the large rivers and lakes. After the Second World War, a great deal of economic growth in combination with a growing population led to increasing pollution of surface waters. In 1960, surface waters were so polluted that several rivers and local lakes suffered from a lack of oxygen and nearly were dead [Dirkzwager (1997)].

40 years later by the year 2000, the sanitary and environmental infrastructure in the Netherlands has become one of the most advanced in the world. Nearly all Dutch households have drinking water supply and 98% of the houses are connected to sewer systems with a total length of more than 82,000 km. 84% the total sewer system are gravity sewers and 16% are mechanically pressurised sewer pipes. Generally, two types of gravity sewers are being used, i.e., seventy-two percent of the gravity sewers are of the so-called '*combined system*'. In this system, wastewater and storm water are transported together to a treatment plant. A part of the storm water is discharged directly to surface water through combined overflow structures (CSO's). Twenty-eight percent of the gravity sewers belong to the '*separate system*': wastewater is fed to the treatment plant, and rainwater is directly discharged into surface waters. Because of remaining unwanted pollution from both systems, an increasing number of '*improved separated systems*' have been added to these systems [Rioned (2001)].

With the enforcement of the Pollution of Surface Waters Act in 1970 a massive building programme of wastewater treatment plants (wwtp's) was started. Water management became particularly directed towards improving the oxygen content and removal of oxygen consuming substances was the only issue. Around 1978, in addition to biological oxygen demand (BOD) reduction, nitrification of ammonium-nitrogen was also required, resulting in a discharge standard for Kjeldahl-nitrogen. The standards were best met by constructing and operating biological treatment plants. Therefore, many biological treatment plants have been built since 1970. Due to the fast development of the activated sludge system, physical-chemical treatment in mechanical treatment plants was replaced by biological systems increasingly. The period after 1985 refers to the control of eutrophication, due to the discharge reduction into surface water of the nutrients phosphate and nitrogen [Nieuwenhuijzen and Graaf (2000)].

Wastewater treatment

Nowadays more than ninety-six percent of the produced municipal and industrial wastewater is treated. In 1998, 409 publicly owned municipal wastewater treatment plants were in operation with a design capacity of nearly twenty-five million population equivalents (see figure 1.1), resulting in an average design capacity per wwtp of 61,000 population equivalents (p.e.). By mid 1990, standards were set for the phosphate concentrations in effluent water to be discharged by wastewater treatment plants. The limits for nitrogen date from 1992. In 1998, about 200 treatment plants were equipped for dephosphatation and increasingly (22%) of the total design capacity is operated with denitrification for N_{total} removal. Increasingly, biological P-removal is applied, with the result that the use of precipitation with metal salts and correlating particle removal in the first treatment stage is declining.

Figure 1.1 shows an overview of various types of operated treatment plants in 1995 and 1998. By comparing the situation in both years, it can be seen that the number of (ultra) low loaded systems increase over the years with the carrousel systems and oxidation tanks. For these systems, also an increasing design capacity is observed. At this moment, the carrousel system is numerically the most applied treatment system. The total number of oxidation ditches decreased from 1995 to 1998. The design capacity increased, however, resulting in an increase of hydraulic load of the centralised treatment plants. The number of aeration basins decreased over the same period, but still remains with the highest overall design capacity in the Netherlands. Trickling filters and mechanical treatment systems have nearly disappeared.

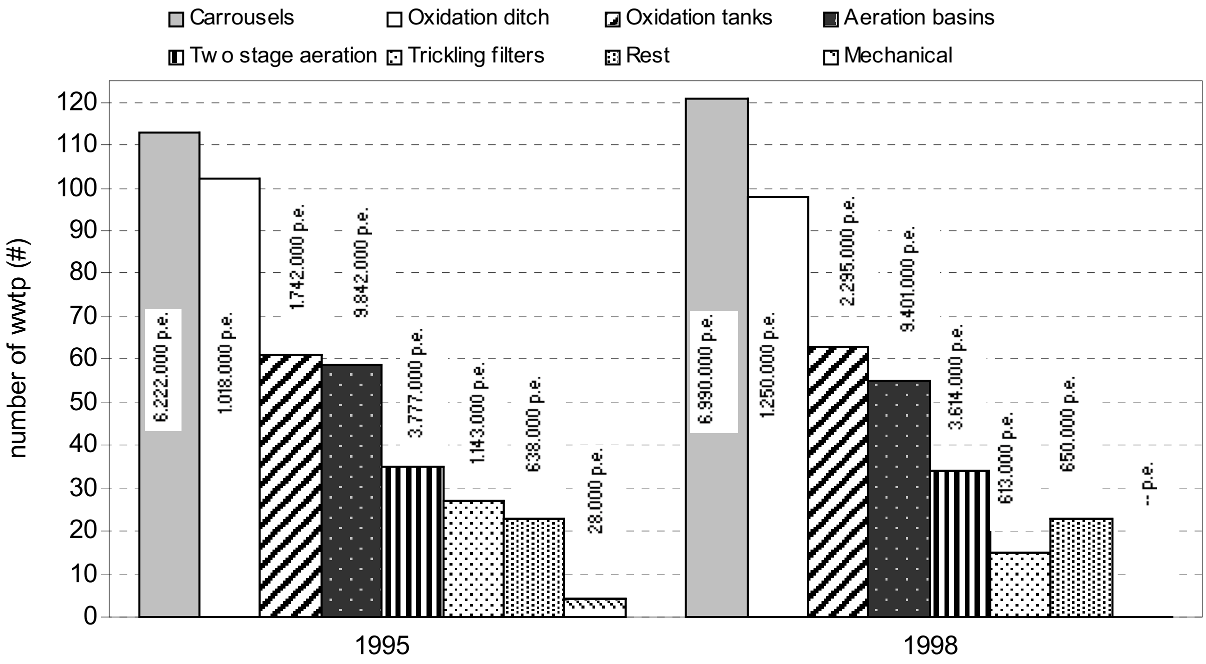

Figure 1.1: Various types of applied wastewater treatment systems in the Netherlands, in numbers of operated plants. The data in the column represent the design capacity [CBS (1995-2000)]

To give an impression of the wastewater quality in the Netherlands, table 1.1 shows the yearly average concentrations and loads for wastewater influent over the period 1992 to 1998. It shows fluctuating average concentrations of pollutants in the wastewater influent over the years, whereas loads remain at the same level. The fluctuation in concentrations is mainly caused by changes in the average daily flows in a year. Variations in the flow depend on relatively wet (1992 – 1994 and especially 1998) or dry years (1996 and 1997).

Table 1.1: Yearly average concentrations in mg per litres and total freights (between brackets and in 1,000 kg per day) on contaminants in municipal wastewater in the Netherlands (1992 - 1998)

parameter	**1992**	**1994**	**1995**	**1996**	**1997**	**1998**
BOD_5 (mg O_2/l)	196 (957)	190 (959)	185 (907)	230 (949)	224 (985)	173 (953)
COD (mg O_2/l)	533 (2,597)	525 (2,656)	510 (2,522)	603 (2,522)	570 (2,508)	456 (2,548)
N_{kj} (mg N/l)	47.5 (231)	46.4 (235)	47.2 (230)	55.4 (227)	53.9 (233)	41.8 (233)
P_{total} (mg P/l)	7.8 (38.1)	7.0 (38.5)	7.6 (38)	8.8 (37.0)	8.6 (37)	6.7 (37)
Total flow (m^3/d)	4,871,000	5,060,000	5,071,000	4,523,000	4,651,000	5,879,000
Precipitation*(mm/y)	845	929	782	632	686	1,109

Wastewater data by CBS [CBS (1995-2000)]
* annual precipitation data by KNMI [KNMI (2001)]

Table 1.2 shows the current national discharge standards for the discharge of treated municipal wastewater to state surface waters.

Table 1.2: National discharge standards: current effluent standards for wastewater treatment plants, in mg/l 10-day average (flow proportional daily composite samples), except for N-total (yearly average); complying with the EU Urban Wastewater Treatment Directive [EEA (1998)].

parameter	standard	applies	applies to
BOD_5	20	now	
suspended solids	30	now	
P_{total}^{a}	2	now	new plants + existing plants < 100,000 p.e.
	1	now	new plants + existing plants > 100,000 p.e.
$N_{kjeldahl}$	20	now	
N_{total}^{a}	10/15^{b}	now	new plants > 20,000 p.e.
	15	now	new plants < 20,000 p.e.
	10	from 2005	existing plants > 20,000 p.e.
	15	from 2005	existing plants < 20,000 p.e.

a: depending on being able to prove that a 75% elimination rate has been achieved within the control area of a water board as an average. b: 10 mg/l; unless phosphate is removed by simultaneous precipitation; in the latter case 15 mg/l applies.

As an indication for today's situation on wastewater treatment in municipal wwtp, table 1.3 shows some characteristic data. It demonstrates clearly that the removal of oxygen consuming compounds, with 89% COD- and 96% BOD_5-removal, and nutrient removal, with 84% $N_{kjeldahl}$- and 73% P_{total}-removal, is on a rather high level.

Table 1.3: Characteristic data for wastewater treatment in municipal wwtp's in 1998 [CBS (2000), Graaf, van der (2001)]; for detailed data see appendix A

wwtp			
- total number	409	#	
- design capacity	24.8	10^6 p.e.	
- actual load	21.7	10^6 p.e.	
- flow	5.8	10^6 m^3/d	
influent composition	(averages)	**effluent quality**	(averages)
COD	456 mg O_2/l	COD	51 mg O_2/l
BOD_5	173 mg O_2/l	BOD_5	7 mg O_2/l
$N_{Kjeldahl}$	41.8 mg N/l	$N_{Kjeldahl}$	6.6 mg N/l
N_{total}	44.1 mg N/l	N_{total}	13.5 mg N/l
P_{total}	6.7 mg P/l	P_{total}	1.8 mg P/l
sludge production			
total	350	10^6 kg d.s./y	
per p.e.	16	kg d.s./y	= 44 g d.s./day
per m^3	165	mg d.s./y	
energy consumption			
total	520	10^6 kWh/y	
per p.e.	24	kWh/y	= 2.7 Watt
per m^3	0.25	kWh	

As can be seen in table 1.3 the annual energy consumption was calculated to be 520 million kWh. Per population equivalent this is a consumption of about 24 kWh per year (equal to a continuous energy demand of 2.7 Watt, equal to a stand-by bulb on a TV-set).

Sludge handling

Major and increasing problems in wastewater treatment are caused by the sludge produced at treatment plants. By applying more stringent regulations on heavy metals for the usage of sludge as a fertiliser on farm land, the application of wastewater sludge in agriculture was eliminated by 1998 [CBS (1995-2000)]. Wastewater sludge will increasingly have to be deposited. As space becomes limited, costs for landfill increase and measures will have to be taken for an extensive reduction in sludge quantity and volume. The volume can be reduced by mechanical dewatering, using centrifuges, filter presses or belt presses, followed by drying, digestion or composting and finally incineration in centralised sludge combustion installations. After incineration, the residual ashes have to be deposited as chemical waste (see appendix A).

Another way of dealing with the residuals is to fix the ashes by cementation or crystallisation techniques and apply the products in construction materials. This form of recycling is extensively applied for drinking water sludge and is being considered for wastewater sludge [Wolde, ten (2000)].

In addition, sludge digestion can be and is used to produce biogas for energy production for on-site use. In some occasions energy from sludge digestion is sold as ‘green energy’ to power companies.

Treatment costs

Total annual treatment costs in 1998 were about € 700 million, equal to approximately € 32.- per year per p.e. or € 0.33 per m^3 wastewater (see also table A.5 in Appendix A).

From the year that the Pollution of Surface Waters Act came into force (1970) until 1995, € 4,350 million have been invested in the construction of municipal waste-water treatment plants, pumping stations and pipelines.

From 1995 until 2000, again approximately € 1,000 million have been invested in wastewater treatment systems. This investment was mainly attributed to measures on phosphate and nitrogen removal. To be able to comply to the more stringent nitrogen standards, € 1,500 million is or will be invested again over the period 1998 to 2005.

1.3 Wastewater treatment systems in the context of sustainable development

From the moment the Brundtland Commission's report *Our Common Future* described *sustainable development* in 1987 as *‘the development that makes use of the available resources without exporting the problems of pollution into time or into space’* [WCD (1987)] sustainability has been a national and international issue regarding water and wastewater systems. *Environmental sustainability* strives to preserve the available natural resources and environmental conditions so that future generations will still be able to subsist. *Sustainable technological development* aims at the development of technologies that decrease the input of not-renewable resources and prevent emissions that are harmful to the ecosystem. The next paragraph gives a short impression of ideas from the literature on sustainability in relationship to urban water systems.

Sustainable development in Dutch wastewater management

Although the existing system for sewerage and wastewater treatment operates properly, policy makers, technologists and scientists in the Netherlands are considering whether, in view of sustainable development, new concepts should be adopted for future developments. In this context, the urban water management should strive for separation at the source, closed urban water cycles and minimisation of energy consumption.

Rulkens and Van Starkenburg proposed in 1993 general environmental starting points for sustainable development of wastewater treatment plants in the Netherlands [Rulkens and Starkenburg, van (1993)]:

- maximisation of the use of valuable components in wastewater and sludge;
- minimisation of energy demand and maximisation of (re)use of energy;
- minimisation of the use of non-renewable resources;
- minimisation of emissions to air and discharge to surface water; and
- minimisation of final inert waste products.

Within a research programme entitled ‘*Sustainable Technological Development*’, an exploratory study has been conducted of the potential of a more sustainable urban water cycle. In this study, so-called unsustainability factors were identified as most important [Graaf, van der *et al*. (1997)]:

- discharge of heavy metals and organic micro-pollutants into water;
- production of wastewater sludge, and
- discharge of oxygen-consuming substances during storm weather overflow from sewers.

In contrast to Dirkzwager, energy consumption and CO_2 emissions resulting from wastewater treatment plants were identified as less important, since these factors have a comparatively low contribution to the total national output.

Roeleveld *et al.* [STOWA (1996) and Roeleveld *et al.* (1997)] concluded from a life cycle analysis (LCA [Heijungs *et al.* (1992)]) of the municipal wastewater treatment systems that, in a situation of high nutrient removal the contribution of municipal wastewater to the total national unsustainability is relatively low. Effluent quality and sludge production were again identified as most important issues regarding sustainable development. Corresponding with Van der Graaf *et al.* (1997), energy demand was not seen as most important.

Van Loosdrecht *et al.* (1997) implement the sustainable approach by investigating the environmental impact of advanced nutrient removal in an advanced modified University of Cape Town (UCT) treatment process.

Furthermore, Jetten, Horn and Van Loosdrecht (1999) present a concept of a more sustainable municipal wastewater treatment system by applying a more efficient energy balance due to sludge maximisation and the introduction of an ingenious nitrogen removal based on partial oxidation to nitrite and a following denitrification with ammonium as electron supplier: a combination of the SHARON and ANAMMOX processes [Jetten *et al.* (1999)].

International approaches

Internationally, many kinds of sustainability related researches and projects on wastewater treatment have been conducted. Some examples will be given here. Harremöes outlines his opinion on sustainable development and wastewater management in a ‘philosophical’ way and concludes that sustainability is often confused with ‘*quality of life*’ [Harremöes (1997)]. He stated that water is a renewable resource by which water use within an available flow is very sustainable [Harremöes (1999)]. He also states that wastewater treatment in general is already a very sustainable activity [Harremöes (2001)]. Lundin *et al.* (1999) proposed a useful set of sustainability indicators for the evaluation of sanitary systems and the assessment of their progress towards sustainable water management with life cycle analysis [Lundin *et al.* (1999)]. Hedberg reflected that sustainability should be the basis of new ways of thinking about water handling and on the other side that pursuing sustainable development opens possibilities for alternative treatment options [Hedberg (1999)]. Others discussed sustainable development in comparable ways, but were more specific about alternative ways of wastewater management regarding sustainability. Otterpohl advised decentralised systems based on anaerobic composting toilets and reed beds [Otterpohl (1997)]. Larsen and Gujer identified urine as bottleneck of sustainable wastewater treatment, but also as a resource; with the separation of anthropogenic nutrient solution (urine) at the source to prevent increasing entropy by diluting valuable resources [Larsen and Gujer

(1996), (1997)]. Finally, Ødegaard (1995) is cited since he calculated positive economical as well as sustainability effects from advanced particle removal by direct chemical treatment of wastewater in the first treatment step in a wwtp [Ødegaard (1995)].

Disputable implementation of sustainability

Sustainable development is an important intent, but sustainability has occasionally become more hype, rather than a real problem solving way of thinking. The term sustainability is difficult to define; it is interpreted and used differently and for different purposes as is shown above. A certain system or product can be more sustainable than another, but the statement '*this item is sustainable*' is difficult to make. Generally, new approaches to optimise products or systems should aim for more sustainability. However, sustainability can be a misleading term to use; it is often generalised or misused in indicating environmental friendly aspects of a system or a product. Sustainability is some times even used as a commercial strategy. Especially outcomes of life cycle analyses can be changed easily towards a favoured product or scenario by giving subjective weight factors to certain criteria. However, weight factors should be handled very carefully. Important criteria to determine sustainable development are subject to local conditions and to changes in time. Sustainability analysis can be used as a tool to evaluate and compare different scenarios on environmental effects, however [Mels *et al.* (1999)]. For this reason, sustainability criteria will be implemented in the following chapters as environmental interventions without subjective weight factors to evaluate and compare different wastewater treatment systems.

1.4 Physical-chemical pre-treatment

Physical-chemical pre-treatment, also called physical-chemical primary treatment aims at the separation of particles or particulates from wastewater within the first step of a wwtp. Advanced particle separation is assumed to be important in wastewater treatment, since a major part of the influent organic matter and nutrients is associated to particulate material. By the removal of suspended of colloidal particles, many pollutants that are incorporated into particulate material or adsorbed to particulate material can be eliminated. With physical-chemical pre-treatment, the post-treatment will be loaded with less contamination, so it might be designed energy efficiently [Loosdrecht et al. (1997), Ødegaard (1995)] and compactly [Ødegaard (1998)].

An important aspect of physical-chemical pre-treatment is that in many cases particles are too small to achieve an efficient particle separation by applying only a physical removal technique. This is illustrated by figure 1.2 wherein only half of the chemical oxygen demand (COD) larger than 1 µm is being removed by primary sedimentation. For most pre-treatment unit processes, coagulation and flocculation will have to be added in order to change the particle size by the formation of floc conglomerations.

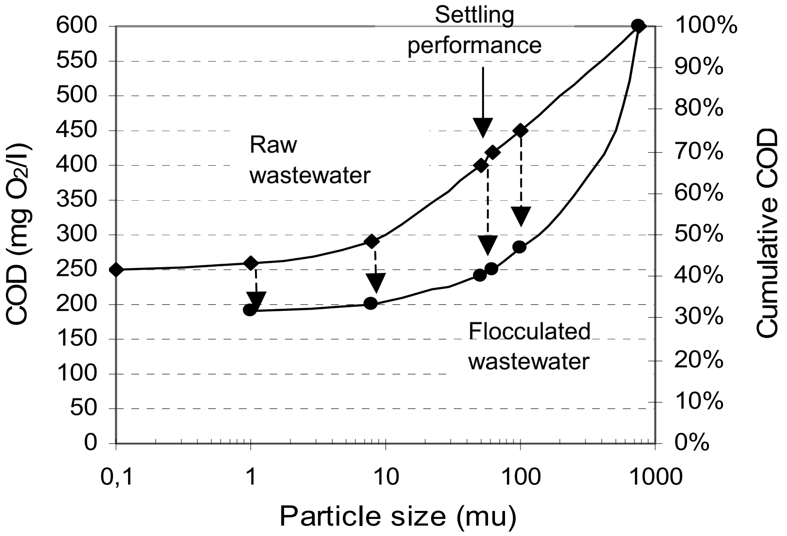

Figure 1.2: Cumulative COD distribution of particle size in raw and flocculated wastewater influent

To achieve coagulation-flocculation, specific flocculating agents will have to be added. Due to the addition of these flocculants, the surface charges of the particles will change and particles will conglomerate due to their mass forces. This results in the formation of larger aggregates, whereby the average size distribution is shifted towards larger particle sizes. Because more, large particle structures are present in the coagulated and flocculated wastewater, the removal efficiency of the physical unit process will increase.

Historical overview

In general, the basic physical-chemical pre-treatment consists of a gravity based separation process, like a primary sedimentation, where chemicals are added to the incoming water to improve the gravitational characteristics of the particles.

Besides sedimentation, flotation, sieving and filtration techniques are particle removal techniques applicable in the pre-treatment step; however, these techniques are not generally applied in municipal wastewater treatment, but are used more often in specific industrial wastewater treatment.

Chemical additives were already used in the early years of water and wastewater treatment as an aid for physical separation technologies. Physical-chemical treatment started for example with the addition of crushed and pulped almonds and beans to (waste)water to improve sedimentation conditions of particles. Later aluminium containing clay or dirt was used as additive.

The first known chemical enhanced wastewater treatment system was applied around 1840 in Paris, France.

Around 1890, more than 200 small-scale wastewater treatment plants, based on chemical treatment processes were operated in England. These treatment plants used lime, aluminium and iron salts as coagulants to lower the suspended solids content in the wastewater.

In 1900, 75 chemical coagulants and flocculants were listed by the German scientist professor König, used in water and wastewater treatment. In a wwtp in Frankfurt (Germany), aluminium sulphate and lime were used in 1884 [Imhoff (1998)].

In the USA, hardly any chemical wastewater treatment was applied until the 1930’s. By the time Rudolfs published an article in 1929 on the advantages of chemical treatment processes at low concentrations of iron chloride regarding particle sedimentation, treatment plants in the USA were adapted towards chemical treatment with metal salts or lime. In the 1960’s, it was considered appropriate to change biological systems completely into chemical and physical-chemical treatment plants. This approach led towards a number of new treatment systems, like the physical-chemical treatment plant at Truckee at Lake Tahoe in California, but never substituted the biological treatment completely, due to relatively high operational costs of the applied chemical technologies.

Starting in 1920 at a wwtp of the city of Tilburg, chemical treatment was introduced relatively late to the Netherlands. Around 1940, lager scale chemical enhanced treatment was operational aiming at the reduction of expensive (re)construction costs. Earlier, in 1900, the German 'Braunkohlenverfahren' came up for a while. In this process, crushed lignite and iron sulphate was added to the wastewater. This led to an improved effluent quality and the residual sludge could be processed further into fire briquettes, to be used in household scuttles instead of turf sods [Groeneveld (1994)].

Actual international and national applications

As described in the short historical overview, mechanically (pre-) treatment is ubiquitous and the first applied technology to treat wastewater. It fell into disfavour, however, because of the required high amounts of chemicals like metal salts or lime. An additional problem was - and still is - the production of large quantities of extra inorganic chemical sludge. Nevertheless, in the 1980's scientists and operators of large treatment plants mainly in the USA rediscovered the potential of chemical enhanced pre-treatment and reintroduced chemical enhanced wastewater treatment and the use of combinations of chemical dosages [Harleman and Murcott (1999), (2001 a/b), Parker *et al.* (2001)]. Wastewater treatment in colder climates, like Scandinavian countries, always has been based on physical and physical-chemical treatment systems. In those regions, these techniques are still being applied on a large scale in combination with additional biological treatment systems [Karlsson (1992), Ødegaard (1987), (1992), (1994), (1998)].

In the Netherlands, advanced particle removal in physical-chemical pre-treatment is not commonly applied at wwtp's anymore. If coagulants are used at all, the main goal is to precipitate phosphates. As earlier stated and as shown in an overview of existing treatment plants in appendix A, phosphorous is increasingly treated by biological dephosphatation, so P-precipitation with metal salts in pre-treatment is less frequently used. In some specific cases, a combined dosage of metal salts and polymeric flocculants is applied, with the aim of lowering the suspended solids content of the wastewater or with the goal to improve settling conditions of the high loaded activated sludge flocs [Schellen (1997)].

1.5 Objective of this dissertation

The overall research objective of the research project was to identify the possibilities of physical and physical-chemical pre-treatment techniques as a basis for the development of more sustainable wastewater treatment scenarios.

In this dissertation, the role of particles in wastewater systems is of particular interest. The specific research question that this dissertation will answer is:

"What is the specific role of particulate matter characteristics of wastewater influent and the impact of advanced particle removal techniques on developing more sustainable wastewater treatment scenarios?”

1.6 Outline of this dissertation

Chapter 2 describes the applied methodology of system and scenario analysis used for identification and evaluation of possible physical-chemical pre-treatment techniques. The developed design and evaluation model DEMAS is demonstrated and finally, promising wastewater treatment scenarios and important research topics are identified by a preliminary evaluation study.

Chapter 3 gives the theoretical background of the presence of particles in wastewater-like suspension.

Chapter 4 focuses on the experimental research into fractionation and characterisation of particulate matter in wastewater influent. The distribution of contaminants over particle size fractions are shown and the impact of physical-chemical treatment processes on the primary effluent quality are presented. Furthermore, energy calculations will indicate the importance and possibilities of advanced particle removal in the first treatment steps.

In chapter 5, applications of organic polymers for coagulation and flocculation of particles in wastewater are dealt with by means of experimental lab-scale and pilot-plant research. Applicability of different types of polymers and dosage optimisation are points of interest in this chapter.

Chapter 6 and 7 describe experimental research and application of direct filtration techniques for advanced particle removal in pre-treatment.

Chapter 6 presents the explorative experimental research activities on large-scale pilot-plants of different applications of coarse media filtration. Continuous and discontinuous, down- and up-flow filtration at varying filtration rates have been tested.

Chapter 7 deals with the exploration of the technological applicability of direct membrane filtration of raw wastewater. Ultra filtration membranes have been applied as pre-treatment technique to filter raw wastewater. The experiment research are described and a possible theoretical application is discussed.

Operation and removal performances of the physical-chemical pre-treatment techniques are starting-points in both chapters.

In chapter 8, a final evaluation is presented wherein different wastewater treatment scenarios based on physical-chemical pre-treatment techniques as investigated in this dissertation are compared. Sensitivity analysis on particle size distributions in wastewater and removal performances of treatment techniques are shown to deepen the knowledge about the influent of particles in physical-chemical pre-treatment.

Finally, in chapter 9, the content of this dissertation is summarised and considerations are made about the implementation of physical-chemical pre-treatment and the role of advanced particle removal in supposed future developments in wastewater treatment.

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CHAPTER 2 METHODOLOGY OF SCENARIO ANALYSIS AND PRELIMINARY EVALUATION

SUMMARY

This chapter describes the methodology of scenario analysis, developed to identify and evaluate the opportunities of different wastewater treatment systems based on physical-chemical pre-treatment[*]. Based on a review of unit processes, scenarios for wastewater treatment were designed and evaluated by means of environmental and financial criteria. To support this methodology arithmetically, a design and evaluation model - entitled DEMAS[+] - was developed, where DEMAS stands for '**D**imensionerings- en **E**valuatiemodel voor **A**fvalwater**S**ystemen'. The basics of this evaluation tool are explained in this chapter. Within the scenario evaluation, special attention was paid to the energy balance, the final sludge production, the space requirement and the economic feasibility of the identified scenarios.

The evaluation study, shows that wastewater treatment systems with physical-chemical pre-treatment can potentially be designed more energy efficient and may designed smaller compared to the commonly applied treatment systems. The total costs of treatment scenarios, which combine physical-chemical pre-treatment with biological post treatment, are in the same range as that of the reference scenario.

The scenario study revealed the following further research needs: (1) Regarding the influence of strength, composition and characteristics of the wastewater on possibilities of pre-treatment, extensive experimental research has to be conducted on wastewater fractionation and characterisation related to particles. (2) Investigations should be carried out to replace (inorganic) metal salts by organic polymers to avoid an increase in sludge production and salinization of the wwtp-effluent. (3) Detailed experimental research on technological operation and performances of direct filtration techniques with coarse media and membranes are to be tested in parallel to flotation of raw wastewater.

Since the methodology and scenario evaluation were developed in close co-operation as described in the preface, some of the content or text phrases of this chapter may resemble Mels' dissertation "Physical-chemical pretreatment as an option for more sustainable municipal wastewater treatment plants" [Mels (2001)].

2.1 Introduction

This chapter describes the methodology of scenario analysis, developed for this research to identify and evaluate the opportunities of different wastewater treatment systems. Scenario analysis is a method that deals with both the complexity and uncertainty of the environment. It is especially suited for forecasting where uncertainty is high and historical relationships are changing. To support this methodology arithmetically, a design and evaluation model - entitled DEMAS$^+$ - was developed, where DEMAS stands for '**D**imensionerings- en **E**valuatiemodel voor **A**fvalwater**S**ystemen'.

Based on the DEMAS methodology, wastewater treatment scenarios consisting of unit processes for pre- and post-treatment are designed and tested with DEMAS$^+$. The identified scenarios are evaluated on environmental and cost criteria and compared to a conventional wastewater treatment plant as a reference scenario. From this starting point, the possibilities to design more sustainable wastewater treatment systems, based on physical-chemical pre-treatment were preliminary evaluated and fields of research are identified.

2.2 Methodology of scenario analysis

The methodology that was developed for evaluating the sustainability of the identified and composed wastewater treatment scenarios, is based on the Life Cycle Assessment (LCA) methodology [Heijungs *et al.* (1992)]. The LCA methodology is a systematic approach to determine the environmental impact of processes and products. A strong point of LCA is, that it also includes environmental impacts due to the production of raw materials and the ultimate discharge of products and processes. Together with extensive cost calculations based on the net present value method [Mol - Van Otterlo (1977)], the environmental impacts were used as evaluation criteria.

Environmental interventions

For the evaluation of treatment scenarios, the Life Cycle Assessment was provisionally limited to the inventory analysis, i.e. the determination and calculation of *environmental interventions*. Environmental interventions are defined within the LCA methodology as "the exchanges between the anthroposphere (the 'economy') and the environment including use of resources and discharges to air, water or soil" [Heijungs e*t al*. (1992)]. Based on sustainability related research in urban water management [Graaf, van der *et al*. (1997), Rulkens and Starkenburg, van (1993)] and earlier studies of the application of LCA in wastewater treatment [Roeleveld *et al*. (1997), STOWA (1996)] the following environmental interventions or sustainability criteria were selected to serve as evaluation criteria:

- energy balance;
- final sludge production;
- consumption of chemicals;
- effluent quality;
- space requirement (footprint).

The environmental impacts related to the construction of the plant (building materials like concrete and steel and final waste products) were not included. This was based on the outcomes of the before mentioned researches [Roeleveld *et al*. (1997), STOWA (1996)], which showed that these items have only very minor contribution to the overall environmental impacts in the Netherlands. Table 2.1 shows an overview of the assessed parameters:

Table 2.1: List of environmental interventions

environmental interventions	**remarks**
energy balance	Result of energy usage and energy recovery (from the sludge digestion). Conducted similar to [Loosdrecht, van *et al.* (1997)].
final sludge production	The sludge production after thickening and digestion. Due to the strict sludge quality standards regarding heavy metals, waste sludge is not allowed to be used on farmland in the Netherlands and has to be incinerated and/or safely dumped.
use of chemicals	Chemicals are used to optimise the treatment processes; reuse of chemicals is included.
effluent quality	The effluent quality has to meet at least the discharge standards in the Netherlands; also high effluent qualities, for the application of effluent (re)use, are taken into consideration.
footprint	In densely populated areas, like the Netherlands, space is scarce.

Cost calculations

Supplemental to the environmental interventions the estimated costs of the scenarios were calculated by means of the net present value method. The net present value (NPV) reflects all aggregated costs over a total depreciation period, with a certain interest rate and inflation rate. The net present value includes the investment costs and the running costs during the total period of operation. Within the net present value method, these costs are all recalculated to the moment of investment [Mol – Van Otterlo, de (1977)].

For the cost calculations of the treatment scenarios, a depreciation period of 30 years was chosen for civil building investments and fifteen years for electro-mechanical investments. The applied interest rate was 5.5% with an inflation rate of 2.5%, resulting in an actual interest rate of 3%.

Equations 2.1 through 2.4 show the methodology for calculating the net present value. Assumptions of investment cost figures and running costs are given in appendix B.

$$NPV_{Investment} = \sum_{n=1}^{30} (\frac{1}{1+i})^{n-1} \cdot [D + i_I] \tag{2.1}$$

where NPV = net present value (€)
i = interest rate (%)
n = number of years (#)
D = yearly depreciation (€)
i_I = interest of remaining investments (€)

The investment costs include the investments for civil structures (I_C in €) and electro-mechanic works (I_{em} in €). So, the NPV for the construction investments are calculated following the equation:

$$NPV_C = \sum_{n=1}^{30} (\frac{1}{1.055})^{(n-1)} \cdot [\frac{I_c}{30} + 0.055 \cdot (I_c - \frac{n}{30} \cdot I_c)] \tag{2.2}$$

The electro-mechanical investment costs are similarly calculated, although a reinvestment after 15 years was taken into account. For this reinvestment (I_{em}* in €) an inflation rate of 2.5% per year was applied.

$$\begin{aligned} NPV_{em} = & \sum_{n=1}^{15} (\frac{1}{1.055})^{(n-1)} \cdot [\frac{I_{em}}{15} + 0.055 \cdot (I_{em} - \frac{n}{15} \cdot I_{em})] + \\ & \sum_{n=16}^{30} (\frac{1}{1.055})^{(n-1)} \cdot [\frac{I_{em}{}^{*}}{15} + 0.055 \cdot (I_{em}{}^{*} - \frac{(n-15)}{15} \cdot I_{em}{}^{*})] \end{aligned} \tag{2.3}$$

where $I_{em}{}^{*} = I_{em}\,(1.025)^{15}$

The operational costs include the total of yearly costs (all in €) for maintenance (M), energy (E), chemicals (C), sludge handling (S) and labour (L) recalculated to the time of investment, including the inflation percentage and interest rate. The cost assumptions for the operational costs are given in appendix B.

$$NPV_{RC} = \sum_{n=1}^{30} (M + E + C + S + L) \cdot \frac{1.025^n}{1.055^n} \tag{2.4}$$

Basic starting conditions

All wastewater treatment scenarios are designed for a wastewater treatment plant (wwtp) of 100,000 population equivalents (p.e.). The hydraulic design parameters are shown in table 2.2. The average influent flow is supplied by the concentrated water, which is rejected in the sludge treatment.

Table 2.2: Hydraulic loading rates for a wwtp with supply of 100,000 p.e.

flow per p.e. ($Q_{p.e.}$)	150 l / p.e./ day	
dry Weather Flow (dwf)	1,000 m^3 / h	(supply during 15 hours per day)
storm weather Flow (swf)	3,000 m^3 / h	
average flow ($Q_{Av.}$)	19,500 m^3 / day	(calculated as 1.3 × dwf × 15 h)

The influent values for COD, BOD_5, N_{total}, P_{total} and suspended solids were preliminary based on average concentrations in wastewater in the Netherlands [CBS (1993), (1995), (1997), (1999), (2000)]. The influent figures were rounded off and are used as theoretical influent composition (see table 2.3). Prerequisite for the design is that the final effluent of the treatment scenarios should at least meet the future effluent discharge standards in the Netherlands.

Table 2.3: Influent composition and effluent standards applied in the analysis

parameter		influent composition	effluent discharge standards
COD	(mg O_2/l)	600	< 50.0 (yearly average)
BOD_5	(mg O_2/l)	220	< 10.0 (yearly average)
N_{total}	(mg N/l)	55	< 10.0 (yearly average)
P_{total}	(mg P/l)	9	< 1.0 (progressive average of 10 successive samples)
susp. solids	(mg TSS/l)	250	< 10.0 (yearly average)

System boundaries

The system boundaries applied in the evaluation, as shown in figure 2.1, include all wastewater and sludge treatment processes. Within the research, the identified wastewater treatment scenarios are combined with a sludge treatment. This sludge handling consists of mechanical and gravitational thickening, followed by sludge digestion and finally dewatering. For all of the wastewater and sludge treatment processes, the selected environmental criteria and costs are calculated.

After dewatering, the final sludge cake is transported to a centralised incineration plant, from where the remaining ashes are landfilled (in general as chemical wastes). These joined processes are called the final sludge handling and disposal unit. Only a cost value is ascribed to this unit, depending on the food-to-micro-organism (F/M) ratio to be disposed. The recycle water from the sludge digestion and dewatering is returned to the influent stream.

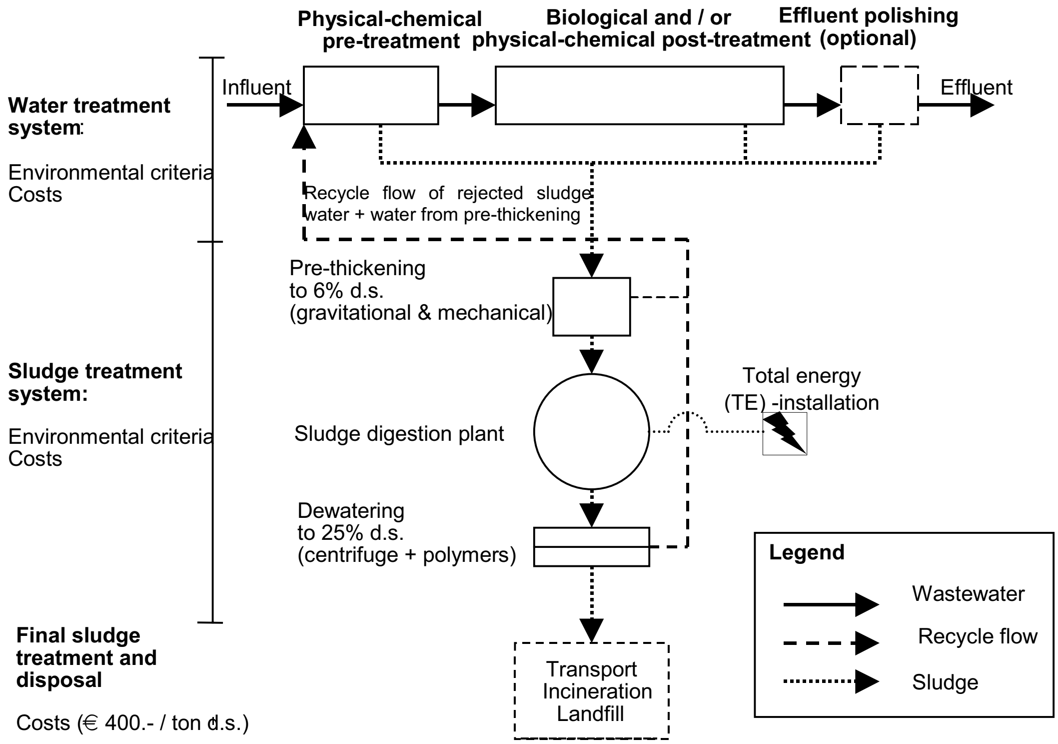


Figure 2.1: System boundaries applied in DEMAS$^+$

2.3 Design and evaluation model DEMAS

For the design and evaluation of the wastewater treatment scenarios a design and evaluation model entitled DEMAS was developed. DEMAS (**D**imensionerigs- en **E**valuatiemodel voor **A**fvalwater**S**ystemen) is the Dutch abbreviation for '**D**esign and **E**valuation **M**odel for W**A**stewater treatment **S**cenarios'. The model consists of a comprehensive and detailed data base of necessary technical information concerning various unit processes (e.g. design parameters, energy consumption, treatment efficiencies, unit operation costs, etc.), which can be selected and combined. In DEMAS, unit processes can be combined to compose complete wastewater treatment scenarios, consisting of a wastewater treatment system and a sludge handling system. Through aggregation, the environmental interventions and the net present values can be determined per unit process or per scenario. From these, individual unit processes as well as complete treatment scenarios can be evaluated and compared. DEMAS was developed further during the research by adding gathered research results, newly found figures from the literature as well as data from practical experiences. Finally, DEMAS$^+$ was created, which was more adequate, flexible and user-friendly. The preliminary evaluation results that will be shown in this chapter were derived by calculations with the first edition of DEMAS. Further on in this dissertation, a final evaluation is described that uses DEMAS$^+$ as calculation and evaluation tool. The design and evaluation model will be explained below as the final DEMAS$^+$ version. The original DEMAS has been extensively described in other earlier published work [Nieuwenhuijzen, van *et al*. (1998), Mels *et al.* (1998), STOWA (1998)].

Structure and flow sheets of DEMAS^{+}

DEMAS was originally created as a supporting calculation tool to design and evaluate treatment scenarios. DEMAS and DEMAS^{+} are build as spreadsheet models in Microsoft® Excel 97 SR-2.

The structure of DEMAS is presented in a flow diagram in figure 2.2 and consists of input and output modules and calculation modules. The individual modules (see figure 2.2) stand for different units of the wastewater treatment system: pre-treatment, post-treatment and sludge handling.

In DEMAS, the first input module concerns the raw wastewater influent composition. Water quality and quantity parameters and particle distribution data can be submitted.

Next to the influent composition, the input module 'pre-treatment' is used to design the desired pre-treatment techniques by adding design parameters and removal efficiencies.

Fed by these input data a calculation module calculates the primary effluent composition and the primary F/M ratio and composition by using mass balances and removal efficiencies. The calculation results are input data for the modules 'sludge treatment' and 'post-treatment' and are completed with external input design data for the post-treatment and the sludge treatment modules.

Pre-treatment, post-treatment and sludge handling system are handles in three individual calculation modules. In addition, the environmental interventions and costs are calculated for each applied unit process in these calculation modules

The pre-treatment and post-treatment modules are linked up to form the water system for which the final effluent quality, the environmental interventions and the costs are derived. The calculated total F/M ratio is looped back into the input module 'sludge treatment'. In parallel, the sludge system is designed by identifying the recycle water flows, the final F/M ratio, the environmental interventions and the costs. The recycle water flows are combined and returned to the influent composition input module. By an iteration process of a maximum of 1,000 calculation steps and a maximum final difference of 0.01, the wastewater influent composition is recalculated and the complete wastewater and sludge treatment system are actualised.

The final output module collects all the environmental interventions and cost figures of the designed wastewater treatment system. The output data can be saved in a data file, from where tables and graphs can be built for further evaluation.

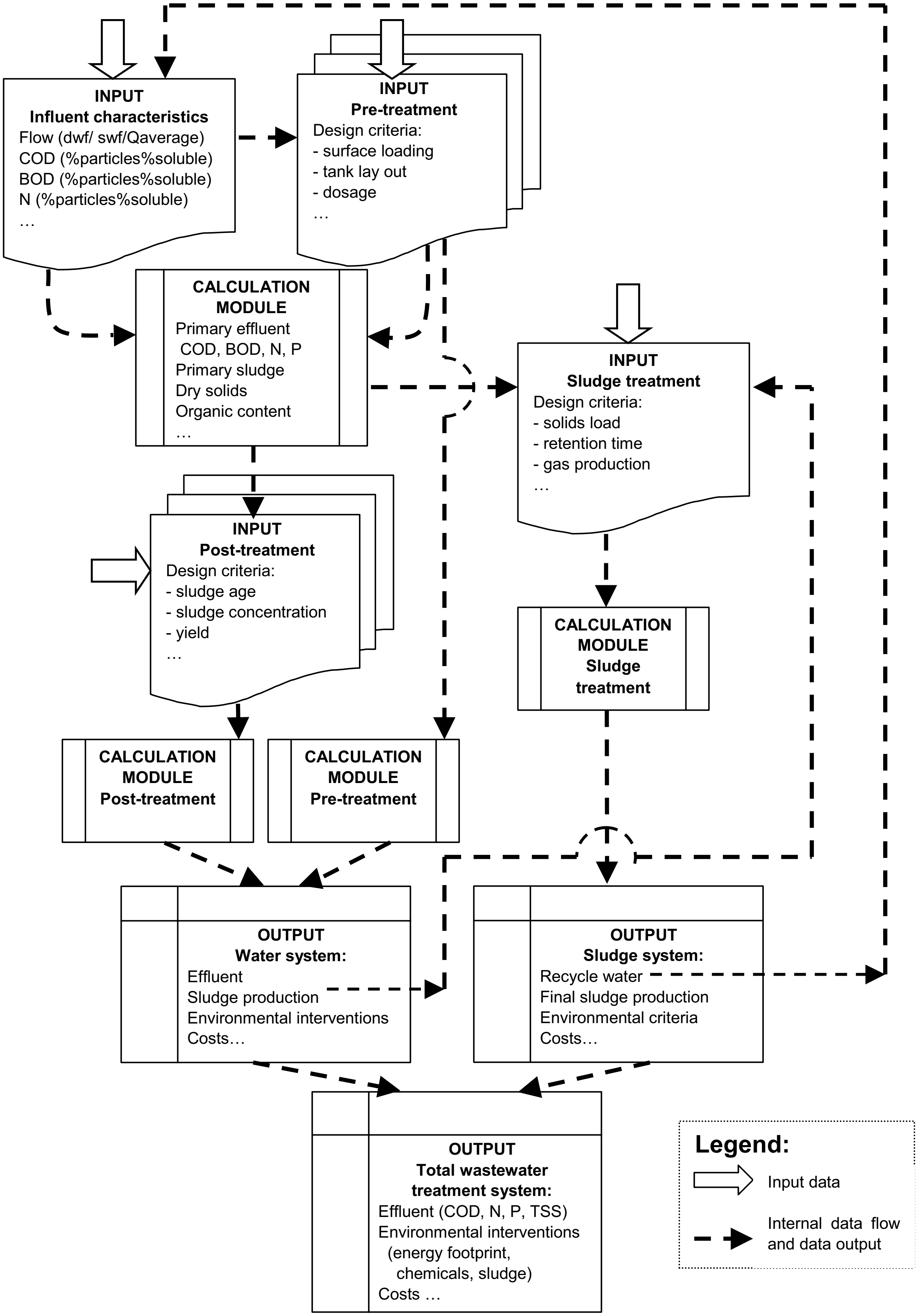
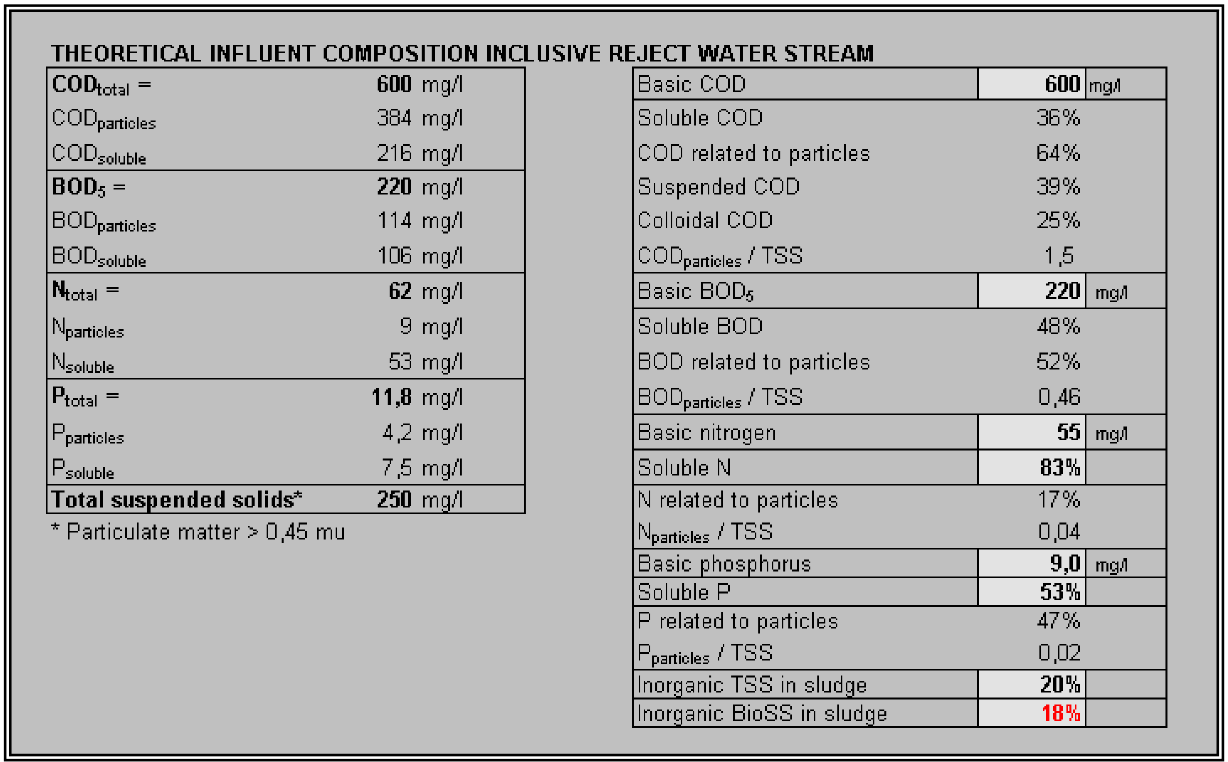


Figure 2.2: Flow diagram of DEMAS^{+}

Input screens

The first input screen, entitled '*Influent composition*' (see figure 2.3), data regarding the composition of the wastewater influent have to be submitted. The basic input data are the concentrations of COD, BOD, nitrogen and phosphorus in the incoming wastewater. These data have to be collected by sampling and analytical tests. Next to the concentrations, the distribution of the contaminants over particle fractions is asked for. If this information is not available, the standard influent characteristics can be used, but an extensive wastewater fractionation and characterisation programme will provide information that is more representative.



THEORETICAL INFLUENT COMPOSITION INCLUSIVE REJECT WATER STREAM

COD$_{total}$ =	**600**	mg/l
COD$_{particles}$	384	mg/l
COD$_{soluble}$	216	mg/l
BOD$_5$ =	**220**	mg/l
BOD$_{particles}$	114	mg/l
BOD$_{soluble}$	106	mg/l
N$_{total}$ =	**62**	mg/l
N$_{particles}$	9	mg/l
N$_{soluble}$	53	mg/l
P$_{total}$ =	**11,8**	mg/l
P$_{particles}$	4,2	mg/l
P$_{soluble}$	7,5	mg/l
Total suspended solids*	**250**	mg/l

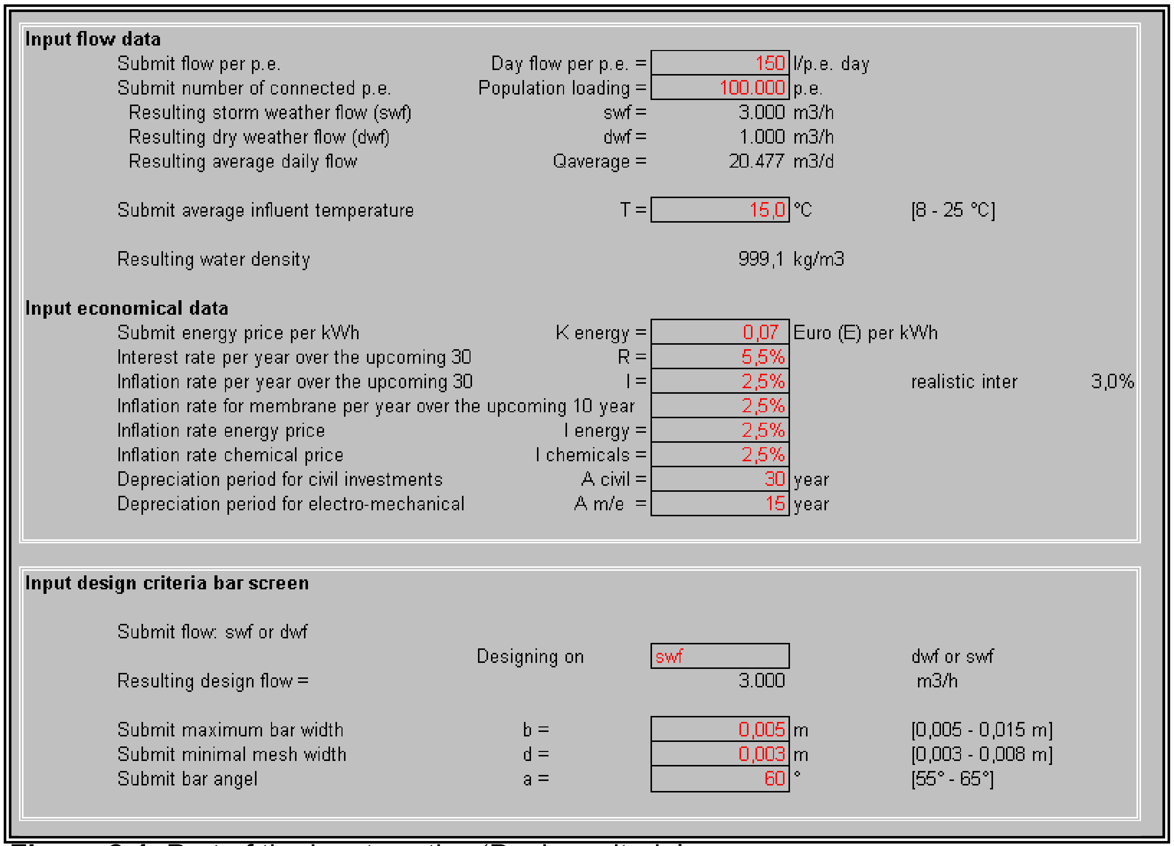
* Particulate matter > 0,45 mu

Basic COD	**600**	mg/l
Soluble COD	36%	
COD related to particles	64%	
Suspended COD	39%	
Colloidal COD	25%	
COD$_{particles}$ / TSS	1,5	
Basic BOD$_5$	**220**	mg/l
Soluble BOD	48%	
BOD related to particles	52%	
BOD$_{particles}$ / TSS	0,46	
Basic nitrogen	**55**	mg/l
Soluble N	**83%**	
N related to particles	17%	
N$_{particles}$ / TSS	0,04	
Basic phosphorus	**9,0**	mg/l
Soluble P	**53%**	
P related to particles	47%	
P$_{particles}$ / TSS	0,02	
Inorganic TSS in sludge	**20%**	
Inorganic BioSS in sludge	**18%**	

Figure 2.3: Example of the input screen 'Influent composition'

From the influent composition screen, the user is led to the input section '*Design criteria*' (see figure 2.4 as an example). In this section, data have to be filled in regarding plant layout and hydraulic capacity. In addition, economical data, e.g. energy costs, inflation figures and interest rates, are requested for the costs calculations. Further on, the design parameters of the unit processes have to be submitted. For each unit process it has to be indicated whether it is to be designed on dry weather flow (dwf) or storm weather flow (swf). Automatically, the design flow is calculated and for each desirable unit process, the lay-out appears on screen, resulting in a provisional layout. Parallel to the design, the cost calculations have been conducted, which result in the net present values for each unit process.

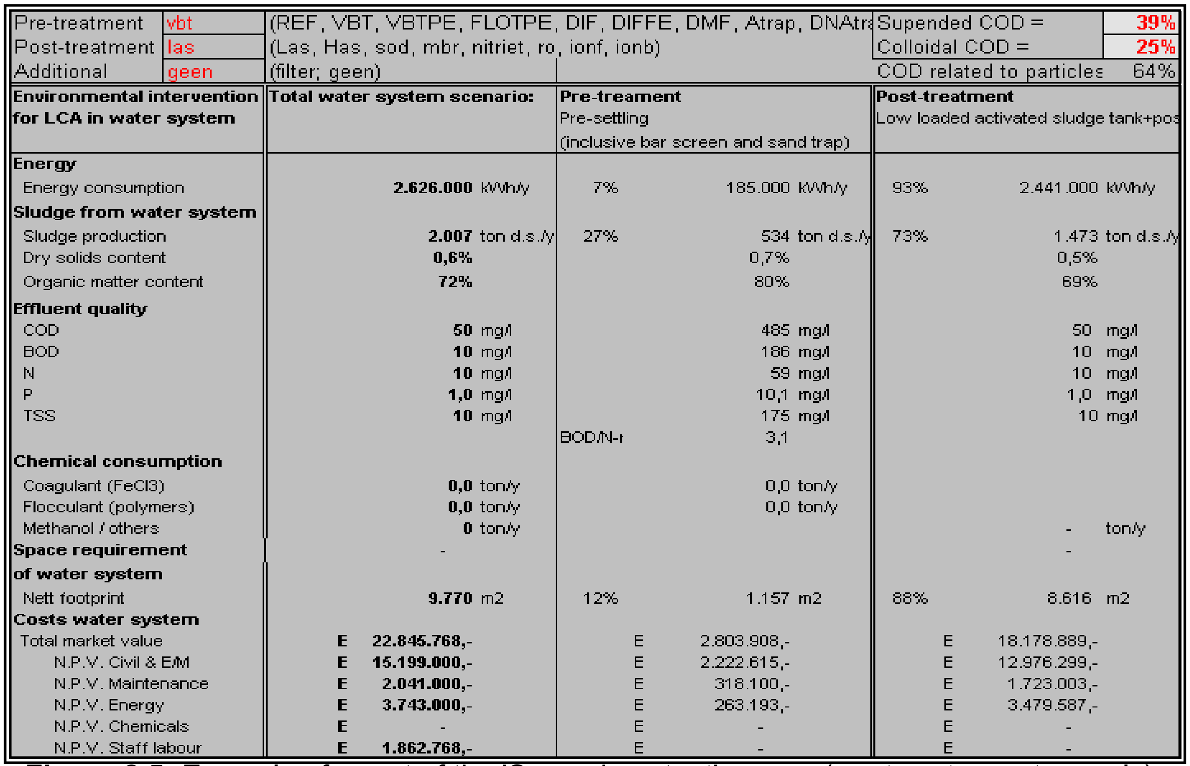
In this section, both, the wastewater system and the sludge handling system can be designed via individual input screens.



Input flow data				
Submit flow per p.e.	Day flow per p.e. =	150	l/p.e. day	
Submit number of connected p.e.	Population loading =	100.000	p.e.	
Resulting storm weather flow (swf)	swf =	3.000	m3/h	
Resulting dry weather flow (dwf)	dwf =	1.000	m3/h	
Resulting average daily flow	Qaverage =	20.477	m3/d	
Submit average influent temperature	T =	15,0	°C	[8 - 25 °C]
Resulting water density		999,1	kg/m3	
Input economical data				
Submit energy price per kWh	K energy =	0,07	Euro (E) per kWh	
Interest rate per year over the upcoming 30	R =	5,5%		
Inflation rate per year over the upcoming 30	I =	2,5%		realistic inter 3,0%
Inflation rate for membrane per year over the upcoming 10 year		2,5%		
Inflation rate energy price	I energy =	2,5%		
Inflation rate chemical price	I chemicals =	2,5%		
Depreciation period for civil investments	A civil =	30	year	
Depreciation period for electro-mechanical	A m/e =	15	year	
Input design criteria bar screen				
Submit flow: swf or dwf				
	Designing on	swf		dwf or swf
Resulting design flow =		3.000		m3/h
Submit maximum bar width	b =	0,005	m	[0,005 - 0,015 m]
Submit minimal mesh width	d =	0,003	m	[0,003 - 0,008 m]
Submit bar angel	a =	60	°	[55° - 65°]

Figure 2.4: Part of the input section 'Design criteria'

In the '*Scenario output'* screen (shown with the reference scenario in figure 2.5) the desired treatment scenario can be built from the available treatment units by selecting a unit process for the pre-, post- and - if necessary - an additional treatment step.



Pre-treatment	vbt	(REF, VBT, VBTPE, FLOTPE, DIF, DIFFE, DMF, Atrap, DNAtra	Supended COD =	39%
Post-treatment	las	(Las, Has, sod, mbr, nitriet, ro, ionf, ionb)	Colloidal COD =	25%
Additional	geen	(filter; geen)	COD related to particles	64%

Environmental intervention for LCA in water system	**Total water system scenario:**		**Pre-treament** Pre-settling (inclusive bar screen and sand trap)			**Post-treatment** Low loaded activated sludge tank+pos		
Energy								
Energy consumption	**2.626.000**	kWh/y	7%	185.000	kWh/y	93%	2.441.000	kWh/y
Sludge from water system								
Sludge production	**2.007**	ton d.s./y	27%	534	ton d.s./y	73%	1.473	ton d.s./y
Dry solids content	**0,6%**			0,7%			0,5%	
Organic matter content	**72%**			80%			69%	
Effluent quality								
COD	**50**	mg/l		485	mg/l		50	mg/l
BOD	**10**	mg/l		186	mg/l		10	mg/l
N	**10**	mg/l		59	mg/l		10	mg/l
P	**1,0**	mg/l		10,1	mg/l		1,0	mg/l
TSS	**10**	mg/l		175	mg/l		10	mg/l
			BOD/N-r	3,1				
Chemical consumption								
Coagulant (FeCl3)	**0,0**	ton/y		0,0	ton/y			
Flocculant (polymers)	**0,0**	ton/y		0,0	ton/y			
Methanol / others	**0**	ton/y					-	ton/y
Space requirement	-						-	
of water system								
Nett footprint	**9.770**	m2	12%	1.157	m2	88%	8.616	m2
Costs water system								
Total market value	**E 22.845.768,-**			E 2.803.908,-			E 18.178.889,-	
N.P.V. Civil & E/M	**E 15.199.000,-**			E 2.222.615,-			E 12.976.299,-	
N.P.V. Maintenance	**E 2.041.000,-**			E 318.100,-			E 1.723.003,-	
N.P.V. Energy	**E 3.743.000,-**			E 263.193,-			E 3.479.587,-	
N.P.V. Chemicals	**E** -			E -			E -	
N.P.V. Staff labour	**E 1.862.768,-**			E -			E -	

Figure 2.5: Example of a part of the 'Scenario output' screen (wastewater system only)

The output screen presents the overall calculated lay-out and environmental interventions as well as the costs calculated as net present values of the complete treatment scenario and for the individual wastewater system or sludge system.

From this output screen, data files can be generated and graphics can be made by which different treatment scenarios can be compared and evaluated.

Removal efficiencies and primary effluent composition

The effluent composition of the applied pre-treatment steps in DEMAS$^+$ is based on the efficiencies found in experimental research, practical application and literature review. The effluent composition of the pre-treatment steps - the so-called primary effluent - determines the pollutant load of the post-treatment and the composition of the primary sludge.

The most actual version of DEMAS$^+$, uses the removal efficiencies and primary effluent compositions per pre-treatment unit process shown in table 2.4.

Table 2.4: Removal efficiencies and primary effluent compositions per pre-treatment unit (excluding N and P addition from recycle water flows)

removal efficiencies of pre-treatment techniques	PST	PST + PE	Flotation + PE	DIF	DCMF + Fe	DMF	A-step + PE	Den. A-step + PE
particle related	30%	80%	85%	50%	60%	100%	60%	70%
‘soluble’	0%	5%	10%	0%	5%	15%	20%	50%

	influent composition	**primary effluents (without additional N and P from recycle water)**							
		PST	**PST + PE**	**Flotation + PE**	**DCMF**	**DCMF + Fe**	**DMF**	**A-step + PE**	**Den. A-step + PE**
COD total (mg O_2/l)	**600**	**485**	**282**	**252**	**408**	**359**	**184**	**239**	**223**
COD particles	384	269	77	58	192	154	0	77	115
COD soluble	216	216	205	194	216	205	184	162	108
BOD_5 (mg O_2/l)	**220**	**186**	**123**	**112**	**163**	**146**	**90**	**128**	**140**
BOD particles	114	80	23	17	57	46	0	23	34
BOD soluble	106	106	100	95	106	100	90	106	106
N total (mg N/l)	**55.0**	**52.2**	**47.5**	**47.1**	**50.3**	**49.4**	**45.7**	**47.5**	**48.5**
N particles	9.3	6.5	1.8	1.4	4.6	3.7	0.0	1.8	2.8
N soluble	45.7	45.7	45.7	45.7	45.7	45.7	45.7	45.7	45.7
P total (mg P/l)	**9.0**	**7.7**	**5.6**	**5.4**	**6.9**	**3.5**	**4.8**	**5.6**	**6.0**
P particles	4.2	3.0	0.8	0.6	2.1	1.7	0.0	0.8	1.3
P soluble	4.8	4.8	4.8	4.8	4.8	1.8	4.8	4.8	4.8
particles (mg TSS/l)	**250**	**175**	**50**	**38**	**125**	**100**	**0**	**50**	**75**
BOD/N	4.0	3.6	2.6	2.4	3.2	3.0	2.0	2.7	2.9

PST = primary sedimentation tank, PE = polymer dosage, DCMF = direct coarse media filtration (discontinuous), DMF = direct membrane filtration, A-step = aerated A-step, Den. A-trap = denitrifying A-step (see appendix C)

N and P content of the recycle water from sludge pre-thickening and dewatering
The nutrient rich water that is rejected in the pre-thickening, digestion and dewatering units in the sludge treatment is recycled as influent flow. The influent volume is increased by the recycle water flow by about 10%, depending of the variable calculated incoming and fixed outgoing (of 25%) dry solids content of the sludge. The recycle water is enriched with ammonia-nitrogen and phosphate-phosphorus, since these components are released in the digestion process as is explained below.

The additional nutrient load from the primary sludge to the wastewater influent is calculated with the following assumptions:

- After removal in the physical-chemical pre-treatment, the primary sludge will be thickened mechanically, without the further release of nutrients.
- In the sludge digestion process 50% of the organic material in the primary sludge is digested following equation (2.5), by which ammonia and phosphates are released:

$$C_aH_bO_cN_dP_e \rightarrow CO_2 + CH_4 + \mathbf{NH_4\text{-}N} + \mathbf{PO_4\text{-}P} + \text{energy} + \text{cellular material} \quad (2.5)$$

- The biomass yield of the anaerobic bacteria in the disgestion process is low, by which the nutrient uptake by these bacteria is assumed neglible.
- Remaining nitrogen and phosphates are released to the water phase and will be partly rejected from the sludge in the final dewatering process.

Based on these assumptions, approximately 50% of the particle related nitrogen and phosphates, removed in the pre-treatment units, will be recycled in soluble form to the influent flow of the wastewater treatment plant.

The additional nutrient load from the secondary biological post-treatment, is calculated as follows:

- In the post-treatment biological sludge is produced, calculated with the individual yields specified per pre- and post-treatment.
- In the digestion process, 30% of the organic fraction of the secondary sludge is digested.
- Again, the nutrient uptake by the anaerobic bacteria is assumed neglible.
- The mass ratio COD:N:P in the secondary biological sludge is set at 150:14:3 [Metcalf & Eddy (1991), Lettinga *et al*. (1998)]. Per 100 g digested COD 9.3 g NH_4-N and 2 g PO_4-P is released to the rejected water phase.
- The inorganic solids content of the grown biomass (BioSS, see figure 2.3) is assumed to be 18%.

Changing F/M ratio due to pre-treatment

In DEMAS$^+$, the applied biological post-treatment units are designed on the food-to-micro-organism ratio or food-to-mass ratio (F/M ratio). In the case of the low loaded activated sludge system, for example, a F/M ratio of 0.060 kg BOD per kg mixed-liquor suspended solids (MLSS) per day is conventionally used in the Netherlands as design value.

In case of the application of an advanced physical-chemical pre-treatment, relatively more inert material is removed from the wastewater; so, an increase of biological activity may be assumed. The applicable F/M ratio for the biological post-treatment step depends on the removal performance for inert material in the pre-treatment. Table 2.4 shows that the different physical-chemical pre-treatment units perform differently regarding the elimination of particles and the inert fraction. Therefore, each pre-treatment produces a specific primary effluent that will have to be treated in a biological post-treatment at a specific F/M ratio.

To make an estimation of the different applicable F/M ratios after the pre-treatment units, the calculation tool *DenNi* (Version 3.22) based on the German HSG model - abbreviation of “Hochschularbeitsgruppe Erfahrungsaustausch Nitrifikation/Denitrifikation" - was used [STOWA (1995)]. The HSG model was developed by German academic institutes and (waste)water management boards, and is the most frequently applied model by engineering companies and water boards in the Netherlands to calculate and design low loaded activated sludge system with nitrification and denitrification.

For the calculations of the F/M ratios the standardised basic values in DenNi were applied. The most important input variation was the influent composition in DenNi. The input data for the influent characteristics, were the, by DEMAS$^+$ calculated, concentrations in the primary effluent per pre-treatment unit, as shown in table 2.4. In DenNi an activated sludge system with pre-denitrification with a capacity of 100,000 p.e. and a hydraulic loading of 19,500 m^3 per day at a design temperature of 10 ºC was designed. From this design, the other F/M ratios were derived. Table 2.5 gives an example of the calculated F/M ratios per pre-treatment step.

The conventional activated sludge system without any pre-treatment, should be designed on a F/M ratio of 0.052 kg BOD/kg MLSS/day, whereas with only a primary sedimentation process, an increased F/M ratio of 0.061 kg BOD/kg MLSS/day may be applied. Due to advanced particle removal by the application of physical-chemical pre-treatment techniques, F/M ratios between 0.076 and 0.111 kg BOD/kg MLSS/day are possible since the suspended solids load to the biological treatment is decreased. The biological treatment following the biological active high loaded aerated A-step (see appendix C) should be designed on a F/M ratio of 0.055.

The significant higher F/M ratios in the case of advanced particle removal in the pre-treatment are remarkable compared to the conventional applied F/M ratio.

Table 2.5: Example of F/M ratios in DEMAS$^+$, calculated by HSG per pre-treatment technique, depending on the removal performance of the technique

pre-treatment technique	applied F/M ratio in activated sludge tank (kg BOD/ kg MLSS / day)
without pre-treatment	0.052
simple sedimentation	0.061
chemically enhanced sedimentation	0.080
chemically enhanced DAF	0.084
direct coarse media filtration	0.076
direct membrane filtration	0.111
A-step (from A/B-verfahren)	0.055

Parallel to the F/M ratio, the sludge yield in the biological system depends also on the suspended solids load of the primary effluent and is thus influenced by the particle removal performance of the applied pre-treatment unit. Additional types of biological post-treatment units were similarly applied with varying F/M ratios, depending on the removal performances of the pre-treatment units.

2.4 Preliminary evaluation

At the start of the research project, a preliminary evaluation was conducted to identify the possibilities of physical-chemical pre-treatment in the Netherlands and to select promising treatment scenarios. From this preliminary evaluation, further research needs could be identified and specific research fields were recommended. This provisional evaluation was conducted with the first basic version of DEMAS, basically identical to DEMAS$^+$, but less sophisticated and accurate. The paragraph refers to the first part of the research project, and assumes the state of the art of the research at that time, which was not as detailed as presently presented in this dissertation. The results of a final evaluation with the most up-to-date DEMAS$^+$ and differences in outcomes of DEMAS and DEMAS$^+$, will be presented in chapter 8 of this dissertation.

Review of unit processes for pre- and post-treatment

The preliminary evaluation started with the inventory of pre-treatment techniques. Table 2.6 shows a brief overview of unit processes, which were identified and studied in the first inventory part of the research. The identified unit processes are the basic elements for the formation of complete wastewater treatment scenarios. By combining these unit operation processes in DEMAS, a large number of treatment scenarios were created.

Table 2.6: Overview of identified and studied unit processes

physical-chemical pre-treatment (removal of particulate material)	**secondary (post) treatment** (removal of soluble COD and (+) N)	**tertiary treatment** (final particle removal)
primary (precipitation) sedimentation	activated carbon adsorption (COD)	rapid sand filtration
flotation (DAF)	ozone oxidation (COD)	membrane filtration
micro screening	ion exchange (N)	- microfiltration
direct coarse media filtration	activated sludge systems (COD + N)	- ultrafiltration
biosorption (A-step)	biofilm systems (COD + N)	
magnetic separation	membrane bio reactor (COD + N)	
advanced settling with micro-sand		

These treatment processes are explained in detail in appendix C.

Reference treatment scenario

In order to compare the composed treatment scenarios with commonly applied treatment systems, a reference scenario was designed. This reference scenario is based on the most common advanced wwtp in the Netherlands with a low loaded activated sludge system for COD, nitrogen and phosphorus removal with primary and secondary sedimentation tanks as shown in figure 2.6.

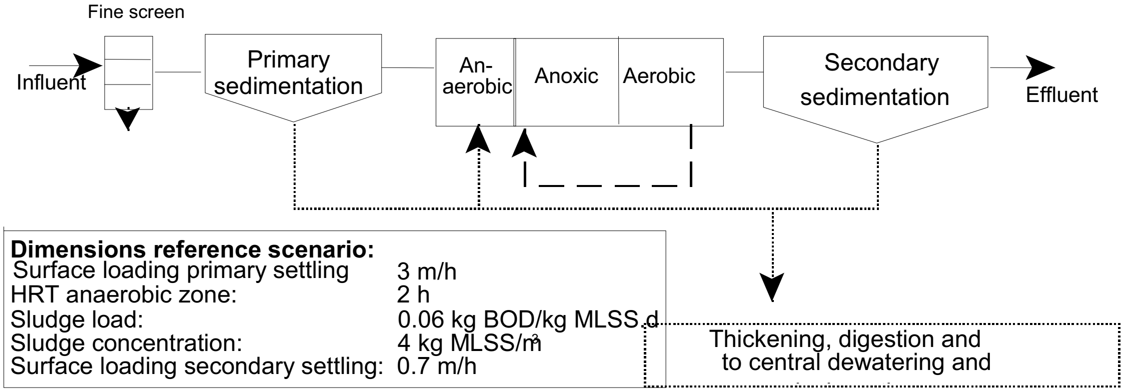


Figure 2.6: Reference scenario: primary sedimentation (without chemical addition) as pre-treatment with a low loaded activated sludge system and secondary sedimentation

The primary sedimentation tank was designed with a hydraulic surface loading of 3 $m^3/(m^2 \cdot h)$. An anaerobic zone was used to create conditions feasible for biological phosphorus removal (hydraulic retention time during dwf = 2 h). The F/M ratio of the aerobic/anoxic zone was preliminary set at 0.06 kg BOD per kg MLSS per day. The sludge concentration is 4 kg MLSS/m^3 and the hydraulic surface loading (at swf) of the secondary sedimentation tank was set at 0.7 $m^3/(m^2 \cdot h)$. The primary and secondary (biological) sludge are collectively treated by the standard sludge handling procedure in the original version of DEMAS, which consisted of thickening, digestion and transport of the remaining sludge to a central dewatering and incineration plant. In the end, the remaining ashes have to be landfilled.

Treatment Scenarios

Within the preliminary evaluation study, eighteen treatment scenarios were created from a selected number of the above mentioned unit processes. In table 2.7, these scenarios are presented. The scenarios are subdivided into three main scenario groups (divisions) - 1, 2 and 3 - based on the applied post-treatment processes.

Table 2.7: Treatment scenarios with pre- and post-treatment steps

scenario	pre-treatment	post-treatment
scenario 1a	flotation (DAF)	sand filter + ion exchange (air stripping) + activated-carbon adsorption (thermal reactivation)
scenario 1b	pre-precipitation	sand filter + ion exchange (air stripping) + activated-carbon adsorption (thermal activation)
scenario 1c	denitrifying biosorption step	sand filter + ion exchange (biological brine treatment) + activated-carbon adsorption (thermal reactivation)
scenario 1d	aerated biosorption step	sand filter + ion exchange (air stripping) + activated-carbon adsorption (thermal reactivation)
scenario 2a	flotation (DAF)	low loaded biofilm system + sand filter
scenario 2b	pre-precipitation	low loaded biofilm system + sand filter
scenario 2c	aerated biosorption step	low loaded biofilm system + sand filter
scenario 2d	flotation (DAF)	low loaded activated sludge system over nitrite + sand filter
scenario 2 e	pre-precipitation	low loaded activated sludge system over nitrite + sand filter
scenario 2f	aerated biosorption step	low loaded activated sludge system over nitrite + sand filter
scenario 3a	flotation (DAF)	biofilm system + sand filter + ion exchange (air stripping)
scenario 3b	pre-precipitation	biofilm system + sand filter + ion exchange (air stripping)
scenario 3c	denitrifying biosorption step	biofilm system + sand filter + ion exchange (biological brine treatment)
scenario 3d	aerated biosorption step	biofilm system + sand filter + ion exchange (with air stripping)
scenario 3 e	flotation (DAF)	high loaded activated sludge system + sand filter + ion exchange (air stripping)
scenario 3f	pre-precipitation	high loaded activated sludge system + sand filter + ion exchange (air stripping)
scenario 3g	denitrifying biosorption step	high loaded activated sludge system + sand filter + ion exchange (biological brine treatment)
scenario 3h	aerated biosorption step	high loaded activated sludge system + sand filter + ion exchange (air stripping)

In division 1, physical-chemical post-treatment is applied. In division 2, biological post-treatment is used and in division 3 a combination of physical-chemical and biological post-treatment is applied.

All treatment scenarios produce a similar effluent. Due to the application of final rapid sand filtration, the effluent is of a higher quality than the required discharge standards. The salt content of the effluent depends on the dosage of metal salts added in the pre-treatment.

Design, environmental and cost criteria for the unit processes were compiled from a literature review (see appendix C and D).

Results of the preliminary evaluation with DEMAS

The calculated outcomes for environmental criteria and costs of the evaluated scenarios from the analysis in DEMAS are summarised in the following paragraphs. A comparing overview of the scenarios is presented with diagrams in appendix E.

Energy balance - From the provisional evaluation it was concluded that energy can be saved through the application of physical-chemical pre-treatment techniques, due to a lower load on the post-treatment (resulting in less required aeration energy) and a higher recovery of energy from bio-gas production by enhanced sludge digestion.

Especially, the scenarios in division 2 are most interesting with respect to energy, with even a theoretical neutral or positive energy balances, so the wwtp may be operated energetically self-sufficient or could even produce energy.

Treatment scenarios based on activated-carbon adsorption demand a lot of energy due to the reactivation of the carbon. In addition, the brine treatment of the ion-exchange process (air stripping or biological regeneration) results in a high energy consumption.

Final sludge production - The treatment scenarios with physical-chemical pre-treatment produce more final sludge after thickening and digestion compared to the reference scenario. The increase of the final sludge quantity is caused by the production of inorganic chemical sludge (primarily metal-phosphate, but also metal-hydroxide as a by-product) due to the dosage of metal salts. An additional increase in sludge production was caused by a higher organic sludge production in the biological post-treatment units. This high organic sludge production has the advantage of a high-energy recovery from the digestion step. On the other hand, due to the necessary transport, central handling and final deposition (at € 400.- per ton dry solids, inclusive central dewatering), high final sludge quantities result in relative high costs.

Chemical consumption - All the scenarios require the addition flocculating agents (in this case $FeCl_3$ and anionic polymers) in the pre-treatment for advanced particle removal. In the cases that biological P-removal is absent, more iron salt has to be added to precipitate phosphorous in the pre-treatment. The treatment scenarios applied with ion exchange (with air stripping) require the addition of substantial amounts of NaOH and H_2SO_4, however this application enables the recovery of nitrogen from the wastewater by producing ammonia sulphate.

The use of chemicals leads towards increasing operational costs and effects the effluent quality. The release of counter-ions causes an increased salt content and fluctuations of pH of the effluent.

Effluent quality - In general, the treatment scenarios produce similar effluent qualities. However, some treatment techniques might improve the final effluent quality, like for example the application of ion exchange and activated carbon adsorption (for details: [Mels (2001)]). Ion exchange is capable to produce effluents with nitrogen contents near zero, whereas activated carbon adsorption may lead toward a more advanced removal of bacteria, micro-pollutants and heavy metals.

The salt content of the effluent increases by using salts in the (pre-) treatment techniques.

Footprint - Compared to the reference scenario all treatment scenarios with physical-chemical pre-treatment can be designed more area-efficiently (compactly). The footprint of the fully physical-chemical scenario 1a (DAF with a sand filter, ion exchange and activated carbon adsorption) is the smallest, with only one-fourth of the space requirement of the reference scenario. The other scenarios have a calculated footprint of around one-third of the space usage of the reference scenario; only scenarios with more than one sedimentation tank (e.g. scenario 2d-2f and 3d-3h) have a relative big footprint.

Costs - The calculations of the net present value analysis show, that through physical-chemical pre-treatment total costs may probably increase. This is in part the result of higher costs for sludge treatment and final waste disposal due to the increase in chemical inorganic sludge production. Treatment scenarios based on ion exchange and activated-carbon adsorption are even two or three times more expensive than the reference scenario. This is mainly caused by the application of the necessary expensive brine treatment and reactivation techniques. Scenarios which combine an advanced physical-chemical pre-treatment with compact (area-efficient) biological (biofilm) systems (scenario 2a-2f) are more cost effective and the calculated costs as net present values can compete with the costs of the reference scenario. The calculations of the net present values have assumedly an accuracy of about ±20%.

2.5 Conclusions

The general conclusion of the first research phase was, that the application of enhanced particle removal results in a smaller and less energy consuming design of complete wastewater treatment systems when compared to commonly applied treatment scenarios. Due to the removal of particles in the physical-chemical pre-treatment, the required post-treatment becomes more efficient. The increased primary sludge production results in a significantly higher bio-gas production and energy recovery by means of sludge digestion. The preliminary calculations show that overall energy savings of 30-100% and space (footprint) savings of 20-80% are possible.

The comparisons of calculated costs shows that the overall costs of treatment scenarios that combine physical-chemical pre-treatment with biological post-treatment may be in the same range as those of the reference scenario.

Wastewater characteristics - The characteristics of the wastewater to be treated were identified to be of major importance for an adequate scenario analysis. The performance of a physical-chemical pre-treatment technique and the overall effect of pre-treatment on the wastewater system are determined to a large extent by the amount of particles in the wastewater and of pollutants related to particle size distributions. Only minor information about wastewater characteristics in relation to particles had been known until now.

Pre-treatment techniques - The literature review shows that different physical-chemical pre-treatment steps are available for the removal of particles. Identified unit processes for pre-treatment are dissolved air flotation, chemically enhanced primary sedimentation, direct wastewater filtration and the aerated or denitrifying A-step. Especially, on the rarely investigated direct wastewater filtration techniques further knowledge has to be gained.

Flocculants - Flocculation is essential in the process of advanced particle removal: with the exception of direct influent filtration all the identified pre-treatment steps require flocculants to obtain a significant level (i.e. >75%) of particle removal. The evaluation shows that the use of metal salts heavily contributes to the overall environmental impacts and costs of treatment systems. The application of metal salts results is an increase of inorganic (chemical) sludge production and in a higher salinity of the effluent.

The increased sludge production causes higher sludge handling costs, since final sludge has to be dewatered and incinerated due to the strict regulations in the Netherlands regarding sludge use in agriculture.

The increased salt concentration (here: chlorine ions) could contribute to the increase of salinity of the receiving surface waters.

Biological post-treatment - A possible bottleneck for the application of pre-treatment is the removal of biodegradable COD that is needed for the denitrification process in a biological post-treatment step. A decrease of the BOD/N ratio by the removal of biodegradable COD may result in richer nitrogen concentrations in the effluent and in an exceeding of the effluent standards.

A process like ion exchange may be an alternative process for nitrogen removal. This technique is especially interesting because of the possible reachable zero nitrogen discharge. The scenario evaluations show that the evaluated techniques for regeneration of the ion exchanger and recovery of the brine (i.e. nitrification and stripping) consume relative large amounts of chemicals and energy and result in relatively high costs. On the other hand, these regeneration techniques offer the possibility of nitrogen recovery for re-use.

Sludge - The scenario study shows that sludge production is a very important issue to consider and should not be seen separately from the wastewater treatment. The costs for sludge handling contribute one-third to half of the total treatment costs.

2.6 Further research

From the provisional research phase, the following further research fields are explored:

- Grounding the theoretical assumptions regarding wastewater, strength, composition and characteristics, extensive experimental research has to be conducted on wastewater fractionation and characterisation related to particles [STOWA (2001c)].

- Investigations into the possibilities of replacing (inorganic) metal salts by organic polymers to avoid an increase in sludge production and salinization of the wwtp-effluent [STOWA (2001a)].

- Detailed experimental research on technological operation and performances of direct filtration techniques with coarse media and membranes is tested in parallel to flotation of raw municipal wastewater. Furthermore, the denitrifying A-step, derived from the aerated A-step, is to be investigated [STOWA (2001a)].

- In addition to the investigation into pre-treatment techniques, research is carried out concerning the effect of advanced particle removal in the pre-treatment on the post-treatment [STOWA (2001c), Mels (2001)].

- The brine treatment of the ion exchange process seems to be the bottleneck of applying ion exchange for advanced nitrogen removal from wastewater. However, an explorative research is carried out into the applicability of this technique, since the potential zero nitrogen discharge into surface water and the nitrogen recovery are of interest [STOWA (2001b)].

Activated-carbon adsorption was identified to be expensive and energy consuming due to the reactivation technique, and will be left out of consideration for further research.

From the above mentioned research items, this dissertation presents conducted research into:

1. wastewater fractionation and characterisation of suspended matter;
2. the applicability of organic polymers for flocculation of raw wastewater;
3. the possibilities of direct coarse media filtration of raw wastewater;
4. the possibilities of direct membrane filtration of wastewater.

Apart from the technological expertise that will be derived from the research activities on the specific research fields, detailed information should be collected to improve the adequacy and applicability of the evaluation model to assist the scenario analysis.

2.7 References

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CHAPTER 3 PARTICLES IN WASTEWATER

SUMMARY

This chapter presents the theoretical background on the presence and behaviour of particles in wastewater-like suspensions, since this are the necessary fundamentals when investigating particles in wastewater and particle removal techniques.

It is known that a major part of the contamination in wastewater can be related to particles, since the parts of the contaminants are adsorbed to or incorporated in particulate matter. However, information about the distribution of contaminants to various particle fractions is rare. From international literature, a survey is made to get an indication about this distribution.

Although the influence of local conditions and habits on the wastewater composition has to be considered, the survey indicates that in average about 30% of the total COD is present in soluble form. 30% of the COD is related to colloidal and suspended particles and about 40% is equal to larger, settleable particulate matter. It is concluded that more information about wastewater fractionations and the characterisation of the particulate matter is needed to determine specific characteristics of an individual wastewater.

3.1 Introduction

A major part of the contamination in wastewater can be related to particles since it is absorbed to or incorporated in particulate matter. Understanding the theory concerning the presence and behaviour of particles in wastewater-like suspensions aids the study of particles in wastewater and particle removal techniques. The particle-related matter in wastewater can be theoretically, but still rather roughly, fractionated as shown in figure 3.1.

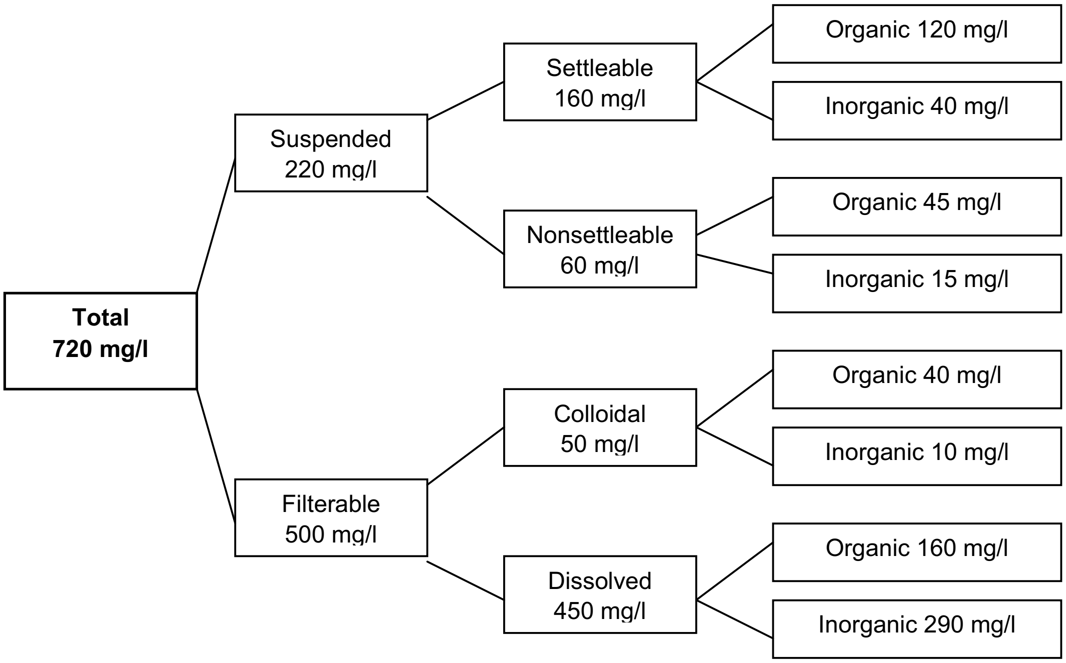


Figure 3.1: Example of a schematic classification of particulate matter in a strong wastewater [Metcalf & Eddy (1991)]

A more accurate information on particle fractions in wastewater is needed to explain and predict the effect of particle removal in wastewater systems. Therefore, this chapter gives an introduction into some colloid chemistry backgrounds, important for understanding the presence and behaviour of particles in wastewater.

3.2 Theoretical characteristics of particles in wastewater

Particles in colloidal stabile suspensions like wastewater are generally negatively charged since the surface charge of the particles is negative [Knoppert and Heide, van der (1990)]. The negative surface charge of the particles is mainly caused by three processes:

(a) reactions at the boundary layer

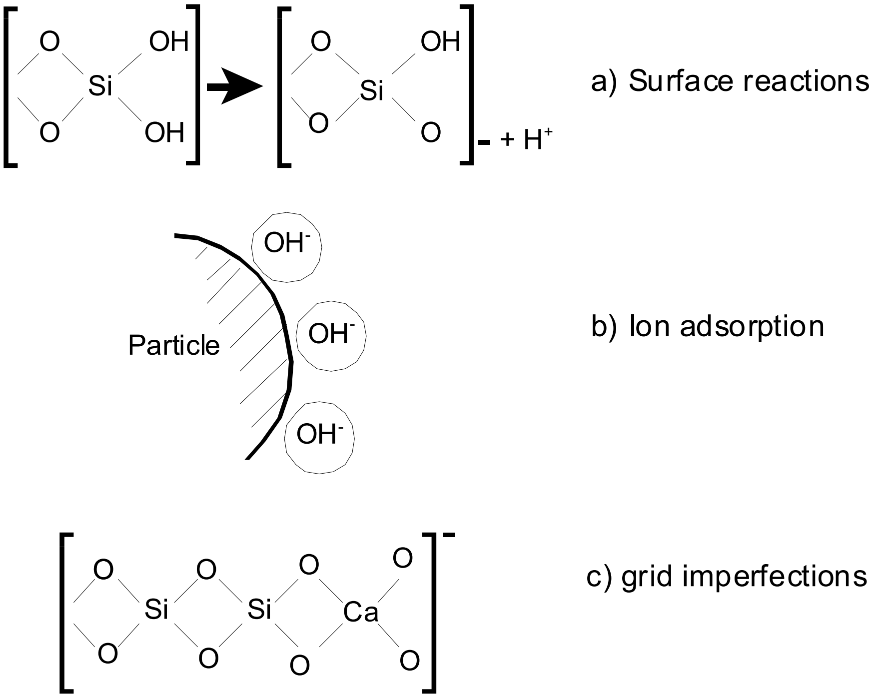
The surface charge is caused by dissolving- and acid/base reactions at the boundary layer of particles. For example, the dissolving reaction during the dissociation of a hydrogen atom from a silanol group causes a negative charge (see figure 3.2a).

(b) adsorbed ions or polymers

Ions or polymers present in the (waste)water may adsorb to the boundary layer of unsolved material (as shown in figure 3.2b). This adsorption may be caused by different mechanisms. Firstly, adsorption can occur because, for example, humic acids are partially hydrophobic. Other options are the formation of affinitive or co-ordinative bonds between ions and the boundary layer of a particle. In addition, various other reactive forces may cause adsorption reactions at the surface of the particle.

(c) grid imperfections

Grid imperfection-type of surface charges are mostly found for clay and clayey substances due to imperfections in the microscopic grid structure of a particle. By substitution of, for example, a silicium atom by a calcium atom (as presented in figure 3.2c) a negative charge is created at the surface.



a) Surface reactions

b) Ion adsorption

c) grid imperfections

Figure 3.2: Negative charge-causing processes

Particles in municipal wastewater are mainly of negative charge, which can be verified by applying an electrical field in a wastewater suspension. Due to the charge, the particles in the wastewater move to the positive electrode. Based on the field potential, the mobility of the particles in the field and the viscosity, the *zeta potential* can be calculated. The zeta potential is the measured negative potential at the hydrodynamic plane of shear between the particle surface and the surrounding water and is measured in conjunction with the movement of the particle in an electric field. For particles in wastewater this zeta potential was experimentally measured from -15 to -40 mV [Henze *et al.* (1995), Elmitwalli (2000)] (see figure 3.3).

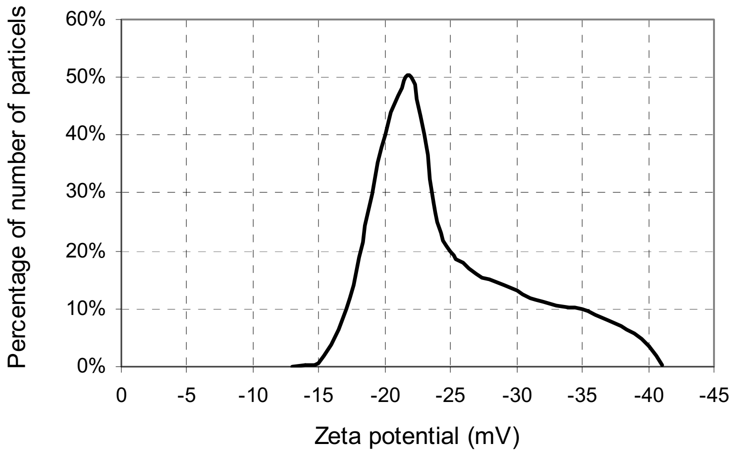


Figure 3.3: Typical zeta potential of particles in wastewater suspensions, redrawn after Henze *et al.* (1995)

Hydrophobicity and hydrophilicity

Particles and colloidals in wastewater can be hydrophilic or hydrophobic. Hydrophilic particles have a strong tendency to bind or adsorb water through the production of hydrogen bonds. The reason for the hydrophilic properties is the presence of water adsorbing groups, like amino (NH_2), hydroxyl (-OH) and acid groups (-COOH).

As stated before, the negative surface charge of organic particles is the result of dissociation of end groups on the surface, and/or adsorption of ions from the water. The first phenomenon usually causes the charge of hydrophilic colloids; the second phenomenon causes the charge of hydrophobic particles. It is assumed that positively charged detergent molecules on the surface of particles contribute to the surface charge of hydrophobic particles. The hydrophobic part of these macromolecules is attached to the surface, while the negatively charged tail is located in the water phase.

The result of the hydrogen bonds is a layer of “*bound water”* around a particle, as shown in figure 3.4, which follows the particles in their movement. The hydrophobic particles do not have this layer, causing repulsion of water molecules.

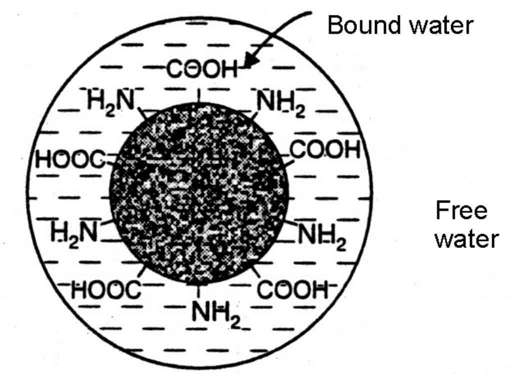


Figure 3.4: Schematic impression of *bound water* caused by hydrogen bonds [Henze *et al.* (1995)]

Electrical double layer

Due to the in general negative surface charge of particles in wastewater, oppositely (positively) charged ions are attracted. These counter ions accumulate in a layer at the surface of the particles, thereby partially neutralising the surface charge. The layer of counter ions is called the *Stern layer* (see figure 3.5). Due to the charge, an electrical potential difference (the zeta potential) prevails between the particle surface and the surrounding water. The hydrodynamic plane of shear is the interface between the proportion of the liquid moving with the particle and the other part of the liquid. For hydrophobic particles, the plane of shear is located close to the surface of the *Stern layer*. For hydrophilic particles, the plane of shear coincides with the outer limit of the "*bound water*".

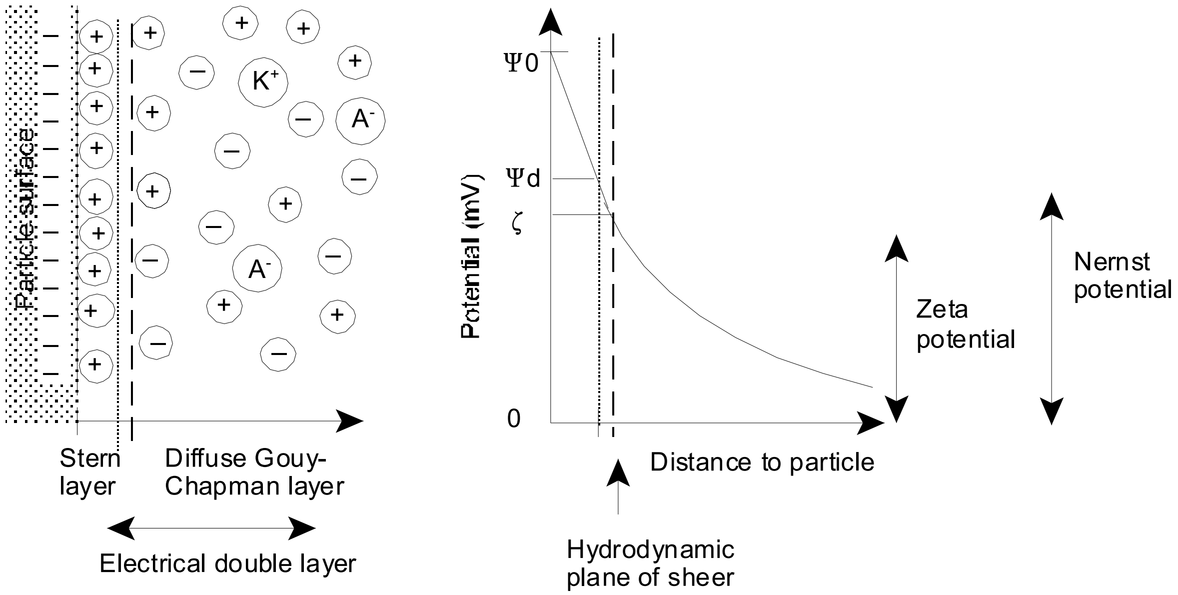


Figure 3.5: Schematic presentation of the *electrical double layer*: Stern layer and Gouy-Chapman layer [Fleer (1993)]

The *Stern layer* is surrounded by a diffuse layer of negative and positive ions. This diffuse layer is called the *Gouy-Chapman layer*. The concentration of counter ions is gradually reduced until it reaches the concentration in the liquid. The *Stern layer* and the diffuse *Gouy-Chapman layer* are referred to as the *electric double layer* [Knoppert and Heide, van der (1990), Fleer (1993)].

Apart from the repulsive electrical forces, colloidal particles also attract each other through the attracting London-Van der Waals' forces. These forces decrease rapidly with increasing particle distance. For particle attachment, the repulsive forces should be neutralised or be overruled, so that the attracting forces of the particles may take effect.

Derjaguin, Landau, Verwey and Overbeek quantified particle stability in terms of energy changes that occur during mutual approach of the particles. The total energy is determined by the sum of the attracting (mass force related) and the repulsive (surface force related) energies within the inter-particulate sphere. Figure 3.6 shows a schematic representation of the so-called DLVO theory and describes the energy pattern in relation to the distance to the particle surface [Knoppert and Heide, van der (1990), Fleer (1993), Schwoyer (1981)]. The curve illustrates the two main conclusions of the DLVO theory:

(a) attractive forces dominate on small and long distances of the particle surface;

(b) repulsive forces dominate in between these two extremes.

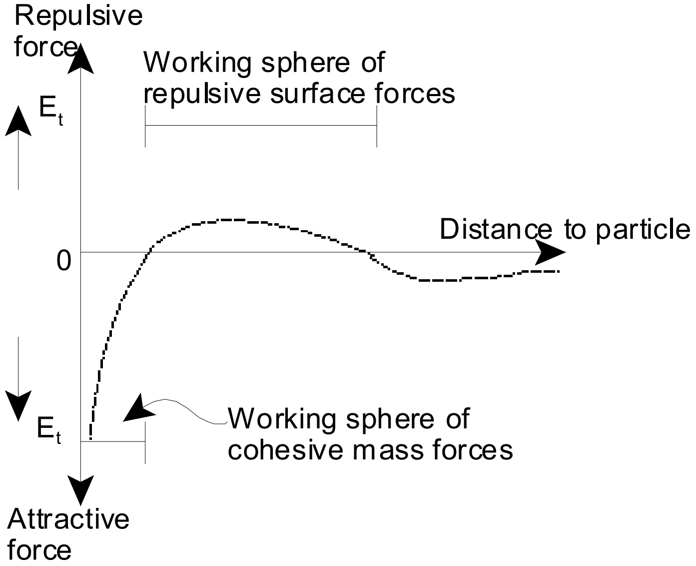


Figure 3.6: Schematic presentation of the Derjaguin, Landau, Verwey and Overbeek (DLVO) theory

3.3 Distribution of contaminants over particle size ranges in wastewater

A major part of pollutants in municipal wastewater is assumedly related to colloidal and suspended particles. In the literature, different size definitions are used for the description of particle fractions. Generally spoken, the separation between dissolved and particle-related fractions is defined somewhere in-between 0.001 and 1 µm. In the Netherlands, as well as internationally, the greater part of the data available on dissolved and particulate pollutants is determined by a separation diameter of 0.45 µm, since this distinction is applied with respect to the biological wastewater characterisation for the IAWQ ASM [Henze (1992), STOWA (1996a), STOWA (1996b), STOWA (1999)].

The paragraph below gives an overview of data from the literature regarding particle-related contaminants in municipal wastewater. The presented data originate from different countries with differences in climatical circumstances, landscape and consuming habits. Together with the characteristics of the applied sewer systems, these specific items cause differences in the wastewater composition and particle distribution in the water. The wastewater characteristics should not be generalised, but may only be interpreted as indications for a certain average wastewater composition.

Wastewater characteristics from literature
Table 3.1 gives an impression of the distribution of COD into a particle fraction and a dissolved fraction of six wastewaters in the Netherlands; $COD_{particles}$ is all COD related to particulate material bigger than 0.45 µm and the remaining COD is entitled as $COD_{soluble}$.

Table 3.1: COD related to particles (> 0.45 µm) and dissolved COD (< 0.45 µm) in wastewaters in the Netherlands calculated from data derived from STOWA (1999)

COD (mg O_2/l)	**wwtp De Bilt**	**wwtp Deventer (atypical)**	**wwtp Nijmegen**	**wwtp Walcheren**	**wwtp Franeker**	**wwtp Gouda**
$COD_{soluble}$	146 (32%)	125 (16%)	154 (29%)	81 (21%)	212 (35%)	120 (27%)
$COD_{particles}$	311 (68%)	674 (84%)	328 (71%)	311 (79%)	395 (65%)	482 (73%)

From table 3.1, it may be expected that about 65% to 80% of the COD present in wastewater in the Netherlands is related to particulate matter.

Similar characteristics were found in recent research on the fractionation of COD in France [Spérandio *et al*. (2000), Anderson *et al*. (2001)] . That research also applied the separation diameter of 0.45 µm to distinguish the soluble and particle-related fractions (see table 3.2).

Table 3.2: Influent characterisation of wastewaters in France; regarding to Spérandio *et al*. (2000), Anderson *et al*. (2001)

COD (mg O_2/l)	**wwtp A**	**wwtp B**	**wwtp C**	**wwtp D**	**wwtp E**	**wwtp F (atypical)**
$COD_{soluble}$	278 (31%)	155 (32%)	228 (39%)	202 (41%)	234 (41%)	461 (60%)
$COD_{particles}$	624 (69%)	329 (68%)	353 (61%)	286 (59%)	332 (59%)	312 (40%)

Although the conditions in France regarding sewers and food consumption are different from those in the Netherlands, the wastewater characteristics in France are quite comparable to the wastewater data in table 3.1. With 60% to 70% of the COD related to particles, the soluble COD fraction is slightly higher than in the wastewaters in the Netherlands.

The supposition that a considerable percentage of the pollutants in municipal wastewater is related to particles and colloids, is illustrated by the following detailed data from research projects in the USA conducted by Levine *et al.* [Levine *et al.* (1985), (1991a/b)]. Levine fractionated wastewater components into four major particle fractions. The applied distribution was divided into a settleable, suspended, dissolved colloidal and a soluble fraction as presented in table 3.3. These particle size distributions were created by means of sedimentation, centrifugal and filtration techniques.

From this survey, approximately 60% of the total COD is present in material larger than 1.0 µm and about 75% of the COD is related to particles and colloidals larger than 0.08 µm.

Table 3.3: Composition of organic material in municipal wastewater in the United States [Levine *et al.* (1985), (1991 a/b)]

parameters	**dissolved**	**dissolved colloidal**	**suspended colloidal**	**settleable**
particle size (µm)	*< 0.08*	*0.08 – 1.0*	*1.0 – 100*	*> 100*
COD (% of total)	25	15	26	34
fat	12	51	24	19
proteins	4	25	45	25
carbon-hydrates	58	7	11	24

Ødegaard [Ødegaard (1987)] quotes Munch [Munch *et al.* (1980)] who presents a wastewater composition with fraction diameters (based on filtration techniques) that differ slightly from Levine's data. From their research, it appears that 36% of the total phosphate-phosphorus present in wastewaters from the wwtp of Saskatoon (Canada) is related to material larger than 0.025 µm (table 3.4). The relatively cold, fresh and dilute wastewater shows a typical nitrogen distribution in wastewater, since 73% of the organic nitrogen is related to particulate matter. Whereas 77% of the kjeldahl-nitrogen is soluble since all ammonium-nitrogen was found to be in dissolved form.

Table 3.4: Distribution of pollutants in municipal wastewater based on Munch *et al.* (1980)

parameter	**dissolved**	**colloidal**	**supra colloidal**	**settleable**
particle size (µm)	*< 0.025*	*0.025 – 3.0*	*3.0 – 106*	*> 106*
BOD	17%	16%	46%	21%
COD	12%	15%	30%	43%
P_{total}	63%	3%	12%	22%
$N_{kjeldahl}$	77%	3%	8%	12%
$N_{organic}$	27%	15%	38%	20%

Table 3.5 shows a survey of organic matter distributions (mostly calculated as COD) in wastewaters related to particle size, as published in the literature. On average, about 30% of the organic material can be regarded as soluble within a fraction smaller than 0.001 µm and approximately 15% is colloidal between 0.001 and 1 µm. 25% of the organics can be related to suspended particles between 1 and 100 µm and 30% of the organic matter is incorporated in or adsorbed to particles bigger than 100 µm.

Table 3.5: Organic matter per particle size range in urban wastewaters, according to different references

< 0.001 µm	**0.001 – 1.0 µm**	**1.0 – 100 µm**	**> 100 µm**	
41%	16%	28%	15%	[Balamt (1957)]
31%	14%	24%	31%	[Heukelekiajn and Balmat (1959)]
38%	13%	19%	30%	[Painter and Viney (1959)]
29%	13%	31%	27%	[Walter (1961 a and b)]
29%	15%	22%	34%	//
25%	14%	27%	34%	[Hunter and Heukelekian (1961)]
18%	15%	25%	42%	//
25%	14%	27%	34%	[Hunter and Heukelekian (1965)]
23%	14%	23%	40%	//
30%	19%	10%	41%	//
50%	9%	18%	23%	[Rickert and Hunter (1967)]
47%	9%	19%	25%	//
40%	10%	21%	29%	[Rickert and Hunter (1970)]
48%	9%	19%	24%	[Helgott *et al.* (1970)]
12%	15%	30%	43%	[Munch *et al.* (1980)]

Schematic particle size distribution

To give an overview of the distribution of pollutants with regard to particle size the schematic graph in figure 3.7 was created, based on wastewater data from the literature [Ødegaard (1987), Levine (1991a)]. The lowest part of the graph shows different parameters in wastewater against particle size. In the top part of the graph a number of treatment techniques are pictured with the specific particle and removal range.

The graph shows that suspended particles in wastewater larger than approximately 1 μm consist mainly of human organic waste and organisms like bacteria, bacteria cell flocs, protozoa and algae. In the small colloidal fraction between 0.001 μm and 1 μm mainly viruses, long DNA strings and cell parts are present. The remaining pollutants are regarded as soluble contaminants and are not related to particulate matter.

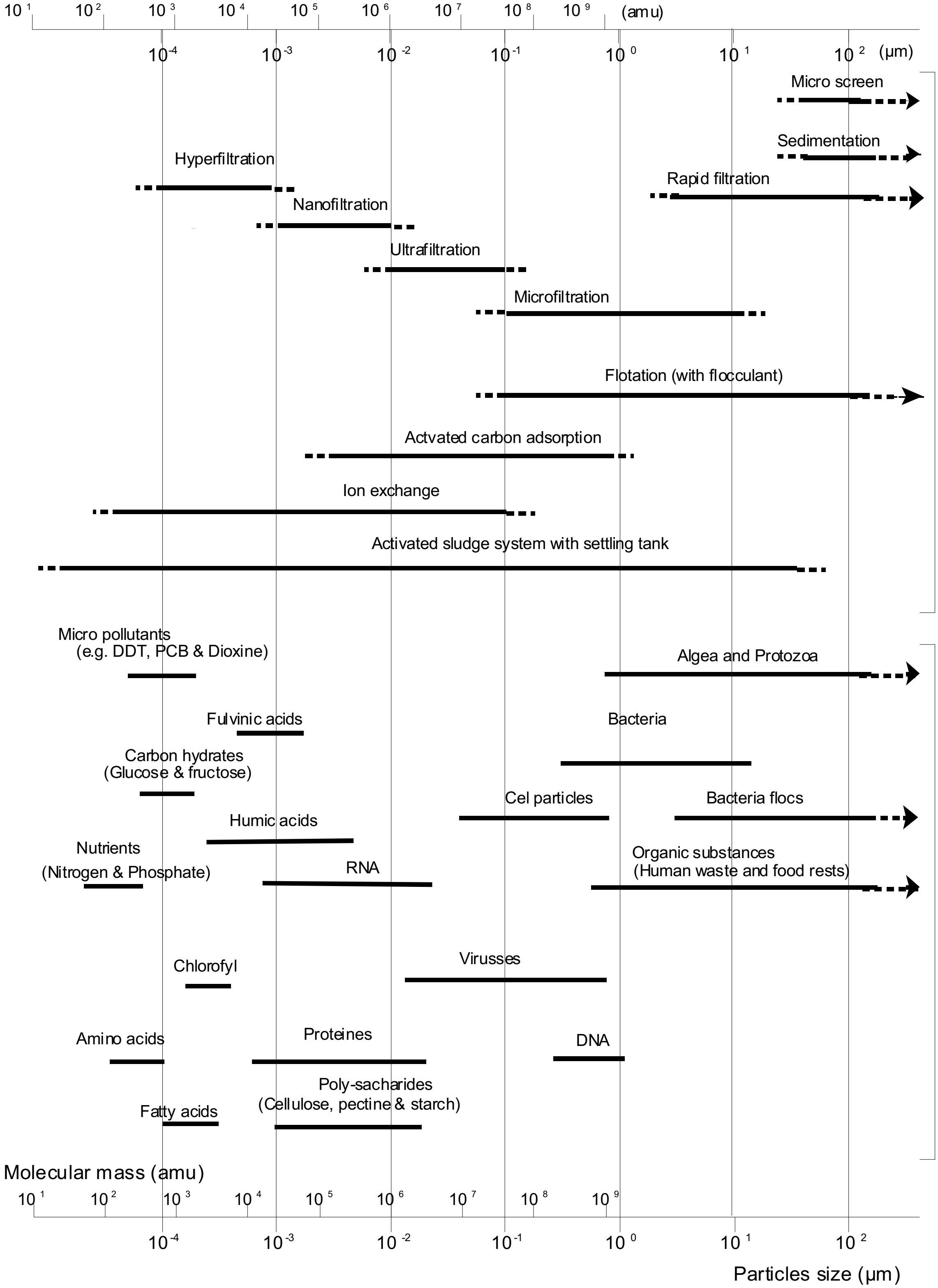


Figure 3.7: Particle size distribution of organic pollutants and particle removal treatment techniques per removal size

The upper part of the schematic figure underlines the importance of particle removal techniques, specifically for the particle size range between 100 µm and 1 µm.

The particle-related pollutants may be removed by the application of physical and physical-chemical treatment unit procedures, but the soluble contaminants are more difficult to treat with these kinds of techniques. The activated sludge system or related biological treatment techniques seem to be most adequate for the elimination of soluble pollutants. In combination with selected particle removal techniques, the whole pollutant range in wastewater can be treated.

3.4 Conclusions

Although the influence of local conditions and habits on the wastewater composition have to be considered and specific wastewater data should not be generalised, the figures and tables indicate that on average about 30% of the total COD is present in soluble form. 30% of the COD is related to colloidal and suspended particles and about 40% is equal to larger, settleable particulate matter. About the distribution of phosphorous and nitrogen on particles of different sizes is hardly any information available.

At the end, it is concluded that more information about wastewater fractionations and the characterisation of the particulate matter is needed to determine specific characteristics of an individual wastewater.

3.5 References

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CHAPTER 4 FRACTIONATION AND CHARACTERISATION OF PARTICULATE CONTAMINANTS IN MUNICIPAL WASTEWATER

SUMMARY

This chapter deals with the experimental research into wastewater fractionation on particle size. The main question in this investigation is how contaminants in wastewater are distributed over particle size. From this understanding, the effect of different particle removal techniques on the wastewater composition can be predicted and the requirements and efficiencies of the necessary post-treatment may be determined.

In the experimental research, eight wastewaters were fractionated into particle fractions using a sieve with a mesh width of 63 µm and membrane filters with different pore sizes (5, 1.2, 0.45 and 0.1 µm). The filtrates were analysed on the components COD, BOD, nitrogen, phosphorus, suspended solids, turbidity and conductivity.

An 'average' wastewater fractionation was statistically calculated. Within this wastewater composition the percentage of oxygen consuming components related to the theoretically determined settleable particle fraction is low, with a maximum of 21% for COD. A major part of BOD (44%), COD (38%) and phosphorous (35%) is present in suspended and supra-colloidal particle fractions with particle diameters between 1.2 and 63 µm. For nitrogen only 4% can be related to settleable particles, 12% to colloidal and suspended fractions; so 83% of the nitrogen is present in soluble form.

By means of the conducted experimental research, the effect and applicability of certain physical-chemical particle removal techniques and of a specific wastewater can be derived. In addition, energy and cost calculations show the importance of particle removal in the pre-treatment down to 1 µm to save energy and costs in the total wastewater treatment system.

The applied testing procedure was very successful, however it should be carried out on a broader scale and has to be optimised. So, it is advised to implement more advanced (online) sampling, separation, analysing and monitoring techniques.

4.1 Introduction

Since it is assumed that a major part of the COD and nutrients in wastewater are related to particulate material, advanced particle removal in a physical-chemical pre-treatment step, results in a lower pollutants load in the following treatment steps. So, the total treatment system can be designed and operated to be more energy efficient and less space consuming [Nieuwenhuijzen, van *et al.* (1998), Mels *et al.* (1999)]. Traditionally, the choice of treatment methods was mainly based on the effluent standards as well as on practical experience with the various techniques by engineering companies and waterboards. Especially the choice of adding pre-treatment steps, mainly primary sedimentation tank, preceding activated sludge systems was not, in general, based on technical knowledge and wastewater characteristics, but was made according to traditional practice and on plant size.

In the preliminary evaluation study, as described in chapter 2, it was identified that a more detailed characterisation of the particulate matter in wastewater is necessary for a better understanding and predicting of removal efficiencies of physical-chemical treatment techniques and the application of optimal chemical dosages. This characterisation should be done on the distribution of contaminants on particle size. Intensive wastewater characterisations have been conducted world-wide to determine biological fractions for activated sludge models (ASM), but few data are available about the particle size distribution of wastewater and the related pollutants.

Thus, a further research was conducted to derive data on the distribution of contaminants on particulate matter in wastewaters in the Netherlands.

The main question in this experimental research phase is how contaminants in wastewater are distributed over particles. From this understanding, the effect of different particle removal techniques on the wastewater composition can be predicted and the requirements and efficiencies of the necessary post-treatment can be determined.

Calculations were made to identify the effect of advanced particle removal in the first treatment step on the biological post-treatment and total wastewater treatment system. The environmental criteria evaluation model DEMAS$^+$ was used to calculate energy and cost potentials of increasing particle removal.

Finally, the wastewaters from different wwtp were compared by means of the wastewater fractionation. Results indicate whether a certain wastewater is efficiently treatable with physical-chemical pre-treatment methods and to what extent.

4.2 Experimental set-up

To find an answer to how the contaminants in wastewater are distributed over particles, experimental research was carried out wherein eight wastewaters were fractionated into particle fractions and analysed on the components COD, BOD, nitrogen, phosphorus, suspended solids, turbidity and conductivity.

Sampling

For each fractionation and characterisation experiment, two grab samples were taken from the wastewater of eight wwtp (Apeldoorn, Arnhem, Bennekom, Berkel, Boxtel, Haarlem, Hoek van Holland and Vlaardingen) specifically at the point behind the coarse grid removal (coarse screen and/or non-aerated sand trap). The sampling location was selected to be just in front of the first treatment facility to determine the wastewater composition directly to be pre-treated since these are the desirable data to be collected. On three different days sampling took place at the same location at the same time (9.00 a.m.) under preferably the same weather conditions (dry weather flow). So, in total (2×8×3 =) 48 samples were taken.

In the first instance, grab sampling was preferred over flow proportional long term sampling since processes like adsorption, degradation or destruction could cause changes in the wastewater composition regarding to particle size distributions. On the other hand, it was recognised beforehand that the single grab samples would only give an instantaneous picture of the total wastewater composition over the day.

The samples were stored in polyethylene containers of 3 to 5 litres and were immediately analysed for temperature and pH. The samples were directly sent to the laboratory for fractionation and further analysis. In case of delay during transport and after reception at the laboratory, the samples were kept under temperature control (at 4 ºC) to prevent changes in the wastewater composition.

Fractionation procedure

At the Laboratory of Sanitary Engineering at the Delft University of Technology, the samples were fractionated over clean, pre-flushed sieves (stainless steel) and membrane filters (cellulose nitrate composite, fabricated by Sartorius) with five different pore sizes (63, 5, 1.2, 0.45 and 0.1 µm) as shown in figure 4.1.

The filtrates of these filtration steps were analysed on water quality parameters like: COD, BOD_5, nitrogen, phosphorus, suspended solids, turbidity, conductivity and biodegradability via extensive BOD analyses on day 1, 2, 3, 5 and 10 ($BOD_{1,2,3,5,10}$). The produced fractions with particle size range and analysed parameters are summarised and summarised in table 4.1.

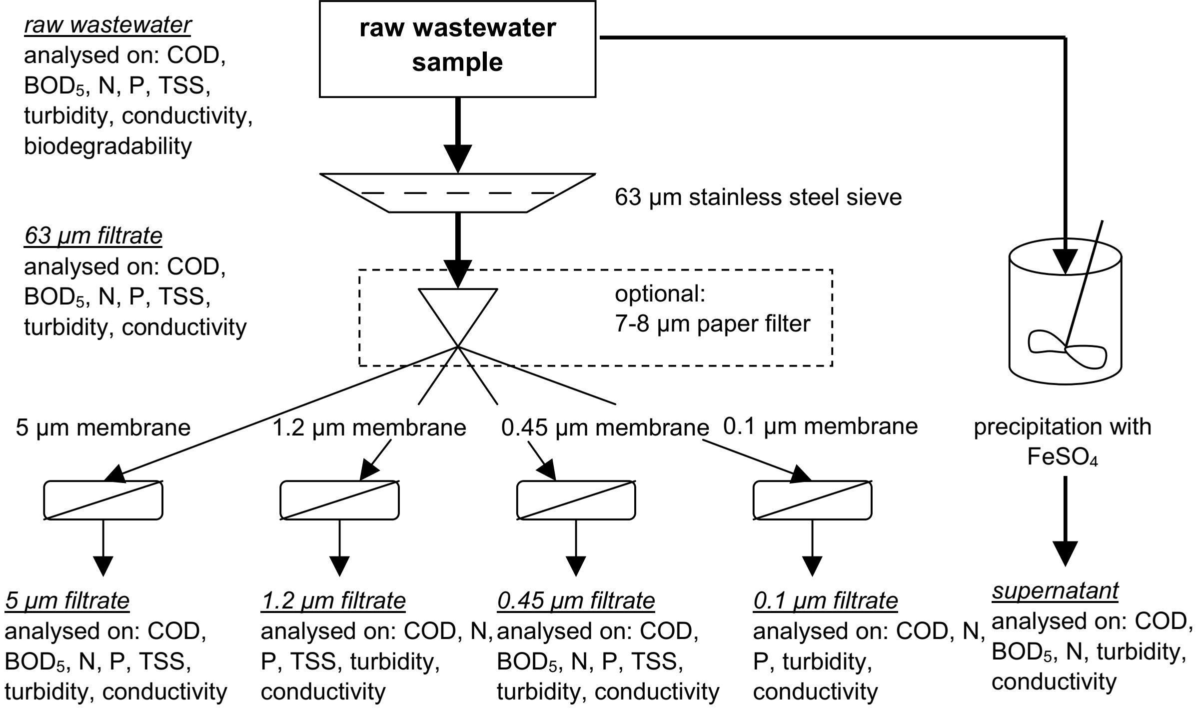

Figure 4.1: Schematic presentation of the fractionation procedure

Table 4.1: Fractions, particle sizes and analysed parameters

fraction	particle size range	analysed parameters
raw wastewater	Whole range	COD, BOD_5, $BOD_{1,2,3,5,10}$, N, P, TSS, turbidity, T, pH, conductivity
settleable-suspended	> 63 µm	COD, BOD_5, N, P, TSS, turbidity, T, pH, conductivity
suspended	5 – 63 µm	COD, BOD_5, N, P, TSS, turbidity, T, pH, conductivity
supra colloidal	1.2 – 5 µm	COD, N, P, TSS, turbidity, T, pH, conductivity
colloidal	0.45 – 1.2 µm	COD, N, P, turbidity, T, pH, conductivity
semi-dissolved	< 0.45 µm	COD, BOD_5, N, P, turbidity, T, pH, conductivity
dissolved	< 0.1 µm	COD, N, P, T, pH, conductivity
dissolved*	supernatant*	COD, BOD_5, N, turbidity, T, pH, conductivity

* Fraction created by sampling supernatant after precipitating raw wastewater with 20 mg Fe^{3+}/l iron-sulphate

The fractions were chosen for different reasons. The *raw wastewater* fraction (figure 4.1: top) is the wastewater influent (containing all waste fractions) that was being analysed extensively to characterise the type of wastewater. The *settleable-suspended* fraction can theoretically be related to the particle fraction that can be removed by primary sedimentation without chemical addition at surface loadings of approximately 3 $m^3/m^2 \cdot h$. In practice, also finer particles than 63 µm can be removed in a primary sedimentation tank at this average surface loading since adsorption and bio-sorption processes may cause the capture of fines and small flocs in larger floc conglomerates. The *suspended* and *supra colloidal* fractions are related to the particle fractions, which can be removed by chemical enhanced primary sedimentation and flotation [Mels *et al.* (2000)]. The *colloidal* fraction can hardly be removed by any conventional pre-treatment technique, but can be eliminated by applying membrane filtration [Nieuwenhuijzen, van *et al.* (2000)].

The differences between the *semi-dissolved* and *dissolved* fraction are chosen for purely academic reasons. The 0.45 μm filters are widely used for characterising the dissolved fractions in the IAWQ activated sludge models (ASM1, 2(d) and 3). Some researchers [STOWA (1999), (2000)] doubt the applicability of this pore size to characterise the soluble part and suggest 0.1 μm as the separation pore size between the colloidal-suspended and the dissolved fraction. To check this assumption the 0.45 or 0.1 μm membrane filter pore sizes were both used in the particle characterisation study. In addition, the supernatant of precipitating the raw wastewater with iron-sulphate (20 mg Fe^{3+}/l, as iron-sulphate to prevent a possible addition of organic material) was used as a third *dissolved* fraction.

Membranes and filtrates

Four membrane filters (see table 4.2) were tested on their leaching potential regarding to COD and nitrogen. Neither COD nor nitrogen could be detected in the leaching tests. To be sure that the produced filtrates would not be polluted by the filter material, each filter was pre-flushed with 0.5 l of de-mineralised water. On average, every filtration step could produce 250 ml of filtrate before the filters would clog. To use the 0.1 μm membrane filter the sieved wastewater had to be pre-treated over a 6 - 8 μm paper filter (Schleicher & Schuell) to prevent fast clogging. By precipitation (with iron sulphate), large volumes of supernatant could be produced to gain a soluble fraction, but the procedure had to be very strict and the optimal sedimentation time took 15 minutes.

Table 4.2: Applied Sartorius cellulose nitrate membranes

nominal pore diameter	type number
5 μm	11342-047N
1.2 μm	11303-047N
0.45 μm	11306-047N
0.1 μm	11358-047N

Analyses

The COD analyses were conducted with a standardised Merck test no. 1.14541 in conformance with ISO 6060. Total Nitrogen (N_{total}) was analysed according to the ISO 11905-1 standard with the Merck test no. 1.14537 and P_{total} with the Merck test no. 1.4729 (ISO 6978-1).

The BOD analyses that were conducted in the Laboratory of Sanitary Engineering in Delft as BOD_5^{20} conform to the NEN standard 3235-5. As inoculation material, effluent of the specific wwtp was used instead of raw wastewater to guarantee a sufficient adapted bacterium population for BOD removal. Besides the BOD_5 analyses of the different filtrates, an extensive BOD analysis was conducted for the raw wastewater to determine the biodegradability of the wastewater. The BOD were determined on day 1, 2, 3, 4, 5 and 10; from these data a BOD curve could be created. By the application of the least-square-method, the degradation parameter k_{BOD} of the raw wastewaters was calculated.

Turbidity was tested as nuclei turbidity unit (NTU) by using the 2100N Turbidimeter from Hach. Suspended solids were measured by vacuum filtration of a paper filter (Schleicher&Schuell) with an average pore diameter of 5 µm. Conductivity of the filtrates was measured in mS/cm by the Metrohm 660 conductometer.

4.3 Results and discussion

Biodegradability of the raw wastewaters

As earlier has been described by different research groups [STOWA (1999), (2000)], the degradation parameter k_{BOD} may not only vary per wastewater of a specific treatment plant but also for one and the same wastewater.

From the analytically derived k_{BOD} values as presented in table 4.3, the biodegradation parameter fluctuates between 0.3 and 0.74 day^{-1}. In correspondence with earlier STOWA research [STOWA (2000)], the value of 0.23 day^{-1}, as used in the IAWQ models, seems incorrect to use as calculation parameter for determining BOD_5 in design and modelling calculations for activated sludge systems, based on the higher values in table 4.3.

Table 4.3: Biodegradation parameters (k_{BOD} value [measurement 1 and 2] in day^{-1}) calculated with the least-square-method from BOD time series on day 1, 2, 3, 5 and 10

wwtp	Apeldoorn	Arnhem	Boxtel	Haarlem	Hoek van Holland	Vlaardingen
$k_{BOD}1$ (d^{-1})	0.67	0.65	0.39	0.3	0.48	-^{2}
$k_{BOD}2$ (d^{-1})	0.74	0.53	0.44	0.42	-^{1}	0.43

^{1} unrealistic value; ^{2} unreliable measurement

Table 4.3 illustrates the limitations of BOD_5 and the determination of the k_{BOD} parameter. Preferably, the unreliable BOD analyses are replaced by more accurate COD tests, that is by nitrogen uptake rate (NUR) and oxygen uptake rate (OUR) tests [Kujawa-Roeleveld (2000)]. Since the BOD analysis is still the most common used analysing technique, it is advised to conduct extensive BOD measurements during a longer period to determine the biodegradability of the wastewater by testing BOD on day 1, 2, 3, 5 and 10 (and optional day 7).

Average wastewater fractions

From the fractionation data from the eight sampled wastewater treatment plants an ‘average’ fractionated wastewater composition was determined, as shown in table 4.4. In total, 44 data sets per fraction are used for this calculation. From the original 48 samples, four samples from the wwtp of Hoek van Holland had to be eliminated due to specific calamities that influenced the wastewater composition. The average results should not be used as generally valid, since local conditions may influence the wastewater composition and particle distribution. To determine the actual particle distribution for a specific wastewater, the influent should be investigated in detail on site.

Table 4.4: Average fractionated wastewater composition (with standard deviation in brackets), calculated from data-analysis of experimental derived influent fractionations

fraction **parameter**	**dissolved** (< 0.1 μm)	**supra dissolved** (0.1–0.45μm)	**colloidal** (0.45-1.2μm)	**supra colloidal** (1.2 – 5 μm)	**suspended** (5 – 63 μm)	***settleable*** (> 63 μm)	**raw influent**
TSS (mg/l)	n.d.	n.d.	n.d.	n.d.	68	62	130
turbidity (NTU)	n.d.	n.d.	8	12	72	24	116
BOD$_5$ (mg O$_2$/l)	82	n.t.	n.t.	24	51	14	170
COD (mg O$_2$/l)	166	14	9	51	124	97	461
N$_{total}$ (mg N/l)	36.3	0.4	1.3	1.7	2.2	1.7	43.7
P$_{total}$ (mg P/l)	4.08	0.23	0.23	0.39	2.31	0.46	7.7
TSS	-	-	-	-	52 (±18) %	48 (±18) %	
turbidity	-	-	7 (±4) %	10 (±7) %	62 (±17) %	21 (±12) %	
BOD$_5$	48 (±12) %	-	-	14 (±6) %	30 (±8) %	8 (±4) %	
COD	36 (±10) %	3 (±4) %	2 (±2) %	11 (± 6) %	27 (±11) %	21 (± 9) %	
N$_{total}$	83 (±25) %	1 (±1) %	3 (±3) %	4 (±2) %	5 (±3) %	4 (±4) %	
P$_{total}$	53 (±18) %	3 (±3) %	3 (±1) %	5 (±2) %	30 (±12) %	6 (±3) %	

n.d. = not detectable; n.t. = not tested

In this ‘average’ wastewater fractionation, the percentage of oxygen consuming components related to the settleable particle fraction is low, with a maximum of 21% for COD. A major part of the total BOD, COD and phosphorous is present in suspended and supra-colloidal particle fractions with particle diameters between 1.2 and 63 μm, namely 44%, 38% and 35%, respectively. For nitrogen, only 4% can be related to settleable particles and 13% to colloidal and suspended fractions; so 83% of the nitrogen is present in the soluble form. The conductivity of the wastewater (not shown) decreased minorly with increasing particle removal.

The fractionation in percentages is graphically presented in figure 4.2. Figure 4.3 shows the water quality components related to particle size (average distribution = solid line; 90-percentile = dotted line).

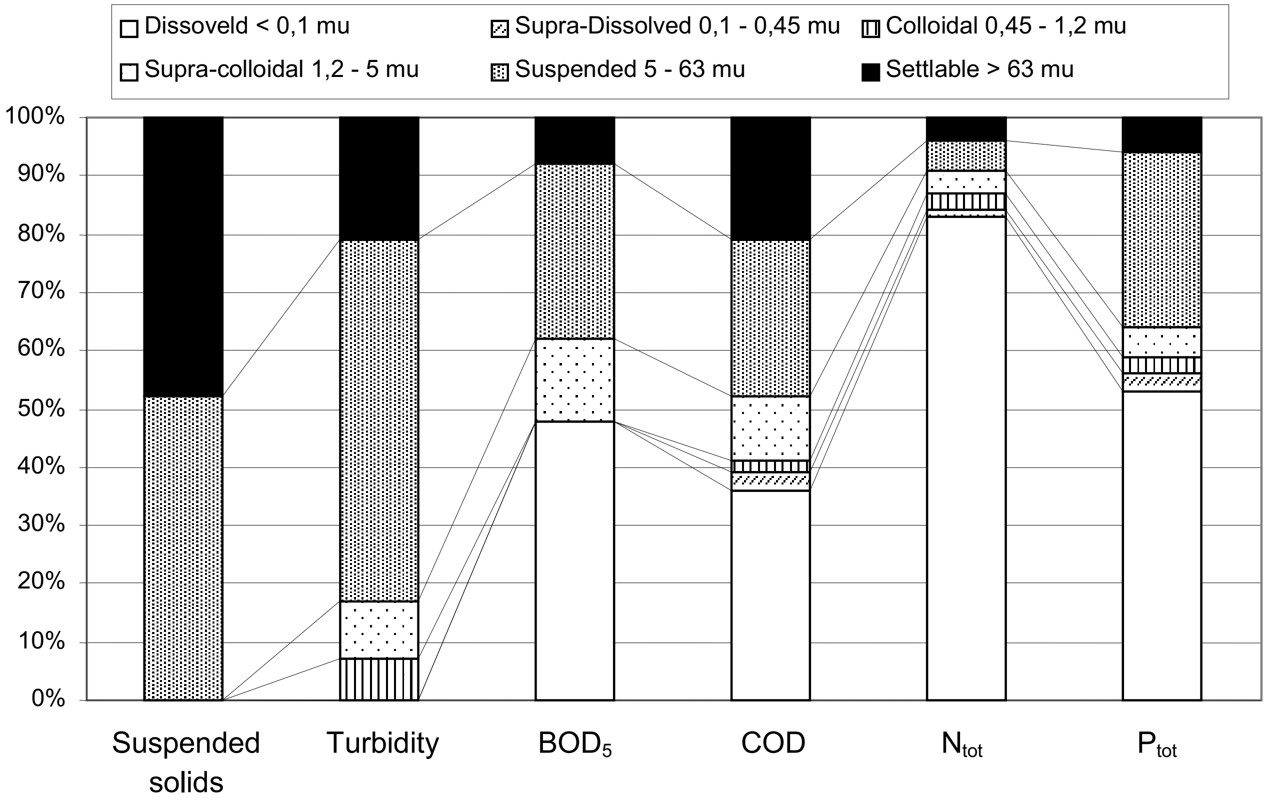

Figure 4.2: Average wastewater fractionation presented in percentages per component

Per sampled wastewater treatment plant, the fractionated and characterised wastewaters are displayed in Appendix F.

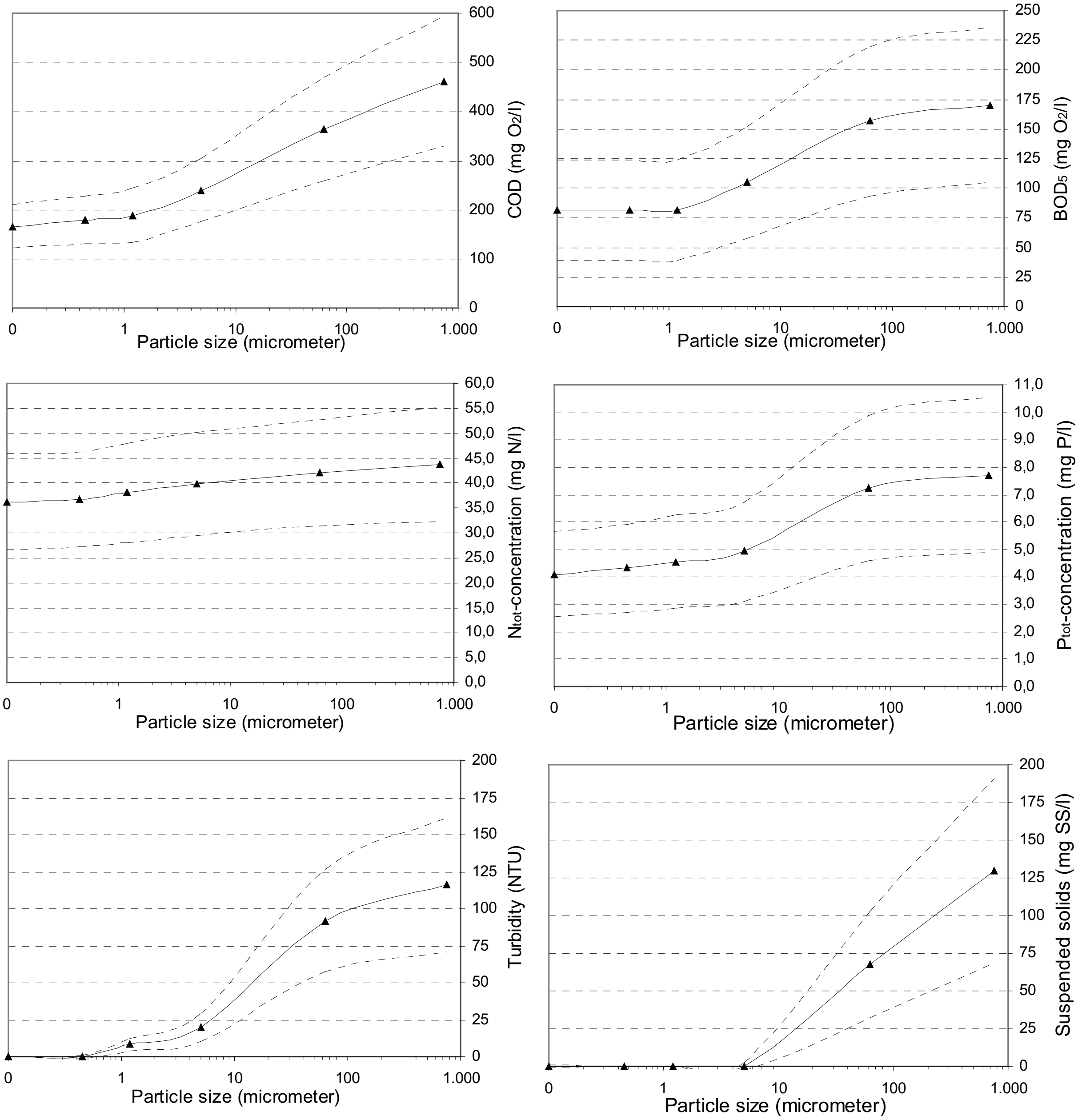

Figure 4.3: Wastewater quality parameters related to particle size

BOD/N ratio per fraction

When the particle removal increases, the BOD/N ratio is declining since BOD is more directly related to particles than nitrogen. The average BOD/N ratios for the average wastewater, the dry weather flow and the storm weather flow are presented in table 4.5.

The average BOD/N ratio decreases from 4.1 in the raw wastewater to 3.6 due to the removal of settable particles down to 63 μm. By total particle removal, the BOD/N ratio decreases to 2.4.

In diluted wastewater (by rain events) the BOD/N-ratio is already low (2.4) in the influent and decreases to below 1 after total particle removal. This will negatively influence the denitrification potential of the remaining wastewater. In more detail, Mels (2001) and Kujawa-Roeleveld (2000) have investigated this problem and made suggestions of solving it.

Table 4.5: Statistically calculated average BOD/N-ratios per fraction for dry weather flow (dwf) and storm weather flow (swf)

filtrates parameter	only dissolved (< 0.1 µm)	after colloidal material removal (< 0.45 µm)	after removal of settable particle fraction (< 63 µm)	raw wastewater
BOD/N$_{average}$	2.4 (±1.34)	2.3 (±1.49)	3.6 (±1.91)	4.1 (±1.80)
BOD/N$_{dwf}$	2.9 (±0.95)	2.9 (±0.91)	4.2 (±1.09)	4.7 (±1.20)
BOD/N$_{swf}$	0.9 (±0.58)	0.8 (±0.59)	2.0 (±0.61)	2.4 (±0.44)

Differences between dissolved fractions

Since wastewater is characterised in the ASM-calculations there are disputes about the proper membrane pore size to fractionate the '*dissolved*' wastewater parameter. In the fractionation and particle characterisation experiments three different 'dissolved' fractions were created to compare the fractions and fractionation method. Next to the 0.45 µm membrane filtration, 0.1 µm membrane filtration and precipitation with iron sulphate was used to produce the 'dissolved' wastewater fraction. Table 4.6 shows the results of the statistical analysis of the filtrates and the supernatant.

From the analysis of 132 data on COD in the different 'dissolved' fractions, it can be concluded that in average, the 0.1 µm membrane filtration produces a similar 'dissolved' wastewater as the precipitation does. The 0.45 µm membrane filtration produces a 'dissolved' wastewater that overestimates the 'real dissolved' COD-fraction of the 0.1 µm membrane and the precipitation by 10%, since small colloidal particles are passing the 0.45 µm membranes.

Table 4.6: Difference between measured COD of the three different 'dissolved' fractions: supernatant from precipitation with iron sulphate and the filtrates of the 0.1µm- and the 0.45 µm-membranes, calculations based on 138 experimental data series.

	difference between measured COD in supernatant and 0.1 µm filtrate	difference between measured COD in supernatant and 0.45 µm filtrate	difference between measured COD in 0.45 µm and 0.1 µm filtrate
average	**-1%**	**-9%**	**8%**
standard deviation	8%	8%	5%
maximum	14%	5%	17%
minimum	-18%	-35%	0%
median	0%	-8%	7%

Especially for the ASM-modelling, where the dissolved parameters have great impacts on the calculations, a proper definition of the dissolved fraction should be made. The experiments lead to the conclusion that the 0.45 μm separation technique should be changed to the more correct fractionation technique of 0.1 μm membrane filtration. Despite the more time consuming analyses, it is advised to apply the 0.1 μm membrane to determine the soluble part in wastewater characterisation and fractionation studies.

The supernatant production by precipitation can easily be confounded by mistakes in procedure or analysis, since the optimal dosage is very important to obtain an excellent particle removal. Added chemicals during precipitation can easily pollute the supernatant and the supernatant recovery should be conducted carefully. Apart from this, the advantage of membrane filtration above precipitation is the guaranty of the specified particle size removal, whereas the supernatant may contain particles. The production of the soluble fraction in large volumes is preferably conducted by the process of precipitation and sedimentation, despite its limitations.

4.4 Evaluation and discussion

Comparison of wastewaters from the Netherlands and Scandinavia

It is stated that contaminants in wastewaters in the Netherlands are in a larger percentage in the dissolved form compared to wastewaters from colder and mountainous regions like for example Norway or Alpine countries. This is caused by the low slopes of the sewer systems in flat countries like the Netherlands, resulting in long retention times [Ødegaard (1998), (2000)]. In oxygen enriched wastewater in steep-slope sewers oxidation of organic matter may occur; whereas (anaerobic) pressure driven sewer pipes (as are frequently used in the Netherlands) can contribute to a higher soluble fraction in wastewater since hydrolysis of components may occur.

The supposed differences in wastewater composition caused by climatical and local conditions are verified by comparing wastewater characteristics. The relation between the total concentration of COD and the particle-related COD is therefore plotted in figure 4.4 for wastewaters from the Netherlands and Scandinavian countries.

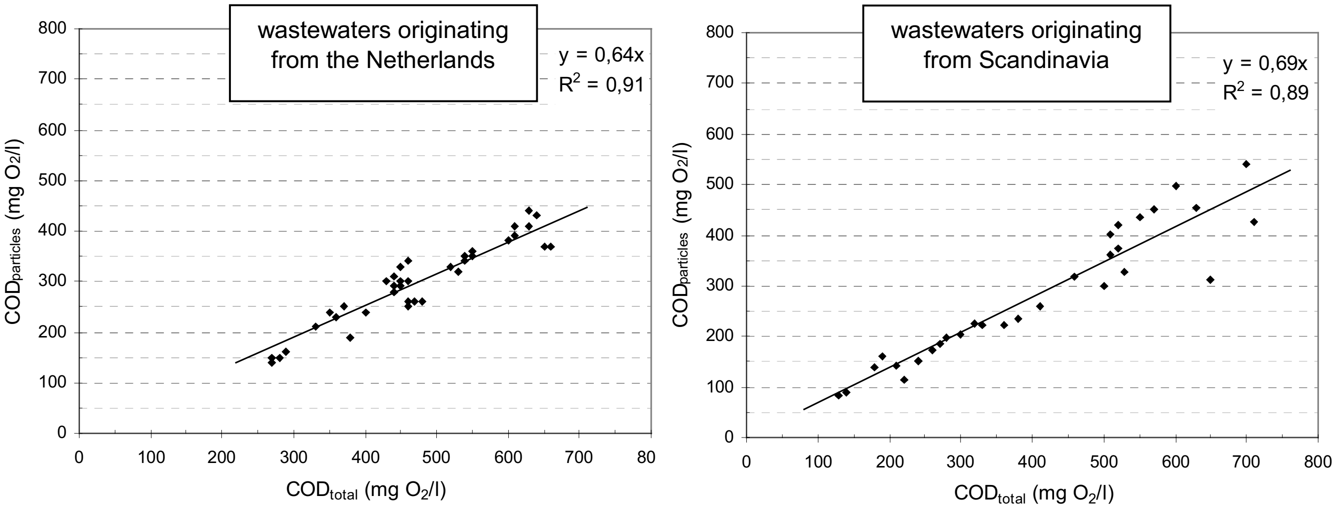

Figure 4.4: $COD_{total}/COD_{particles}$ relationship in wastewaters in the Netherlands (left) and Scandinavia (right, based on [Ødegaard (1999), (2000)])

The left graphic in figure 4.4 presents the relation between COD_{total} and $COD_{particles}$ in the wastewaters as fractionated and characterised in the preceding experiments. The diagram on the right side of figure 4.4 shows the same ratio in wastewater from Scandinavian countries, created with influent data from Norway, Sweden and Finland as published by Ødegaard [Ødegaard (1999), (2000)]. As can be deduced from the graphics, the average $COD_{particle}$ percentage of wastewaters from the Netherlands is 64% and for the wastewaters in Scandinavia 69%. Therefore, it can be concluded that the differences for COD seem to be limited.

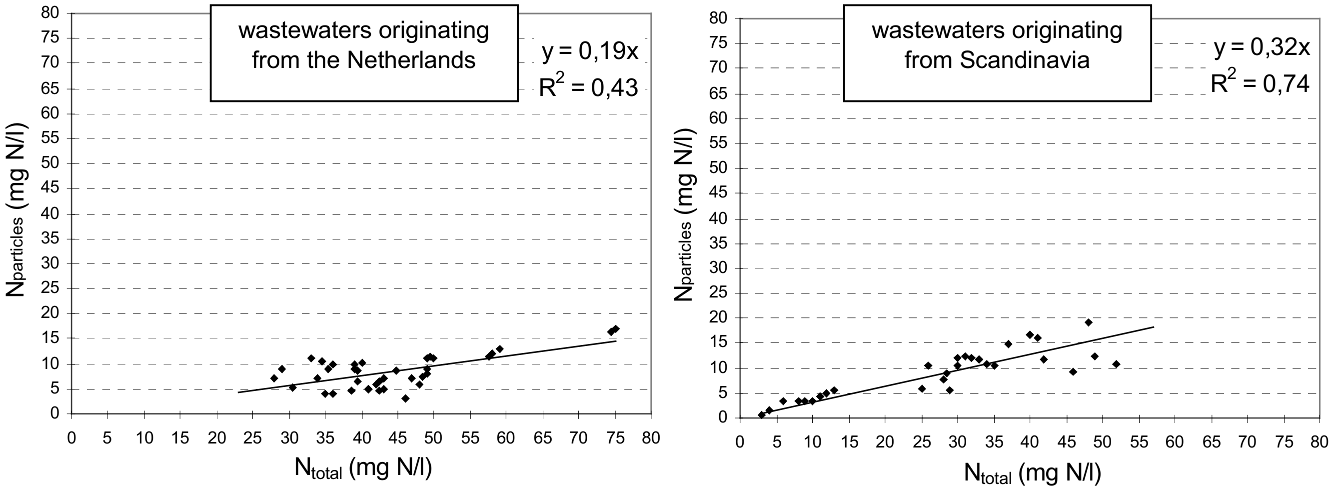

Figure 4.5: $N_{total}/N_{particles}$ ratio in wastewaters in the Netherlands (left) and in Scandinavia (right, based on [Ødegaard (1999)])

For nitrogen, the wastewaters are not similar. In wastewaters in the Netherlands approximately 19% of the total nitrogen is related to particles (see left diagram in figure 4.5), whereas Ødegaard (1999) states that the suspended fraction of total N in wastewaters from Scandinavia varies between 0.2 and 0.5, but is in most cases around 30% (the trend line in the right diagram in figure 4.5 even determines an average $N_{particle}$ percentage of 32%).

Comparison of wastewater fractionations per wwtp

Advanced particle removal affects treatment processes downstream of the pre-treatment. Wastewater fractionation studies based on particle size distributions can show the usefulness of a certain treatment technique and can provide knowledge about the capacity of pre-treatment of the specific type of wastewater. The fractionation data may predict the ability of a certain wastewater and the efficiency of a considered physical-chemical particle removal technique. For both COD and nitrogen, the fractionation has to be carried out to determine the energy and cost effects of advanced particle removal. By comparing the possible particle-related removal efficiencies for COD and nitrogen per wwtp statements may be made whether a particle removal is useful and which pre-treatment technique should be applied.

In this example case the maximum potential removal efficiency for COD and nitrogen down to the particle size of 0.1 µm are being calculated for described in the following formulas.

$$\pi\ \mathrm{COD}_{\text{fraction}} = \frac{(\mathrm{COD}_{\text{influent}} - \mathrm{COD}_{1\ \mu\text{m filtrate}})}{\mathrm{COD}_{\text{influent}}} \cdot 100\% \qquad (4.1)$$

$$\pi\ \mathrm{N}_{\text{fraction}} = \frac{(\mathrm{N}_{\text{influent}} - \mathrm{N}_{1\ \mu\text{m filtrate}})}{\mathrm{N}_{\text{influent}}} \cdot 100\% \qquad (4.2)$$

where:

$\pi\ \mathrm{COD}_{\text{fraction}}$ = potential removal efficiency for COD over the fraction

$\mathrm{COD}_{\text{influent}}$ = COD in the raw wastewater

$\mathrm{COD}_{1\ \mu\text{m filtrate}}$ = COD in the 0.1 µm filtrate

$\pi\ \mathrm{N}_{\text{fraction}}$ = potential removal efficiency for total nitrogen over the fraction

$\mathrm{N}_{\text{influent}}$ = total nitrogen in the raw wastewater

$\mathrm{N}_{1\ \mu\text{m filtrate}}$ = total nitrogen in the 0.1 µm filtrate

As an example, the efficiencies for the wastewater of wwtp Apeldoorn (see appendix F) are conducted. In the raw wastewater of wwtp Apeldoorn 470 mg O_2/l of COD was found and 34 mg per litre total nitrogen was measured. In the filtrates of the 1 µm membrane filter 220 mg O_2/l of COD and 28 mg N/l remained. The calculation, corrected for the height of the influent concentrations, is as follows:

π COD = [(470 - 220) / 470] x 100% = 53%

π N = [(34 - 28) / 34] x 100% = 18%

Table 4.7 gives the pre-treatment potentials of the sampled wwtp's with an increasing maximum potential removal efficiency.

The maximum possible COD removal due to particle removal is present in the wastewater of wwtp Berkel with 67%. The possible nitrogen removal efficiency for wwtp Vlaardingen is the highest of all wwtp with 23%.

Table 4.7: Calculated maximum possible removal efficiencies through advanced particle removal in the pre-treatment for COD and nitrogen per wwtp

wwtp	**COD**	**nitrogen**
Hoek van Holland	not determinable	not determinable
Arnhem	58%	7%
Bennekom	52%	17%
Boxtel	58%	13%
Apeldoorn	53%	18%
Haarlem	62%	10%
Berkel	67%	14%
Vlaardingen	61%	23%

Specific limits for the pre-treatment potential are assumed to be set by local conditions since environmental criteria and costs will determine the applicability and efficiency of a pre-treatment. To determine whether there is a certain π below which particle removal in the pre-treatment is not efficient anymore, a theoretical approach could be followed.

Energy calculations based on wastewater fractionations

Calculating the potential energy consumption of the remaining wastewater after removal of certain particle fractions can show the effect of advanced particle removal on the energy efficiencies of a wwtp.

First the energy consumption in a theoretical activated sludge system (load from 100,000 p.e.) with COD- and N-removal was calculated based on the wastewater fractionation (figure 4.6: graph bottom left).

With the evaluation model DEMAS$^+$, the total energy balance of the water treatment and sludge handling system for 100.000 p.e. and a flow of 20,000 m^3/day was calculated, resulting in the graph right below in figure 4.5. Since nitrogen will hardly be removed by particle removal and COD is strongly related to particles, the graph left below looks like the graph top left in figure 4.6. The total energy balance (bottom right) shows a steeper decline in energy consumption with more advanced particle removal, due to the fact that sludge production and handling depend strongly on particles and can possible lead to a positive energy input. The highest energy saving is possible by the removal of the settable fraction down to 63 µm (34% off the total energy savings) and the suspended particle fraction between 63 µm and 5 µm (46% of the total energy savings). Due to the removal of the remaining colloidal particles between 5 and 0.1 µm again 20% of energy can be saved.

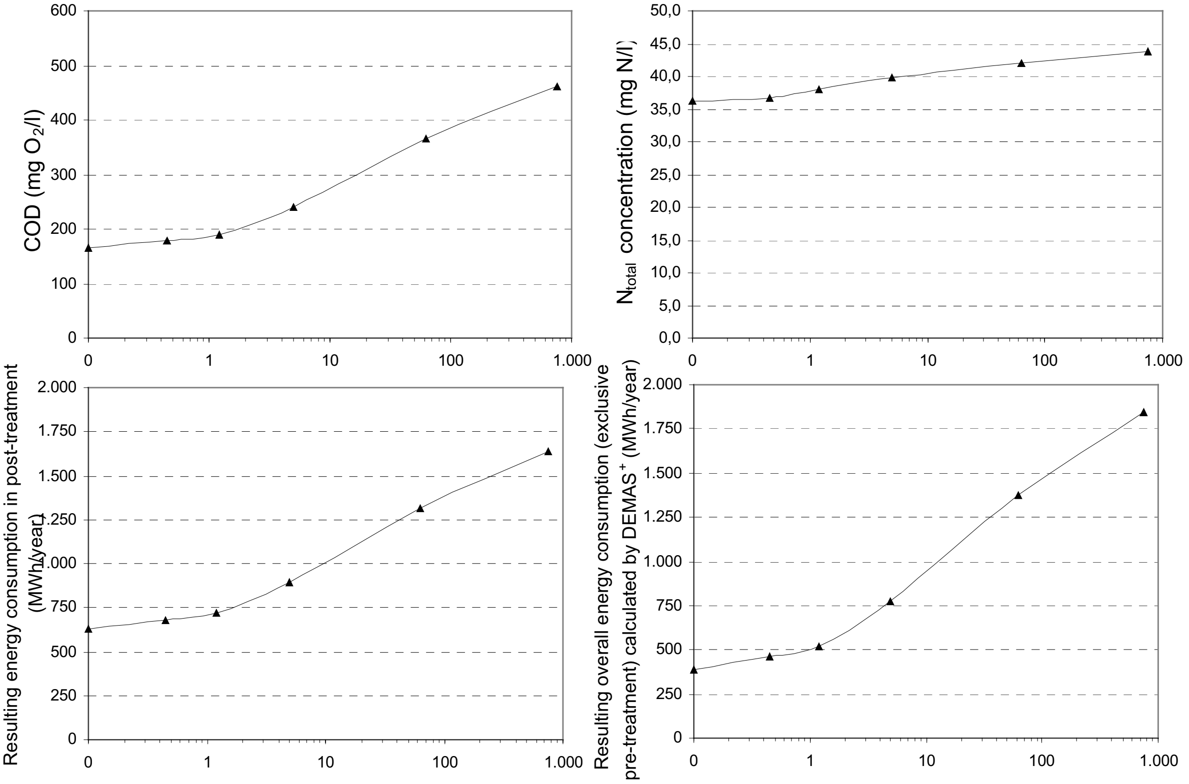

Figure 4.6: Energy consumption in the post-treatment (bottom left) and the total water and sludge treatment (bottom right), based on a 100,000 p.e. wwtp with a daily average flow of 20,000 m^3/day (particle size in µm on x-axis)

Table 4.8 shows the possible energy saving potential for an activated sludge plant per particle size fraction. If all particle fractions down to 0.1 µm are removed, 1,106 MWh per year of energy is saved (out of 1,843 MWh per year). Primary sedimentation removes particles down to 63 µm, which results in energy savings of 364 MWh/y. Due to an additional removal of suspended particles down to 1.2 µm after primary sedimentation again 644 MWh/y can be saved. On the other hand, the removal of the colloidal particle fraction between 1.2 and 0.45 µm will only lead to an additional energy saving of 28 MWh/y.

Table 4.8: Calculated overall energy savings (exclusive pre-treatment step) by fractional particle removal (MWh/y) at a 100,000 p.e. reference activated sludge system [read from particle size in column to particle size in row]

particle diameter	Raw	63 µm	5 µm	1.2 µm	0.45 µm	0.1 µm	
Raw wastewater	-	**364**	822	1,008	1,036	1,106	MWh/y
63 µm		-	**458**	644	672	742	MWh/y
5 µm			-	**186**	214	284	MWh/y
1.2 µm				-	**28**	98	MWh/y
0.45 µm					-	**70**	MWh/y
0.1 µm						-	MWh/y

The energy consumption of each particle removal technology has to be compared with the overall energy savings due to its removal of specific particle sizes. So the application of a certain pre-treatment technique can be counterbalanced to the resulting overall energy savings in the wwtp.

Cost calculations based on wastewater fractionation

In addition to energy calculations, also space requirements, chemical use, final sludge treatment, and effluent quality can be calculated based on wastewater characterisation on particle size. Together this can be summed up in the total treatment costs (see figure 4.6) calculated as net present values.

From figure 4.7, the financial effective investment costs per pre-treatment technique can be derived. For break even, a particle removal technique down to particle of 63 µm (like primary sedimentation) may cost about € 5 million (net present value). For a pre-treatment technique that removes particles down to 5 µm, € 13 million may be invested to compete with the primary sedimentation. An additional removal to 0.1 µm allows an additional investment of less than € 4 million.

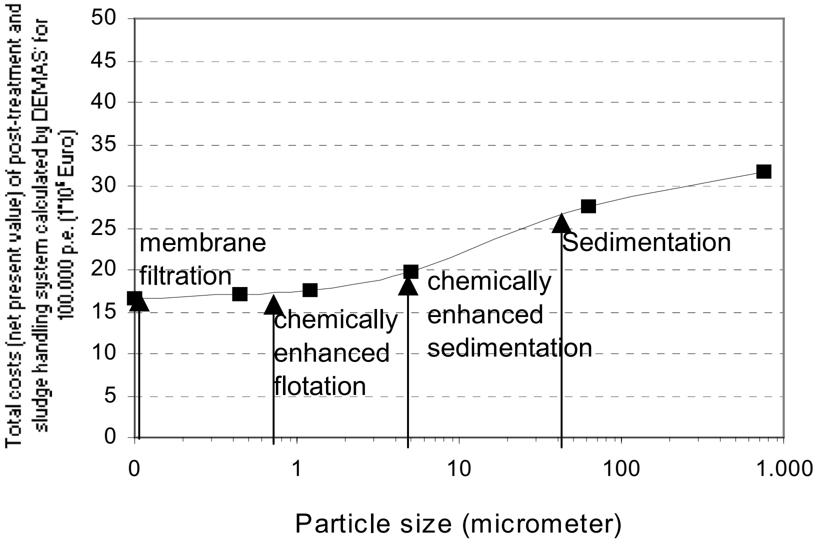

Figure 4.7: Indicative relationship between remaining treatment costs and size of removed particles (with pre-treatment techniques and particle size removal range)

4.5 Conclusions and recommendations

Conclusions

In the ‘average’ wastewater fractionation, the percentage of oxygen consuming components related to the theoretically determined *settleable* particle fraction is low, with a maximum of 21% for COD. A major part of BOD (44%), COD (38%) and phosphorous (35%) is present in *suspended* and *supra-colloidal* particle fractions with particle diameters between 1.2 and 63 µm. For nitrogen only 4%

can be related to settleable particles, 13% to colloidal and suspended fractions; so 83% of the nitrogen is present in soluble form.

By means of the conducted experimental research the effect and applicability of a certain physical-chemical particle removal technique, the so-called 'pre-treatment potential' of a specific wastewater can be derived. In addition, energy and cost calculations show the importance of particle removal in the pre-treatment down to approximately 5 à 1 μm, depending on the wastewater, to save energy and costs in the total wastewater treatment system.

Recommendations

From the fractionation experiments the primary effluent composition and the amount and composition of the produced primary sludge could be derived. Besides this, it could help predict the optimum type or dosage of a coagulant or flocculant to be added to the pre-treatment. These topics should be investigated in more detail.

The experiments show that the 0.45 μm separation technique should be changed to the more correct fractionation technique of the 0.1 μm membrane filtration. In case of the necessity of large volume of soluble fraction, is is advised to use precipitation-sedimentation.

The applied testing procedure was successful and should be carried out on a broader scale. More advanced (online) sampling, separation, fractionation and analysing techniques should be implemented to optimise the fractionation and characterisation tests.

Furthermore, the remaining filtrates should be tested in more detail for treatability with different kinds of post-treatment techniques. The nitrogen uptake rate (NUR) and oxygen uptake rate (OUR) tests are to be considered for this purpose.

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CHAPTER 5 ORGANIC POLYMERS FOR COAGULATION AND FLOCCULATION OF RAW WASTEWATER

SUMMARY

The objective of this chapter is to investigate the application of organic polymers as an alternative to metal-based flocculants by means of a literature review and jar test experiments.[*]

Two types of organic polymers are most frequently applied in (industrial) wastewater treatment and sludge handling, (1) low to moderate molecular weight polyamines and poly-DADMACs and (2) high molecular weight (HMW) polyacrylamides. Jar test experiments revealed that especially the cationic HMW polyacrylamides appeared to be interesting for practical application, as they got effective at relatively low additions (5-10 mg/l). For a low molecular weight cationic polyamine also high turbidity removals were found, however at higher required additions (20-30 mg/l). For both types of polymers, the experiments resulted in turbidity removals of 65-90% and a total suspended solids removal of more than 90%.

Due to the addition of HMW cationic polymers, large floc conglomerations with a high settleability are produced. The flocs formed after addition of a low weight cationic polyamine appeared to be significantly smaller and settle less fast.

Additionally it was found that the addition of organic polymers does not affect the pH of the treated water and results in only a minor increase of the salt content of the primary effluent.

Some parts of this chapter have been published in Dutch by STOWA (2001). A chapter with similar content is part of the dissertation by Mels (2001)

5.1 Introduction

In the provisional scenario study in chapter 2 it was concluded that wastewater treatment plants based on enhanced particle removal in the pre-treatment can be designed as more energy efficient and smaller compared to commonly applied systems. Moreover, if physical-chemical pre-treatment is combined with a biological post-treatment the costs of the overall treatment system are in the same range as those of currently applied scenarios and may thus become a realistic alternative. The evaluation study and the research into wastewater fractionation showed that the application of physical-chemical pre-treatment involved some drawbacks that are merely related to the application of (inorganic) metal salts for coagulation-flocculation. Due to the application of metal salts, the production of inorganic (chemical) sludge is increased, resulting in increased sludge handling costs and in a significant amount of remaining ashes after sludge incineration. A second problem was an increase in the salinity of the effluent, due to the release of counter ions by the flocculants. In case of the addition of $FeCl_3$ for example, about 1.5 mg Cl^- per mg Fe^{3+} was added, resulting in almost 30 g Cl^- per m^3 effluent for ferric additions that are used in practice.

Organic polymers may be an alternative for the inorganic (metal-) flocculants that were used in the scenario study. Chemical sludge production is absent when organic polymers are applied for particle flocculation and the increase in salt concentration of the effluent is small. This chapter discusses the possibilities of replacing metal salt coagulants by organic polymers; the result of a literature review and experimental research. Firstly, the theoretical flocculation processes are described. Secondly, the methods and results of the experimental research are presented and finally, the outcomes are discussed.

5.2 Theoretical background of coagulation and flocculation with organic polymers

Coagulation and flocculation processes

As explained in chapter 3, particles in water-like suspensions, like municipal wastewater, are mainly negatively charged. The particles in these suspensions are stabilised due to the electrical double layer of ions and the resulting negative zeta potential. In order to realise mutual attachment of particles by attracting mass forces destabilisation of the particles' repulsive surface charge is necessary. Regarding these destabilisation and attachment processes, a general distinction is made between *coagulation* and *flocculation*.

Coagulation indicates the process of charge neutralisation resulting in destabilisation of the particles. The term flocculation is used to indicate the process of complex formation that is succeeding the destabilisation [Fleer (1993)]. In understanding the coagulation and flocculation processes, three main mechanisms will be described [see also Knoppert and Heide, van der (1990)]:

(a) electrostatic coagulation

Electrostatic coagulation is the most important process when metal salts are applied. Electrostatic coagulation is caused by an increase in the electrolyte concentration in the wastewater suspension through the addition of the metal salts. Hereby the amount of counter ions present in the diffuse part of the electrical double layer (see chapter 3) in the stable colloidal suspension increases, which causes destabilisation of the particles in suspension. Compared to the situation without the addition of metal salts, the higher ionic strength results in a faster decrease of the zeta potential, thus decreasing the size of the Gouy-Chapman layer. Due to the decrease of the electrical double layer, the zeta potential decreases and the particles can approach each other closer. If high amounts of metal salts are applied, the concentration of counter ions in the diffuse layer may increase to such a level that the zeta potential shifts to a positive charge. In this case the particles will be positively charged and again be colloidal stable in suspension.

(b) precipitation coagulation (or *sweep coagulation*)

In addition to electrostatic coagulation, precipitate coagulation (or ‘sweep coagulation’) is an important coagulation process. In precipitation coagulation, particles are entrapped by flocculent metal-hydroxides. This mechanism is in fact the most important mechanism of turbidity removal, when metal salts are added.

(c) adsorptive coagulation (or *bridging flocculation*)

The process of adsorptive coagulation is defined as the destabilisation by adsorption of polymers or long hydroxide chains to the particle surface and was first described by Stumm and O’Melia [Stumm and O’Melia (1968)]. The adsorption is caused by mutual attraction of opposite charge and/or London-Van der Waals’ mass forces. The surface charge of the particles is partially or completely neutralised by the adsorbed polymer or hydroxide chain.

A special type of adsorptive coagulation is entitled as bridging flocculation. In bridging flocculation, macromolecules or long polymer chains are adsorbed to the surface of particles. Due to their length, the polymer chains are able to attach to different particles, thus causing a ‘*bridging*’ effect (figure 5.1). An important characteristic of bridging flocculation is that lower electrolyte additions can be applied compared to destabilisation and flocculation by electrostatic and precipitate coagulation.

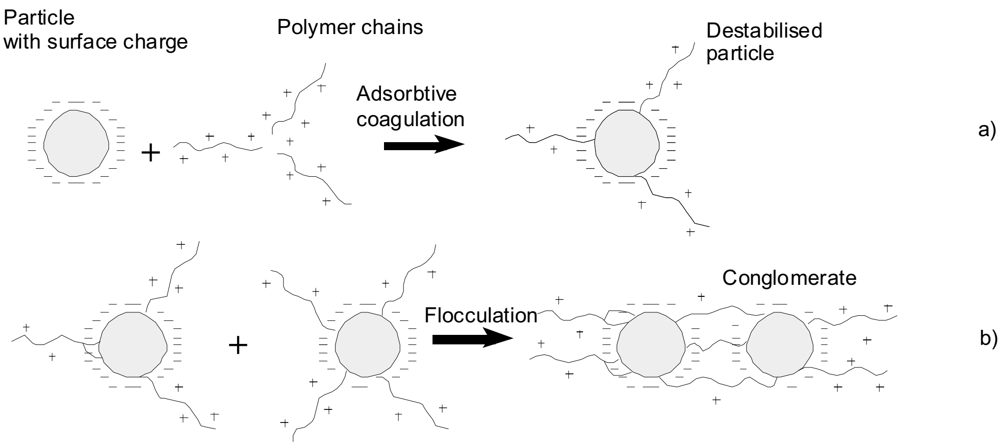


Figure 5.1: *Adsorptive coagulation* (a) and *bridging flocculation* (b) due to the addition of a high molecular weight cationic polymer

Similar to polyelectrolyte overdose in electrostatic coagulation, an overdose of polymer may result in restabilisation of particles. The adsorption capacity of the colloidal particle under restabilisation would be completely used by several polymer chains and could not longer be used for bridging with other particles (see figure 5.2).

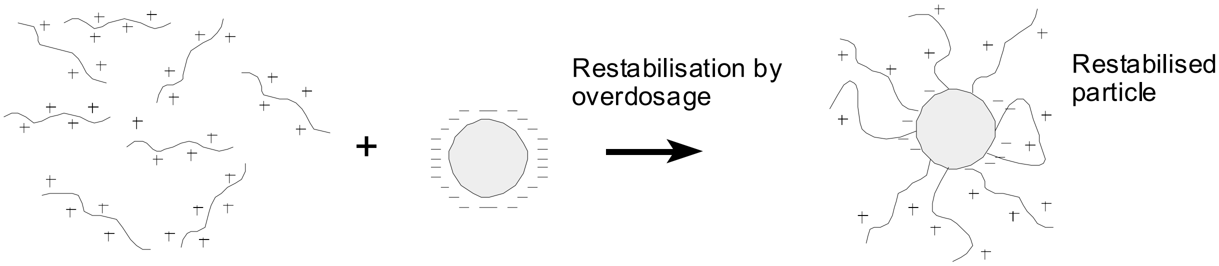


Figure 5.2: Restabilisation of particles due to overdose of organic polymer

Organic polymers as coagulants and flocculants

Organic polymers are generally characterised by two main properties: their molecular weight and the amount of ionic charge.

The molecular weight (or, more properly, molecular mass) is an indication for the amount of monomers and thus the length of the polymer chain. The molecular masses of polymers are subdivided in low molecular weight (LMW), mid molecular weight (MMW) and high molecular weight (HMW) (see appendix G). The molecular weight can be measured by rheological, osmotic and light scattering techniques. Most applied are viscosity measurements of polymer solutions in order to determine the average molecular mass.

Concerning ionic charge, polymers can be cationic, anionic and non-ionic, i.e. respectively positively, negatively and neutrally charged. The charge density of the polymer indicates the amount of charge available to accomplish particle destabilisation and flocculation. The cationic or anionic charge density is normally expressed as the (weight) percentage of charged monomers. Polymers with charge percentages of 5 to 100 weight-percentage are produced for different applications. This means that 5 to 100% of the total weight of these polymers consists of cationic monomers. The charge density of polymers can be measured by potentio-metric, colloidal or counter-ion titration.

Additional to the molecular mass and the charge density, the structure of the polymer is import. Besides the linear configuration, polymers can be manufactured cross-linked or branched. Cross-linked or branched polymers are especially used in sludge dewatering by centrifuges because of their ability to resist high shear forces. Based on the molecular weight and the knowledge on the structure, the length of the polymer chain can be calculated. As mentioned earlier, this chain length determines the coagulation-flocculation mechanism that will occur.

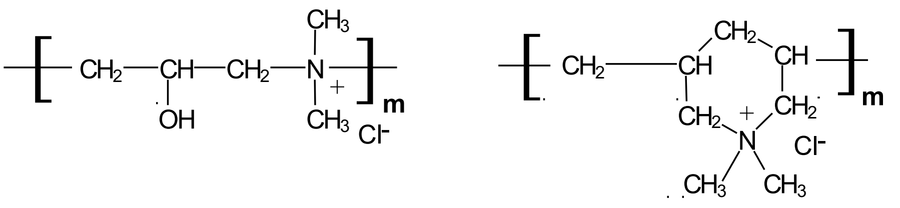
Synthetic organic polymers used in clarification of water and wastewater and dewatering of sludges are most commonly sold in the form of dry powders or inverse (water-in-oil) emulsion and are categorised into two major types:

- low to mid molecular weight cationic coagulants based on (non-) quaternised dimethylamine (polyamines, figure 5.3a) and diallyldimethyl ammonium chlorides (DADMACs, figure 5.3b); and

- very high molecular weight polyacrylamides (PAMs) flocculants, which may be anionic, cationic or nonionic (see figure 5.4 a to c).

The two commonly applied chemical structures of low to moderate molecular weight cationic polymers are shown in figure 5.3. Both polymers are homo-polymers based on one type of monomer. Structure a. in figure 5.3 depicts a quaternary cationic polyamine, while structure b. shows a cationic polymer based on a DADMAC monomer. Both polymers are frequently used for coagulation processes due to their high cationic charge and their low molecular mass (resulting in short chains).

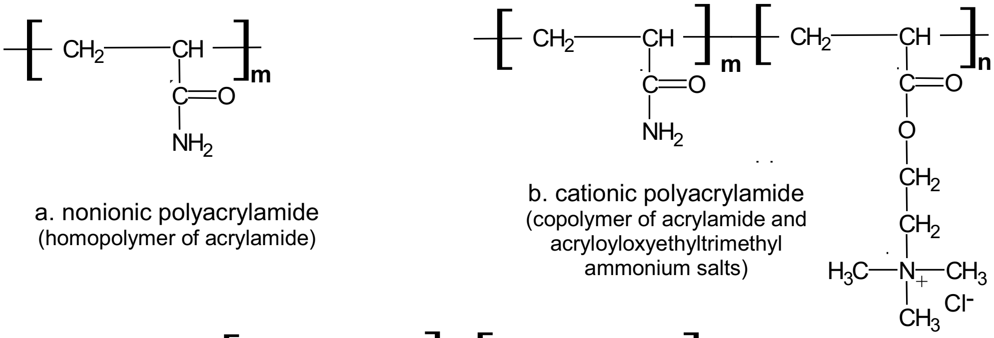
The monomers shown in figure 5.3 can also be co-polymerised with acrylamide to create a variety of high weight copolymers.



a. polyamine

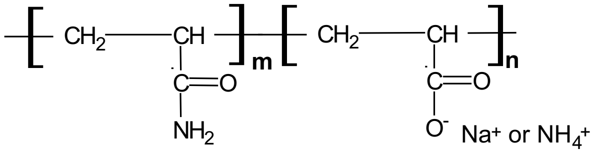
b. poly-DADMAC

Figure 5.3: Chemical structures of low to moderate molecular weight cationic polymers



a. nonionic polyacrylamide
(homopolymer of acrylamide)

b. cationic polyacrylamide
(copolymer of acrylamide and
acryloyloxyethyltrimethyl
ammonium salts)



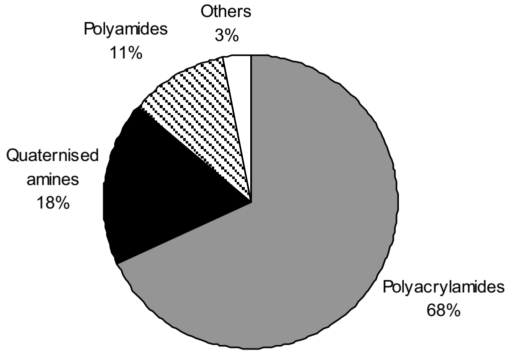
c. anionic polyacrylamide
(copolymer of acrylamide and
a salt of acrylic acid)

Figure 5.4: Chemical structure of polyacrylamide based polymers [Barvenik (2000)]

Structure a. in figure 5.4 shows the basic acrylamide monomer, which after polymerisation yields non-ionic polyacrylamide. Non-ionic polymers are hardly applied in wastewater treatment or sludge handling. Structure 5.4b depicts the most common acrylamide based cationic polymer, consisting of a non-ionic monomer (acrylamide) and a cationic monomer (quaternised aminomethylated acrylamide). The 'm' and 'n' indicate he ratios of non-ionic and cationic monomers and determine the charge densities of the polymer. Structure 5.4c represents an anionic polyacrylamide, containing an acrylic acid monomer. These polymers are frequently employed as coagulant aids in water treatment. The anionic polymers are relatively pH dependent in comparison with the cationic polyacrylamides, due to the acidic nature of the monomer. At a pH lower than 4 the anionic polymer is uncharged.

Figure 5.5 [see also Will (2000)] illustrates the amount of polymers used globally for water and wastewater treatment. Two-third of the volumetric global market in speciality coagulants and flocculants is occupied by PAMs, followed by quaternised ammonium salts and polyamides.

Apart from the synthetic polyacrylamides, polyamines and quats different organic polymers exist that are based on natural sources. Examples are cationic quaternised starche polymers, condensed tannin [Gucciardi (2000)], cellulose and galactose products, microbial polysaccharide products, gelatine based products, seeds of the Moringa Olefeira [Ndabigengeere and Narasiah (1996), Kalogo and Verstraete (2000), Okuda *et al.* (2001) and Chitine based products like Chitosan [Fettig *et al.* (1998), Pan *et al.* (1999), Gucciardi (2000)].



Total Global Market per 1998
(Value: €1.4 billion; Volume: 250 kmt)

Figure 5.5: Global market shares of coagulants and flocculants based on [Will (2000)]

Aquatoxicity of organic polymers

The past years a lot of attention was paid by manufacturers and research institutes to the possible consequences of polyacrylamides and polyamines and (by- and degradation products) on the aquatic environment. The concern refers especially to the persistence and toxicity of biodegradation and degradation products of polyacrylamides and the toxicity of the type of by-product of emulsified polymers [STOWA (1995)].

Extensive research has been conducted regarding the biodegradation and degradation pathways of PAMs and the potential toxicological effects of acrylamide monomers to plant and animals. It was concluded that PAMs do not degrade to yield acrylamide monomers, but that acrylamides have to be handled carefully [Barvenik (2000)].

A part of the assortment of polymers is available in emulsified form. For most of these products, 50% consists of active polymer while the other 50% is added for emulsification. The major anxiety concerns the emulgators that are added, the alkyphenolethoxylates (APEO's). These compounds are toxic to water organisms. One of the degradation products of APEO is Nonylfenol.

Nonylfenol is toxic and affects the hormonal system of fish. In 1998 a voluntary agreement was signed by European manufacturers to stop the use of APEO's, because of these hazardous effects [Umwelt Bundesamt (1997)].

Mixing aspects

When polymers are applied for coagulation-flocculation, mixing time and mixing intensity are of major importance for the effectiveness of the flocculation process. Good mixing results in an even distribution of the polymer over the particles in the wastewater suspension. If the mixing zone is not properly designed, the effectiveness of the flocculation process may be sub-optimal due to an inefficient distribution of polymers and particles [Amirtharajah and Jones (1996)]. Mixing after the addition of flocculants generally consists of two successive phases:

- a short, intensive mixing phase aimed at optimising conditions for particle-flocculant interactions; and
- a longer, less intensive phase aimed at flocculation of the activated particles to grow strong, large and separable flocs.

The mixing intensity is usually expressed in the velocity gradient G (s^{-1}) [Camp and Stein (1947), Culp (1967)], although this parameter is under discussion [Grader (1985)]. Table 5.1 contains an overview of rapid and flocculation mixing conditions that were found in the literature.

Table 5.1: Literature information on mixing times and velocity gradients when dosing organic polymers

application	mixing time and intensity	flocculation time and intensity	reference
jar test experiments with municipal wastewater	5 min. (G = 700 s^{-1})	15 min. (G = 23 s^{-1})	Fettig *et al.* (1991)
jar test experiments with municipal wastewater	1 - 2 min.	10 - 25 min.	Udaya Bhaskar and Gupta (1986)
jar test experiments for drinking water treatment	10 s (G = 1,180 s^{-1})	20 min. (G = 40 s^{-1})	Meijer *et al*. (1984)
jar test experiments for sludge handling	10 s	3 min.	According to manufacturers

5.3 Materials and methods

In order to determine the feasibility of cationic organic polymers for flocculation of municipal wastewater, jar tests under different process conditions were conducted.

Types of organic polymers

For coagulation-flocculation one low molecular weight (LMW) cationic polyamine, four linear high molecular weight (HMW) polyacrylamides and two HMW anionic polyacrylamides (all *CYTEC* products) were tested (see table 5.2). A stock polymer solution was prepared by dissolving the basic polymer powders or emulsions. Shortly before the experiment, these stock solutions were diluted to create a working solution that could be immediately added to the wastewater sample.

Table 5.2: Characterisation of organic polymers used in the study

type of polymer	description	molecular weight (g/mol)	degree of charge (weight%)	calculated chain length (µm)*
cationic low molecular (# 1)	- linear polyamine - water-in-oil emulsion	$0.25 \cdot 10^6$	(+) 100%	0.6
cationic high molecular (# 4)	- linear polyacrylamides - powders	$4.0 \cdot 10^6$	(+) 24%	18.5
		$6.0 \cdot 10^6$	(+) 24 / 38%	27.8 / 25.3
		$8.0 \cdot 10^6$	(+) 24%	37.0
anionic high molecular (# 2)	- linear polyacrylamides - powders	$6.5 \cdot 10^6$	(-) 20%	35.6
		$18.5 \cdot 10^6$	(-) 30%	98.2

* theoretical chain length, calculated based on molecular weight (full linear stretch is unlikely)

Additional comparative tests were carried out with CIBA and ONDEO Nalco products, with similar polymer characteristics (see Appendix G).

Materials

The jar tests were conducted in a set up with 6 stirred beakers each with a capacity of 2.5 litres (see schematic presentation in figure 5.6). The beakers were filled with 2.0 litres of raw unsettled wastewater. Portions of dissolved polyelectrolyte were applied simultaneously to all beakers under rapid coagulation mixing conditions (mixing intensity 300 r.p.m.; G = 800 s^{-1}; 120 s for LMW polymers; 20 s for HMW polymers). After coagulation mixing, slow flocculation mixing was applied (50 r.p.m.; G x t = 50 s^{-1} x 180 s). The supernatant was analysed for turbidity, pH and TSS after 900 s sedimentation time.

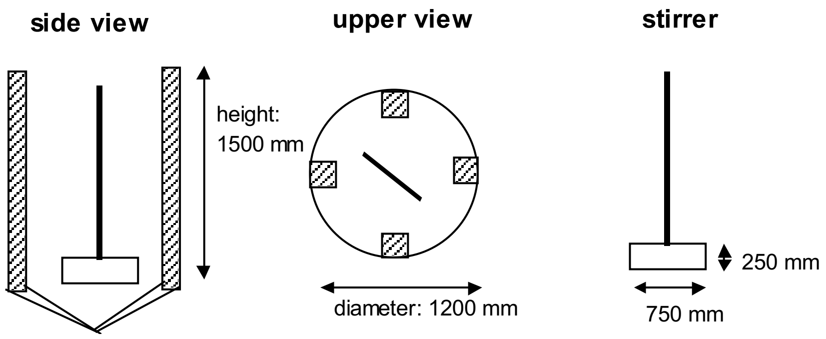


Figure 5.6: Schematic presentation of the used jar test apparatus with dimensions

Analysis

Turbidity was measured with a HACH turbidimeter in nuclei turbidity units (NTU). COD, total suspended solids, pH and temperature were measured according to Standard Methods [APHA (1998)]. The COD was determined for three wastewater samples: untreated, filtered through a 5.4 μm filter (Schleicher & Schuell, S&S) and filtered through a 0.45 μm membrane filter (S&S). For characterisation purposes, total COD was fractionated into $COD_{particles}$, $COD_{colloidal}$ and $COD_{soluble}$. Herein $COD_{particles}$ = COD in raw wastewater – COD in 0.45 μm filtrate; $COD_{colloidal}$ = COD in 5.4 μm filtrate – COD in 0.45 μm filtrate; and $COD_{soluble}$ = COD in the 0.45 μm filtrate.

Wastewater characteristics

For the experiments, wastewater from the municipal wwtp of the village of Bennekom (the Netherlands) was used. Table 5.3 gives an overview of relevant parameters.

Table 5.3: Wastewater characteristics of the influent of wwtp Bennekom (June 1998 - March 1999)

parameter	average concentration	fluctuation
turbidity	130 NTU	75 – 220 NTU
COD_{total}*	525 mg O_2/l	220 – 670 mg O_2/l
total suspended solids (TSS)	160 mg TSS/l	100–300 mg TSS/l
pH	7.9 -	7.4 - 8.5 -

* 59% as particle-related COD larger than 0.45 μm

Sludge analyses

To study the filterability (or dewaterability) of the primary wastewater sludge produced by polymer flocculation, the average specific (sludge) cake resistance (SCR) was measured with the technique of the Filtration-Expression cell (FE cell, [Heij, la *et al.* (1996)]). The average SCR is a measure for the average filtration resistance of a sludge cake. The FE cell is an automated measuring device that can be used to record the dynamic dewaterability behaviour of sludges. Figure 5.7 shows a schematic set-up of the device.

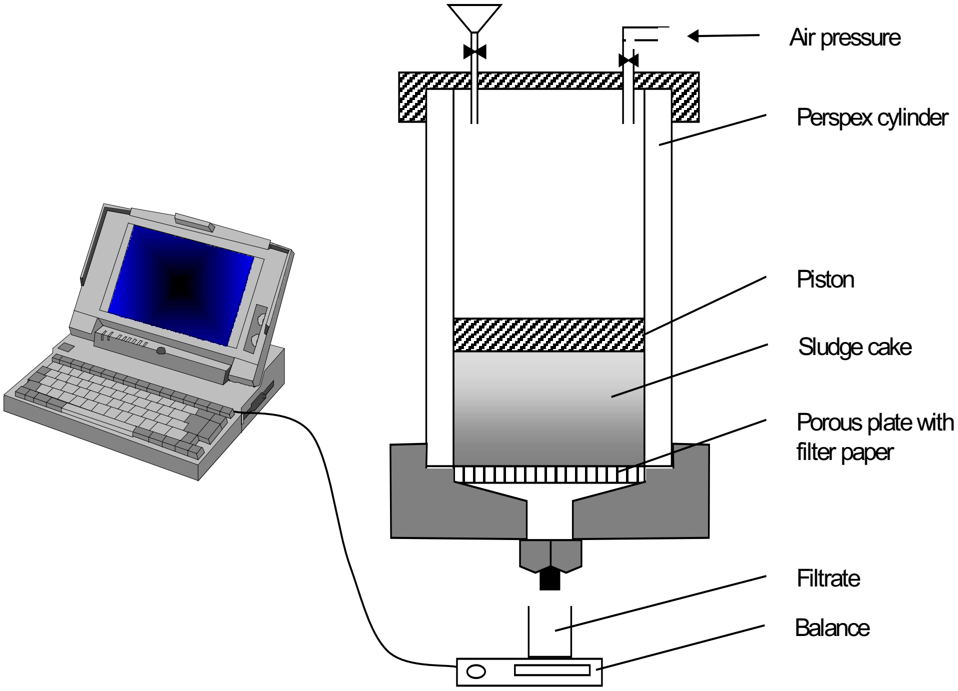


Figure 5.7: Schematic presentation of the FE cell

The FE cell consists of a perspex cylinder with a porous bottom plate. Before filtration, a filter paper is placed on the porous plate and a sludge sample is introduced into the cell. A non-porous piston is placed on top of the sample. When the gas pressure is applied the dewatering of the sample starts. The filtrate is collected in a beaker on a balance. The balance registers the mass of the released filtrate as a function of time. After the experiment the dry solids content of the sludge sample is measured. The filtration process is monitored continuously by a computer.

Based on the assumption that the sludge cake behaves incompressible, the SCR can be calculated by means of the integrated Darcy equation [Heij, la *et al.* (1996)].

$$t = SCR_s \cdot \frac{\eta \cdot c_v}{2A^2 \cdot \Delta p} \cdot V^2 + \frac{\eta \cdot R}{A \cdot \Delta p} \cdot V \qquad (5.1)$$

where: SCR_s = average specific cake resistance (mg/kg)
η = viscosity of the filtrate (Pa·s)
A = surface of the filter medium (m^2)
c_v = concentration dry solids in suspension (kg/m^3)
Δp = pressure difference (Pa)
t = time (s)
V = filtrate volume (m^3)
R = resistance of the filter medium (1/m)

5.4 Experimental results

Jar tests were conducted to determine the feasibility and effectiveness of organic polymers for flocculation of municipal wastewater.

Additional to the polymer-added samples, each time a sample without the addition of polymer was tested to investigate the specific sedimentation condition of non-flocculated components in the raw wastewater. So, turbidity and suspended solids removal were monitored by sedimentation. The settleable part of turbidity and suspended solids are shown in the following graphics as grey blocks and are entitled as the '*settleable*' part of the raw wastewater.

Coagulation-flocculation with a LMW cationic polymer

First, the results of the coagulation-flocculation tests with a low molecular weight polymer are presented. Figure 5.8 shows the turbidity and suspended solids content of the supernatant after flocculation and sedimentation with the tested LMW cationic polymers from table 5.2. The turbidity decreased from 225 NTU to 20 à 40 NTU and suspended solids were removed from 330 mg TSS/l to below 15 mg TSS/l.

The settleable part of turbidity and suspended solids were 65 NTU and 225 mg TSS/l, respectively.

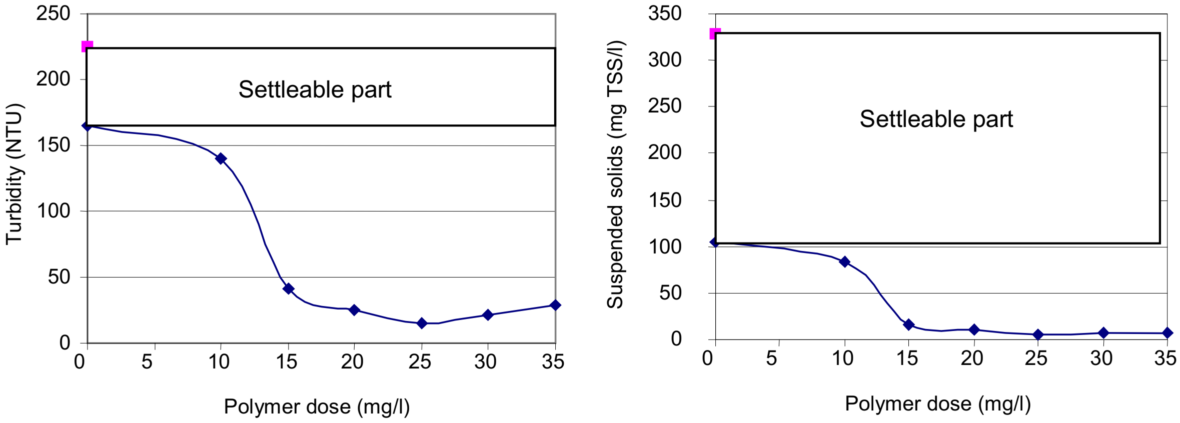


Figure 5.8: Turbidity (left) and suspended solids content (right) in the supernatant after flocculation and sedimentation with a LMW cationic polymer ($0.25 \cdot 10^6$ g/mol; charge +100%)

Combination addition - When the combination addition is applied the produced sludge floc is relatively small and settles slowly. An optional improvement to enhance the sedimentation is the addition of anionic flocculants. A significant improvement of the sedimentation properties of the flocs due to the addition of anionic flocculants is not obvious in figure 5.9 however.

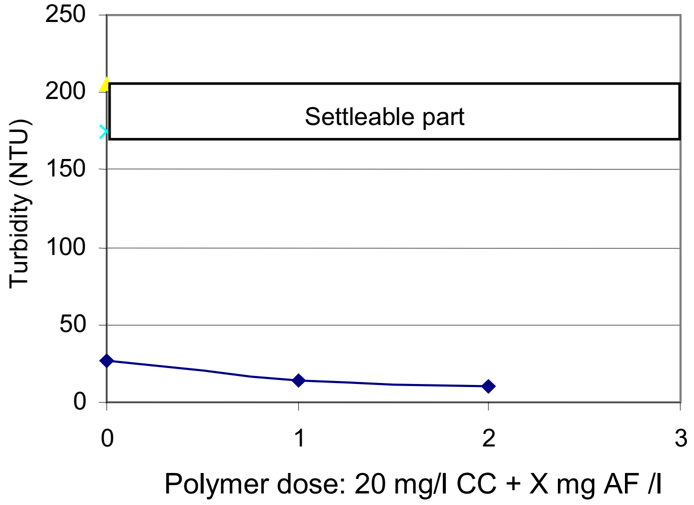


Figure 5.9: Increase of the turbidity removal by applying a LMW cationic polymer ($0.25 \cdot 10^6$ g/mol; indicated with CC – cationic coagulant) in combination with an HMW anionic polymer ($6.5 \cdot 10^6$ g/mol; indicated with AF – anionic flocculant)

Overdosing - For the LMW cationic polymer, overdosing may result in an increase of the turbidity as shown in figure 5.10. In this example, the optimum addition is somewhat higher than 30 mg/l; after which the turbidity of the supernatant increases again. The experiments showed that the optimum addition might differ depending on the strength of the wastewater. For the tested wastewater, the average optimum addition was 25 mg/l.

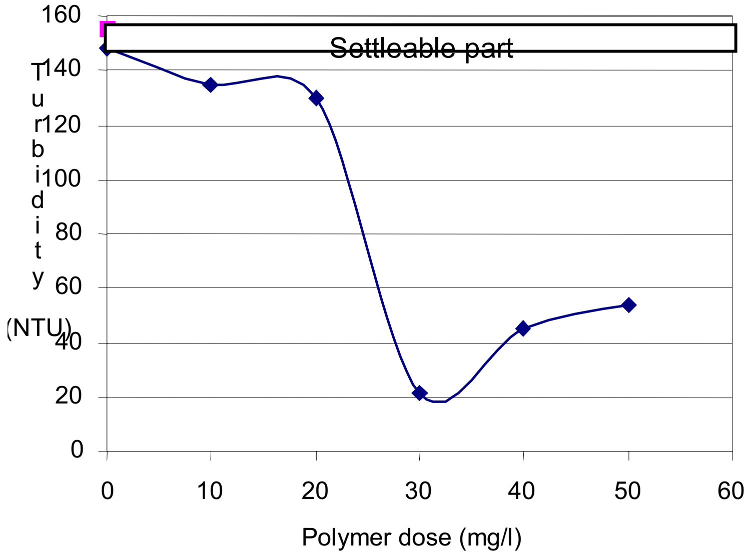


Figure 5.10: The effect of overdose when dosing a LMW polyamine ($0.25 \cdot 10^6$ g/mol)

Flocculation with HMW cationic polymers

Figure 5.11 shows the result of a flocculation jar test experiment with a HMW cationic polymer of 6·10^6 g/mol. The figure shows that this type of polymer results in a significant particle removal at relatively low dosing rates. At an addition of 5 mg active polymer per litre, the turbidity of the supernatant decreases to below 40 NTU, while the suspend solids content amounts less than 10 mg TSS/l. In the conducted experiments, overdosing with HMW cationic polymers was not detected at additions up to 40 mg polymers per litre.

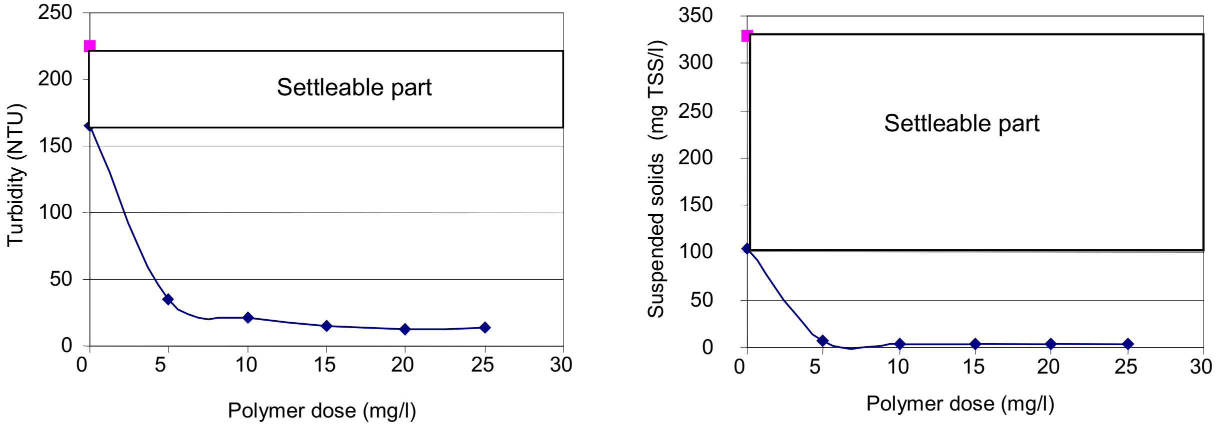


Figure 5.11: Turbidity (left) and suspended solids content (right) in the supernatant after flocculation and sedimentation with a HMW cationic polymer with a molecular weight of 6·10^6 g/mol and +24% charge

Molecular weight

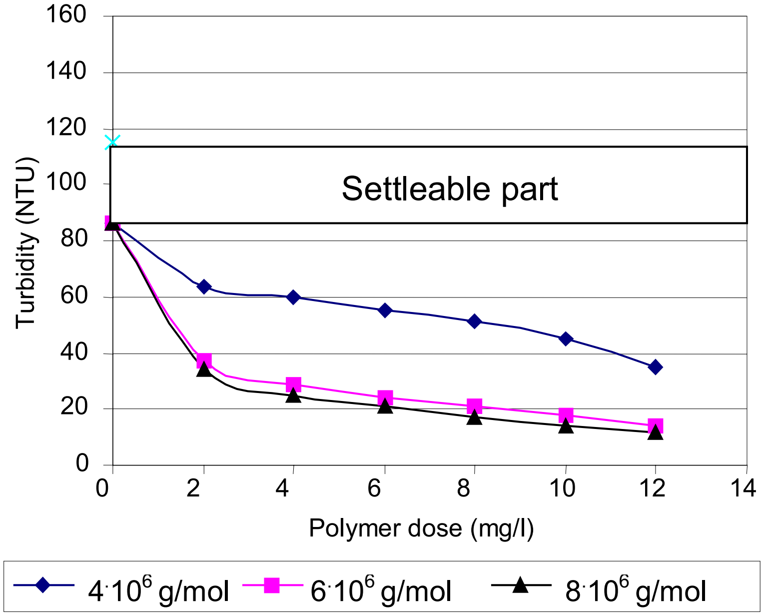


Figure 5.12: Turbidity in the supernatant after flocculation and sedimentation with HMW cationic polymers with rising molecular weight (4·10^6, 6·10^6 and 8·10^6 g/mol; charge 24%)

In figure 5.12, a comparison is made for three tested HMW polymers with different molecular weights. As explained before, for a linear polymer the chain length is an import factor in the size of the floc that is formed and for the effectiveness of the particle removal. The chain length is determined by the specific molecular weight of the polymer. As shown in the figure the polymer with the lowest molecular weight ($4 \cdot 10^6$ g/mol) shows a lower turbidity removal than both other polymers.

Charge density - The degree of charge determines the number of available places on the polymer chain that are active to bind to particles, flocs or other polymer conglomerations. The degree of charge may influence the ultimate particle removal. Figure 5.13 shows the turbidity and suspended solids removal of a jar test experiment with two HMW polymers with identical molecular mass and different charge densities and +24 and +38 weight percentages, respectively. The outcome of the experiment showed for these polymers only a small difference in favour of the +24 weight percentage polymer.

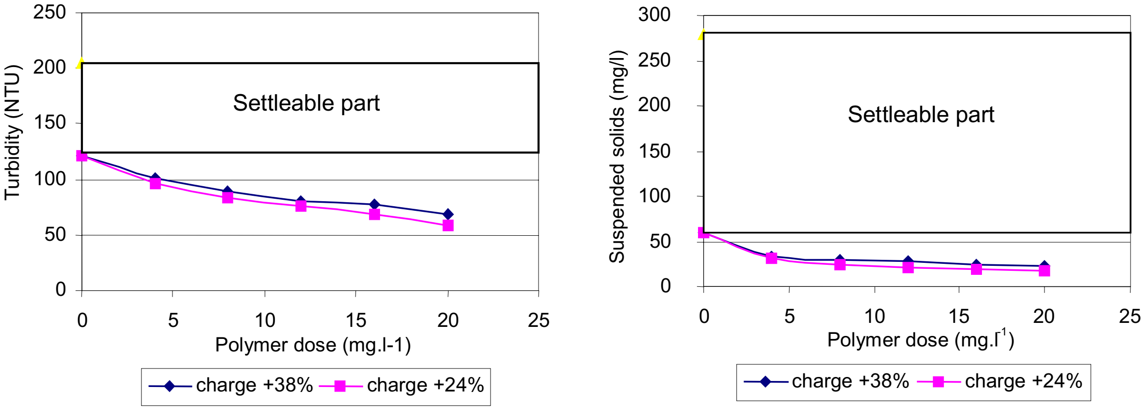


Figure 5.13: Effect of charge density: turbidity (left) and suspended solids content (right) for two cationic polymers with a different charge density and similar molecular weight ($6 \cdot 10^6$ g/mol)

An additional interesting observation from both figure 5.12 and 5.13 is that in the lower addition ranges the removal of turbidity increases gradually with an increasing addition.

Flocculation with HMW anionic polymers

In figure 5.14, the turbidity and suspended solids removal after flocculation and sedimentation with a HMW anionic polymer of $18.5 \cdot 10^6$ g/mol are shown. For this polymer, hardly any effect is measured. Based on the before mentioned fact that particles in municipal wastewater are mainly negatively charged it is obvious that anionic polymer will have hardly any effect in flocculation.

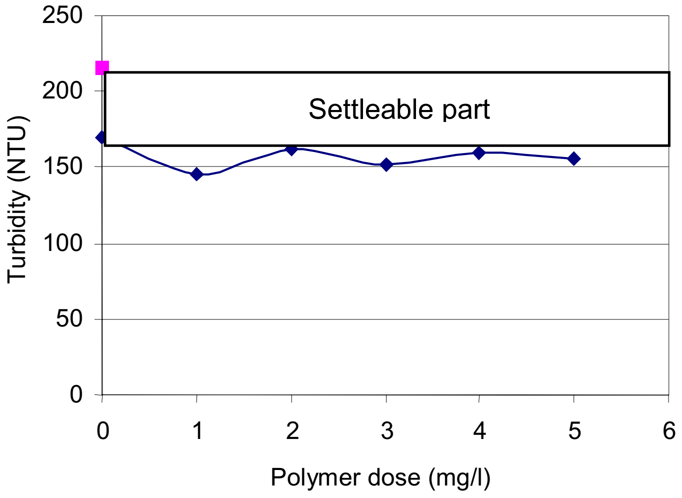


Figure 5.14: Turbidity removal after flocculation and sedimentation with an anionic polymer (molecular weight: 18.5·10^6 g/mol; charge: –30 weight%)

COD and pH of the flocculated wastewater

Figure 5.15 shows the result of measurements of COD in the supernatant after flocculation and sedimentation with a HMW cationic polymer. Both the removal of COD_{total}, as well as the removal of the fractions $COD_{particles}$, $COD_{colloidal}$ and $COD_{soluble}$ is given (left, and right, respectively). The COD of the influent decreases from 525 mg O_2/l (settled 420 mg O_2/l) to 276 mg O_2/l at a addition of 4 mg polymer per litre influent. The fractionation shows that for the given addition range the $COD_{particles}$ shows a removal of 42% (no addition) to more than 90% at a addition of 4 mg/l, while the $COD_{colloidal}$ removal increases from 2% to 35%.

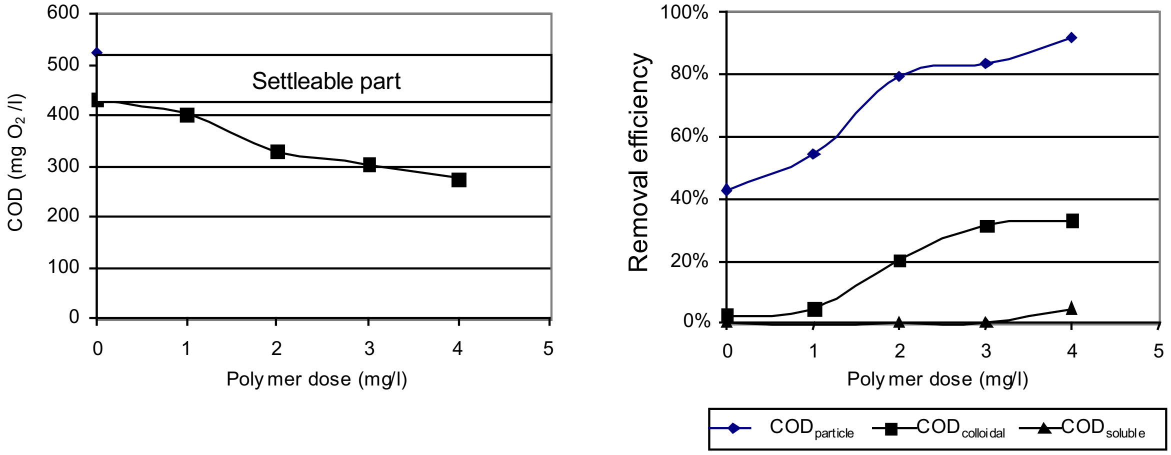


Figure 5.15: COD in the supernatant after flocculation and sedimentation with increasing additions of a HMW cationic polymer (8·10^6 g/mol, +24%); left: COD_{total} removal; right: removal of COD fractions

In contrast to inorganic metal salts, it was found that the addition of organic polymers did not affect the pH of the wastewater. This statement is illustrated by figure 5.16 that shows the pH after flocculation and sedimentation with an organic HMW cationic polymer (left) and an inorganic poly-aluminium-chloride (PAC) (right graph). The pH of the supernatant in case of the polymer addition, stayed constant at about 7.8, whether the pH of the remaining wastewater after PAC addition decreased from 7.8 to less than 5. The addition of a LMW polymer neither showed a pH effect.

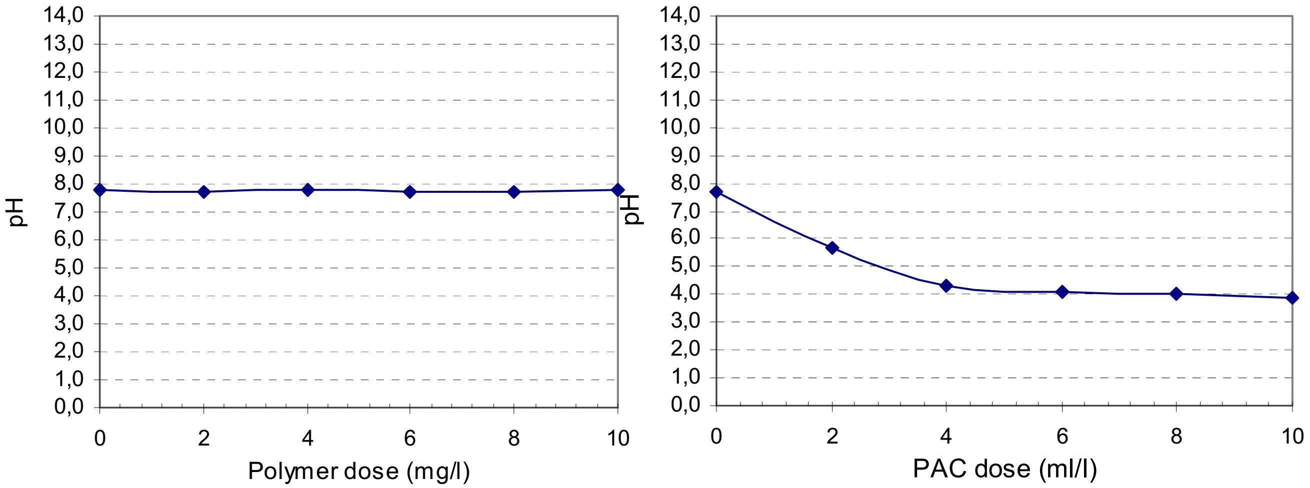


Figure 5.16: pH of the supernatant after flocculation and sedimentation with (left graph) increasing additions of a HMW cationic polymer ($8 \cdot 10^6$ g/mol, +24%) and (right graph) addition of poly-aluminium-chloride (PAC).

Floc characteristics

Photo analysis - Figure 5.17 shows three microscopic photographs of flocs created by the addition of $FeCl_3$ (photo A) and two cationic organic flocculants (B and C). The photo analysis entitled B, pictures a floc of a LMW cationic organic polymer, photo C shows a floc formed by the addition of a HMW cationic polymer (with molecular weights of respectively $0.25 \cdot 10^6$ g/mol and $8 \cdot 10^6$ g/mol). In comparing the flocs, the effect of chain length in flocculation is made clear. The LMW polymer has a theoretically calculated chain length of 0.6 µm, while the HMW polymer has a theoretical chain length of 37 µm. The photographic analysis shows that coagulation with $FeCl_3$ and flocculation with the LMW polymer results in relatively small flocs (pin flocs) compared with floc structure of the HMW polymer. The sludge floc produced by the HMW polymer is a large floc with a width of circa 5 mm, a length of more than 10 mm and a theoretically calculated diameter of 0.5 mm [Dentel *et al.* (2000)].

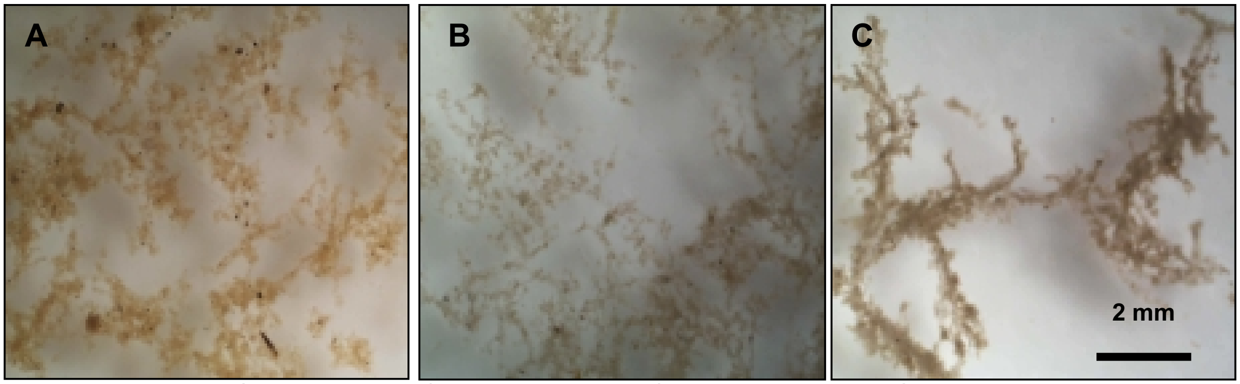


Figure 5.17: Comparison of three sludges flocculated by $FeCl_3$ and two cationic organic flocculants; A: $FeCl_3$ (15 mg Fe/l); B: LMW cationic polymer $0.25 \cdot 10^6$ g/mol, +100% (30 mg/l); C: HMW cationic polymer $8 \cdot 10^6$ g/mol, +24% (10 mg/l).

Settleability of flocs - The graphic in figure 5.18 presents a sedimentation curve for the flocs of the LMW and HMW cationic polymers and for untreated influent. In this experiment, samples were taken from the supernatant at 10 cm beneath the water surface. The curve illustrates the differences in floc sizes and densities produced by the different polymers, resulting in different sedimentation velocities. The figure shows that sludge flocs that are formed by the HMW cationic polymer have a higher sedimentation velocity than the flocs formed by the LMW cationic polymer (combined with a addition of 1 or 2 mg HMW anionic polymer). This experiment indicates that in the case that flocs are produced by HMW polymers, a sedimentation tank may be designed with a higher surface load to achieve comparable removal performances than with LMW polymers.

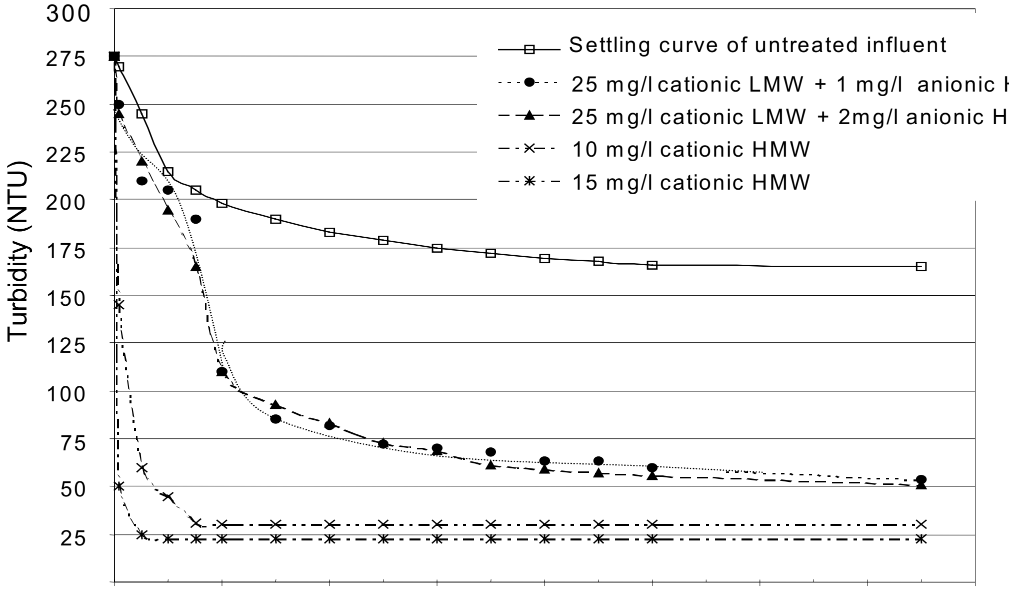


Figure 5.18: Sedimentation curves of untreated influent and influent treated with LMW and HMW polymers; (LMW cationic polymer: $0.25 \cdot 10^6$ g.mol^{-1}, +100%; HMW anionic polymer $6.5 \cdot 10^6$ g.mol^{-1}, –30%, HMW cationic polymer: $6 \cdot 10^6$ g.mol^{-1}, +24%)

Dewaterability of the polymer flocs - In figure 5.19, the average specific cake resistance of the produced sludge flocs, measured by the FE cell, is given as a function of the flocculant addition. The addition is recalculated to the flocculant dosage per gram of sludge solids (mg/g TS). The average specific (sludge) cake resistance (SCR) was measured for settled sludge flocs from experiments with raw, untreated wastewater as well as from experiments with increasing additions of HMW cationic flocculant. Additional, sludge flocs of wastewater coagulated with 15 mg/l Fe^{3+} and a sludge sample of activated sludge were tested.

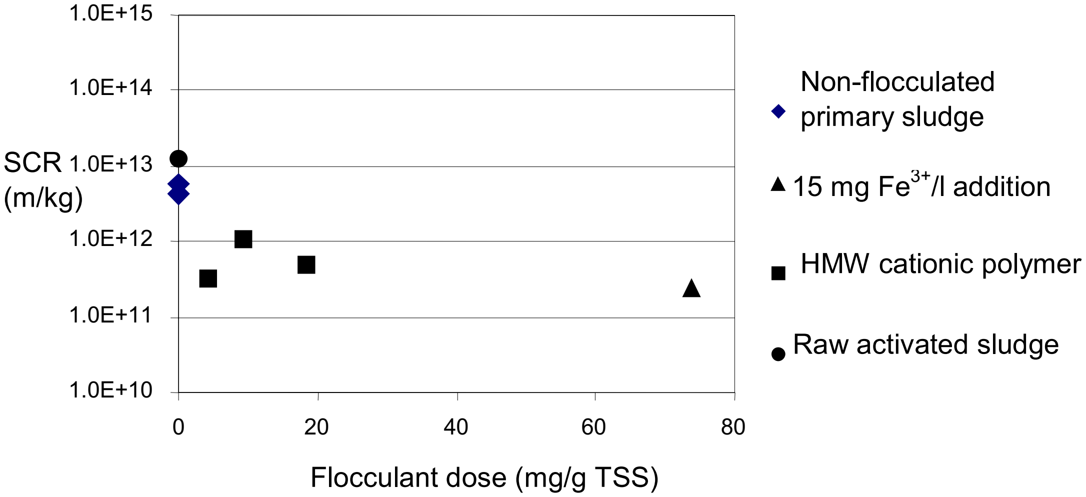


Figure 5.19: Average specific cake resistance (SCR) for flocculated and non-flocculated floc and sludge samples, measured by the Filtration-Expression cell.

The figure shows that the average SCR of the flocculated sludge samples is lower than the average SCR of non-flocculated primary sludge. The results of the average SCR show that both polymer and ferric addition in the pre-treatment are favourable for the dewaterability characteristics of the produced sludge. Similar to the application of flocculants in sludge handling, coagulation-flocculation results in (partial) charge neutralisation and into the formation of larger aggregates.

5.5 Discussion

The conducted jar test experiments show that cationic organic polymers can be used for flocculation of raw municipal wastewater. As expected, the addition of an anionic polymer did not show significant turbidity removal. Table 5.4 gives a short overview over the gathered experimental results.

Table 5.4: Applicable organic polymers for flocculation of wastewater

type of polymer	assumed coagulation-flocculation mechanism	dosage (mg/l)*	achievable final value for turbidity (NTU)	achievable final value for suspend solids (mg TSS/l)
LMW cationic ($0.25 \cdot 10^6$ g/mol)	electrostatic coagulation	15 - 30	15 - 25	< 5
HMW cationic (> $6 \cdot 10^6$ g/mol)	bridging flocculation	2 - 5	50 - 70	10 - 20
		10 - 15	15 - 50	< 10

* Indicative value: based on the wastewater used in the experiments

Assumed coagulation-flocculation mechanisms

The jar test experiments show an overdose of LMW cationic polyamine results in increased turbidity compared to the optimum addition (figure 5.10). After a certain optimal minimum concentration (25 - 35 mg/l) a sudden increase in the turbidity of the supernatant was measured. Considering the short chain length of this polymer and the high cationic charge it can be concluded that this polymer shows an electrostatic coagulation effect. The polymer that is attached to the surface of the particles is causing an increase of charge resulting in an increased zeta potential. In case of overdose the zeta potential is becoming positive causing a restabilisation effect.

The high weight cationic polyacrylamides behaved according to the mechanism of bridging flocculation. The study shows that the HMW cationic polymers are most feasible for practical applications since they can be applied at relatively low dosing concentrations. Increasing concentrations of polymer addition led to increasing removal rates of turbidity.

The effect of polymer addition on pH and salinity of the primary effluent

pH - Unlike the addition of metal salts, no pH effect to the remaining primary effluent was found when dosing with polymers. The addition of metal salts may result in a strong decrease of the pH in the supernatant as has been proven in parallel experiments. A pH drop of 7.8 to 3.8 was found when 8 to 10 ml PAC (poly-aluminium-chloride) per litre wastewater was applied The cause is the general complexation after addition with metal salts as explained by equation 5.2:

$$MeCl_3 + 6\ H_2O \Rightarrow Me(H_2O)_6^{3+} + 3\ Cl^- \qquad (5.2)$$

resulting into acidification according to the reaction equation 5.3:

$$Me(H_2O)_6^{3+} \Rightarrow Me(H_2O)_5(OH)^{2+} + H^+ \qquad (5.3)$$

A pH drop may have a strong effect on the biological post-treatment, since especially nitrification is sensitive to a low pH.

Salinity - As shown in equation 5.2, the addition of metal salts leads to an increase of the salt content of the primary effluent, due to the release of counter-ions, in this case chloride, from the metal coagulant. Per active metal ion, three chloride ions are released into the water. With the addition of 1 g/m^3 of active Fe^{3+} with a molecular weight of 55.8 g/mol, which equals $1.79 \cdot 10^{-2}$ mol $5.38 \cdot 10^{-2}$ mol Cl^-, equal to 1.9 g Cl^-/m^3 is added to the effluent.

As can be calculated, the addition of organic polymers has less impact on the salt content of the remaining primary effluent. Three out of the four HMW cationic polymers that were used in the experiments had a charge density of +24 weight percentages. This means that 24% of the total weight of these polymers consists of cationic monomers. Irrespective of the molecular weight of the entire polymer the chlorine content can be calculated based on the weight of the monomers. The molecular weight of a cationic monomer is 163.4 g/mol, while a non-ionic monomer weighs 71 g/mol. 24 weight percentages equals $1.47 \cdot 10^{-3}$ mol of cationic monomer per gram polymer. Since per cationic monomer one chlorine ion is released the chlorine content per mg cationic polymer is $1.47 \cdot 10^{-3}$ x 35.4 g Cl^-/mol = 0.05 g Cl^-. Thus, the addition of 1 gram of active polymer per litre results in only 2.7% of the increase in salinity compared to the addition of ferric chloride.

Phosphate removal

It should be noticed that if metal salts are replaced by organic polymers precipitation of phosphate does no longer occur in the pre-treatment. Only the 20 - 30% of the phosphorus that is related to particulate material will be removed by flocculation. In applying organic polymers in the pre-treatment an additional phosphate removal step in the post treatment should be designed in order to meet the effluent requirement for P (< 1 mg P/l). In case of a biological post-treatment, the introduction of biological phosphorous removal would probably be most suitable.

Sulphide removal

In the case that sulphides may be present in the wastewater, these will not be precipitated anymore when metal salts are replaced by organic polymer flocculants. In specific cases, when high loads of sulphates originating from industrial activities (e.g. fish processing industries) are discharged into the sewer system, corrosive, smelly and even toxic sulphide concentrations may occur in the wwtp influent due to hydrolysis in anaerobic sewers. The sulphides have to be removed to protect wwtp personnel towards toxic sulphide concentrations, to prevent corrosion of metal objects at the wwtp and to prevent odour problems at the treatment plant and its surroundings. Biological sulphur or physical sulphide removal techniques instead of the metal precipitation have to be applied to solve this problem.

Toxicity of by- or degradation products of organic polymers

As earlier mentioned, by-products or (bio)degradation products of synthetic organic polymers may be toxic for the aquatic environment and humans. Polymers should be used and handled carefully. Optimal addition and mixing is essential for a proper flocculation process and prevents that active polymers are released to surface water. Dosing equipment should be safe to handle by employees and should prevent the diffusion of (powder) polymers to the air.

Turbidity controlled dosing strategy

An interesting consequence is that for HMW polyacrylamides the turbidity removal increases gradually with an increasing polymer addition. This phenomenon may offer a possibility to control the particle removal of unit operations for pre-treatment.

Mels [Mels (2001)] expands this theory with the investigation into the possibilities of using online-turbidity measurements to monitor the particle concentration in the influent and effluent of a pre-treatment unit operation with organic polymers. In addition, the application of a feed-forward polymer dosing strategy based on a fixed polymer to influent-turbidity ratio was tested (see figure 5.20).

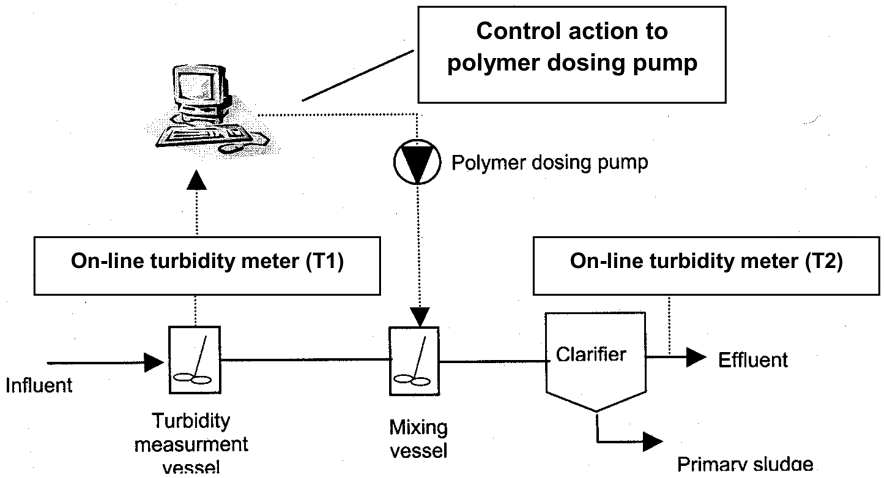


Figure 5.20: Schematic impression of the tested turbidity controlled dosing strategy

The described experiments confirm that online turbidity measurements can be used to quantify the particle concentration. For both, influent and flocculated primary effluent a linear relation was found in a wide range of $COD_{particles}$ (100 - 900 mg O_2/l) and turbidity (50 - 450 NTU). Online turbidity measurements showed that the particle concentrations in the tested municipal wastewater varied significantly. During dry weather conditions the turbidity fluctuated from 100 to 400 NTU, while in rainy periods fluctuations of 100 to >1000 NTU were measured.

The experiments with turbidity-related polymer dosing showed that this method is technically possible and results in higher removal efficiencies compared to the addition of a fixed concentration per litre. Turbidity-related dosing results in a more stable dosing system because overdose at low influent turbidity is prevented. Turbidity-related dosing offers a tool to control the $COD_{particles}$ and $BOD_{particles}$ removal and therefore may be interesting from the perspective of denitrification in a down-stream biological treatment system.

5.6 Conclusions

The experimental study shows that cationic organic polyelectrolytes can be used for flocculation of municipal wastewater. The addition of an anionic polymer did not show significant turbidity removal. From the tested cationic polyamine- and polyacrylamide-based organic polymers, especially the HMW polyacrylamides are interesting for practical application because they are effective at relatively low additions (5 - 10 mg/l). For a LMW polyamine-based polymer also high turbidity removals were found, but at higher required additions (20 - 30 mg/l).

Flocculation of particles in wastewater is generally described by two main coagulation/flocculation mechanisms, electrostatic coagulation and bridging flocculation. Considering the removal pattern of both polymers the LMW polymer shows an electrostatic coagulation type of behaviour, while flocculation by HMW polymers is governed by bridging flocculation.

The addition of HWM cationic polymers results in large, well settling flocs. The sludge flocs formed by the addition of LMW cationic polymers are significantly smaller. Filterability tests with the Filtration-Expression cell showed that, , compared to regular primary sludge, the dewaterability of the flocculated sludge improves due to the addition of polymers and ferric chloride in the pre-treatment.

In contrast to metal salt additions, the addition of organic polymers does not affect the pH of the primary effluent.

If 1 mg/l of active HWM cationic polymer (+24 weight%) is applied, the increase in salt concentration of the primary effluent is in the order of +0.05 mg Cl^-/l. This in contrast to the addition of +1.9 mg Cl^-/l for dosing 1 mg Fe^{3+} per litre.

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CHAPTER 6 DIRECT COARSE MEDIA FILTRATION AS A PRE-TREATMENT TECHNIQUE FOR ADVANCED PARTICLE REMOVAL

SUMMARY

This chapter presents a description and the results of explorative experimental research into the application of direct coarse media filtration (DCMF) as a first treatment step in a wastewater treatment system. The aim of the experimental research was to investigate the feasibility of DCMF as a pre-treatment step for advanced particle removal. Various filter configurations were investigated in large scale pilot-plant filters at the wwtp Leiden-Noord. The removal characteristics for suspended and colloidal material were investigated as well as operational conditions for discontinuous and continuous filters in up-flow and down-flow.

The experiments revealed that the discontinuous filters, operated at maximum filtration rates of 10 $m^3/m^2 \cdot h$, had relatively short runtimes due to clogging. The continuous filter could be operated at filtration rates of 12.5 $m^3/m^2 \cdot h$ without clogging.

The filters produced a filtrate with a consistent quality of low concentrations of solids and low turbidity. When no coagulants were added, only suspended solids were removed and most of the colloidal material past the filter columns. Adding metal salt coagulants resulted in phosphorous precipitation and improved total particle removal. Addition of organic polymeric flocculants resulted in the formation of large structures that caused cake layer formation on the top or bottom layer of the filter, resulting in fast clogging.

Discontinuous direct influent filtration is not advised as a pre-treatment technique, based upon the research results. To make discontinuous direct wastewater filtration suitable for practical application, much more research and technological adjustments have to be done.

When coarse media filtration can be applied continuous filtration would be preferred since the removal efficiencies of the continuous and discontinuous filter configurations did not differ significantly. Further research on the continuous operation and specifically, the clogging prevention by means of filter control and water quality monitoring is recommended in order to develop a possible practical application of this technique.

6.1 Introduction

In the previous chapter, increasing the average particle floc size in wastewater was investigated through flocculation processes. From the preliminary evaluation study in chapter 2, different kinds of physical-chemical pre-treatment techniques, including direct raw wastewater filtration, were recommended for advanced particle removal and the removal of flocculated particle formations. This chapter describes explorative research into the application of direct coarse media filtration for advanced particle and floc removal.

Experiences with direct wastewater filtration
Research on filtration techniques for pre-treatment of wastewater is rather rare and application in practice is hardly known. Only a small number of research activities into direct influent filtration with up-flow floating filters have been reported e.g., in Norway, Australia, Japan and the USA [Bigot *et al*. (1999), Becker *et al.* (2000), Booker *et al*. (1996), Gray *et al.* (1998), EPA (1979), Liao (1998), Mouri and Niwa (1993), Ødegaard *et al*. (1998), (2000), (2001); Tanaka *et al*. (1995)]. Filtration for the treatment of primary effluent from sedimentation tanks has been investigated more often [e.g., Brown and Wistrom (1999)] while direct filtration is commonly known in drinking water production [e.g., Dijk, van, and Huisman (1996), Gary (1991)].

The pilot-plant investigations into direct filtration of municipal wastewater are limited and hardly useful for the Dutch situation. The research activities focused mainly on different small and large scale pilot-plant filtration experiments wherein columns (2 – 4 m in height) were half-filled with coarse and porous filter material (diameter = 1 – 8 mm). In addition, plastic rings, tubes (2 – 4 mm in diameter) or biofilm carriers have been tested as floating or immersed filter material. Up-flow and down-flow filtration test were conducted, in some cases with addition of coagulants or flocculants.

From the investigations, sedimentation, (bio)flocculation and adsorption or attachment of particles and flocs to the filter material were determined as major filtration processes. The results of the experiments reported in the literature were positive. High filtration rates (5 – 40 $m^3/m^2 \cdot h$) were applicable with resulting removal efficiencies for suspended solids of 80-90%.

The backwash frequency was dependent on the applied filtration rate and the suspended solids content of the influent. At high rates, every 5 to 10 hours the filter columns had to be backwashed. In case of filtration rates of about 5 $m^3/m^2 \cdot h$, the filter had to be cleaned every 55 to 65 hours [Mouri and Niwa (1993), Tanaka *et al.* (1995) and Ødegaard *et al.* (1997), (1998), (2000)].

Table 6.1 shows average removal efficiencies found from experiments with filtration of raw wastewater.

Table 6.1: Average removal efficiencies of experimental direct filtration of wastewater

parameter	removal efficiency
suspended solids	80 – 90%
BOD	40 – 50%

based on [Mouri and Niwa (1993), Tanaka *et al.* (1995) and Ødegaard *et al.* (1997), (1998), (2001)]

Despite the limited experimental investigations and the lack of practical experiences, direct wastewater filtration may have advantages over the commonly used sedimentation. At first, it is possible to achieve a high particle removal even without the use of chemicals. Secondly, filtration can be applied very compactly, which can be an important aspects in densely populated areas like The Netherlands.

Aim of the experimental investigation

The aim of the experimental research as described in this chapter was to explore the possibilities and applicability of direct influent filtration as a pre-treatment step for advanced particle removal. The removal efficiencies for suspended and colloidal material as well as operational and process conditions of different filter configurations were investigated.

6.2 Materials and methods

Two types of large-scale pilot-plant filters were tested at the wwtp of Leiden-Noord in the Netherlands. The first one was a discontinuous filter system that could be loaded in up- and down-flow mode. The second filter was an up-flow continuous washed filter. Both filter systems were tested with raw, unsettled wastewater with coarse non-floating filter materials.

Wastewater composition

The influent of the filter systems was the raw unsettled wastewater of wwtp Leiden-Noord, the Netherlands after coarse material had been removed by a 6 mm screen (without sand removal). The quality of the wastewater was influenced by daily and weekly variations by climatical changes and water use e.g., storm weather flows played a dominant role.

Table 6.2 gives an overview of the wastewater characteristics during the testing period from October 1998 to June 1999.

Table 6.2: Indicative wastewater composition at wwtp Leiden-Noord

parameter	indicative value	fluctuations
turbidity (NTU)	125	75 – 250
suspended solids (TSS/l)	110	50 – 250
COD_{total} (mg O_2/l)	380	250 – 650
BOD_5 (mg O_2/l)	140	120 – 210
P_{total} (mg P/l)	6.5	3.0 – 8.0
$N_{kjeldahl}$ (mg N/l)	40	30 – 50
pH (-)	7.2	6.5 – 7.8
temperature (°C)	14	11 – 19

Discontinuous pilot-plant filter

For the experimental research, a large-scale pilot-plant was used consisting of a perspex filter column with an internal diameter of 0.44 m (filtration surface: 0.152 m^2) and a total column height of 3.5 m (see figure 6.1).

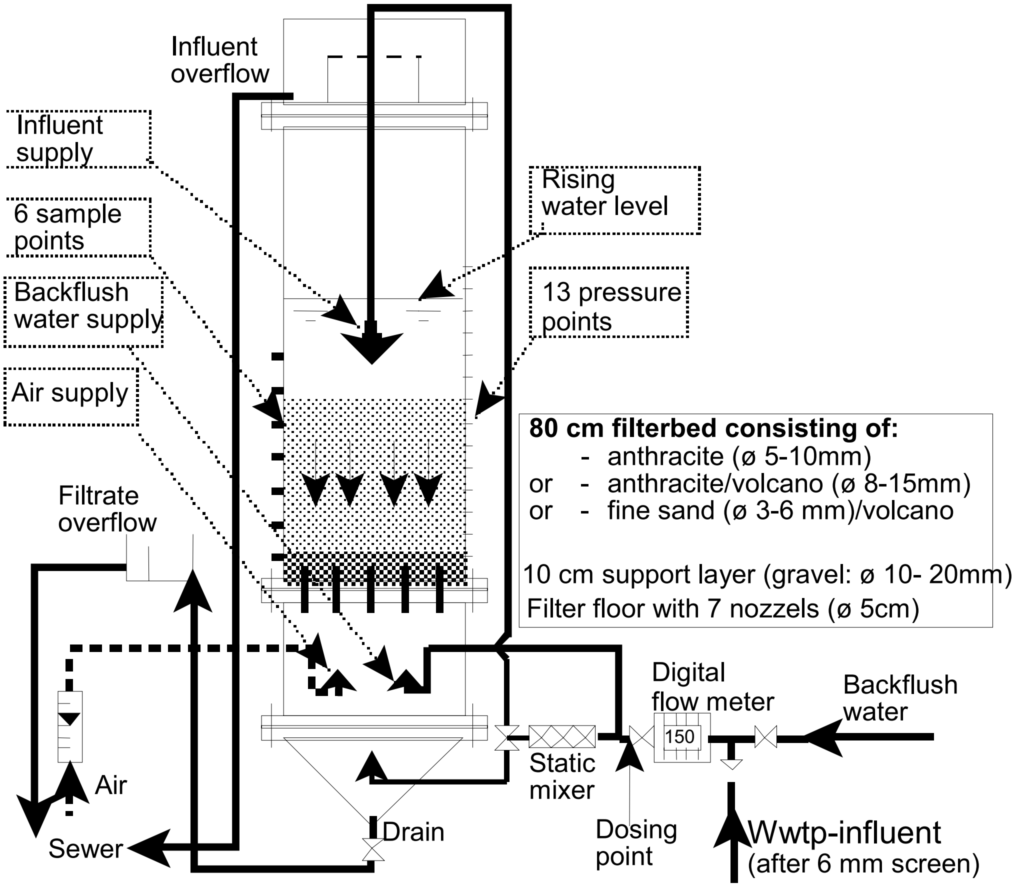

Figure 6.1: Schematic drawing of the pilot-plant for discontinuous direct wastewater filtration, applied as down or up-flow filter

This pilot-plant could be operated as an up-flow and down-flow filter. For up-flow filtration, the influent was fed at the filter bottom through seven nozzles with 5 mm passage holes to flow in upward direction through a support layer of coarse gravel (100 mm) into the filterbed. The filtrate was leaving the system by the overflow system at the top of the filter column. The up-flow filtration rate was set and controlled by a digital flow-meter. For down-flow filtration the wastewater was pumped up to an inlet pipe and was introduced 200 mm above the filterbed. With a rising water level in the filter column, the flow through the filterbed was kept at constant velocity. The water was passing through the (multi-media) filterbed and the filter bottom into a filtrate overflow. The nozzles in the filter floor were protected against clogging by the gravel support layer.

The inlet pipe was equiped with a static pipe mixer for coagulant or flocculant addition. No flocculator was applied since filtration requires small flocs in the inlet, which will flocculate within the filterbed [e.g. Ødegaard (1987)].

Within the filter column, different filterbed configurations were tested. In contrast to previous conducted investigations into filtration of raw wastewater based on [Mouri and Niwa (1993), Tanaka *et al.* (1995) and Ødegaard *et al.* (1997), (1998), (2001)], not floating but high-density filter material was used. The applied filter materials with the characteristic applications, grain size, density and the average porosity are presented in table 6.3.

Table 6.3: Characteristics of applied filter media

filter material	**place of application**	**flow direction**	**grain size** (mm)	**density** (x1,000 kg/m^3)	**average porosity** (%)
gravel	support layer	Down / up	10 - 20	±2.6	> 65
quartz sand, fine	bottom layer	Down	3 - 6	±2.6	±40
quartz sand, coarse	bottom or single layer	Up	5 - 8	±2.6	±45
hydro-anthracite	single or top layer	Down / up	5 - 10	±1.4	±50
aqua-volcano**	top layer	Down	8 - 15	±1.1	>50

*all products supplied by Aqua-Techniek BV, Rockanje, the Netherlands; ** expanded clay product.

After halting the filtration process due to clogging or breakthrough, the filterbed was cleaned by up-flow backwashing with air (3 to 5 bar) and water (at backwash rates up to 80 $m^3/m^2 \cdot h$) or a combination of both. The concentrated backwash water was analysed and returned to the wwtp influent.

The development of the headloss over the filterbed was measured by 13 sample points connected to a manometer system where the pressure drop could be monitored. The quality changes in the filterbed could be analysed through 6 sampling points over the filterbed. The influent and filtrate were sampled separately to analyse quality parameters; turbidity could be measured on-line during the filter run.

Up-flow coarse media filtration - For the up-flow filter, two filterbed configurations were tested. The first filterbed consisted of a 900 mm coarse sand (grain ø 5 - 8 mm) bed. The second bed with a height of 900 mm consisted of coarse hydro-anthracite (ø 5 - 10 mm). Both filters were tested during three month (October to December) for removal efficiencies and operational conditions at three flows: 0.6, 1.1 and 1.5 m^3/h, corresponding to filtration rates of 4, 7 and 10 $m^3/m^2 \cdot h$. All filter runs were conducted without the addition of any chemicals.

During the filter run the water quality parameters of influent and filtrate as well as the headloss over the filterbed were monitored. The maximum tolerated headloss over the up-flow filterbed was influenced by the type of filter material and the applied bed height. Above these tolerable headlosses the whole filterbed was lifted up and transported in upward direction through the filter column.

Down-flow coarse media filtration - Three different filterbeds were investigated in down-flow mode with and without the addition of a coagulant ($FeCl_3$) or a cationic polymer (Cytec C492 HMW: $8.0 \cdot 10^6$ g/mol, 24% charge density), as described in table 6.4.

Table 6.4: Applied filterbed configurations for down-flow filtration

name of filterbed	filterbed configuration	dosage
ANTHRA	single layer: 800 mm of hydro-anthracite	one
ANTHRA + VOLC	top layer: 200 mm of volcano bottom layer: 550 mm of anthracite	none
ANTHRA + VOLC + Fe	top layer: 200 mm of volcano bottom layer: 550 mm of anthracite	10 mg Fe^{3+}/l
SAND + VOLC	top layer: 250 mm of volcano bottom layer: 450 mm of fine sand	none
SAND + VOLC + Fe	top layer: 250 mm of volcano bottom layer: 450 mm of fine sand	10 mg Fe^{3+}/l
SAND + VOLC + PE	top layer: 250 mm of volcano bottom layer: 450 mm of fine sand	1-2 mg PE/l

PE = cationic HMW polymer (see chapter 5)

During the six-month testing period with down-flow filtration, the applied filtration rates were: 4, 7 and 10 $m^3/m^2 \cdot h$.

The tolerated headloss over the down-flow filterbed was set to be 900 mm; nevertheless, it was possible to increase the headloss to more than 2 m (total column height above the filterbed).

Continuous pilot-plant filter

The continuous flushed filter system - type *ASTRASAND* AS-070 (from Paques, The Netherlands) - is a variation on the Dyna-sand filter system [Kramer et al. (1989)] and consists of a steel filter column with a height of 4 metres and an internal diameter of 950 mm (see figure 6.2). This type of filter system has been used on practical scale in the drinking water production and wwtp effluent polishing.

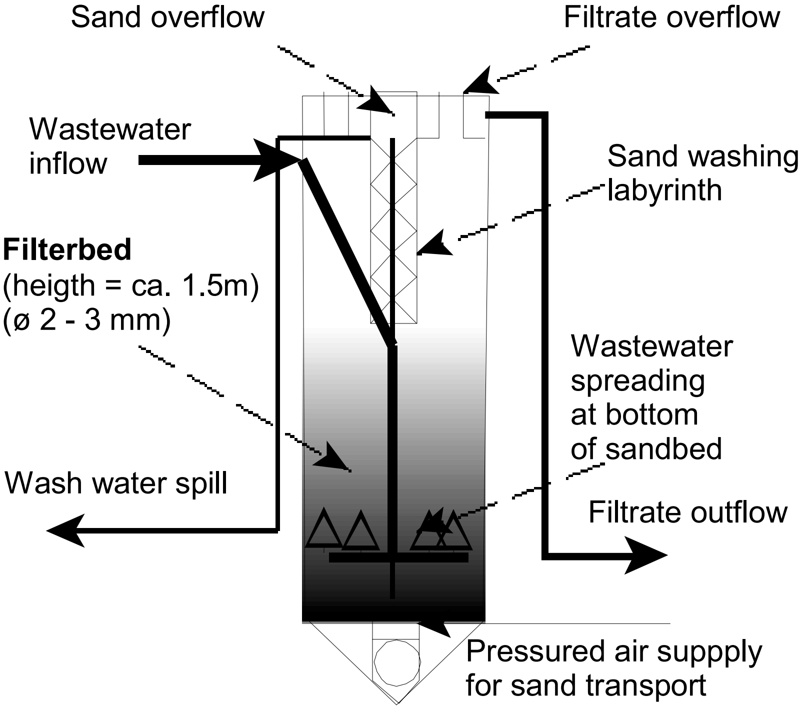

Figure 6.2: Schematic presentation of the continuous filter system

The column was filled with 1.5 m of sand (with an average grain diameter of 2 – 3 mm). The resulting effective filter surface is 0.70 m^2.

The filter is fed with pre-screened (mesh width = 6 mm) wastewater at the bottom of the filterbed and is flowing in upward direction through the filterbed. At the top the filter column, the treated filtrate is extracted from the filter system.

In this system, the filterbed is continuously washed and circulated from bottom to top. From the bottom of the filter column, the polluted filter sand is removed and is transported via a pressured air (with 0.7 – 1.4 Nm^3/h at 5 bar) driven (mammoth) pump to the sand washing labyrinth at the top of the filter column. In the labyrinth the sand is flushed and cleaned by a counter stream of water. The concentrated wash water is rejected from the system and has to be treated as primary sludge. The clean sand is returned to the filter at the top of the filterbed.

The wash water consumption is constant at a flow of 0.7 m^3/h. The filtration rate could be regulated by changing the influent flow via a pump. The sand circulation velocity can be controlled between the advised settings of 2 and 15 mm per minute by changing the air flow trough the mammoth pump.

During the testing period, filtration rates of 4, 7, 10 and 12,5 $m^3/m^2 \cdot h$ were applied.

The headloss over the continuous cleaned filterbed was measured via a pressure tube next to the filter column. The sand circulation velocity was determined by measuring the downward motion of the sand with an introduced rule.

Analysis

During the experiments, the filter influents as well as the filtrates were analysed for turbidity (HACH Turbidimeter, with batch and on-line via surface scanners), suspended solids, COD_{total}, paper filtered COD (COD analysis of filtrate from SS-test < 4.4 μm) and P_{total} (all chemical analysis by MERCK Photometer SQ200). On some occasions additional analysis on $COD_{dissolved}$ (membrane filtered < 0.1 μm), BOD_5 and $N_{Kjeldahl}$ were conducted.

6.3 Experimental research results

Discontinuous filtration

Up-flow filtration – The filtration tests started with up-flow filtration, since the natural stratification of the filterbed (fine filter material at the top and coarse material at the bottom) is most favourable for up-flow filtration [Graaf, van der, and Nieuwenhuijzen, van (1998), Metcalf&Eddy (1991)]. Filter runtimes were terminated by clogging of the filterbed with up-flow filtration through a sand and followed by an anthracite bed. No breakthrough of solids was observed during testing the up-flow filtration system. During several tests, complete clogging (maximum tolerable headloss) of the filter was reached; this was followed by the break up of the filterbed). This spectacular phenomenon occurred with the sand filter at a filtration rate of 10 $m^3/m^2 \cdot h$ after an average runtime of 220 minutes. Because of the even lower maximum tolerable headloss of the filter bed compared to the sand bed, the runtimes of the anthracite filter were already finished after 160 minutes (at a rate of 10 $m^3/m^2 \cdot h$). The length of the possible filter runtime was largely assessed by clogging due to higher incoming suspended solids loads. Especially, shortly after a rain fall event the filters clogged rapidly.

The filterbeds were only loaded with solids over the first 100 to 150 mm from the bottom part of the bed of the (sand?) bottom. The remaining filterbed layer did not remove particulate material and never became clogged.

Up-flow backwashing for cleaning the filter was very difficult. Large quantities of air and backwash water had to be used for cleaning the beds after the filtration runs were terminated by clogging. Especially the sand bed was inefficiently cleaned by backwashing. The ratio of necessary wash water to the amount of produced filtrate reached up to 60% for the up-flow sand filter. Because of these inefficient filtration runs, it was decided to replace the sand bed by an anthracite bed and later on to change from up-flow to down-flow filtration.

Removal efficiencies of the sand and anthracite filter, both without chemical dosage, were similar. The produced filtrate had low concentrations of suspended solids and a constant composition over time. Because of the inefficient filtration process with up-flow sand filters, only the up-flow anthracite bed (ANTHRA UP) is compared to the down-flow filter configurations in the following description of the results of filtration tests at a velocity of 10 $m^3/m^2 \cdot h$ (see figures 6.3, 6.4 and 6.5).

Down-flow filtration - After changing the flow direction through the anthracite bed from up-flow to down-flow, the average runtimes were shortened from 160 minutes to 70 minutes, as is shown in the first two black bars in figure 6.3. This was caused by faster clogging of the finer top layer of the filterbed (due to stratification of the filterbed) and the formation of a cake layer above the filterbed due to the settling of suspended material.

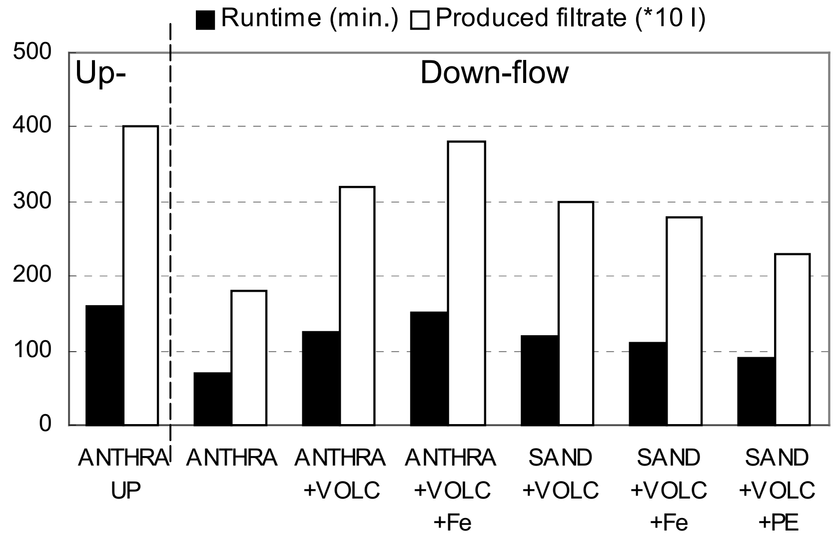

Figure 6.3: Average runtimes (minutes) and filtrate production (×10 l) per tested filter configuration, at a filtration rate of 10 $m^3/m^2 \cdot h$

To solve the clogging problem, an extra layer of 250 mm of coarse aqua-volcano (a type of expanding clay) was added to the anthracite bed. The dual media volcano-anthracite filter performed much better with regard to runtimes (t = 125 min.), even with the addition of iron chloride (t = 150 min.), as is shown with the third and forth black bar in figure 6.3. Cake formation above the filterbed did not occur anymore. The whole vulcano layer and the first 10 to 15 centimetres of the anthracite layer showed particle deposition. After metal salt addition, the complete height of the filterbed was used.

To increase the removal efficiency of the filter, the anthracite bed was replaced by a bed of finer sand. With this volcano-sand filter, runtimes of approximately 120 minutes at a rate of 10 $m^3/m^2 \cdot h$ were achieved. Addition of 10 mg Fe^{3+} per litre resulted in similar runtimes (t = 110 minutes). After replacing the iron chloride by a cationic high molecular polymer (2 ppm C492HMW), the runtimes were reduced to 90 minutes due to clogging of the top zone of the bed and settling of flocs above the filterbed.

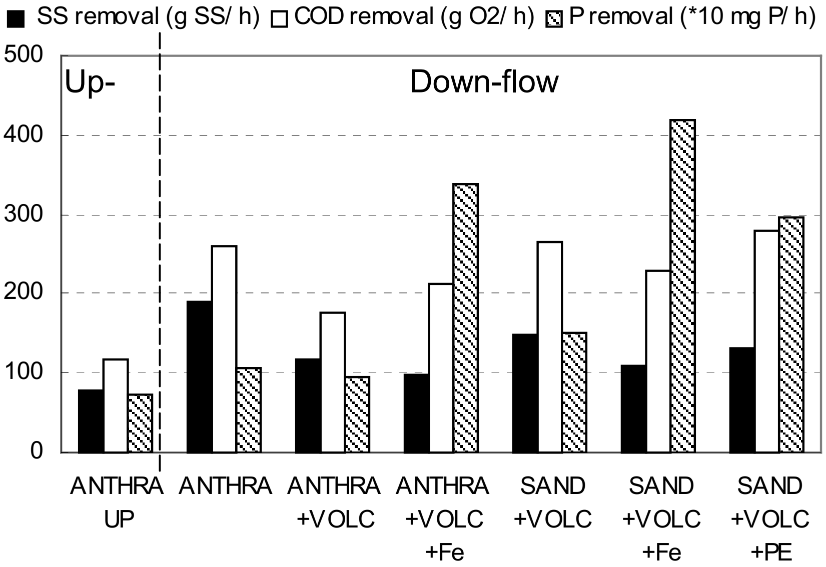

Figure 6.4: Removal rates per tested filter configuration, in loads per hour (g or ×10 mg/h), at a filtration rate of 10 $m^3/m^2 \cdot h$

The removal characteristics of the filter configurations are presented in figure 6.4. The first three bars represent the removed loads per hour of suspended solids, COD and phosphorus for the up-flow anthracite filter. Compared to the down-flow filters, the up-flow filter performed less well under these conditions. The change from up-flow to down-flow anthracite filtration resulted in an increased and faster removal of suspended solids (from approximately 80 g/h to 190 g/h), COD (from 120 g/h to 260 g/h) phosphorus (750 mg/h to almost 1,100 mg/h) and turbidity (also shown in figure 6.5).

The extension of the filter with the volcano layer did not result in better removal efficiencies. The replacement of the coarse anthracite by the finer sand as the bottom layer resulted in slightly increased removal rates for COD and phosphorus.

The addition of iron chloride led to an increase in COD and phosphorus removal due to more advanced particle removal and precipitation. The analysis of suspended solids showed a lower removal rate with iron addition compared to filtration without the addition of chemicals (see figures 6.4). This was mainly caused by the presence of active Fe^{3+} in the filtrate and the (re)formation of hydroxide and phosphorus flocs in the sampling bottles. Polymer addition resulted in high removal rates for suspended solids (135 g/h) and, especially, COD (280 g/h). The increased phosphorus removal was mainly caused by the elimination of colloidal and suspended material.

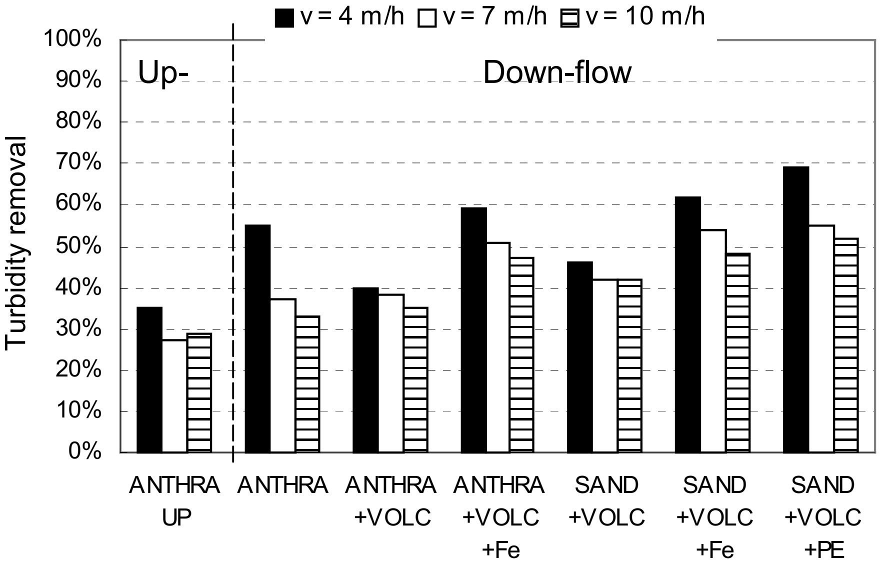

Figure 6.5: Turbidity removal efficiency per tested filter configuration, at filtration rates of 4, 7 and 10 $m^3/m^2 \cdot h$

The influence of the applied filtration rate on the turbidity removal is shown in figure 6.5. The removal efficiency for turbidity was slightly decreasing with increasing filtration rates. The strongest decrease resulted from the change in velocity from 4 to 7 $m^3/m^2 \cdot h$. The increase in the filtration rate from 7 to 10 $m^3/m^2 \cdot h$ had hardly any impact on the turbidity removal efficiency.

Backwashing - One of the major reasons for changing the filter configuration from up-flow to down-flow filtration was the difficult cleaning procedure. As mentioned before, the up-flow sand filter used up to 60% of the produced filtrate flow for backwashing of the filter (see example). The replacement of coarse sand by anthracite reduced the wash water/filtrate-ratio to approximately 30% providing a more effective bed expansion of the anthracite layer. With down-flow filtration, the wash water consumption could be further decreased to a more efficient 10% (with the volcano-sand filter) of the produced filtrate.

Example of the backwash water consumption (not optimised)

During operating the filterbed with coarse sand at a filtration rate of 4 $m^3/m^2 \cdot h$ (equal to 608 l/h) a runtime of 5 hours was reached. So, the production of filtrate was **3,040 litres**.

Within the backwashing procedure an air/water flush was applied and a single 8½ minute water flush at a backwash rate of 65 $m^3/m^2 \cdot h$ (equal to 9.800 l/h) was applied. The amount of **consumed backwash water** for this backwash was approximately **1,390 litres** of clean filtrate.

The resulting **backwash water percentage** is equal to 1,390 / 3,040 * 100% = **46%**

Concentrated filtration sludge - Pre-thickening the wash water in the filter column itself after only flushing the sludge from the filterbed by single air washing, resulted in a concentrated sludge stream with a dry solids content of more than 1%. This filtration sludge is assumedly similar to primary sludge from sedimentation tanks, although the content of small particles or even colloidal fractions in the filtration sludge would be higher.

Detailed removal efficiencies by characterisation of influent and filtrate
Similar to the fractionation of wastewater as described in chapter 4, the influents and filtrates of the experiments with the volcano-sand filter were analysed in more detail to gain a better understanding of the filtration process and the removal characteristics. The fractionation was done by splitting the COD into three particle-related fractions. First, total COD was measured, then the filtrate from the paper filter of the suspended solids analysis (pore size = approximately 4.4 μm) was measured and finally the filtrate from a membrane filter (with pore size of 0.1 μm) was analysed for COD. The results of this COD fractionation are presented for the filter influent and the produced filtrate in table 6.5.

Table 6.5 and figure 6.6 show the detailed removal efficiencies of filter runs without the addition of chemicals. 140 mg O_2/l (33%) of the total 430 mg O_2/l on COD in the influent were present as suspended material (> 4.4 μm = COD_{total} - $COD_{<4.4\mu m}$). In this example, the colloidal COD fraction, between 0.1 μm and 4.4 μm, consisted of 110 mg O_2/l (26%). In the influent 42% of the COD_{total} was soluble ($COD_{<0.1\mu m}$ = 180 mg O_2/l) and 58% of COD_{total} was related to all suspended and colloidal particles.

Table 6.5: Detailed analysis of influent and filtrate without chemical addition

analysed parameter	influent	filtrate	removal efficiency
turbidity (NTU)	117	58	50%
suspended solids (mg SS/l)	103	43	58%
COD_{total} (mg O_2/l)	430	340	21%
$COD_{<4.4\ \mu m}$ (paper filter, mg O_2/l)	290	280	3%
$COD_{<0.1 \mu m}$ (membrane, mg O_2/l)	180	180	0%
BOD_5 (mg O_2/l)	145	140	3%
$N_{kjeldahl}$ (mg N/l)	44	44	0%
P_{total} (mg P/l)	6.8	6.2	9%
down-flow volcano-sand filter at v = 7 $m^3/m^2 \cdot h$; pH = 7; T = 18.5°C			

In the volcano-sand filter (at a filtration rate of 7 $m^3/m^2 \cdot h$), 21% of the total COD was removed. In the filtrate 60 mg O_2/l was analysed as suspended material (> 4.4 µm), resulting in a removal efficiency of 57% on suspended COD. The colloidal COD fraction in the filtrate was determined at 90 mg O_2/l, corresponding with 10% removal of colloidal COD in the filter. Soluble COD was not removed, whereas 3% of BOD_5 was eliminated in the filter.

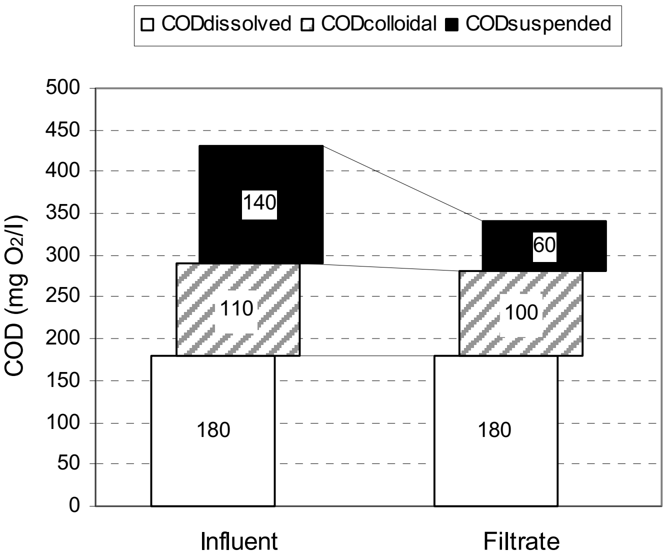

Figure 6.6: Fractionated COD in the filter influent and the produced filtrate (as shown in table 6.5)

The ratio of suspended COD to the suspended solids content can be calculated to be 1.36 in the influent and 1.39 in the filtrate. The BOD_5/N-ratio in the influent was 3.3 and in the filtrate 3.2. Phosphorus was hardly removed.

Table 6.6 presents the characterisation of the influent and the filtrate of the volcano-sand filter with a dosage of 10 mg Fe^{3+}/l at a filtration rate of 7 $m^3/m^2 \cdot h$. The removal efficiencies increased due to the addition of the inorganic coagulant compared to the filter performance without chemical addition.

Especially the removal of the colloidal fraction increased due to coagulation and/or flocculation.

As is shown in table 6.6 and figure 6.7, the influent contained 160 mg O_2/l (35% of COD_{total}) suspended COD and 100 mg O_2/l colloidal COD (22% of COD_{total}). 43% of the COD was present in soluble form and 57% was bound to or incorporated in particulate matter. In the filtrate, 40 mg O_2/l (15% of COD_{total}) was analysed as suspended COD and of this suspended COD 75% is removed in the filter. 50 mg O_2/l (19% of COD_{total}) was present in the filtrate as colloidal COD, resulting in a removal efficiency for this fraction of 50%. Even soluble COD was removed (15%), probably due to the removal of small colloidal particles, just like 13% of BOD_5.

Table 6.6: Detailed analysis of influent and filtrate with $FeCl_3$ addition

analysed parameter	influent	filtrate	removal efficiency
turbidity (NTU)	123	53	57%
suspended solids (mg SS/l)	115	39	66%
COD_{total} (mg O_2/l)	460	260	43%
$COD_{<4.4\ \mu m}$ (paper filter, mg O_2/l)	300	220	27%
$COD_{<0.1\mu m}$ (membrane, mg O_2/l)	200	170	15%
BOD_5 (mg O_2/l)	160	140	13%
$N_{kjeldahl}$ (mg N/l)	48	46	4%
P_{total} (mg P/l)	7.2	3.4	53%

down-flow volcano-sand filter with 10 mg Fe^{3+}/l at v = 7 $m^3/m^2 \cdot h$; pH =7.5; T = 18°C

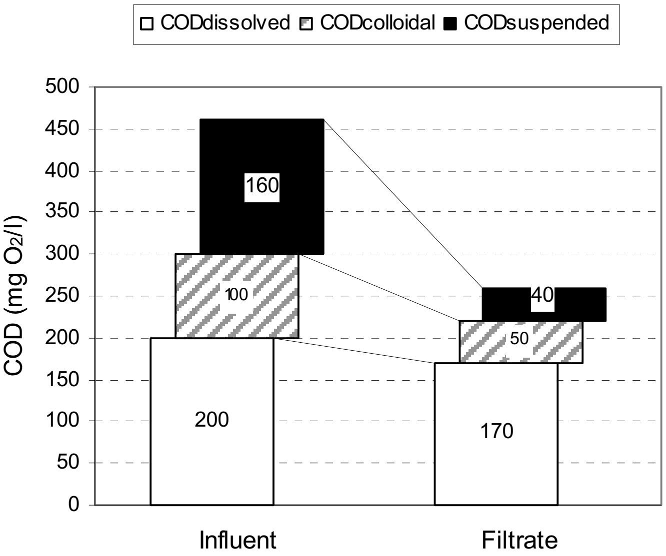

Figure 6.7: Fractionated COD in the filter influent and the produced filtrate of a down flow filter with chemical addition (see table 6.6).

The $COD_{suspended}$/SS-ratio in the influent in this experiment was 1.39, whereas in the filtrate it was calculated to be 1.02. The BOD_5/N-ratio decreased after filtration from 3.3 in the influent to 3.0 in the filtrate, indicating a higher removal of suspended or colloidal nitrogen components. Due to phosphorus precipitation with iron, P-removal was measured to be 53%.

Continuous filtration

Operational results – The continuous filter could be operated at a filtration rate of 12.5 $m^3/m^2/h$ with a wash water consumption of 7% of the produced filtrate flow and a sand circulation velocity of 8 mm per minute. 35% of turbidity was removed and average removal efficiencies of 45% for suspended solids and 18% for COD were measured. Increasing the filtration rate from 7 to 10 and 12.5 $m^3/m^{2\cdot}h$ did not influence the removal efficiencies of the filter negatively. A decrease in sand circulation led to increasing removal efficiencies for turbidity, suspended solids and COD (for details, see appendix H).

The measured headloss over the filterbed was limited, but was dependent on the applied filtration rate and the sand circulation velocity. Table 6.7 shows the influence of the filter settings on the headloss over the filterbed.

Table 6.7: Measured headloss over the filterbed at different filter settings

filter settings (filtration rate and sand circulation velocity)	**measured headloss of over the filterbed**
7 m/h and 4 - 5 mm/min.	< 50 mm
7 m/h and 5 - 6 mm/min.	not detectable
10 m/h and 4 – 5 mm/min.	200 mm
10 m/h and 5 – 6 mm/min.	150 mm
10 m/h and 6 – 7 mm/min.	120 mm
10 m/h and 8 – 9 mm/min.	100 mm
12.5 m/h and 8 - 9 mm/min.	250 mm

The limited headloss indicates a possible further increase of the filtration rate or a further decrease of the sand circulation velocity, since the tolerable headloss in this system with a filter bed depth of 1,500 mm is about 1 m (in compliance with a safety factor of 1.5).

Clogging of the sand wash labyrinth - Principally, the continuous filter should not be allowed to clog during operation since it is continuously washed. Nevertheless, in case of high suspended solids loads in the influent, the sand wash labyrinth can be overloaded with sticky sand and mud balls. During the testing period this occurred in some cases after intensive rain events with resulting high suspended solids content in the incoming wastewater (see the example in figure 6.8). Under those circumstances, the sand circulation was cancelled and the filtration process stopped and the sand wash labyrinth had to be cleaned manually.

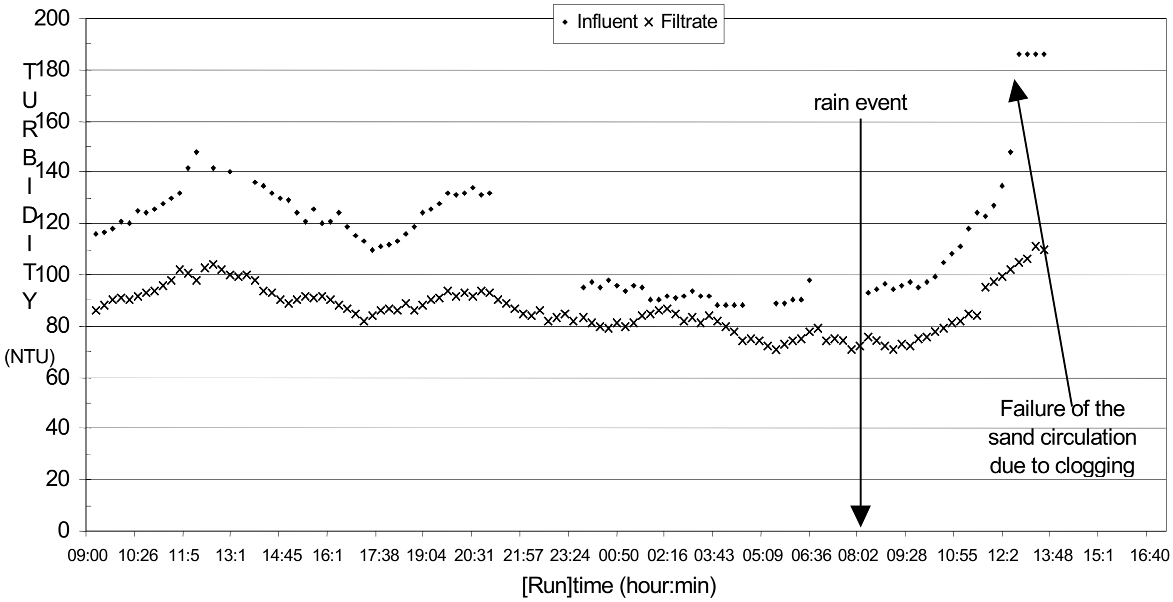

Figure 6.8: Example of the development of the turbidity in influent and filtrate of the continuous filter. This example shows the influence of the increase of turbidity in the influent (probably caused by a rain event) on the turbidity of the filtrate and indicates the failure of the sand circulation system at 13:00 h.

Detailed removal efficiencies by means of COD fractionation - To gain insight in the distribution of the COD over the particle fractions and the removal efficiencies of the continuous filter on the specific particle fractions, a similar COD fractionation was carried out as described before.

In this specific filtration run 24% of COD_{total} was removed by the filter, from 490 mg O_2/l in the influent to 370 mg O_2/l in the filtrate (see figure 6.9). The total COD in the influent consisted of 180 mg O_2/l of suspended COD (36% of total), 110 mg O_2/l of COD (27%) was related to colloidal particles and 37% (180 mg O_2/l) was dissolved COD. In the filtrate, COD_{total} could be attributed to 100 mg O_2/l of the suspended (27%), 100 mg O_2/l (27%) of the colloidal and 170 mg O_2/l (47%) of the dissolved fraction.

Of the suspended COD fraction 44% was removed by the continuous filter. The removal efficiency for the colloidal COD was 23%, while 7% of dissolved COD (probably related to ultra fine colloidals) was removed.

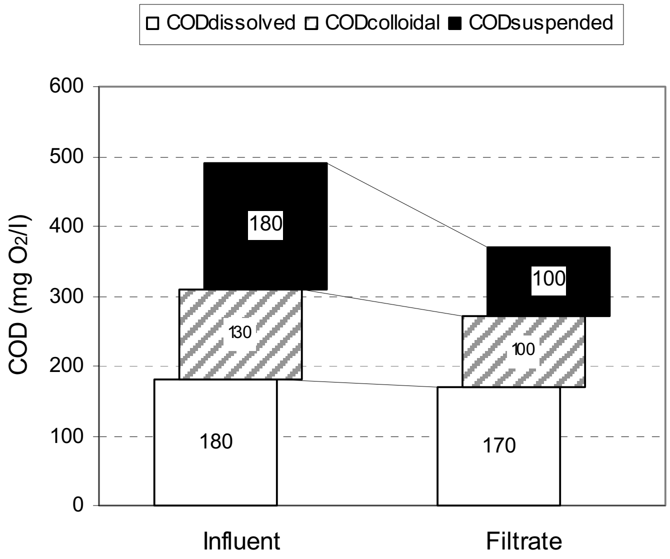

Figure 6.9: Fractionated COD in the filter influent and the produced filtrate of the continuous filter

6.4 Discussion

Practical operation – From an operational point of view, the tested filter configurations performed differently.

For application, the discontinuous filter is not easily suitable since the washing procedure is complicated, time consuming and water consuming. Especially the up-flow filter configurations had a high loss of water due to the difficult cleaning of the filterbed. The down-flow filters clogged rapidly with a resulting high backwash frequency.

The continuously washed filter is more comfortable to operate, since almost all settings are self-regulating. In case of practical application, automatic control of the filter and online influent quality monitoring is necessary to prevent the failure of the sand circulation, but overall the filters perform satisfactory.

Pre-treatment efficiency of direct coarse media filtration

In chapter 4, the maximum removal efficiency for particle related contaminants was introduced as a specific characteristic of the wastewater. The basic purpose of that theoretical efficiency is to predict if a certain wastewater can efficiently be pre-treated by means of physical-chemical particle removal techniques. On the other hand, it can be used to determine an achievable pre-treatment efficiency of a certain pre-treatment technique like direct coarse media filtration.

Based on the removal efficiencies of the filter configuration the achievable pre-treatment efficiency for COD can be calculated. Table 6.8 enumerates the pre-treatment efficiencies for the discontinuous down-flow filter without addition of iron chloride (as shown in table 6.5), with iron chloride (as shown in table 6.6) and the continuous filter without chemical addition.

Table 6.8: Achievable pre-treatment efficiency of direct coarse filtration configurations

filter configuration	**achievable pre-treatment efficiency for COD**
discontinuous down-flow filter	21%
discontinuous down-flow filter with 10 mg Fe^{3+}/l	43%
continuous up-flow filter	24%

The maximum possible removal efficiency for COD related to particles down to a size of 0.1µm of the wastewater of the wwtp Leiden-Noord is on average 59%. The direct filters without chemical addition only remove 35% to 40% of the maximum achievable particle removal. The filter configuration with the dosage of 10 mg Fe^{3+}/l results in an additional removal of almost 75% of the achievable pre-treatment potential of the wastewater.

6.5 Conclusions and recommendations

Conclusions

Discontinuous filter - From the experimental research into the techniques and the application of direct influent filtration, conducted with a large scale pilot-plant filter at wwtp Leiden-Noord, it was concluded that filtration of raw wastewater is feasible. In total eight filter configurations were tested with a discontinuous filter, showing the filtration runtimes were, being one to several hours, which was relatively short due to clogging of the filterbeds.

The filters produced a filtrate with a constant quality, however, with low concentrations of suspended solids and low turbidity. Without chemical addition hardly any colloidal and soluble material was retained on the filters, only suspended particulates were removed. After additions of ferric iron or a cationic polymer was it possible to remove more finer grained suspended matter and a high amount of colloidal material.

Continuous filter - The *ASTRASAND* filter could be operated continuously with a wash water consumption of 7% of the produced filtrate flow at a filtration rate of 12.5 $m^3/m^2/h$.

The maximum tolerable headloss of 1 m was never reached. In some occasions, the sand circulation was hindered by high suspended solids contents in the wastewater.

For the continuous filter the following removal efficiencies were found:

- 30% of turbidity was removed in average at filtration rates of 10 and 12.5 $m^3/m^{2\cdot}/h$ and removal efficiencies of 50% were measured at rates of 7 $m^3/m^{2\cdot}h$;
- The average removal efficiency for suspended solids was determined at 40% at a filtration rate of 12.5 $m^3/m^{2\cdot}h$ and approximately 50% at filtration rates of 10 $m^3/m^{2\cdot}h$ and 7 $m^3/^{2\cdot}h$;
- 15% of the incoming COD was removed by the continuous filter at filtration rates of 10 $m^3/m^{2\cdot}h$ and 12.5 $m^3/m^{2\cdot}h$. At filtration rates of 7 $m^3/m^{2\cdot}h$, more than 20% of COD was removed.

A decrease of the sand circulation had a positive effect on the removal efficiencies.

Recommendations for application

Discontinuous direct influent filtration is not advised as a pre-treatment technique based on the research results. To make discontinuous direct wastewater filtration suitable for practical application, much more research and technological adjustments have to be done.

Continuous filtration would be preferred from an operational point of view when direct coarse media filtration is applied, since the removal efficiencies of the continuous or discontinuous filter configurations did not differ significantly, Further research on the continuous operation and specifically on clogging prevention by means of filter control and water quality monitoring is recommended such that a possible practical application of this technique can be developed.

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CHAPTER 7 DIRECT MEMBRANE FILTRATION FOR ADVANCED PARTICLE REMOVAL FROM RAW WASTEWATER

SUMMARY

This chapter presents pilot-scale research into direct membrane ultrafiltration of raw wastewater. The aim of this experimental research is to explore the characteristics of direct membrane filtration as a pre-treatment technique for advanced particle removal. Removal performances and operational process conditions were investigated with a pilot-plant membrane installation operated discontinuously with maximum run times of 7 hours.

From the explorative experiments it can be stated that pre-treatment of raw wastewater with membranes is possible from an operational point of view and has remarkable prospects regarding permeate quality and reuse potential. The investigations indicate an average normalised production flux of 300 $l/m^2 \cdot h \cdot bar$ at a trans-membrane pressure of 0.2 to 0.4 bar and a cross flow rate of 2.4 m/s. Without backflushing, it is even possible to keep the flux at a constant level, implicating a limited fouling of the membranes. The cross-flow velocity plays an important role in this process, mainly because of the shear stress it causes along the membrane surface.

Removal performances are excellent since all suspended and colloidal substances are removed by the membranes with pores of 30 nm; 65% of COD, 50% of BOD, 30 % of phosphorus and about 20% of nitrogen were removed.

In addition, direct ultrafiltration of raw wastewater yields a useful permeate because it is assumedly free of bacteria and viruses and contains a considerable amount of easily accessible nutrients. It can thus be used for irrigation and might supply the nutrient needs of different types of crops.

7.1 Introduction

The limited particle removal by coarse media filtration and the recommendations for further research in the preliminary evaluation study led to the initiation of experimental research into the application of direct membrane filtration of raw wastewater. An additional hypothesis is that direct membrane filtration might eliminate the use of flocculants. Possible negative side effects resulting from the dosage of chemicals as described in chapter 2 and 5, can be prevented by applying direct membrane filtration. As described before, research on filtration techniques for pre-treatment of wastewater is rather scarce and application in practice is hardly done. Only a small number of research activities into direct influent filtration, mainly with coarse media up-flow floating filters, have been reported (see chapter 5). Besides a South Korean small-scale research on hotel wastewater [Kyu Hong Ahn *et al.* (1998)] and trial set-ups in Sydney, Australia at the wwtp's Malabar and Cronulla [Hudman *et al.* (1995], investigations into direct membrane filtration of raw municipal wastewater have hardly been described so far.

Nevertheless, direct filtration of wastewater may have specific advantages over the commonly used sedimentation process. Firstly, it is possible to achieve an advanced particle removal even without the use of chemicals. Secondly, filtration techniques can be applied in a very area-efficiently way that may be an important aspect in highly populated areas like the Netherlands.

After the investigations into direct coarse media filtration, a further step was taken by applying membranes as filtration media [STOWA (2001)]. The aim of this experimental research is to explore the characteristics of direct membrane filtration as a pre-treatment technique for advanced particle removal. Removal performances and operational process conditions were investigated for different configurations.

7.2 Theoretical background

Membrane filtration is defined as the filtration process of a liquid or gas through a membrane. This membrane is a microfilter consisting of semi-permeable material. By means of membranes, suspensions as wastewater or slurries can be separated into a concentrated stream (the *concentrate*) and a clarified flow, entitled as the *permeate*.

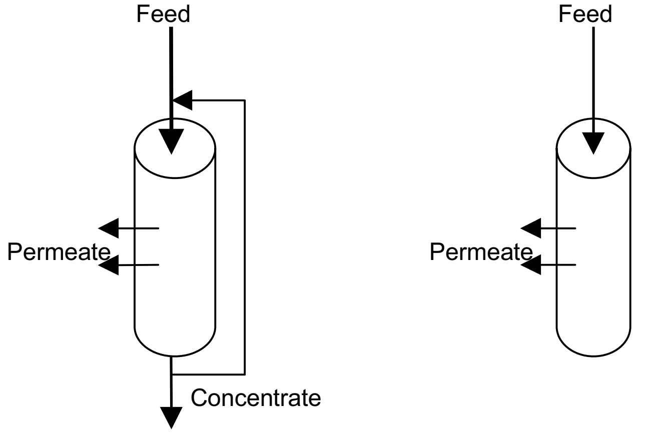
Membrane filtration is subdivided into different sub-processes determined by the nominal pore size of the applied membranes. Table 7.1 gives an overview of different filtration processes plotted against the effective pore size of the membrane: microfiltration, ultrafiltration, nanofiltration and hyperfiltration.

Table 7.1: Membrane filtration processes

pore size	Nm	0.1	1	10	100	1000			
	µm	0.0001	0.001	0.01	0.1	1	10	100	1000
components			ions/ molecules		macro-molecules	colloids		particles	
			salts	viruses	bacteria				sand
filtration processes		hyperfiltration---|							
			|--nanofiltration--|						
				|----ultrafiltration---|					
					|-----microfiltration----|--------sand filtration------→				

Membranes in water and wastewater treatment are manufactured as tubes, plates, spiral-winded configurations. Most applied in wastewater effluent filtration are tube membranes [Günder and Krauth (1998)].

Another subdivision in membrane filtration is made regarding the flow of the influent through the membrane tube. In case of a longitudinal continuous flow (the cross-flow) of feed water through the tube, the modification is called cross-flow filtration. A part of the influent flow is continuously recycled to the inflow or treated as concentrate, while the other part is filtered through the membrane as permeate (see figure 7.1a). The other modification is entitled as dead-end filtration (figure 7.1b). In this application the influent flow is driven through the membrane and leaves the installation as permeate. In this case the concentrate is produced by frequent flushing of the membrane tube, a process that is also applicable in cross-flow filtration. Since the tested membranes were driven in cross-flow configuration, the following paragraphs will focus on this type of membrane filtration.



7.1a Cross-flow configuration **7.1b Dead-end configuration**

Figure 7.1: Membrane configurations: cross-flow (7.1a) and dead-end (7.1b)

Fouling of the membrane

During the filtration process, colloids, particles and other substances accumulate on the membrane surface. The membrane pores become clogged during the filtration process by this accumulation and the resistance over the membrane increases. This phenomenon is called membrane fouling. Fouling is defined by as "*the process resulting in loss of performance of a membrane due to deposition of suspended or dissolved substances on its external surfaces, at its pore openings, or within its pores*" [Koros *et al.* (1996)]. Fouling can be subdivided in essentially five mechanisms, each monitored as a build up of resistance [Roorda and Graaf, van der (2001), Bourgeous *et al.* (2001)]:

- adsorption inside the membrane pores;
- blocking of the membrane pores;
- concentration of foulants near the membrane surface, also called (concentration) polarisation;
- deposition on the membrane surface forming a cake layer; and
- compression of the cake layer.

These fouling mechanisms may occur successively or simultaneously during the filtration process.

The fouling process has to be prevented by means of pre-treatment or the fouling layer has to be removed during or after the filtration process. This is done by backflushing, chemical cleaning and in case of cross-flow filtration, continuously with the cross-flow.

Cross-flow filtration

The feed (F in m3/h), in this case the wastewater influent, is pumped through the membrane tube with certain *cross-flow velocity* and is submitted to the membrane under a certain pressure (p1 in bar). The cross-flow is needs to be that high such that it is preferably turbulent. The created high shear stress at the membrane wall results in the removal of the deposition.

The driving force in membrane filtration is the pressure difference over the membrane, the so-called *trans-membrane pressure* (TMP in bar). Since the permeate flow is generally from the inside to the outside in cross-flow configurations, the pressure outside the membrane (p2 in bar) is lower compared to the pressure inside the membrane tube (p1), so the TMP is equal to p1 minus p2 (see figure 2).

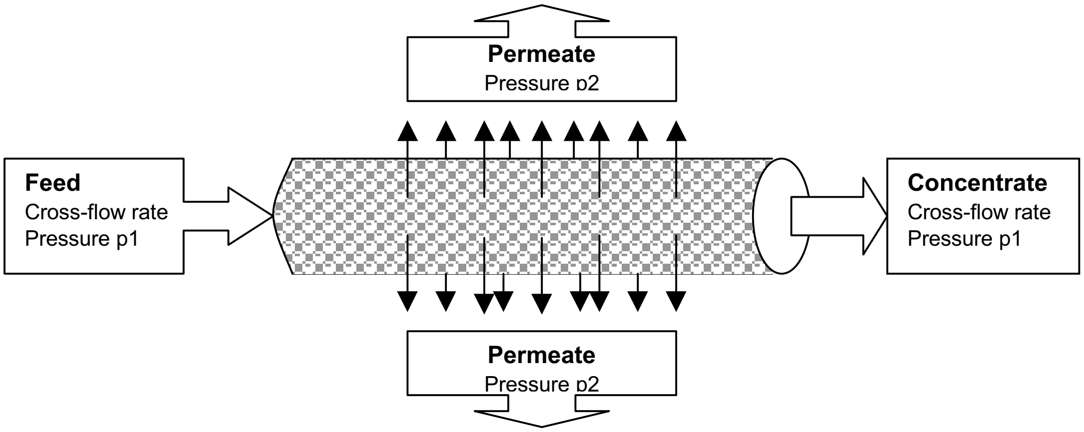


Figure 7.2: Parameters concerning membrane filtration in cross-flow configuration

The necessary TMP to produce a certain quantity of permeate is determined by:

- the pore size of the membrane;
- the concentration of salts and organics in the feed water; and
- the pollution of the membrane.

The produced permeate flow is entitled as the permeate flux and is calculated in litres per square-meter membrane per hour (J in $l/m^2 \cdot h$).

By dividing the flux with the TMP, normalised flux or *permeability* of the membrane is calculated (P in $l/m^2 \cdot h \cdot bar$).

Since the pressure-flux relation and the permeability depend on the viscosity of the feed flow, these parameters are determined by the temperature of the influent. To avoid this temperature effect, the parameters are recalculated to a standardised temperature [Koros *et al.* (1996), Roorda and Graaf, van der (1999)].

7.3 Experimental Set-up

Material

The investigations were carried out with a membrane unit (figure 7.3), consisting of two, 1 inch wide and one metre long, membrane modules, each containing 12 membrane tubes with an internal diameter of 5.2 mm. The flow direction through the membranes was outside-in.

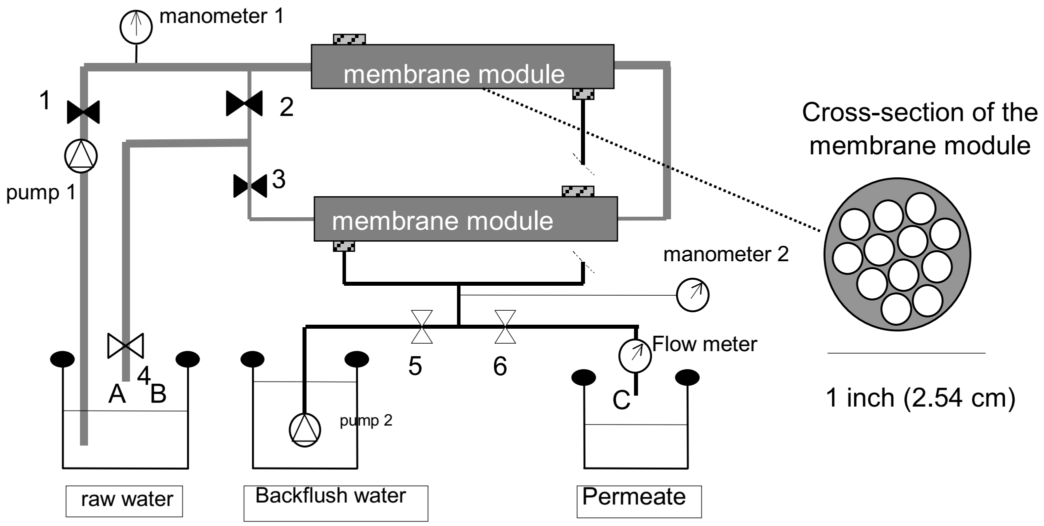


Figure 7.3: Scheme of the membrane test unit

The total effective membrane area was 0.17 m^2 per module; the membrane pore size was 30 nm. The applied membranes were of the type STORK FX4385, a hydrophylic polyvinylidenefluoride membrane on a supporting layer of polyester-polyolefine fabric. According to the technical specifications, the initial flux (tested with distilled water at 25 °C) at 1 bar should be higher than 1,000 $l/m^2 \cdot h$ (see appendix I for more details).

Operation

The two modules were coupled in series as shown in figure 7.3 and the installation is driven in cross-flow. The cross-flow velocity and the TMP were induced by pump 1. The TMP could be adjusted by operating valves 1 and 3. Pump 2 was used for cleaning of the membranes by backflushing at a pressure of 0.8 bar.

The membrane unit was operated in the cross-flow mode at different flow rates. Every ten minutes the production was interrupted for a backflush, using filtered tap water. The backflush time varied from 30 s to 120 s. The experiments were performed at a TMP ranging from 0.2 to 0.5 bar.

Since the membrane filtration unit was not capable of continuous operation, the average filtration runtime was seven hours. Continuous operation is still not tested.

Temperature correction – Because the pumps used the feed water as cooling water, the temperature of the feed water to the membranes increased during a test period. To prevent this, the feed water flow was constantly refreshed with fresh influent with the original temperature. Nevertheless, temperature changes of the feed water during a testing period could not be erased completely.

To avoid the influence of the temperature on the test results, the fluxes and TMP's were recalculated to a standardised temperature of 15 ºC.

The temperature effect is caused by the relationship between the flux and the viscosity of the feed water. The flux is dependent on the kinematic viscosity (*v*) as stated in equation 7.1.

$$v = \frac{497 \cdot 10^{-6}}{(42.5 + T)^{1.5}} \tag{7.1}$$

where: v = kinematic viscosity (m^2/s)
T = temperature (ºC)

Since the flux is inversely proportional to the kinematic viscosity, the measured flux with a certain temperature is corrected with the standardised temperature as shown in equation 7.2:

$$J_{corrected} = J_{measured} \cdot \frac{(42.5 + T_{standard})^{1.5}}{(42.5 + T_{measured})^{1.5}} \tag{7.2}$$

where: $J_{corrected}$ = by $T_{standard}$ corrected flux ($l/m^{2} \cdot s$)
$J_{measured}$ = flux derived from measurements ($l/m^{2} \cdot s$)
$T_{standard}$ = standardised temperature (in this experiments 15°C)
$T_{measured}$ = measured temperature of the feed water during testing (°C)

The TMP can be corrected equally to the flux, but in this case the TMP is proportional to the viscosity of the feed water.

Cross-flow velocity – Within cross-flow membrane filtration the cross-flow is continuously cleaning the feed side of the membrane. To achieve an efficient cleaning process, the cross-flow should be sufficiently high that a turbulent flow is reached and a minimum shear stress guarantied.

The turbulence criterion is set by the Reynolds number (Re) of the flow through the membrane tube as described in equation 7.3. This Reynolds number has to be more than 4,000.

$$Re = \frac{u \cdot d}{v} > 4{,}000 \tag{7.3}$$

where: u = water velocity in the membrane tube (m/s) = $Q/(¼\, v \cdot d^2)$
d = diameter of the membrane tube (m)
v = kinematic viscosity (m^2/s)

In the case of the experimental set-up, the membrane tubes had an internal diameter of 5.2 mm. Calculated by equation 7.3 the minimum cross-flow velocity has to be 0.88 m/s to meet the turbulence criterion at a feed water temperature of 15 °C.

Besides the turbulence criterion, a minimum shear stress (τ in Pa) on the membrane is necessary to optimise the removal of a dirt or fouling layer by the cross-flow [Elmaleh and Abdelmoumni (1998), Bourgeous *et al.* (2001)]. The shear stress is determined by the cross-flow velocity through the membrane tube and the flow pattern (see equation 7.4). Elmaleh and Abdelmoumni (1998) advise an optimal range for the shear stress between 20 and 30 Pa.

$$\tau = \left(f/2\right) \cdot \rho \cdot u^2 \tag{7.4}$$

where τ = shear stress (Pa)
ρ = liquid (water) density [kg/m^3]
u = cross-flow velocity [m/s]
$^f/_2$ = friction factor, depending on the Reynolds number Re:
for Re ≤ 2,500 - 4,000: $^f/_2 = 8/Re$
for 5,000 ≤ Re ≤ 200,000: $^f/_2 = 0.023 * Re^{-0,20}$

Resistance over the membrane - In a second testing period, experiments were done in which the resistance (R in m^{-1}), caused by fouling and the membrane, was monitored [Roorda and Graaf, van der (1999), Evenblij (2001)]. The resistance is calculated by equation 7.5:

$$R = \frac{TMP}{J \cdot \mu} \tag{7.5}$$

where: R = resistance [m^{-1}];
TMP = applied trans-membrane pressure [Pa];
J = resulting permeate flux [m^3/m^2·s];
μ = dynamic viscosity of the permeate [Pa·s].

Feed water characteristics

As feed water for the operational tests with the membrane unit, wastewater was taken from the inlet of the municipal wastewater treatment plant Bennekom (18.000 p.e.), in the Netherlands. The temperature of the wastewater in the 80 litre feed water tank was kept as constant as possible by applying a continuous bleed. During the operational testing period, the feed water was analysed as summarised in table 7.2.

Table 7.2: Characteristics of the feed water during technological experiments

parameter		**average concentration**
total suspended solids	(mg TSS/l)	130
turbidity	(NTU)	130
COD	(mg O_2/l)	680
BOD_5	(mg O_2/l)	210
P_{total}	(mg P/l)	11
$N_{Kjeldahl}$	(mg N/l)	72
temperature	(°C)	21

Further tests were carried out to investigate the removal performance of direct membrane filtration with wastewater from wwtp Berkel, wwtp Beverwijk and wwtp Tilburg (all in the Netherlands)

Testing procedure

During the experiments, flux, TMP and temperature were measured and samples off the feed and permeate were taken for analysis. All fluxes and TMP presented were recalculated to a standard temperature of 15 °C.

Each filtration runtime started with measuring the permeability of the membranes by using clean tap water. The permeability characterises the condition of the membrane.

After this first permeability test, the filtration of raw wastewater was started. During several hours the wastewater was fed to the membrane unit, producing permeate. The membrane filtration unit was operated at a specific cross-flow velocity and a constant TMP and the changes of the flux during the filtration run time were measured. At certain intervals samples off the feed and produced permeate were taken to be analysed with respect to COD, N, P, turbidity and suspended solids.

Immediately after an experiment with raw wastewater, the permeability of the membrane was measured again with tap water, in order to determine the condition of the membranes.

Chemical cleaning and analysis

As a chemical cleaning, the membranes were exposed to a chlorine solution of 350 mg/l, this being 75% of the maximum concentration allowed for this specific membrane.

Turbidity was measured with a HACH turbidimeter in NTU. COD, suspended solids, pH and temperature were measured according APHA (1998).

7.4 Experimental results

General overview

The first result from this explorative research was that membrane filtration of raw wastewater is operational possible. Over the three month testing period with relatively short filtration runtimes, no membrane failure was monitored and the installation produced a clear, constant quality permeate. A steady-state configuration was reached for the applied membrane unit at a TMP of 0.2 to 0.4 bar with a production time of 10 minutes and a 1-minute backflush. With this configuration the average normalised flux over the three month testing period was calculated to be 300 $l/m^2 \cdot h \cdot bar$, decreasing from about 600 $l/m^2 \cdot h \cdot bar$ at the start of the tests to 180 $l/m^2 \cdot h \cdot bar$ three months later. The research results are indicative of the potential operation of direct membrane filtration. However, the membranes were not tested in continuous operation. Up-scaling and continuous operation of the direct membrane filtration installation will make clear if this technique is applicable in practice.

Cross-flow rate versus flux - In the process of cross-flow membrane filtration, the cross-flow velocity is one of the most important parameters. The cross-flow causes a turbulent flow in the membrane tube and a shear stress on the membrane surface to hinder or remove deposition of particles and flocs. Therefore, fouling of the membrane is (partially) prevented or limited.

Figure 7.4 shows the effect of an increase of the cross-flow velocity on the permeate flux. Raising the cross-flow velocity (v_c) from 1 m/s to 2.5 m/s the average flux increased from 70 l/m^2·h to 110 l/m^2·h.

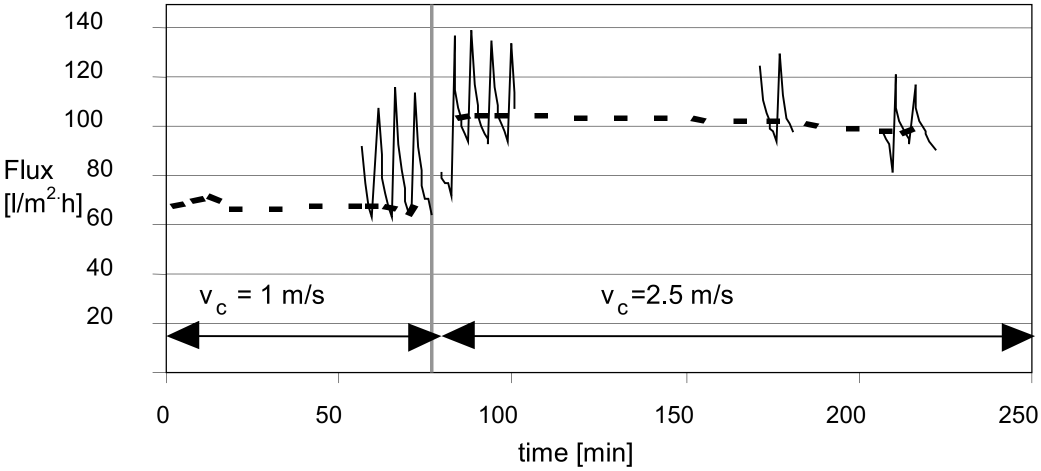


Figure 7.4: Effect of the increasing cross-flow velocity on the flux at a TMP of 0.5 bar

The increase in flux is caused by a more intensive removal of fouling and/or cake layer from the membrane. Because the flow conditions in the membrane tube of 5.2 mm were already turbulent at the original cross-flow rate of 1 m/s (remember: Re > 4,000 at 0.88 m/s), the effect was not caused by the increased turbulence. The decisive parameter is supposedly the shear stress on the membrane surface, which becomes efficient above 20 Pa [Elmaleh and Abdelmoumni (1998), Bourgeous *et al.* (2001)]. In case of the applied membrane tubes, it can be calculated that the minimum shear stress is reached at a cross-flow rate of 2.4 m/s.

Production time versus backflush time - Within the experimental set-up, the production of permeate was interrupted every ten minutes for a backflush, in order to clean the membranes. To investigate the influence of backflush cleaning on the membrane filtration process, the flux was recorded just before each start of a backflush and immediately after a backflush. In this way, a curve of start-fluxes (J_s) and a curve of end-fluxes (J_e) was obtained. In figure 7.5, these curves are plotted versus a number of production runs.

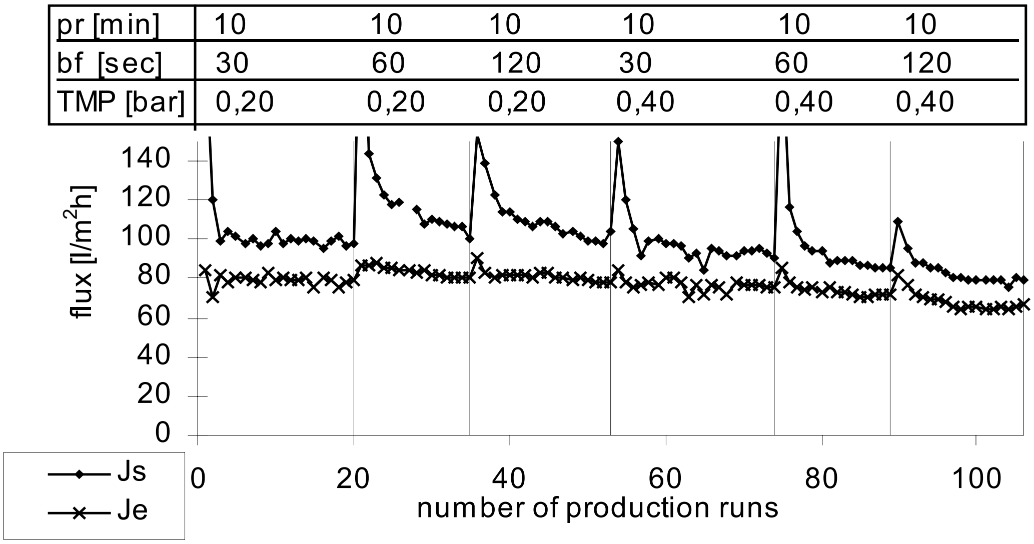


Figure 7.5: Influence of backflush (b_f), versus production time (p_r) and TMP on the start-flux (J_s) and end-flux (J_e)

In the first three sections of the graph, the backflush time (b_f) is increasing from 30, over 60 to 120 seconds at a standard TMP of 0.2 bar and a constant production time of 10 minutes. By doubling b_f from 30 to 60 seconds, the start-fluxes were increased as well. The end-fluxes hardly changed, resulting in higher permeate production per filtration run. A further increase of b_f up to 120 seconds did not improve the value of the permeate fluxes.

By raising the TMP from 0.2 to 0.4 bar (see difference between section 3 and 4), the start- and end-fluxes hardly changed. So, converted into pressure normalised fluxes, the permeability or so-called permeance [Koros *et al.* (1996)] of the membrane was halved by doubling the TMP. The increase in backflush time did not influence the production fluxes at the TMP of 0.4 bar. This may indicate the formation of a more resistant fouling or cake layer on the membrane due to the higher pressure.

With the applied backflush-TMP of 0.8 bar a 1-minute backflush was enough to reach the maximum effect, preferably at a TMP of 0.2 bar.

Figure 7.5 also shows that the end-fluxes are almost constant for all experiments. At the beginning of an experiment, the start-flux decreased rapidly within a couple of production runs to an almost constant level. After these first few production runs clogging of the membranes apparently decreased and the permeate flux reached a constant value. This indicates that independently of the process parameters, the flux after ten minutes reaches a specific value. Fouling of the membranes manifested itself as a decrease in start-fluxes, and less in a decrease in end-flux.

Flux development without backflush - As described in the last paragraph, the flux decrease occurred over the first minutes of the test run, and after that, the flux was going to a constant value. So, at a certain flux value, in combination with the cross-flow velocity and the TMP, permeate fluxes do not decrease in time anymore. This assumption was tested in a production run that lasted over 1.5 hours, without being interrupted by a backflush. Figure 7.6 presents the resulting flux pattern.

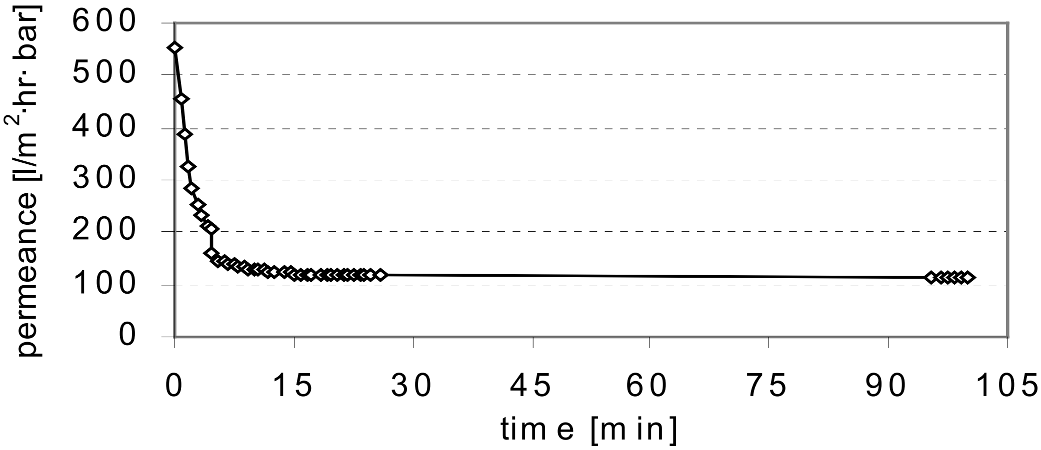


Figure 7.6: Flux pattern during production run without backflushing

As in the normal test runs, the flux at the beginning of the permeate production decreased rapidly from 550 litre per m^2 per hour per bar to less than 200 l/m^2·h·bar, but reached a constant permeability of 120 l/m^2·h·bar after 15 minutes. The flux stays constant for more than one hour, without backflushing. Assumedly, an equilibrium of cake layer formation and removal has been reached.

Resistance due to fouling - An important factor in fouling of the membrane is the flux (induced by the TMP). An increase in flux results in an increase in fouling rate, since more suspended matter and particles are transported towards the membrane. Because of the permeate flux through the membrane, substances with dimensions larger than the pore size of the membrane will accumulate near the membrane. Although this layer is permeable, it forms an additional resistance for the permeate.

Besides the concentration of foulants, the properties of the foulants are important. The dimensions of the particles play an important role, as this determines the transport mechanism. A relatively big particle will be more exposed to the scouring effect of the cross-flow than a smaller one. Apart from the dimensions, the hydrophobicity of the particles or colloids determines the fouling capacity of the solution to be filtrated [Evenblij *et al.* (2001)].

Membrane fouling and cleaning – As stated before, during the filtration process the membranes polluted. The easily removable fouling or cake layer was flushed away by the regular backflush; so, formation and removal are in balance. In-between the tests, the membranes were chemically cleaned. Backflushing and chemical cleaning should remove the reversible fouling completely. However, after chemical cleaning, a certain fouling component remains attached to the membrane: an irreversible type of fouling. This fouling can be quantified by determining the permeability of the membrane at certain points in time during the filtering process over the total experimental period. Figure 7.7 shows the permeability development over the tests.

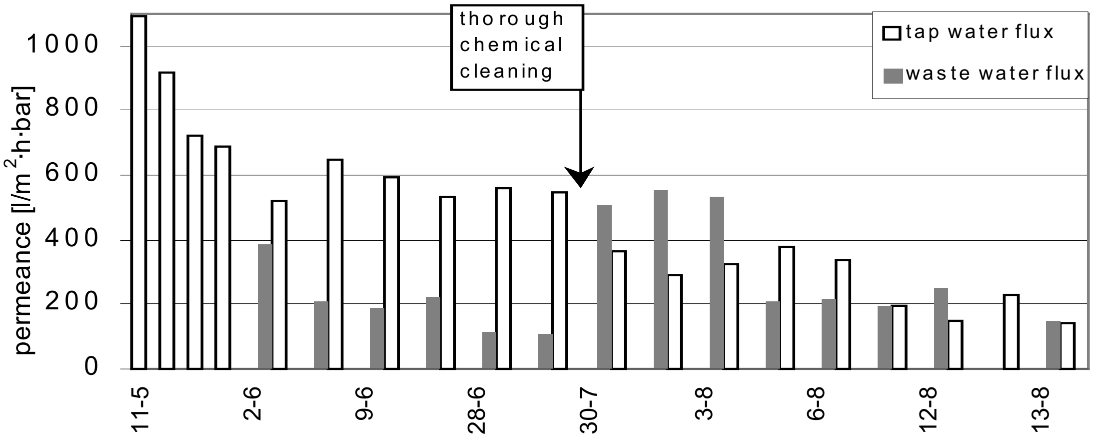


Figure 7.7: Permeability of the membranes over the testing period

Before any experiments with wastewater were done, the membrane was only fed with clean tap water to test the membrane unit. Over this period, the permeability of the membrane decreased considerably, from 1,100 $l/m^2 \cdot h \cdot bar$ to 650 $l/m^2 \cdot h \cdot bar$. This may be ascribed to adsorption of ions to the membrane surface. It seems likely that this process went on during the whole series of experiments with wastewater, thus causing a more or less constant decrease in permeability from 650 $l/m^2 \cdot h \cdot bar$ to less than 200 $l/m^2 \cdot h \cdot bar$. This adsorption process caused a fouling that could not be removed effectively by the normal chemical cleaning procedure.

Earlier research found that a high cross-flow velocity resulted in a stagnation of the fouling [Elmaleh and Abdelmoumni (1998)]. This assumption could not be verified in this research, because the cross-flow velocity could not be raised above 2.5 m/s. Bouhabila *et al.* (1998) and Megat Johari et al (1996) found that the decrease in flux stopped when 20-30% of the initial flux of the membrane was reached. The described results from these investigations are in accordance with the outcomes of our own experiments, as was the result we found that with a TMP of 0.2 bar a constant flux could be reached.

The experimental results of filter runs without backflush showed that under certain conditions an equilibrium could be reached between fouling and cleaning by the cross-flow. Bouhabila *et al.* (1998) describes that the adsorption process could even be eliminated and fluxes could be becoming constant in time. This phenomenon is assumed to be related to a (secondary) critical flux, as stated by Field *et al.* (1995).

Removal performances

The presented removal performances of the direct membrane filtration unit are all derived with a direct ultrafiltration installation as described in the experimental set-up applied membranes with a pore size of 0.03 µm (30 nm). As presented in table 7.3, turbidity and suspended solids could not be detected in the produced permeates since all particulate material had been removed by the membranes.

Table 7.3: Removal performances of direct ultrafiltration over a 0.03 µm membrane in cross-flow mode.

		wwtp Beverwijk			**wwtp Bennekom**			**wwtp Berkel**			**wwtp Tilburg**		
		infl	perm	R	infl	perm	R	infl	perm	R	infl	perm	R
TS solids	mg TSS/l	204	n.d.	99.9%	n.t.	n.t.	n.t.	212	n.d.	99.9%	n.t.	n.t.	n.t.
turbidity	NTU	98	n.d.	99.9%	195	n.d.	99.9%	140	n.d.	99.9%	206	n.d.	99.9%
COD	mg O_2/l	430	210	51%	650	225	65%	490	220	55%	740	270	64%
BOD_5	mg O_2/l	170	90	47%	240	115	52%	200	100	50%	265	130	51%
N_{total}	mg N/l	42**	37	12%	73	59	19%	49	43	13%	63	49	22%
P_{total}	mg P/l	4.4	2.1	53%	6.8	5.6	18%	7.6	5.3	30%	8.0	5.2	35%
conductivity	mS/cm	0.91	0.88	3%	0.90	0.85	6%	1.17	1.14	2%	1.15	1.11	4%
BOD/N ratio		4.05	2.43		3.28	1.95		4.08	2.33		4.21	2.65	

infl = concentration in influent, perm = concentration in permeate; R = removal percentage.

n.d. = not detectable; n.t. = not tested

* 1 mS/cm ≈ 750 mg NaCl/l

** recalculated concentration

The removal performances of the membrane unit show differences for each treated wastewater depending on the raw wastewater characteristics of the specific wwtp. COD is removed between 50% (wwtp Beverwijk) and 65% (wwtp Bennekom and Tilburg) and BOD_5 between 44% and 52%. 12% (wwtp Beverwijk) to 22% (wwtp Tilburg) of total nitrogen was removed from the wastewater and 18% (wwtp Bennekom) to 53% (wwtp Beverwijk) of phosphorus could be removed. Conductivity removal was limited to 2% to 6%.

The absolute concentrations in the permeate indicate that the permeate of the direct membrane filtration unit needs further treatment regarding oxygen consuming components (BOD and COD) and nutrients (N and P) to meet the European and National effluent discharge standards (see appendix A). An important item for this further treatment is the BOD/N ratio of the permeates. It is generally stated that the biological denitrification process requires a minimum BOD/N ratio of 2.5 [STOWA (1998a)] to proceed properly, whereas Mels [Mels (2001)] found that a BOD/N ratio of 2 could be sufficient. Table 7.3 shows that the permeates, besides for the tested wastewater of wwtp Tilburg, do not reach this original required BOD/N ratio of 2.5 and the permeate of wwtp Bennekom even does not meet the desired BOD/N ratio of 2.

7.5 Discussion

From a technological point of view, direct membrane filtration (DMF) is applicable as pre-treatment for wastewater; however, further research has to be done on long-term application and optimisation of the process.

Post-treatment

The implementation of direct membrane filtration in the (existing) wastewater treatment system is a point of concern. As explained before, the permeate has to be treated further to remove oxygen consuming components and nutrients. Because the permeate of DMF is particle free and has a low BOD/N ratio, biological systems seem not to be suitable for this purpose, since biological activity will pollute the clear permeate with (active) sludge particles and denitrification is difficult to maintain. In relation to these topics, further physical-chemical post-treatment seems more adequate. However, as described in chapter 2, these techniques are relatively expensive an energy consuming. Other post-treatment techniques or applications for the permeate have to be considered to apply direct membrane filtration successfully.

In spite of the shortcomings of the biological post-treatment systems, direct membrane filtration will be evaluated in chapter 8 in combination with an activated sludge system, a biofilm system and a membrane-bio-reactor.

Direct reuse of the DMF permeate

Reuse of wastewater is an interest-gaining subject, since in many regions water shortage is becoming a problem and/or different kinds of water types are requested. For example in the Netherlands, groundwater extraction is increasingly limited, and so even in this wet country reuse of wastewater comes important [STOWA (1998b)]. In addition, in arid climates, the availability of water is increasingly becoming a problem and the reuse of wastewater is considered or is already applied. Usually, this is done by advanced treatment of the effluent from wastewater treatment plants. The purpose of this treatment then, is to separate the water from pollutants to get water with a purity as high as possible and the reuse of the water part of the wastewater is aimed at.

However, besides water itself, many useful substances can be found among the pollutants in wastewater, e.g. organic materials and nutrients.

Therefore, reuse of wastewater with respect to fertilising substances is interesting. Applied in irrigation, the permeate from DMF provides the necessary amount of water as well as the nutrients that are needed for cultivation. In this application, biological treatment should be omitted, to avoid the biodegradation of useful organics and nutrients. By applying direct membrane filtration, all particulate matter, bacteria and most of the viruses can be separated from the wastewater and the soluble, easily accessible nutrients will remain in the water for further use [Evenblij *et al.* (2001)].

7.6 Conclusions and recommendations

Taking into account the technical limitations of the tested membrane unit, the following conclusions can be drawn and recommendations can be made.

Conclusions

Pre-treatment of raw wastewater with membranes is possible from an operational point of view and has remarkable prospects regarding permeate quality and reuse potential. From the explorative experimental research it can be said that the removal performances of direct membrane filtration are excellent; together with all colloidal and suspended material, 65% of COD, 50% of BOD, 30 % of phosphorus and about 20% of nitrogen can be removed. The produced permeate is of a constant quality, only dissolved pollutants remained in the permeate. During the three month testing period with average discontinuous filter runtimes of seven hours, an average normalised flux of approximately 300 $l/m^2 \cdot h \cdot bar$ was possible. Steady state could be reached with a permeate flux of about 100 $l/m^2 \cdot h \cdot bar$. Practical values for TMP are between 0.2 and 0.4 bar, since higher values would cause a compact, more resistant cake layer on the membrane surface.

Without backflushing, it is possible to keep the flux at a constant level, implicating a limited fouling of the membranes. The cross-flow velocity plays an important role in this process, mainly because of the shear stress it causes along the membrane surface. There seems to be an optimal cross-flow velocity, at a certain TMP, above which no flux decreasing fouling occurs. This could be related to the (secondary) critical flux.

Direct ultrafiltration of raw wastewater yields a useful permeate. It is assumedly free from bacteria and viruses and contains a considerable amount of easily accessible nutrients. Thus, it can be applied in irrigation and supply the nutrient need of different types of crops.

Recommendations
Subject of further investigations should be the up scaling of the results. A continuous pilot plant should be set up, which supports automatically monitoring of parameters, such as flux, TMP, cross-flow rate, turbidity and COD. The mentioned existence of a (secondary) critical flux should be verified.

With respect to the membranes, research should be done to investigate the efficiency of cleaning agents and procedures, and the necessity of cleaning at all.

Another field of investigation is the further treatment of the permeate and the retentate, produced by the membrane process. It seems likely that for some applications, the permeate can be used in a regular treatment step. It should be kept in mind, that the permeate is free of particles and that it can easily be polluted by a subsequent treatment step. Concerning the retentate, it can be said that it seems appropriate to concentrate it in further membrane processes, to achieve high dry solids concentrations. Thus, one may think of a cascade of membrane steps, in which the retentate is thickened within every step, and each step contributes to a certain amount of particle free permeate.

The practical application of the proposed direct use of the permeate as fertilising irrigation water should be proved by means of field tests.

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CHAPTER 8 UPGRADED SCENARIO ANALYSIS WITH DEMAS$^+$

SUMMARY

This chapter describes, the final comparative evaluation study, conducted with an upgraded version of DEMAS, of selected wastewater treatment scenarios, based on the physical-chemical particle removal pre-treatment techniques. Next to the reference treatment scenario, scenarios with polymer enhanced primary sedimentation as investigated in chapter 5 are applied. In addition, two configurations of direct coarse media filtration, as well as direct membrane filtration are used as pre-treatment processes as they were investigated in chapter 6 and 7. The scenarios are arranged into three divisions, differentiated by means of the applied post-treatment technique. In addition to the evaluation, a sensitivity analysis (carried out with DEMAS$^+$) is carried out to investigate the effect of changing particle fractionations in the wastewater on the treatment scenarios as was identified and described in chapter 4.

As was concluded in the preliminary evaluation in chapter 2, the upgraded scenario analysis reveals that advanced particle removal in the physical-chemical pre-treatment result in wastewater treatment scenarios that consume less energy, can be compactly designed and are cost effective when the post-treatment is based on biological suspended activated sludge systems or biofilm systems.

The comparison shows that the scenarios using polymer enhanced primary sedimentation are most attractive regarding energy consumption, sludge production and costs. The application of direct membrane filtration results in the most-compact treatment systems and low sludge production.

In the scenarios it is shown that energy consumption decreases when more particles can be removed in the pre-treatment, which will be especially important when increasing percentages of particle-related contamination in wastewater are present. The sensitivity of the scenarios to the particle adhering colloidal oxygen demand ($COD_{particle}$) percentage depends on the particle removal efficiency of the applied pre-treatment process; a higher efficiency results in an increased sensitivity.

The influence of the $COD_{particle}$ percentage on the costs is divers. The investment and energy costs decrease whereas the sludge treatment and occasionally the chemical costs increase with an increasing $COD_{particle}$ percentage.

8.1 Introduction

In chapter 2, the scenario analysis methodology and the evaluation tool DEMAS$^+$ were described. At the end, chapter 2 presented a preliminary evaluation study to identify the possibilities of the composed treatment scenarios and the further research needed. By implementation of the research results, the model was upgraded during the progress of the project, resulting in the modified version DEMAS$^+$.

This chapter describes the final comparative evaluation study, conducted with the upgraded DEMAS$^+$, of wastewater treatment scenarios that were selected based on the experimental physical-chemical particle removal pre-treatment techniques explored in chapters 5 through 7. The purpose of the calculations is to evaluate the treatment scenarios on environmental interventions and cost, as it was preliminary done in chapter 2.

Furthermore, a sensitivity analysis using DEMAS$^+$ was carried out to investigate the effect of changing particle fractionations in the wastewater on the treatment scenarios as identified and described in chapter 4.

At the end, conclusions are drawn.

8.2 Methodology

The methodology applied for the final evaluation study corresponds to the method described in chapter 2. All assumptions and technological data mentioned in that chapter are valid for this final exercise. The design and evaluation model DEMAS$^+$ as described in chapter 2 is used to calculate the environmental interventions and the costs by means of the net present value.

Selected treatment scenarios

Eleven treatment scenarios were selected out of the original 32 that had been evaluated and compared in the research project [STOWA (2001a)]. The selection criteria were as follows:

1. The reference treatment scenario is based on the commonly applied primary sedimentation tank; followed by advanced low loaded activated sludge, as presented in chapter 2.
2. Since chapter 5 deals with polymer addition for flocculation, one of the applied pre-treatment techniques is primary sedimentation enhanced with polymer addition.
3. Two configurations of direct coarse media filtration, as well as direct membrane filtration are also applied as pre-treatment processes since they have been investigated in chapter 6 and 7.

The scenarios are arranged into three divisions, differentiated by means of the applied post-treatment technique. Division 1 works with the low loaded activated sludge system, division 2 is equipped with a biofilm system and division 3 connected to a membrane-bio-reactor (mbr). Table 8.1 gives an overview of the selected treatment scenarios.

Table 8.1: Selected treatment scenarios

	scenario	pre-treatment technique	post-treatment technique
division 1	Ref	primary sedimentation (PST)	low loaded activated sludge + secondary sedimentation
	1a	primary sedimentation + polymers (PST + PE)	low loaded activated sludge + secondary sedimentation
	1b	direct coarse media filtration (DCMF)	low loaded activated sludge + secondary sedimentation
	1c	direct coarse media filtration + Fe (DCMF + Fe)	low loaded activated sludge + secondary sedimentation
	1d	direct membrane filtration (DMF)	low loaded activated sludge + secondary sedimentation
division 2	2a	primary sedimentation + polymers + Fe	biofilm system (post-denitrifying) + rapid sand filter
	2b/c	direct coarse media filtration + Fe	biofilm system (post-denitrifying) + rapid sand filter
	2d	direct membrane filtration + Fe	biofilm system (post-denitrifying) + rapid sand filter
division 3	3a	primary sedimentation + polymers + Fe	membrane-bio-reactor
	3b/c	direct coarse media filtration + Fe	membrane-bio-reactor
	3d	direct membrane filtration + Fe	membrane-bio-reactor

Design criteria

Primary sedimentation - Next to the reference scenario wherein primary sedimentation is applied as pre-treatment, the scenarios marked with an a (1a, 2a and 3a) are designed with a primary sedimentation tank with addition of a low dosage (4 g active polymer per m^3 wastewater) of an organic cationic HMW polymer (table 8.2). The primary sedimentation tank within the reference scenario is designed with a surface loading at maximum flow (swf) of 3 $m^3/m^{2}\cdot h$. The maximum surface loading of the chemically enhanced primary sedimentation tank is set at 6 $m^3/m^{2}\cdot h$, since the polymer addition leads to improved settling characteristics of the produced flocs (see chapter 5).

Direct coarse media filtration - The pre-treatment technique 'direct coarse media filtration' is designed in two variations: b and c. The direct filter configurations differ due to the ferric addition in the scenarios entitled with c. 10

mg Fe^{3+}/l is added to this filter to achieve a higher particle removal efficiency. In addition to the increased particle removal, phosphates are precipitated in the filter. In both configurations, a filtration rate of 10 $m^3/m^{2}\cdot h$ at swf is applied (see chapter 6 and table 8.2).

Direct membrane filtration - The applied flux for the direct membrane filtration installation is conservatively assumed to be 0.1 $m^3/m^{2}\cdot h$ at a TMP of 0.8 bar, with a permeate recovery of 75% (see chapter 7). The membrane installation is designed in cross-flow mode with frequent backflushing. It is theoretically assumed that the backflush frequency, as modelled in $DEMAS^+$, depends on the particle content of the incoming wastewater. Since direct membrane filtration removes all particle-related material down to the applied membrane pore size, chemical addition is not necessary to gain more advanced particle removal efficiencies (table 8.2). However, cleaning chemicals are used for intensive cleaning. The frequency of this chemical cleaning and the renewal period is assumed to depend on the particle load to the membranes.

Table 8.2: Design criteria of selected pre-treatment techniques

technique	chemical addition	design criterion
primary sedimentation	--	surface loading = 3 $m^3/(m^{2}\cdot h)$
primary sedimentation with polymer addition	4 g/m^3 cationic HMW polymer	surface loading = 6 $m^3/(m^{2}\cdot h)$
direct coarse media filtration	--	filtration rate = 10 $m^3/(m^{2}\cdot h)$
direct coarse media filtration with metal addition	10 g Fe^{3+}/m^3	filtration rate = 10 $m^3/(m^{2}\cdot h)$
direct membrane filtration	--	flux = 0.1$m^3/m^{2}\cdot h$; TMP = 0.8 bar; permeate recovery 75 %

Post-treatment - The post-treatment techniques: activated sludge system, biofilm system and membrane-bio-reactor are designed following the criteria from chapter 2 and Appendix C.

All the applied post-treatment techniques are low loaded systems designed for COD and nitrogen removal. The suspended activated sludge system also removes phosphorous biologically.

Frequently it is stated that the BOD/N ratio of the wastewater that is to be denitrified, should be at least 2.5 to gain a satisfactory denitrification process. However, experimental research by [Mels (2001)] revealed that a well-designed activated sludge system, receiving primary effluent with an average BOD/N ratio below 2.5, still contains sufficient denitrification capacity to reach total-nitrogen concentrations in the final effluent below the effluent standards. This may be partly the result of relative high amounts of easily biodegradable organic matter in the primary effluent remaining after advanced particle removal. This could be especially the case for wastewater originating form flat sewer systems where hydrolysis occurs [Ødegaard and Karlsson (1994)]. Based on these results, the minimal required BOD/N ratio in DEMAS$^+$ is fixed at 2 for denitrification. In case the BOD/N ratio reaches a value lower than 2, extra carbon is added. In DEMAS$^+$ methanol is used for this purpose, since it is most commonly applied in practice.

The applied biofilm system is designed as a post-denitrifying system since the DEMAS$^+$ calculations showed that this configuration is in this case the most suitable regarding to environmental impact and costs. Yet, the consequence is a low BOD/N ratio (< 2) in the wastewater that has to be denitrified. As it is done in the activated sludge system, methanol is dosed to fulfil the required BOD/N ratio of 2.

Within the scenario analysis, it is conservatively assumed that biological phosphorous removal in the biofilm system and the mbr is not possible. Due to this fact, phosphorous has to be precipitated to meet the effluent requirements. So, in the divisions 2 and 3, iron salt is dosed to the applied pre-treatment steps. For the phosphorous precipitation, a Fe/P ratio of 1 mol/mol is used resulting in a necessary iron addition depending on the phosphorous concentration in the wastewater.

Removal efficiencies and primary effluent quality

The applied removal efficiencies of the applied pre-treatment techniques as shown in table 8.3, are determined by the results of experimental research (chapter 5 to 7) and literature reviews (chapter 2 and 3).

The removal efficiency is subdivided into a particle fraction and a soluble fraction. The applied removal efficiency for the soluble fraction is not actually related to dissolved contaminants, but is determined by the removal of fine colloidal material. The primary sedimentation tank of the reference scenario and the direct coarse media filtration without metal addition do not remove this fraction. The chemically enhanced primary sedimentation tank and the coarse media filtration with iron addition, as well as the direct membrane filter remove the soluble part for 5%, respectively 15% each.

For further treatment in the biological post-treatment, the BOD/N ratio of the produced primary effluent is important to guaranty a sufficient denitrification. As described before, the requested BOD/N ratio is set at 2. As can be derived from the bottom row in table 8.3, only the primary effluent of the direct membrane filtration does not meet this requirement.

Table 8.3: Removal efficiencies and primary effluent (inclusive $N_{soluble}$ and $P_{soluble}$ from the recycled sludge water) compositions per scenario

scenarios		**Ref**	**1a, 2a, 3a**	**1b**	**1c, 2b/c, 3b/c**	**1d, 2d, 3d**
removal efficiency of pre-treatment unit		PST	PST + PE	DCMF	DCMF + Fe	DMF
particle fractions		30%	80%	50%	60%	100%
'soluble' fractions		0%	5%	0%	5%	15%
	raw influent composition	**primary effluents**				
COD total (mg O_2/l)	**600**	**485**	**282**	**408**	**359**	**184**
COD particles	384	269	77	192	154	0
COD soluble	216	216	205	216	205	184
BOD_5 (mg O_2/l)	**220**	**186**	**123**	**163**	**146**	**90**
BOD particles	114	80	23	57	46	0
BOD soluble	106	106	100	106	100	90
N total (mg N/l)	**55.0 + $N_{recycle}$**	**59.4**	**54.6**	**57.6**	**56.5**	**52.5**
N particles	9.3	6.5	1.8	4.7	3.7	0.0
N soluble	45.7 + $N_{recycle}$	52.9	52.8	52.9	52.8	52.5
P total (mg P/l)	**9.0 + $P_{recycle}$**	**10.1**	**8.3**	**9.4**	**7.5**	**7.5**
P particles	4.2	3.0	0.8	2.1	1.7	0.0
P soluble	4.8 + $P_{recycle}$	7.1	7.5	7.3	5.8	7.5
particles (mg TSS/l)	**250**	**175**	**50**	**125**	**100**	**0**
BOD/N ratio	4.0	3.1	2.3	2.8	2.6	1.7

8.3 Results

This paragraph presents the results of the scenario analysis by comparing the treatment scenarios as described in table 8.1. The calculated results are presented in diagrams, wherein the scenarios are arranged in the three divisions and are marked as in table 8.1. First, the calculated results of the environmental interventions are presented, followed by the costs.

Appendix I presents the calculation results numerical in more detail.

Environmental interventions

Energy balance - Figure 8.1 presents the overall energy balance within the system boundaries (see figure 2.1 in chapter 2). When the scenarios from division 1 are compared with the reference scenario, it can be concluded that the scenarios with advanced physical-chemical pre-treatment techniques, with exception of direct membrane filtration, consume less energy. This is mainly due to the saving of aeration energy in the activated sludge system since more COD is removed in the pre-treatment. Secondly, more biogas is yielded from the digestion process since more easily digestible primary sludge is produced in the pre-treatment.

Application of the polymer enhanced primary sedimentation (1a), results in energy savings of 66% compared to the reference scenario. The application of direct coarse media filtration without chemical addition (1b) and with metal salt addition (1c) saves 15%, respectively 25% of the energy consumed by the reference scenario. The treatment scenario applied with direct membrane filtration (1d), uses 27% more energy than the reference scenario. Since the post-treatment of this scenario uses less energy due to the absence of particles in the primary effluent, the high overall energy consumption is due to the relatively high (pump) energy consumption of the in cross-flow driven membrane filtration installation.

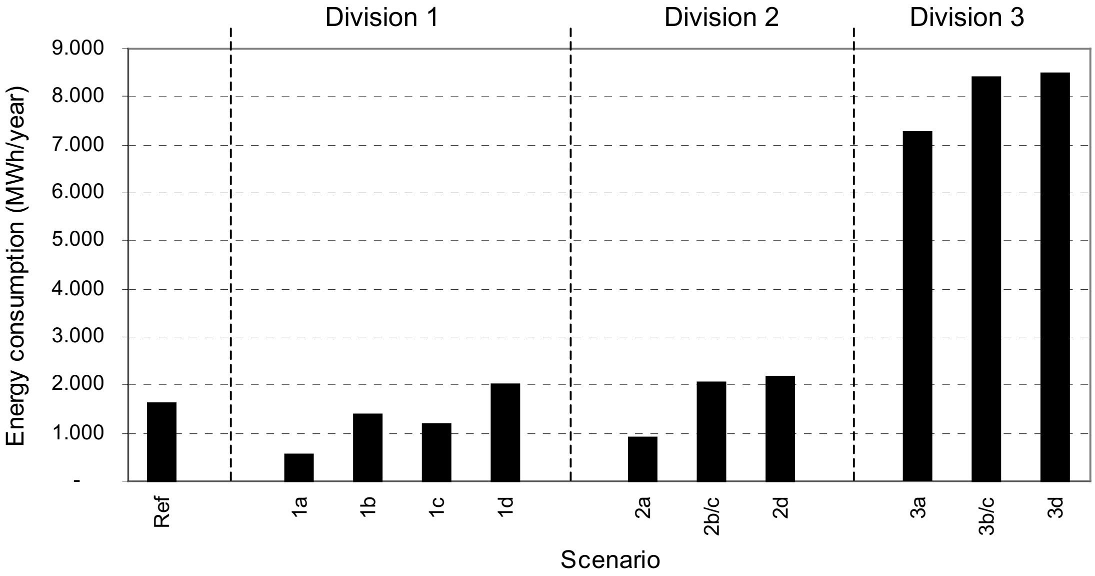

Figure 8.1: Energy consumption per evaluated treatment scenario

In division 2, only scenario 2a (chemically enhanced primary sedimentation combined with a biofilm system) consumes less energy than the reference scenario, with energy savings of 43%. The scenarios with direct filtration techniques (2b/c and 2d) in this division use approximately 30% more energy than the reference scenario.

The scenarios of division 3 (configured with the membrane-bio-reactor as post-treatment), have a calculated energy consumption of 4.5 to 5.3 times the energy demand of the reference scenario. The increased energy consumption is mainly due to the membrane-bio-reactor's energy use.

Space requirement - Figure 8.2 reflects the net space requirement per treatment scenario. As was concluded before in chapter 2, the diagram shows clearly that the scenarios with advanced particle removal in the pre-treatment can be designed with a much less footprint than the reference scenario. The scenarios with direct membrane filtration are most area-efficient, followed by the scenarios with polymer enhanced primary sedimentation and the direct coarse media filtration scenarios.

For example, treatment scenario 1a (polymer enhanced primary sedimentation with an activated sludge system) uses 25% less space than the reference scenario does. The space saving is caused by two reasons. In the first place the application of a less area using compact primary sedimentation tank and possible higher applicable surface loading accounted for the improved settleability of the produced polymer flocs (see chapter 5). Secondly, it is a result of the more compact activated sludge tanks since a mayor part of the COD is already removed by the chemically enhanced primary sedimentation and because of the possibility of the application of a higher F/M-ratio due to the lower particle load of the post-treatment (compare with chapter 2, table 2.5).

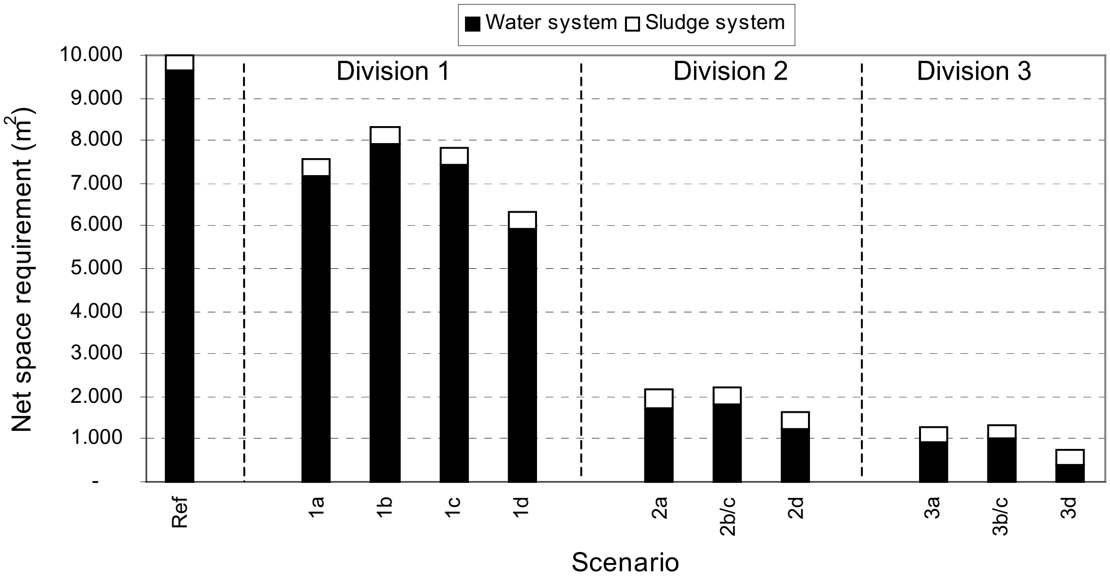

Figure 8.2: Space requirement per scenario

Major differences in net space requirement are present between the three divisions. The scenarios based on the activated sludge system (applied with a secondary sedimentation tank) are using four times more space than the scenarios with biofilm systems (with a rapid sand filter for final solids removal). The scenarios with the membrane-bio-reactor are approximately seven times smaller than the scenarios from division 1. This is mainly due to the space-consuming activated sludge tanks and most importantly due to the large secondary sedimentation tanks (4.300 m^2) in the scenarios of division 1.

The major space use within the system boundaries is caused by the wastewater system. The net space requirement of the sludge system depends on the amount and composition of the sludge produced in the wastewater system but is relatively constant compared to the space use of the wastewater system. For division 1 scenarios, the sludge treatment only uses 4% to 5% of the total net required space. For scenarios in division 2 and 3 this percentage increases to 18% - 25%, respectively to 20% - 48% due to a decreasing space use for the wastewater system in these scenarios.

Sludge production - Before comparing the scenarios as to the final sludge production, the effect of physical-chemical pre-treatment on the total sludge production is summarised. Due to the advanced particle removal in the physical-chemical pre-treatment, the scenarios with this pre-treatment have a relatively high primary sludge production. Since the organic matter and nutrient load to the biological post-treatment is decreasing with an increasing particle removal in the pre-treatment step, the biological sludge production is less. Having in mind that primary sludge is easily digestible and biological sludge is more difficult to digest, the total digested sludge production after digestion (with omission of possible chemical sludge production) is decreasing with advancing particle removal in the pre-treatment.

When inorganic coagulants and flocculants are added in the pre-treatment, the final sludge production is increasing by the amount of inorganic chemical sludge, produced in the form of metal hydroxides, metal phosphates and other inorganic conglomerations. This extra inorganic sludge is passing the complete sludge treatment and the incineration plant, without being converted or used efficiently.

On the other hand, organic flocculants do not produce extra inorganic sludge as metal salts do.

In case of a lack of carbon for a proper denitrification, an external carbon source is added. This dosage of methanol is causing an increase in biological sludge production, since a larger amount of organic matter is converted into cell material. This increasing biological sludge production results in a higher final sludge production.

Taking these aspects in mind, the optimal way of minimising the final sludge production is to remove particles in the pre-treatment as far as possible without endangering the denitrification process without dosing inorganic chemicals.

Comparing the scenarios, it can be seen that the scenarios in division 1, with the exception of scenario 1c that uses metal salts resulting in chemical sludge production, have a slightly less final sludge production than the reference scenario. The original amount of inorganic material in the wastewater causing the basic inorganic sludge production is for all scenarios the same. However, the incorporation of inorganic components in the bacteria cells in the biological post-treatment unit processes leads to an increase of the inorganic sludge fraction. Since, for the scenarios in division 1 the sludge yield of the activated sludge system is set on 0.4 kg dry solids per kg $COD_{removed}$, the differences in the production of organic and inorganic sludge between the scenarios is caused by the differences in removal efficiencies in the pre-treatment step and the resulting amount of COD, removed in the activated sludge system. A comparable line of argument can be made for the sludge production in the scenarios of division 2.

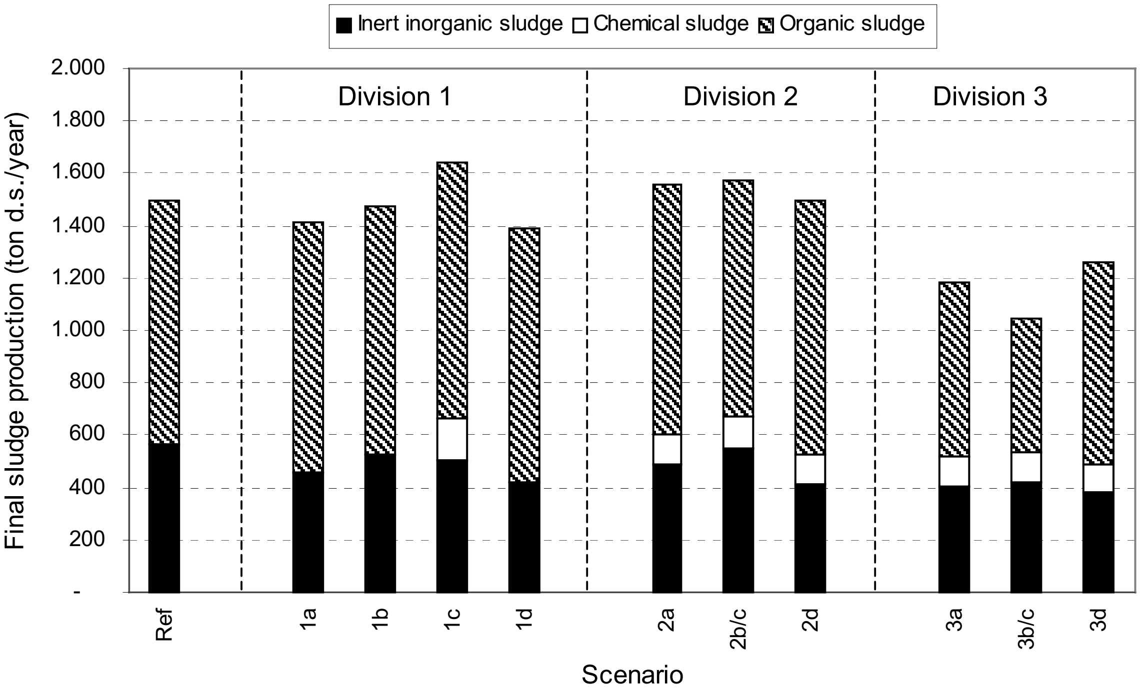

Figure 8.3: Sludge production after thickening, digestion and dewatering

As in scenario 1c, a part of the total sludge production in the scenarios of division 2 is caused by the production of chemical sludge due to the addition of metal salts for phosphorous removal. The differences in sludge production between the scenarios in division 2 are caused by the specific removal efficiencies of the applied pre-treatment unit processes.

As can be seen in figure 8.3 the overall sludge production for the scenarios in division 3 is less than the production for the scenarios in division 1 and 2 and the reference scenario. The reason for this is the assumed sludge minimising function of the membrane-bio-reactor as it is designed in the evaluation model. The sludge yield of the membrane-bio-reactor was supposed to be 0.1 kg dry solids per kg removed COD.

However, retrospectively, the low sludge production factor of the membrane-bio-reactor seems not to be applicable on semi-full scale as STOWA research recently revealed from comparative large scale pilot-plant testing at the wwtp of Beverwijk, the Netherlands [STOWA (2001b)]. Therefore, in practical application, the same sludge yield as applied in the activated sludge system should be applied. This would result in the same sludge production for the membrane-bio-reactor as for the scenarios of division 1 and 2.

Comparing the sludge production of the scenarios within division 3, more final sludge is produced when applying more advanced particle removal processes as pre-treatment step (increase from 3b/c to 3a to 3d). This is in contrast to the scenarios in division 1 and 2, where the sludge production decreases due to increasing particle removal efficiencies in the pre-treatment (from 2b/c to 2a to 2d; and 1b to 1a to 1d).

The deflecting sludge production in division 3 is caused by the earlier mentioned assumed sludge minimising function of the membrane-bio-reactor through which the organic matter in the primary effluent is more or less mineralised and the secondary sludge production from the biological post-treatment is reduced. To minimise the total final sludge production in these scenarios, the primary sludge production has to be limited by decreasing the particle removal in the pre-treatment. Starting from the assumption of the sludge minimising operation of the membrane-bio-reactor by applying a yield of 0.1 kg d.s./ kg $COD_{removed}$, advanced particle removal as pre-treatment should be omitted to produce less sludge; this in contrast to the scenarios in division 1 and 2.

External carbon source - As described before, in case of a lack of carbon in the primary effluent for a proper denitrification process in the post-treatment, methanol is dosed. Figure 8.4 presents the amount of methanol to be dosed per scenario. In the scenarios applied with direct membrane filtration (1d, 2d and 3d) the BOD/N ratio of the primary effluent has to be raised from 1.7 to 2.

The biofilm system (division 2) is designed with post-denitrifying filters, since this was identified as the most cost-effective configuration. In this process design the organic matter is degraded before the nitrate-nitrogen denitrifies. As a result of the BOD oxidation the BOD/N ratio decreases and has to be supplied by an external carbon source. This causes the methanol consumption in the scenarios in division 2. The differences in the methanol consumption of the scenarios within division 2 can be carried back to the differences in nitrogen load in the wastewater. The nitrogen concentration in the influent is changing due to the recycle flow from the sludge digestion and dewatering processes as described in the sub-paragraph ‘N and P content of the recycle water from sludge dewatering’ in paragraph 2.3. Higher nitrogen concentrations in the primary influent (see table 8.2) result in a higher external carbon consumption.

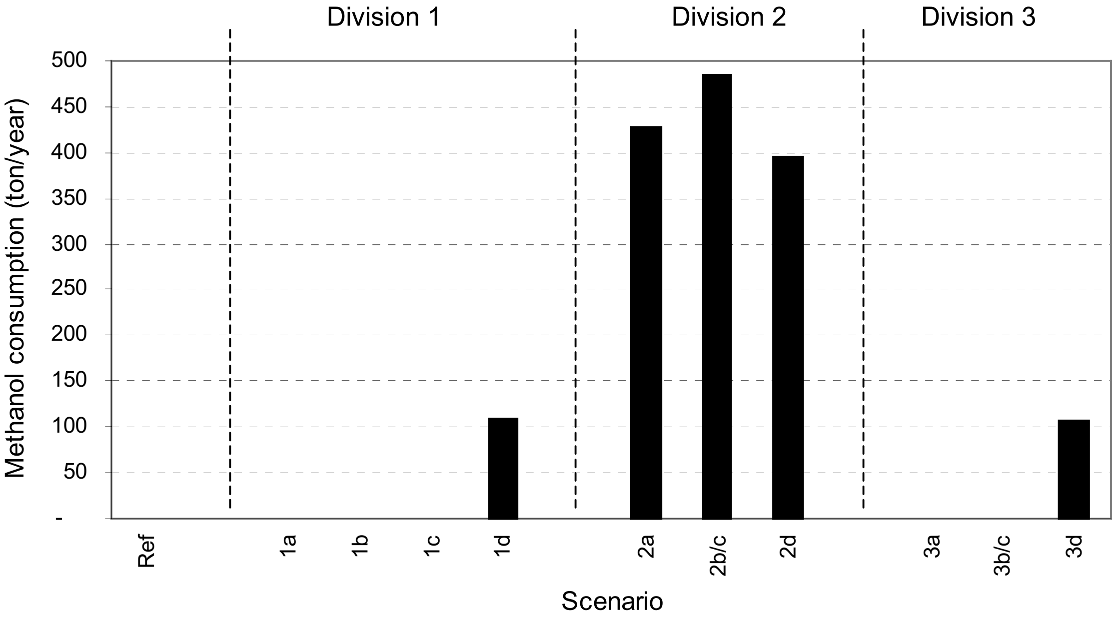

Figure 8.4: Calculated methanol consumption per scenario

Costs calculations

Figure 8.5 presents the net present value per scenario, subdivided into the investment costs and the variable operational costs. Figure 8.6 shows the relative costs per scenario related to the total net present value of the reference scenario, whereby the net present value of the reference scenario is fixed at 100%.

Compared to the reference scenario, only the scenarios applied with the polymer enhanced primary sedimentation tank combined with the activated sludge system (1a) and the biofilm system (2a) are more cost effective. However, all scenarios of division 1 and 2 are within the accuracy range of the cost calculations of 20% (see figure 8.6).

The fact that the scenarios based on direct membrane filtration are less expensive than the scenarios that are applied with the direct coarse media filters is remarkable. The more compact post-treatment and the more cost-effective sludge treatment in case of the direct membrane filtration mainly cause this cost difference.

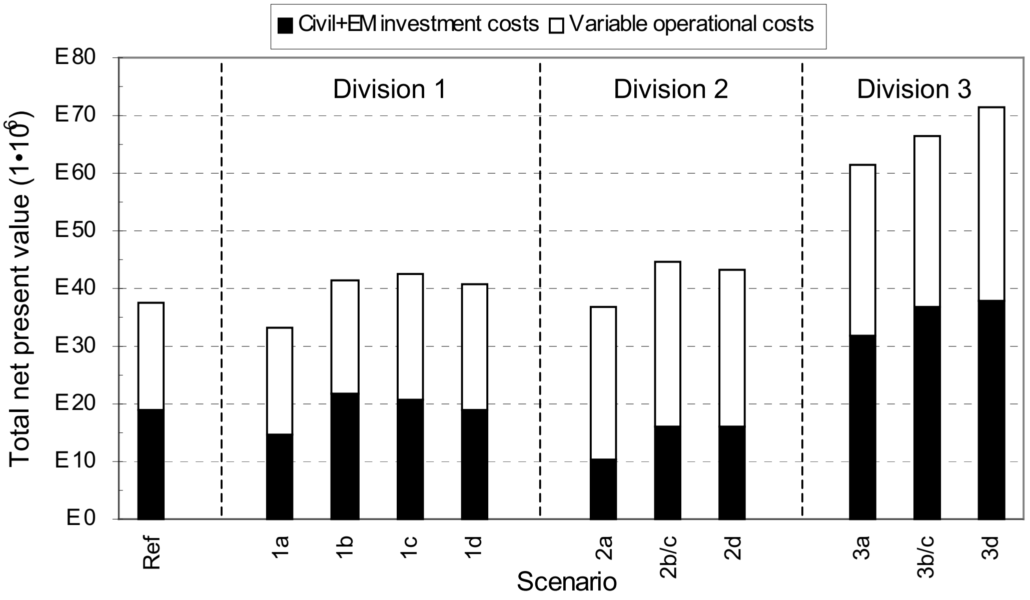

Figure 8.5: Net present values per scenario (in million €)

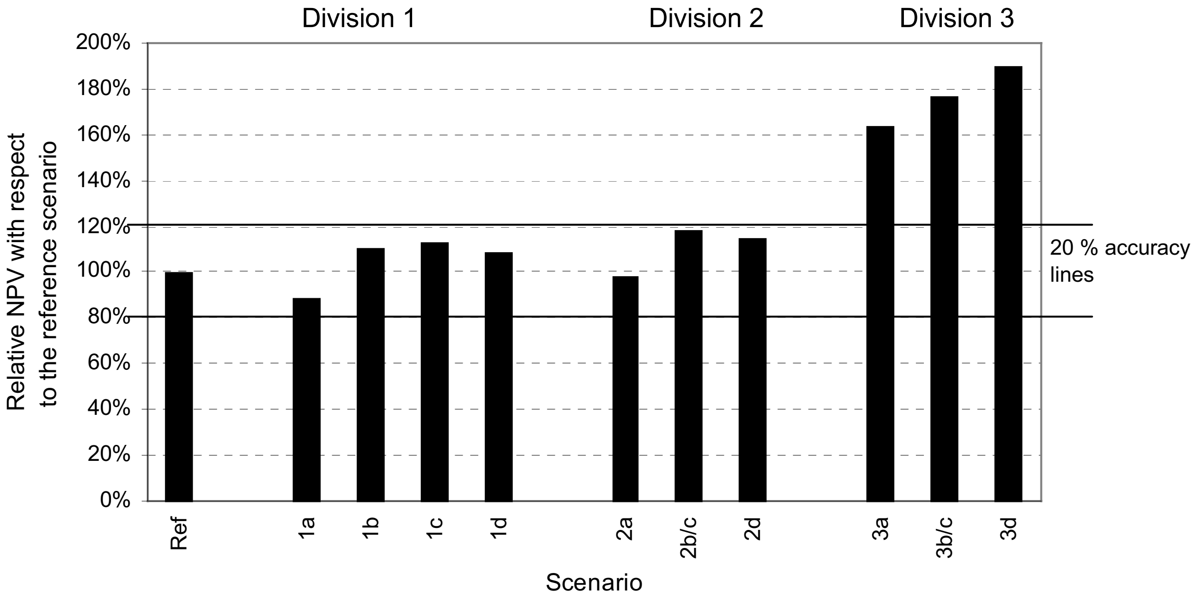

Figure 8.6: Relative cost comparison. The reference scenario is set at 100%

The scenarios based on the membrane-bio-reactor (division 3) are calculated to be approximately 1.6 to 1.9 times more expensive compared to the reference scenario, mainly caused by the application and operation of the membranes. As described earlier, the membrane-bio-reactor is designed at a low sludge yield instead of 0.4 kg d.s./kg $COD_{removed}$ as used in the activated sludge system and the biofilm system. The low sludge production results in significant lower sludge handling costs. In case the sludge yield is increased to 0.4 kg d.s./kg $COD_{removed}$ as is advised by STOWA [STOWA (2001b)] the sludge production increases and so do the costs of these scenarios.

8.4 Sensitivity analysis: influence of the particle-related COD

In order to determine the effect of the particulate matter characteristics in the wastewater on the energy consumption and the costs, the reference scenario and the scenarios from division 1 are recalculated with a varying percentage of particle-related COD. For the reference scenario and the scenarios of division 1, the calculations are conducted in DEMAS$^+$ with calculation steps of 5% over a range of the $COD_{particle}$ percentage between 50% and 75%. As described in chapter 4 and shown in table 8.3 the original percentage of COD related to particles was 64% (36% of COD is soluble). The range of 50% to 75% was selected based upon the identified bandwidth in the explorative investigation into wastewater fractionation (see chapter 4, table 4.4).

Relationship between energy consumption and $COD_{particle}$ percentage

Figure 8.7 presents the effect of the $COD_{particle}$ percentage on the overall energy consumption of the scenarios. The dotted line represents the original $COD_{particle}$ percentage of 64% as it was used in the evaluation study.

Figure 8.7 shows for all scenarios that if an increasing amount of COD in the wastewater is related to particles, the energy consumption declines. As described before, this is caused by saving aeration energy in the activated sludge system combined with the extra energy production from the digestion process. So, the increased removal of particles in the pre-treatment saves energy in the post-treatment and produces energy in the sludge treatment system.

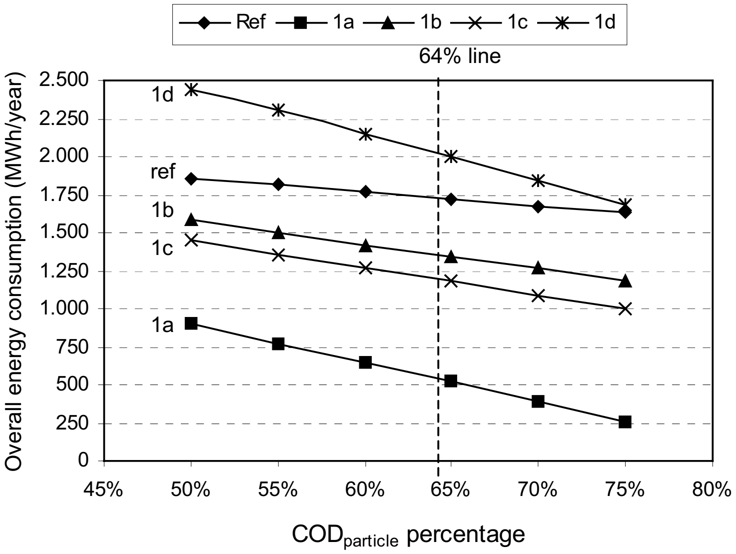

Figure 8.7: The impact of the $COD_{particle}$ percentage on the overall energy consumption per scenario

The slope of a relationship line can be interpreted as a sensitivity indicator to the changing $COD_{particle}$ percentage; high slopes indicate a large influence of the particle related COD. The slope of the lines in figure 8.7, indicate that the effect of the $COD_{particle}$ percentage on the energy consumption is stronger in case a more advance particle removal technique is applied. The slope of the relationship lines of scenario 1d and 1a are the steepest, since the particle removal efficiency in the pre-treatment of the scenario in question is 100%, respectively 80%. The energy consumption of the reference scenario, with only 30% of particle removal in the primary sedimentation tank, is less influenced by the $COD_{particle}$ percentage as is indicated by the flat slope of the relationship line.

Influence of the $COD_{particle}$ percentage on the costs

Figure 8.8 presents the development of the costs with a varying $COD_{particle}$ percentage. With exception of scenario 1d, the net present values of the scenarios do not change drastically with an increasing percentage of particle-related COD.

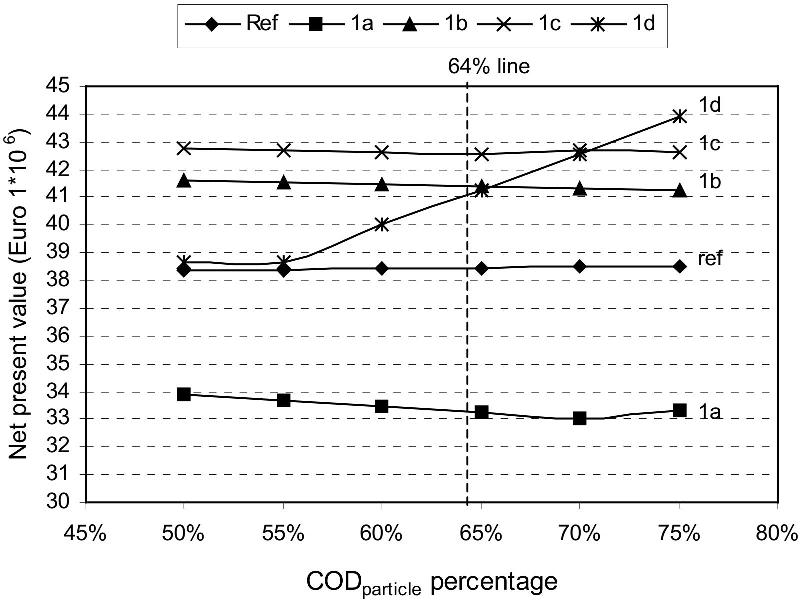

Figure 8.8: The impact of the $COD_{particle}$ percentage on the total net present value per scenario

Within the net present value of all scenarios, the investment costs shift with an increasing $COD_{particle}$ percentage from the post-treatment to the pre-treatment. This is caused by the higher particle load on the particle removing pre-treatment processes. Since the particle removal efficiency of the pre-treatment stays constant, the post-treatment is less loaded with organic matter and nutrients. This leads to a smaller required post-treatment unit, resulting in lower investment costs.

Due to a moderately increased sludge production (with increasing sludge treatment costs) caused by a shift from secondary sludge to primary sludge production, the overall costs of the reference scenario rise slightly with an increasing $COD_{particle}$ percentage.

A comparable cost development is found for the scenarios applied with the direct coarse media filter (1b and c). However, for scenario 1b the cost saving for civil structures, maintenance and energy use counterbalances the increasing sludge handling and final removal costs, so the net present value decreases very slightly.

The cost development over the $COD_{particle}$ percentage of the scenario applied with the direct membrane filtration (1d) deflects from the course of costs of the other scenarios. The costs of this scenario rapidly increase with an increasing $COD_{particle}$ percentage in the wastewater. Due to the increased particle load to the membranes the membranes assumedly have to be cleaned (with chemicals) more frequently, so the chemical consumption increases.

In addition, the increasing particle load to the membranes makes a more frequent replacement of the membrane modules necessary, so the reinvestment costs increase. With increasing particle-related contaminants, also the sludge handling and deposit costs increase slightly. Only the energy costs decrease.

The costs of scenario 1a vary slightly over the $COD_{particle}$ percentage. First, the costs decrease with the rising $COD_{particle}$ percentage. The descent reaches a minimum at a $COD_{particle}$ percentage of 70%, beyond which the net present value increases again.

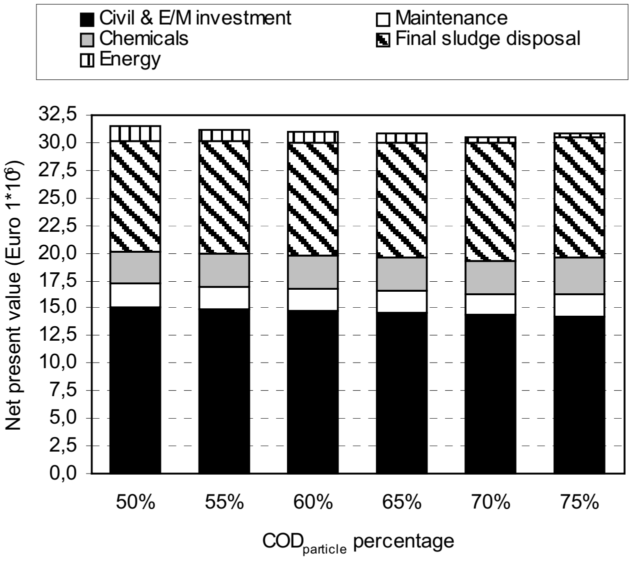

Figure 8.9: The impact of the $COD_{particle}$ percentage on the different costs for scenario 1a

A differentiated cost analysis of scenario 1a (figure 8.9) reveals that with an increase in $COD_{particle}$ percentage the investment costs and energy costs decrease. This is because the less loaded post-treatment can be designed on a smaller scale with less consumption of aeration energy. As mentioned before, the net present value rises beyond the $COD_{particle}$ percentage of 70%, after a gradual decrease. This increase in costs is caused by rising chemical costs, a result of the decreasing BOD/N ratio of the primary effluent. Exceeding 70% of $COD_{particle}$ the BOD/N ratio of the primary effluent reaches a lower value than the required 2 due to the advanced removal of particle-related COD in the pre-treatment. To guarantee a sufficient denitrification, methanol has to be added. This results in higher chemical costs and rising costs due the increase in sludge production.

8.5 Conclusions of the final evaluation

The evaluation study by means of DEMAS+ calculations reveals that the application of advanced particle removal through physical-chemical pre-treatment processes result in wastewater treatment scenarios that consume less energy, can be compactly designed and are cost effective when post-treatment is based on biological suspended activated sludge systems or biofilm systems.

The scenario using polymer enhanced primary sedimentation has a low energy consumption and low costs and has a relatively low final sludge production. The application of direct membrane filtration leads towards treatment systems with a small footprint and may have a low final sludge production.

The application of the activated sludge systems and the biofilm system as post-treatment system are from an environmental and cost efficiency point of view relatively similar and exchangeable. If space is scarce, the compact biofilm systems could be applied instead of the space consuming activated sludge system.

Due to an increasing percentage of particle-related contamination in the wastewater, the energy consumption of the scenarios decreases since more particles can be removed in the pre-treatment. The sensitivity of the scenarios to the $COD_{particle}$ percentage depends on the particle removal efficiency of the applied pre-treatment process; a higher efficiency results in an increased sensitivity.

The influence of the $COD_{particle}$ percentage on the costs is diverse. The investment and energy costs decrease whereas the sludge treatment and occasionally, the chemical costs increase with an increasing $COD_{particle}$ percentage.

8.6 References

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CHAPTER 9 EPILOGUE

9.1 Summary

The research dealt with in this dissertation had the objective identifying the specific roles of particulate matter in wastewater influent and its inherent characteristics. In addition, advanced particle removal techniques for the development of more sustainable treatment scenarios based on physical-chemical pre-treatment of wastewater were researched. The specific research theme was part of a research project with the overall objective identifying the possibilities of physical and physical-chemical pre-treatment techniques as basis for the development of more sustainable wastewater treatment scenarios in the Netherlands. Within this research, the role of particles in wastewater systems was of particular interest. Knowledge was gained about the contribution of particles regarding the pollution of wastewater due to adsorbed or incorporated pollution fractions on or in particulate material. Furthermore, the effect of particle removal in the pre-treatment on the performances of total wastewater treatment systems and the related environmental criteria and costs were explored.

Chapter 1

Chapter 1 defined the scope of the research, the topics of sustainable development regarding wastewater management and the history of physical-chemical pre-treatment of wastewater.

Since potable water was supplied in the Netherlands as of the year 1853 and sewer systems were constructed in the early 1900, surface waters have become increasingly polluted by increasing human and industrial discharges. In 1960, surface waters were so polluted that several rivers and local lakes suffered from an oxygen shortage and were considered nearly dead. With the enforcement of the Dutch Pollution of Surface Waters Act in 1970, a massive building program of wastewater treatment plants started. Water management to fight eutrophication of surface water was introduced, at first directed towards the removal of oxygen consuming substances and later towards advanced nutrient removal. In 1998, 409 municipal wastewater treatment plants with a design capacity of nearly twenty-five million population equivalents were in operation treating more than 96% of the nation-wide produced wastewater.

Although the presently existing system for sewerage and wastewater treatment operates properly, policy makers, technologists and scientists in the Netherlands are considering whether, in view of sustainable development, new or other collection and treatment concepts should be adopted for future developments. Advanced particle removal by means of physical-chemical pre-treatment is part of these developments.

Physical-chemical pre-treatment aims at the separation of particles or particulates from wastewater within the first step of a wwtp. Advanced particle removal is assumed to be important in wastewater treatment, since a major part of the influent organic matter and nutrients is associated with particulate material. By the removal of suspended colloidal particles, many pollutants incorporated into the particulate material or adsorbed to the particulate material can be eliminated. With physical-chemical pre-treatment, the post-treatment will be loaded with less contamination, and might therefore be less energy consuming and require less space and/or land.

In this context, a research project was conducted with the objective of identifying the possibilities of physical and physical-chemical pre-treatment techniques as basis for the development of more sustainable wastewater treatment scenarios. Within this investigation, the role of particles in wastewater systems is of particular interest. This dissertation answered specific questions about the impact of advanced particle removal in the pre-treatment on the development of more sustainable wastewater treatment scenarios.

Chapter 2

In chapter 2, a methodology of scenario analysis was described. The methodology was developed to identify and evaluate different wastewater treatment scenarios based on physical-chemical pre-treatment. The wastewater treatment scenarios composed and designed to meet the discharge standards in the Netherlands were evaluated by means of environmental and financial criteria. To support this methodology arithmetically, a design and evaluation model, entitled DEMAS$^+$, was developed. Within the scenario analysis, special attention was paid to the energy balance, the final sludge production, the space requirement and the economic feasibility of the identified scenarios.

The preliminary evaluation study shows that wastewater treatment systems with physical-chemical pre-treatment can potentially be designed with less energy consumption and may be designed smaller compared to the commonly applied treatment systems. The total costs of treatment scenarios, which combine physical-chemical pre-treatment with compact biological post-treatment, are in the same range as that of the reference scenario, consisting of primary sedimentation in combination with a low loaded activated sludge system and final sedimentation.

The preliminary scenario study revealed research needs into:

- extensive experimental research on wastewater fractionation and characterisation related to particles;
- investigations to replace (inorganic) metal salts by organic polymers;
- detailed experimental research on the technological operation and performance of direct filtration techniques with coarse media and membranes.

Chapter 3

Chapter 3 presented the theoretical background on the presence and behaviour of particles in wastewater-like suspensions.

It is known that a major part of the contamination in wastewater can be related to particles, since contaminants are adsorbed to or incorporated in particulate matter. However, information about the distribution of contaminants over various particle size fractions is rare. From the international literature, a survey was made to get an indication about this distribution.

Although the influence of local conditions and household and industrial habits on wastewater composition has to be considered, the survey indicates that on average about 30% of the total COD is present in soluble form. Another 30% of the COD is related to colloidal and suspended particles and approximately 40% is related to larger, settleable particulate matter. It is concluded that more information about wastewater fractionations and the characterisation of the particulate matter is needed to determine specific characteristics of an individual wastewater.

Chapter 4

Chapter 4 dealt with the experimental research into wastewater fractionation with regard to particle size. The main question in this investigation was how contaminants in wastewater are distributed over particles. This knowledge should help to predict the effect of different particle removal techniques on the primary effluent composition and may determine the requirements of the necessary post-treatment.

In the experimental research, eight wastewaters were fractionated into particle fractions using a sieve with a mesh size of 63 µm and four membrane filters with different pore sizes (5, 1.2, 0.45 and 0.1 µm). The filtrates were analysed on the components COD, BOD, nitrogen, phosphorous, suspended solids, turbidity and conductivity.

Within an 'average' wastewater, the percentage of oxygen consuming components related to the theoretically determined settleable particle fraction was low, with a maximum of 21% for COD. 44% of the BOD, 38% of the COD and 35% phosphorous was present in suspended and supra-colloidal particle fractions with particle diameters between 1.2 and 63 µm. For nitrogen only 4% could be related to settleable particles and 13% to colloidal and suspended fractions; so 83% of the nitrogen was present in soluble form.

By means of the conducted experimental research, the effect and applicability of certain physical-chemical particle removal techniques could be derived. In addition, energy and cost calculations showed the importance of particle removal in the pre-treatment down to 1 µm to save energy and costs in the total wastewater treatment system.

The successfully applied testing procedure should be carried out on a broader scale. However, the procedure can still be optimised by implementing more advanced (online) sampling, separation, analysing and monitoring techniques.

Chapter 5

The investigation into the application of organic polymers as an alternative to metal-based flocculants by means of a literature review and jar test experiments was presented in chapter 5.

Two types of organic polymers are most frequently applied in (industrial) wastewater treatment and sludge handling:

1. low to moderate molecular weight polyamines and poly-DADMACs; and
2. high molecular weight (HMW) polyacrylamides.

Conducted jar test experiments revealed that especially the cationic HMW polyacrylamides were interesting for practical application, as they are effective at relatively low dosages (5-10 mg/l). For a low molecular weight cationic polyamine, high turbidity removals were also found; this required higher dosages (20-30 mg/l), however. For both types of polymers, the experiments showed turbidity removals of 65-90% and a total suspended solids removal of more than 90%.

Due to the addition of HMW cationic polymers, large floc conglomerations with a high settleability could be produced. The flocs formed after addition of a low weight cationic polyamine appeared to be significantly smaller and less settleable, however.

Additionally, it was found that the addition of organic polymers did not affect the pH of the treated water and resulted only in a minor increase of the salt content of the primary effluent.

Chapter 6

Explorative experimental research into the application of direct coarse media filtration (DCMF) as a first treatment step in a wastewater treatment system was set up in chapter 6. The aim of the experiments was to investigate the feasibility of DCMF as a pre-treatment step for advanced particle removal. Various filter configurations were investigated in large-scale pilot-plant filters at the wwtp Leiden-Noord. The removal characteristics for suspended and colloidal material were investigated as well as operational conditions for discontinuous and continuous filters operated in up-flow and down-flow.

The experiments revealed that the discontinuous filters, operated at maximum filtration rates of 10 $m^3/m^{2\cdot}h$, had relatively short runtimes due to the occurrence of clogging. The continuous filter could be operated at filtration rates of 12.5 $m^3/m^{2\cdot}h$ without clogging.

The filters produced a filtrate with a constant quality of low concentrations of solids and low turbidity. Without the addition of coagulants only suspended solids were removed and most of the colloidal material passed the filterbeds. Addition of metal salt coagulants resulted in phosphorous precipitation and improved the total particle removal. Addition of organic polymeric flocculants resulted in the formation of large structures that caused cake layer formation on the top or bottom layer of the filter, resulting in fast clogging.

Based on the research results, discontinuous direct influent filtration is not yet advised as a pre-treatment technique. To make discontinuous direct wastewater filtration suitable for practical application, more research has to be done and more technological adjustments experimented with.

Since the removal efficiencies of the continuous or discontinuous filter configurations did not differ significantly, continuous filtration would be preferred from an operational point of view in the case of (a practical application of) direct coarse media filtration. Further research into the continuous operation and specific the clogging prevention by means of filter control and water quality monitoring is recommended in order to develop a possible practical application of this technique.

Chapter 7

Chapter 7 described pilot-scale research into direct membrane ultrafiltration of raw wastewater. The aim of this experimental research was to explore the characteristics of direct membrane filtration as a pre-treatment technique for advanced particle removal. Removal performances and operational process conditions were investigated with a pilot-plant membrane installation operated discontinuously with maximum runtimes of seven hours.

From the explorative experiments it can be stated that pre-treatment of raw wastewater with membranes is possible from an operational point of view and that it has remarkable prospects regarding permeate quality and reuse. The investigations indicated a possible average normalised flux of 300 $l/m^2 \cdot h \cdot bar$ at a trans-membrane pressure of 0.2 to 0.4 bar and a cross-flow rate of 2.4 m/s. Without backflushing, it was even possible to keep the flux at a constant level, implicating a limited fouling of the membranes. The cross-flow velocity played an important role in this process, mainly due to the shear stress it caused along the membrane surface.

Removal performances were excellent since all suspended and colloidal substances were removed by the membranes with pores of approximately 30 nm; 65% of COD, 50% of BOD, 30 % of phosphorous and about 20% of nitrogen were removed.

In addition, direct ultrafiltration of raw wastewater yielded a useful permeate. It was assumedly free from bacteria and viruses and contained a considerable amount of easily accessible nutrients. Thus, it can be applied in irrigation and supply the nutrient needs of different types of crops.

Chapter 8

Chapter 8 discussed the upgraded comparative evaluation study of selected wastewater treatment scenarios based on the physical-chemical particle removal pre-treatment techniques. Next to the reference treatment scenario, scenarios with polymer enhanced primary sedimentation as investigated in chapter 5 were calculated. In addition, two configurations of direct coarse media filtration, as well as direct membrane filtration were used as pre-treatment processes as they were investigated in chapter 6 and 7. The scenarios were arranged into three divisions, differentiated by means of the applied post-treatment technique. In addition to the evaluation, a sensitivity analysis was carried out to investigate the effects of changing particle fractionations in the wastewater on the treatment scenarios as identified and described in chapter 4.

As in the preliminary evaluation in chapter 2, the upgraded scenario analysis revealed that advanced particle removal in the physical-chemical pre-treatment results in wastewater treatment scenarios that consume less energy, use less space and are cost effective when the post-treatment is based on biological suspended activated sludge systems or biofilm systems.

From the scenario comparisons it could be concluded that when polymer enhanced primary sedimentation has taken place energy consumption, sludge production and costs are most attractive. The application of direct membrane filtration resulted in the most compact treatment systems and low sludge production.

With increasing percentages of particle-related contamination in wastewater, the energy consumption of the scenarios decreases since more particles can be removed in the pre-treatment. The sensitivity of the scenarios to the $COD_{particle}$ percentage depends on the particle removal efficiency of the applied pre-treatment process; a higher efficiency results in an increased sensitivity.

The influence of the $COD_{particle}$ percentage on the costs is diverse. The investment and energy costs decrease whereas the sludge treatment and occasionally the chemical costs increase with an increasing $COD_{particle}$ percentage.

9.2 Final remarks and considerations regarding future developments

Within the following paragraphs, final remarks are made and considerations are given with regard to future developments in the wastewater system.

Physical-chemical pre-treatment in the context of sustainability

The original overall objective of the conducted research project was to identify the possibilities of physical-chemical pre-treatment techniques such that more sustainable wastewater treatment could be developed. As is presented in this dissertation, advanced particle removal by physical-chemical pre-treatment leads towards less energy consuming treatment plants, which can be designed at a smaller footprint. So, within the scope of the research project, advanced particle removal in physical-chemical pre-treatment was identified of having positive effects on the overall wastewater treatment system. Especially when starting from the commonly applied physical-chemical pre-treatment, it is identified that the replacement of inorganic metal salts by organic polymers has positive effects on the final sludge production and the salinity of the effluent.

However, the question remains whether physical-chemical pre-treatment really contributes to an increase in sustainability of the wastewater treatment, in virtue of only savings of energy and space requirement. As set out in chapter 1, sustainability is subject to local situations and time. So, the effects of physical-chemical pre-treatment should be regarded in the context of time and local conditions. Having in mind that wastewater treatment in general is already a very sustainable process [Harremöes (2000)], that effluent quality and sludge production are most important, and that energy consumption is seen as less important regarding sustainable development of wastewater systems, physical-chemical pre-treatment seems to contribute only minorly to the increase of sustainability.

However, advanced particle removal in the pre-treatment can be very effective to optimise wastewater treatment operations concerning energy consumption and space requirement. In situations (determined by time and local conditions) that these environmental interventions are of major importance, advanced particle removal in the pre-treatment is favourable from operational point of view and could lead towards more sustainable wastewater treatment processes.

Energy - Research by for example Ødegaard (1995) and Van Loosdrecht (1997) had also identified the energy saving capacity of physical-chemical pre-treatment. Energy saving is considered recommendable since energy resources such as oil and gas are finite and energy production and use is related to CO_2 emissions. However, wastewater treatment is, compared to other activities, a relative low energy demanding operation with an average continuous consumption per population equivalent of 2.7 Watt. Since energy consumption was identified of being only a minor sustainability criterion [Roeleveld *et al.* (1997), Graaf, van der, *et al.* (1997)], occasionally, the application of high energy demanding techniques can improve the sustainability of wastewater treatment, if other, more important sustainability criteria (as effluent quality and sludge production) are optimised by these techniques.

However, within the scope of the applications of alternative, ‘more sustainable’ ‘green’ energy sources, optimising the energy balance at wastewater treatment plants could be interesting, since wastewater treatment plants may even produce ‘green’ energy themselves by sludge digestion; a process that is positively affected by physical-chemical pre-treatment.

Footprint - While energy consumption is a time-related sustainability criterion, space requirement is a location specific item. In regions where space is precious due to dense population or due to specific soil types, footprint is of major importance. In densely populated urban areas like the western part of the Netherlands, application of compact, low space requiring treatment systems can sometimes be favourable from the space-saving point of view, but it is also based on economical motives. However, it should be kept in mind that the net space requirement of wastewater treatment plants is of minor importance, since the ‘real’ space impacted by wastewater treatment systems is mostly determined by the odour contours, which exceed the footprint of the civil structures.

An example of a wastewater treatment plant where area use is of importance is the VEAS wwtp near Oslo, Norway. This wwtp is built completely underground into the hard rock, so the soil type determines the footprint. Their treatment concept is based on compact chemically enhanced primary settling in combination with nitrifying and denitrifying biofilm filters [Sagberg *et al.* (1998)]. The VEAS treatment plant is comparable to the evaluated treatment scenario 2a in DEMAS$^+$.

Heavy metals - An identified topic of importance regarding the increase in sustainability of the wastewater treatment system is the discharge of micro-pollutants to the surface water [Graaf, van der *et al.* (1997), Roeleveld *et al.* (1997)]. In relation to this, advanced particle removal in the pre-treatment, and especially complete particle removal as in direct membrane filtration, could be of major importance since heavy metals can be related largely to particulate matter [Ødegaard (1987), Mels (2001)]. This research topic was not explored in detail within this project. Advanced particle removal in wastewater treatment systems could, however, contribute to the improvement of sustainability due the reduction of heavy metal discharge to surface water.

Remarks about polymer addition for flocculation

In chapter 5 of this dissertation, organic polymers were identified to be very effective for flocculation of particles in raw wastewater, with resulting high achievable removal efficiencies for particulate matter in separation techniques. The high particle removal efficiency results in very effective treatment systems. Compared to the addition of metal salts, organic polymer addition is favourable from an operational point of view since extra chemical sludge production is absent, the salinity of the primary effluent will not increase and the pH of the pre-treated wastewater will not be influenced. Nevertheless, some comments have to be made about the application of synthetic organic polymers.

Toxicity effects - As described in chapter 5, uncertainties are present about the fate of by-products and residuals of the applied organic polymers [Umwelt Bundesamt (1997), Barvenik (2000)]. Eco-toxicity effects of monomers related to organic flocculants are suspected, but no closely-reasoned conclusions have been made so far. However, if toxicity questions remain unanswered, large-scale application of synthetic polymers should be reconsidered and further research into the toxicity of these products conducted. An alternative to the use of synthetic polymers is the application of biosynthesised polymers as were described in chapter 5.

Production energy of polymers - The high energy demand of the production process of synthetic polymers is an important aspect of its use. The production of synthetic polymers demands eight times more energy per load active product than the production per load active Fe^{3+} [Hellström (1998)]. In spite of the the lower required dosage and the decrease in energy consumption at the site of the wwtp due to the application of polymeric flocculants in the pre-treatment, the high energy demand for the production might result in an overall negative energy balance when broadening the system boundaries [STOWA (2001)]. On the other hand, as reasoned before, the increase of energy input could be beneficial to the sustainability of the wastewater treatment system since the sludge production does not increase.

Polymers in combination with metal salts - As described earlier, in some cases the complete replacement of metal salts by organic polymers is not possible. When biological phosphorous removal is not applicable, precipitation should be applied to guarantee the effluent quality [Harleman and Murcott (2001 a/b), Parker *et al.* (2001)]. In these cases, an optimised combination of metal salts and polymers might be applied such that the particles are removed with polymers and the sulphides and phosphorous with metal salts in an optimal way.

In addition, when sulphides are present in the wastewater and physical or biological removal techniques are not applicable, precipitation by addition of metal salts is most favourable.

(Re)use of wastewater and recovery of valuable components

Within the scope of sustainability of the wastewater treatment system, (re)use of water and recovery of valuable components from wastewater was earlier identified as being important [Rulkens and Starkenburg, van (1993)]. Physical-chemical pre-treatment could contribute to the (re)use and recovery concepts.

(Re)use of wastewater - One example of a technique that enables direct reuse of wastewater is direct membrane filtration. Direct membrane filtration produces a permeate that can be used directly for irrigation purposes. The permeate is free of bacteria and viruses and has an ideal C:N:P:K ratio for the cultivation of various crops (e.g., corn, soybeans and wheat). Then, not only the water is used efficiently but also the available nutrients are usefully applied [Evenblij *et al.* (2001)]. However, salts and dissolved contaminants are not removed by direct membrane filtration.

Advanced particle removal by means of physical-chemical pre-treatment can also have a positive effect on the operation of final effluent treatment techniques like rapid sand filtration or membrane filtration since the major part of inert particles are already removed in the pre-treatment and do not disturb the final filtration processes [Koning, de, and Nieuwenhuijzen, van (1999)]. The improved operation of these processes results in an increase in effluent quality at lower costs.

P-recovery - Considering recovery of nutrients, phosphorous recovery from wastewater is an issue brought up by the phosphorous producing industries since the quantity of good quality “clean phosphorous” from $Ca_3(PO_4)_2$ ores are limited. The phosphate producing industries prefer concentrated, clean, non-metal bound phosphorous, so P-precipitation from wastewater with metal salts is not favoured. Other ways of removing phosphorous from wastewater and concentrating it in suitable forms are therefore under investigation [CEEP/CEFIC (2001)].

Physical-chemical pre-treatment enhanced with organic polymers could be of importance in wastewater treatment concepts wherein P-recovery is an option. Treatment concepts that combine denitrification of nitrate with biological P-removal by denitrifying phosphorous bacteria are most suitable for this purpose [Loosdrecht *et al.* (1998)]. In these treatment concepts, concentrated phosphorous can be recovered from the wastewater for example by applying a P-stripper at a point where the phosphate concentrations in the wastewater are the highest, i.e. at the end of the anaerobic zone [Brandse (2000), Klapwijk (2001)].

It was identified that treatment plants based on nitrogen and phosphorous removal with denitrifying phosphorous bacteria are operated most efficiently when advanced particle removal by means of physical-chemical pre-treatment is applied [Loosdrecht, van *et al.* (1997), Brandse (2000)]. On the other hand, phosphorous should remain in the primary effluent to be recovered, and not be precipitated by metal salts. To meet these requirements, advanced particle removal with organic polymer enhanced pre-treatment techniques is favourable.

Physical-chemical pre-treatment and possible future developments

Sludge treatment - Sludge production is seen as one of the most important bottlenecks of wastewater treatment systems [Graaf, van der *et al.* (1997), Roeleveld *et al.* (1997)]. Physical-chemical pre-treatment is often related to an increased sludge production due to the formation of metal containing inorganic sludge. This problem is tackled by applying organic polymers instead of metal salts.

In addition, sludge could be regarded as a source of valuable products. First of all, as energy source, but also as a source of nutrients and carbon. Physical-chemical pre-treatment increases the ratio between primary and biological sludge, resulting in an increasing biogas yield from the sludge digestion. Besides digestion, sludge treatment techniques based on hydrolysing processes may create the possibility of recovering valuable products as nutrients and liquid carbon sources from sludge [Andersson (2000), Machenbach (2000)].

Urine separation - Another interesting development in wastewater treatment is the exploration into the possibilities of separation of urine from the wastewater. Since the major part of the soluble nitrogen and phosphorous in wastewater originates from urine, the remaining wastewater after urine separation contains mainly particle-related contaminants [STOWA (2001b)]. The remaining wastewater is perfectly suitable for physical-chemical pre-treatment and even complete physical-chemical treatment could be applied.

9.3 Overall conclusion

The research question of this dissertation, as explained in chapter 1, can be answered as follows: *Since a major part of contaminants in wastewater are related to particles, advanced particle removing techniques play an important role in optimising the wastewater treatment process.*

Through the evaluating scenario analysis, it is identified that through advanced particle removal processes in the physical-chemical pre-treatment, wastewater treatment plants can be operated using less energy than treatment plants without pre-treatment and may be designed on a smaller scale.

Wastewater fractionation and characterisation of particulate material in wastewater give a deeper understanding of the effect and applicability of a considered physical-chemical particle removal technique. By wastewater fractionation, it can be determined down to which particle size pre-treatment should be applied to optimise the treatment system.

Due to the replacement of metal salts by organic polymers, advanced flocculation of particles can be achieved without the production of inert chemical sludge and without an increasing salt content in the primary and final effluent. Due to the addition of high molecular weight cationic polymers, large strong flocs with excellent sedimentation properties can be produced. The potential eco-toxicity and the lifecycles of the organic polymers should be researched, however, before large-scale operations are warranted.

Filtration techniques for advance particle removal from raw wastewater could be applied as pre-treatment step instead of sedimentation. Especially since these techniques are capable to remove particles down to small particle sizes without the addition of coagulants and flocculants. Direct membrane filtration of wastewater is identified by means of experimental research to be applicable from operational point of view. This pre-treatment technique removes all colloidal and particulate material down to the applied membrane pore size, whereby completely new treatment concepts become possible and direct reuse of wastewater seems to be applicable.

Last but not least, the developed scenario analysis in combination with the evaluation tool DEMAS$^+$ is an important end product of this research. Scenario analyses based on environmental and cost criteria should be used more often when comparing different wastewater treatment options. The design and evaluation model for wastewater treatment scenarios (DEMAS) is a major support tool in conducting these analyses. Further development of DEMAS into sludge treatment and applicability is recommendable.

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Scenariostudies naar vergaande deeltjesverwijdering in de fysisch-chemische voorzuivering van afvalwater

SAMENVATTING

Inleiding - Het in dit proefschrift beschreven onderzoek gaat in op de specifieke rol van deeltjes in stedelijk afvalwater bij het streven om tot een minder milieubelastende, c.q. meer duurzame, afvalwaterbehandeling te komen. Daarbij wordt de toepassing van fysisch-chemische voorzuivering als eerste stap bij de zuivering van stedelijk afvalwater beschouwd. Deze voorzuiveringsstap heeft tot doel om onopgelost materiaal (deeltjes en colloïdale stoffen) af te scheiden van de waterstroom en zodoende een groot deel van de vervuilende stoffen relatief eenvoudig te verwijderen. Volgende zuiveringsstappen, die veelal biologisch van aard zijn en efficiënt opgeloste verontreinigingen kunnen verwijderen, worden hierdoor minder zwaar belast met inert materiaal. Het gehele zuiveringssysteem zou hierdoor efficiënter moeten functioneren.

Probleemdefinitie - Binnen het algemene onderzoeksthema dat de identificatie van de mogelijkheden van fysische en fysisch-chemische voorzuiveringstechnieken als basis voor een meer duurzaam zuiveringssysteem in Nederland behelsde, was de rol van deeltjes in deze systemen van bijzonder belang (hoofdstuk 1). Binnen het onderzoek is kennis vergaard omtrent de bijdrage van inert materiaal aan de vervuilingsgraad van afvalwater door adsorptie of insluiting van vervuilende stoffen in of aan deeltjes en colloïdalen. Tevens zijn de effecten van deeltjesverwijdering in de voorzuivering op de prestaties van het gehele zuiveringssysteem en de invloed op milieucriteria en kosten verkend.

Methodologie en scenarioanalyses - Voor het onderzoek is de methodologie van scenarioanalyses gebruikt. In dit geval werd een specifieke methode ontwikkeld waarbij verschillende afvalwaterzuvieringsscenario's gebaseerd op fysisch-chemische voorzuivering worden geïdentificeerd en vervolgens geëvalueerd worden op gedefinieerde milieucriteria en economische mogelijkheden. Voor deze methode werd een model ontwikkeld, genaamd DEMAS$^+$, waarin de zuiveringsscenario's gedimensioneerd kunnen worden en een evaluatie op basis van de milieucriteria en kosten plaats vindt. Uit een eerste evaluatie kwam naar voren dat vergaande deeltjesverwijdering in de fysisch-chemische voorzuivering tot ruimte- en energiebesparing kan leiden, betere zuiveringsprestaties behaald kunnen worden en mogelijk waardevolle componenten uit afvalwater teruggewonnen kunnen worden. Dit is mogelijk tegen vergelijkbare kosten als die van de huidige conventionele zuiveringssystemen (hoofdstuk 2). Voor een nauwkeurige analyse werden de volgende onderzoekspunten geïdentificeerd:

- onderzoek is vereist naar vergaande fractionering van afvalwater en karakterisering op deeltjes;
- de mogelijkheden van de vervanging van anorganische metaalzouten door organische polymeren als flocculant dienen uitgezocht te worden;
- gedetailleerd experimenteel onderzoek is nodig naar de mogelijkheden van vergaande deeltjesverwijderingstechnieken zoals directe grove-mediafiltratie en membraanfiltratie.

Samenstelling van afvalwater - Om kennis te vergaren omtrent de samenstelling van stedelijk afvalwater en het belang van deeltjes hierin, is een uitgebreid literatuuronderzoek uitgevoerd (hoofdstuk 3). Ondanks dat de lokale omstandigheden en gewoontes in huishoudens en industrie de samenstelling van afvalwater beïnvloeden, kon gesteld worden dat gemiddeld 30% van het totale CZV in opgeloste vorm in afvalwater aanwezig is. Verder kan 30% van het totale CZV gerelateerd worden aan colloïdale en gesuspendeerde deeltjes en ongeveer 40% wordt geassocieerd aan grote, bezinkbare deeltjesmateriaal. Voor andere vervuilingscomponenten zijn vergelijkbare grove kengetallen beschikbaar. Uit de literatuurstudie blijkt dat meer informatie over de samenstelling en fractionering van afvalwater op deeltjesverdeling, met name voor de Nederlandse situatie, nodig is om uitspraken te kunnen doen over het nut van vergaande deeltjesverwijdering in de voorzuivering en om zuiveringsmodellen gedetailleerder uit te breiden met fysisch-chemische verwijderingtechnieken.

Influentfractionering - In hoofdstuk 4 werd op bovengestelde kennisleemte ingespeeld met de behandeling van een uitgebreid experimenteel onderzoek naar fractionering van afvalwater op deeltjesgrootte. De onderzoeksvraag hierbij was hoe verontreinigingen in afvalwater verdeeld zijn over deeltjesgrootte. Deze kennis draagt bij aan de voorspelling van het nut en het effect van vergaande deeltjesverwijdering in de voorzuivering. In het experimentele onderzoek werden acht verschillende afvalwaters gefractioneerd door zeven en membraanfilters met afscheidingsdiameters van 63; 5; 1,2; 0,45 en 0,1 µm. De filtraten werden geanalyseerd op onder anderen CZV, BZV, stikstof, fosfaat, zwevendestof, troebelheid en geleidbaarheid.

Door statistische berekeningen is een ‘gemiddelde afvalwatersamenstelling en fractionering’ opgesteld. Hieruit bleek dat het percentage van zuurstofconsumerende stoffen dat gerelateerd kon worden aan een theoretisch bezinkbare fractie slechts 21% van het totale CZV bedroeg. 44% van het totale BZV, 38% van het totale CZV en 35% van het fosfaat bleek aanwezig te zijn in gesuspendeerde en supra-colloïdale vorm, met deeltjesdiameters tussen 1,2 en 63 µm. Stikstof bleek voor slechts 4%

geassocieerd te kunnen worden aan bezinkbare deeltjes en 13% aan colloïdale and suspendeerde fracties. Zodoende is 83% van de totale stikstof aanwezig in opgeloste vorm.

Toepassing van organische polymeren als flocculant - Het onderzoek naar de toepassing van organische polymeren als alternatief voor anorganische metaalhoudende flocculanten werd uitgevoerd door middel van literatuuronderzoek en experimentele bekerglasproeven (hoofdstuk 5).
Twee typen polymeren worden regelmatig toegepast in de (industriële) afvalwaterzuivering en slibbehandeling:

3. laag- tot middelmoleculaire (LMW en HMW) polyamines en poly-DADMACs;
4. hoogmoleculaire (HMW) polyacrylamides.

Uit de uitgevoerde bekerglasproeven bleek dat vooral kationische hoogmoleculaire polyacrylamides potenties bieden voor praktijktoepassingen, omdat deze polymeren reeds bij relatief lage doseringen (5-10 mg/l) zeer goed flocculatie-eigenschappen vertonen en een helder deeltjesvrij supernatant produceren. Met laagmoleculaire kationische polyamines konden hoge verwijderingsrendementen voor troebelheid behaald worden, waarbij echter hogere doseringen (20-30 mg/l) nodig waren. Voor beide typen polymeren toonden de experimentele bekerglasproeven verwijderingsrendementen aan van 65-90% voor troebelheid en > 90% voor zwevendestof.

Door toevoeging van HMW kationische polymeren aan afvalwater worden grote, sterke vlokstructuren geproduceerd met hoge bezinksnelheden. De vlokken die geproduceerd werden met LMW kationische polymeren waren aanzienlijk kleiner en minder goed bezinkbaar.

In aanvullende proeven werd gevonden dat door toevoeging van organische polymeren de pH van het afvalwater niet beïnvloed werd en slechts een marginale verhoging van het zoutgehalte van het water tot gevolg had.

Grove-mediafiltratie van ruw afvalwater - Een uitgebreid verkennend experimenteel onderzoek aan grootschalige pilot-plants naar de toepassingsmogelijkheden van continue en discontinue directe (grove-media)filtratietechnieken als eerste zuiveringsstap werd beschreven in hoofdstuk 6.

Gebaseerd op de onderzoeksresultaten kon momenteel discontinue influentfiltratie nog niet geadviseerd worden voor praktijktoepassing aangezien de korte haalbaar geachte looptijden die bepaald werden door verstopping van het filterbed. Om discontinue influentfiltratie toepasbaar te maken voor de praktijk zijn aanpassingen aan ge beproefde opstellingen nodig en dient verder onderzoek uitgevoerde te worden.

Omdat de verwijderingsprestaties van de continue en discontinue filters nauwelijks verschilden en de bedrijfsvoering van continue filters meer opties biedt voor een storingsvrije bedrijfsvoering, wordt continue filtratie geprefereerd boven discontinue directe filtratie. Echter dient ook voor deze configuratie verder onderzoek uitgevoerd te worden om tot een praktijktoepassing te komen.

Directe membraanfiltratie - Hoofdstuk 7 lichtte een verkennend onderzoek op pilot-plantschaal toe naar de toepassingsmogelijkheden van directe membraanfiltratie van ruw afvalwater Het doel van dit onderzoek was om de mogelijkheden en karakteristieken van directe membraanfiltratie als eerste zuiveringsstap voor vergaande deeltjesverwijdering te identificeren aan de hand van korte-duur testen.

Uit het onderzoek kan in eerste instantie geconcludeerd worden dat vanuit operationeel oogpunt voorzuivering van ruw afvalwater door membranen mogelijk is en dat hierdoor een filtraatkwaliteit geleverd wordt die geschikt lijkt voor direct hergebruik in plaats van verdere verwerking in een biologisch zuiveringssysteem. Het onderzoek openbaarde mogelijke gemiddelde genormaliseerde fluxen van 300 $l/m^2 \cdot h \cdot bar$ bij een transmembraandruk van 0,2 tot 0,4 bar en een cross-flowsnelheid of 2,4 m/s. Zonder terugspoeling van het membraan kon een evenwichtssituatie gecreëerd worden waarbij de flux constant bleef bij gelijkblijvende druk. Dit impliceert het continue afspoelen van de gevormde koeklaag door de cross-flowsnelheid en een gelimiteerde persistente vervuiling van het membraan. De cross-flowsnelheid is hierbij van groot belang door de opwekking van de schuifspanning op de membraanwand.
De verwijderingsprestaties van directe membraanfiltratie waren, zoals verwacht, uitstekend. Alle gesuspendeerde en colloïdale deeltjes groter dan de poriëndiameter van het membraan (30 nm) worden samen met 65% van het CZV, ruim 50% van het BZV, 30% fosfaat en circa 20% stikstof uit het afvalwater verwijderd. Alle in het permeaat aanwezige organische verbindingen en nutriënten zijn in gemakkelijk opneembare opgeloste vorm aanwezig. Tevens is het permeaat vrij van bacteriën en bevat nauwelijks nog virussen. Het zodoende geproduceerde permeaat blijkt van zulke samenstelling dat het direct nuttig zou kunnen worden gebruikt als nutriëntenhoudend irrigatiewater voor verschillende soorten gewassen.

Evaluatie en gevoeligheidsanalyse - Uiteindelijk werd in hoofdstuk 8 een samenvattende verbeterde evaluatiestudie uitgevoerd waarin geselecteerde afvalwatersystemen gebaseerd op de onderzochte fysisch-chemische voorzuiveringstechnieken werden vergeleken op milieu-ingrepen en kosten. Hiervoor werden in het evaluatiemodel DEMAS$^+$ de uit het experimenteel onderzoek verkregen gegevens geïntegreerd. Naast een referentiescenario dat een huidig conventioneel zuiveringssysteem vertegenwoordigt, werden scenario's met polymeergedoseerde vergaande voorbezinking zoals onderzocht in hoofdstuk 5 ontwikkeld. Daarnaast werden zuiveringsscenario's opgesteld op basis van zowel een continue als een discontinue uitvoeringsvorm van directe grove-mediafiltratie (hoofdstuk 6) en directe membraanfiltratie (hoofdstuk 7).

De ontwikkelde scenario's werden ingedeeld in drie categorieën, onderscheiden door het type toegepaste nazuivering (actiefslib, slib-op-drager en membraanbioreactor). Naast de evaluatiestudie werden enkele gevoeligheidsanalyses uitgevoerd waarin het effect van veranderende deeltjesgrootteverdelingen in het afvalwaterinfluent (zoals onderzocht in hoofdstuk 4) op de prestaties, milieu-ingrepen en kosten van de zuiveringsscenario's werd bepaald.
De eerste voorlopige evaluatiestudie uit hoofdstuk 2 bevestigend identificeerde de opgewaardeerde evaluatiestudie de potentie van fysisch-chemische voorzuivering om door vergaande deeltjesverwijdering energie en ruimte te besparen. Dit kan zeer kosteneffectief indien de vergaande deeltjesverwijdering in de voorzuivering gecombineerd wordt met geoptimaliseerde actief-slibsystemen of slib-op-dragersystemen. Uit de scenariovergelijkingen bleek verder dat door toepassing van polymeerondersteunde voorbezinking de meest efficiënte zuiveringsscenario's gecreëerd kunnen worden. Gebruik van directe membraanfiltratie resulteerde in de meest compacte zuiveringssystemen en lage slibproducties.

Uit de gevoeligheidanalyse op deeltjesverdeling in afvalwater bleek dat door een toenemend percentage van aan deeltjes gerelateerde verontreinigingen het energieverbruik van de zuiveringssystemen daalde omdat relatief meer aan deeltjes gerelateerde verontreinigen reeds in de voorzuivering energie-efficiënt verwijderd worden. De afhankelijkheid van de zuiveringsscenario's voor het $\text{CZV}_{\text{deeltjes}}$-percentage wordt in grote mate bepaald door de deeltjesverwijderingsprestaties van de toegepaste voorzuiveringstechnieken; een hoger verwijderingrendement in de voorzuiveringsstap leidt tot een toenemende afhankelijkheid voor de deeltjesverdeling in het afvalwater. De invloed van het $\text{CZV}_{\text{deeltjes}}$-percentage op de kosten is van diverse aard. De investeringskosten en energykosten nemen af bij een toenemende $\text{CZV}_{\text{deeltjes}}$-percentage, terwijl de kosten voor slibverwerking en deels de chemicaliënkosten toenemen.

Conclusie - Door experimenteel verkennend onderzoek aan deeltjesverwijderingstechnieken in de voorzuivering en evaluerende scenarioanalyses werd bepaald dat toepassing van vergaande deeltjesverwijdering in de fysisch-chemische voorzuivering kan leiden tot afvalwaterzuiveringssystemen die ontworpen kunnen worden met een lager ruimtegebruik en bedreven kunnen worden met een lager energieverbruik vergeleken met afvalwaterzuiveringsinrichtingen zonder fysisch-chemische voorzuiveringsstappen. Dit is mogelijk tegen vergelijkbare kosten.

A.F. van Nieuwenhuijzen, januari 2002

TERMINOLOGY AND ABBREVIATIONS

BioSS:
inert solids content of the grown biomass; in DEMAS assumed to be 18%.

BOD = biochemical oxygen demand (mg O_2/l):
the oxygen equivalent of the organic matter that can be oxidised biochemically by micro-organisms. Mostly determined as BOD_5^{20} (oxygen demand over 5 days at 20 ºC).

coagulation:
the process of charge neutralisation resulting in destabilisation of the particles.

COD = chemical oxygen demand (mg O_2/l):
the oxygen equivalent of the organic matter that can be oxidised by using a strong chemical oxidant in an acidic medium (usually potassium dichromate).

DAF = dissolved air floatation

DCMF = direct coarse media filtration

DEMAS = 'Dimensionerings- en Evaluatiemodel voor AfvalwaterSystemen':
dutch for 'design and evaluation model for wastewater treatment systems'; calculation and evaluation tool developed to conduct scenario analysis.

DMF = direct membrane filtration

d.s. = dry solids:
dried residue of insoluble components from wastewater or wastewater sludge after evaporation at 105 ºC.

dwf = dry weather flow

effluent:
outgoing flow of treated wastewater.

environmental intervention:
defined within the LCA-methodology as 'the exchanges between the anthroposphere ('economy') and the environment including resource use, emissions to air or soil and discharge to water'.

flocculation:
the process of complex formation that is succeeding the destabilisation (coagulation).

F/M ratio = food to mircro-organismn ratio or food-to-mass ratio (1/d):

commonly used parameter to design the activated sludge process; defined as influent load of BOD per load mixed-liquor suspended solids (MLSS) per day.

footprint:

net space or area requirement of a process or scenario.

influent:

incoming flow of water or wastewater.

LCA = life cycle analysis or assessment:

LCA refers to the analysis of the total environmental impact of a product through every step of its life - from obtaining raw materials, through production, use, and disposal.

N = nitrogen:

all nitrogen components in wastewater: ammonia, nitrate, nitrite.

$N_{kjeldahl}$:

total ammonia nitrogen and organic bound nitrogen.

NPV = net present value:

reflects all aggregated costs over a total depreciation period, with a certain interest rate and inflation rate.

nutrients:

nutrients are any chemical elements or compounds that supports normal reproduction, growth, lactation, or maintenance of life processes. In wastewater, the most important nutrients are phosphorous (P), nitrogen (N) and potassium (K).

P = phosphorous:

all phosphorous components in wastewater.

P_{total}:

total of phosphates present inn wastewater.

p.e. = population equivalent:

the oxygen equivalent to the amount of oxygen demand related to one inhabitant. For publicly owned wwtp calculated as 54 g BOD_5^{20} per day.

PE = polyelectrolyte also entitled as polymer

physical-chemical pre-treatment:

pre-treatment by means of physical separation technology possibly enhanced by chemical aids.

polymer:

organic compound with (high) molecular weights and composed of repeating chemical units (monomers).

post-treatment:

treatment of pre-treated wastewater by biological, physical, chemical or combined treatment technologies.

pre-treatment:

treatment of wastewater in the first step of a wwtp.

reject water:

rejected (waste)water from pre-thickening and dewatering processes in the sludge treatment; mostly returned as internal wastewater flow to the influent of the wwtp.

settleability:

the (cap)ability of particles to sedimentate.

scenario:

set of one or more unit processes. In this case: wastewater treatment scenario, consisting of a wastewater treatment unit processes and sludge handling unit processes.

scenario analysis:

a methodology of strategic planning, composing and evaluating which works solving predictable problems en route. Scenario analysis is a method that deals with both the complexity and uncertainty of the environment. Scenario analysis is especially suited for forecasting where uncertainty is high and historical relationships are changing.

STOWA:

Dutch Foundation for Applied Research on Water Management.

SS or TSS = (Total) suspended solids

sustainable development:

the Brundtland Commission's report *Our Common Future* described sustainable development in 1987 as ‘the development that makes use of the available resources without exporting the problems of pollution into time or into space’.

swf = storm weather flow

turbidity:

a measure of light-transmitting properties of (waste)water indicating the quality of (waste)water with respect to residual suspended matter.

unit process:

a separate unit wherein a technique or a process is conducted.

wwtp = wastewater treatment plant:

generally a publicly owned municipal wastewater plant.

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CURRICULUM VITAE

Arjen Frans van Nieuwenhuijzen was born the 15th of March 1971 in Dordrecht, the Netherlands. After attending elementary school and a part of secondary school (Gymnasium) in Hamburg, Germany, and graduating from the Dutch secondary school (Atheneum) he started his university study in 1990 at the Delft University of Technology at the Faculty of Civil Engineering. He obtained his title of civil engineer (ir) in 1996 by defending the M.Sc.-thesis 'Optimisation of Flocculation Filtration of wwtp Effluent' at the Department of Sanitary Engineering. He continued his career at the Delft University of Technology by joining the Department of Sanitary Engineering as contract researcher to conduct a five-year research project initiated and funded by STOWA - the Dutch Foundation for Applied Research on Water Management. The work carried out in this project formed the basis of this dissertation. Since September 2001, he is employed at the engineering and consultancy firm Witteveen+Bos in Deventer, the Netherlands.

APPENDICES

APPENDIX A: STATE OF THE ART - Figures 1981 through 1998

In this appendix, the following data are being presented to base the state of the art of wastewater treatment systems in The Netherlands:

- design capacity and applied treatment methods;
- technical operating data of the wastewater treatment plants;
- quantities, composition and destination of wastewater sludge;
- investments and operating costs in wastewater treatment plants and transport systems.

Table A.1 shows the number of public wastewater treatment plants from different points of view. Table A.2 shows the total load and the average concentration of different parameters in influent and effluent of the public wastewater treatment. The following parameters are listed in the table:

- chemical oxygen demand (COD);
- biochemical oxygen demand (BOD_5);
- organically bonded Kjeldahl-nitrogen (N-Kj);
- and total phosphates as phosphorus (P-tot).

Furthermore, an estimation has been made of heavy metal loads in influent and effluent of public wastewater treatment plants. The results of these estimations are listed in table A.3.

Data on the quantity, destination and composition of the produced wastewater sludge have been processed into loads per destination, loads per sludge-component and average concentrations. Table A.4 shows the quantity of removed wastewater sludge per destination. Sludge that is used as fertiliser in agriculture has to meet the Dutch standards concerning the concentration of heavy metals (BOOM). As a result of these standards only a minor quantity of sludge from private wastewater treatment plants may still be used for this purpose.

Finally, table A.5 gives the investment and operating costs of the public wastewater treatment plants. These costs are represented as a total and as costs per p.e. on the basis of design capacity, quantity supplied and quantity extracted.

TABLE A.1: Public wastewater treatment plants in the Netherlands per 31th of December 1998

Number						**Design capacity**						
1981	**1985**	**1990**	**1995**	**1997**	**1998**	**1981**	**1985**	**1990**	**1995**	**1997**	**1998**	
(#)						*(million p.e.)*						
504	487	469	424	412	409	20.0	22.7	23.7	24.4	24.8	24.8	total among which
												type of treatment
47	30	14	4	2	-	1.9	1.8	0.1	0.0	0.0	-	mechanical
81	62	48	27	18	15	2.2	2.0	1.7	1.1	1.0	1.0	trickling filters
57	63	64	59	59	55	7.7	8.8	10.2	9.8	9.8	9.4	aeration basins
73	80	97	113	123	121	3.8	4.3	4.9	6.2	6.9	7.0	carrousels
												class of capacity
170	131	100	66	58	55	0.4	0.3	0.2	0.2	0.1	0.1	up to 5,000 p.e.
40	58	63	66	66	67	12.0	13.8	14.4	15.0	15.1	15.1	100,000 p.e. or more
												auxiliary functions
14	19	36	133	177	199	0.8	0.9	1.7	12.3	16.2	19.0	dephosphatation
												sludge stabilisation
303	298	301	291	290	290	8.9	9.1	8.6	9.5	10.2	10.4	simultaneous[1]
111	123	125	113	107	106	8.9	11.4	14.2	14.0	13.8	13.7	warm digestion
11	10	8	6	6	4	0.5	0.5	0.3	0.3	0.3	0.1	separate aerobic stabilisation
												sludge dewatering
327	271	208	89	69	37	6.9	6.6	5.0	2.4	1.3	1.0	drying beds
166	195	227	229	221	232	5.0	6.0	9.0	7.1	6.9	7.8	wet removal [2]
67	75	99	117	116	118	8.8	9.0	12.6	16.6	16.6	16.1	mechanical
												receiving surface water
82	85	92	87	92	92	4.8	6.0	7.5	7.9	8.3	8.2	fresh nat.waters
6	8	8	10	10	10	1.5	1.9	1.7	1.7	1.8	1.8	marine nat.waters

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1) including public wastewater treatment plants without or with unknown sludge stabilisation.

2) after 1986: including public wastewater treatment plants of which sludge is treated in separate sludge dewatering plants.

TABLE A.2: Public wastewater treatment plants: influent and effluent

Influent						**Effluent**								
1981	**1985**	**1990**	**1995**	**1997**	**1998**	**1981**	**1985**	**1990**	**1995**	**1997**	**1998**			
flow (million rn^3/day)														
3.5	3.9	4.6	5.1	4.7	5.9	3.8	3.9	4.6	5.1	4.6	5.8			
daily load (1,000 kglday)													*efficiencies in 1998*	
													aa	*wa*
1,904	2,192	2,556	2,522	2,508	2,548	396	368	360	275	244	283	COD	88%	89%
780	834	955	907	985	953	109	79	65	37	33	37	BOD_5	95%	96%
166	192	299	230	233	233	68	76	69	46	44	44	N_{Kj}	85%	81%
48	51	39	38	37	37	28	30	17	10	9	9	P_{total}	73%	76%
concentration (g/m^3)														
551	573	595	510	570	456	112	103	88	60	57	51	COD		
222	220	222	185	224	173	29	22	18	9	9	7	BOD_5		
53	54	52	47	54	42	18	18	14	9	9	7	N_{Kj}		
15	15	9	8	9	7	10	9	4	2	2	2	P_{total}		

aa = arithmetical average
wa = weighted average

TABLE A.3: Public wastewater treatment plants: heavy metals in influent and effluent

Influent						**Effluent**						
1981	**1985**	**1990**	**1995**	**1997**	**1998**	**1981**	**1985**	**1990**	**1995**	**1997**	**1998**	
metals (1,000 kg)												
130	161	201	189	157	166	39	48	60	23	22	20	copper
62	55	43	32	17	21	25	22	17	7	5	6	chromium
460	511	509	451	368	445	138	153	153	124	93	123	zinc
139	145	119	81	47	59	56	58	47	10	10	8	lead
2.6	2.4	2.1	1.5	0.8	1.0	1.0	1.0	0.8	0.4	0.3	0.3	cadmium
47	44	45	31	26	30	33	31	31	13	16	14	nickel
0.9	0.8	1.0	0.7	0.5	0.5	0.3	0.3	0.3	0.2	0.1	0.1	mercury
2.5	3.3	4.9	5.7	5.4	6.9	1.2	1.6	2.4	2.6	2.5	3.2	arsenic

TABLE A.4: Public wastewater treatment plants: removal of wastewater sludge

Sludge						**Dry solids**						
1981	**1985**	**1990**	**1995**	**1997**	**1998**	**1981**	**1985**	**1990**	**1995**	**1997**	**1998**	
loads (million kg)												
						184	243	320	362	359	358	*production*
3,622	3,750	4,860	2,135	1,512	1,406	180	227	315	360	347	350	*total removed*
												of which to
1,401	1,474	1,621	-	-	-	69	70	82	-	-	-	agriculture
-	-	-	440	367	224	-	-	-	22	23	12	VerTech [1]
439	548	447	472	240	192	38	71	63	80	56	48	preparation of compost. black earth
1,720	1,620	2,705	765	470	255	65	77	158	179	165	101	landfill[2]
35	51	78	454	421	702	5	7	12	79	98	162	incineration
27	56	10	4	14	33	3	3	1	0	6	27	others, reuse

1) wet oxidation in VerTech plant.
2) 1981-1990: including sludge removal into the North Sea.

TABLE A.5: Public wastewater treatment costs

1981	**1985**	**1990**	**1995**	**1997**	**1998**	
investments (million €)						
€ 83.9	€ 163.4	€ 157.0	€ 296.8	€ 142.5	€ 171.1	*wastewater and sludge treatment*
€ 40.8	€ 58.1	€ 49.0	€ 39.5	€ 38.6	€ 38.1	*transport systems*
€ 124.3	€ 221.4	€ 206.0	€ 336.3	€ 181.1	€ 209.2	total
operating costs (million €)						
€ 250.0	€ 366.7	€ 479.2	€ 650.3	€ 677.0	€ 697.5	*total operating costs among which*
€ 202.4	€ 288.6	€ 376.2	€ 542.7	€ 559.1	€ 576.8	*wastewater and sludge treatment*
costs per p.e. (€)						*wastewater and sludge treatment costs per p.e. on the basis of*
€ 10.0	€ 12.7	€ 15.9	€ 22.2	€ 22.7	€ 23.1	design capacity
€ 13.2	€ 18.6	€ 21.3	€ 32.2	€ 30.9	€ 32.7	quantity supplied

APPENDIX B: ASSUMPTIONS IN DEMAS - Cost calculation data

Table B.1: Assumptions for the operational costs (all costs in €)

electricity	€ 0.068 per kWh
$FeCl_3$ (41 %)	€ 150 per m^3
anionic polymer	€ 5,600 per 1,000 kg
fast dosing	€ 5,500 per 1,000 kg for cationic polymer
	€ 3,100 per 1,000 kg for anionic polymer
methanol	€ 180 per 1,000 kg
final sludge disposal	€ 400 per 1,000 kg dry solids
staff labour	€ 36,500 per staff member per year

Table B.2: Assumptions for the construction (C)* and electro-mechanical (EM)* costs (all in €)

Physical-chemical pre-treatment techniques	C (€)	*EM* (€)	Maintenance: percentage of C / EM (€ per year)
fine screen	135,750 (total)	226,250 (total)	0.5% / 1.5%
flocculation	450 per m^3	340 per m^3	0.5% / 2.0%
primary and secondary sedimentation tank	270 per m^2	70 per m^2	0.5% / 1.5%
cover primary sedimentation tank	90 per m^2	--	--
flotation	1,130 per m^2	2,260 per m^2	0.5% / 2.0%
aerated / denitrifying a-step	270 per m^3	180 per m^3	0.5% / 1.5%
Post-treatment techniques			
activated sludge system	140 per m^3	90 per m^3	0.5% / 1.5%
biofilm system	1,360 per m^2	1,580 per m^2	0.5% / 2.0%
membrane-bio-reactor (membrane costs: 36 per m^2)	13 per m^2 membrane	51 per m^2 membrane	after 5 year EM reinvestment ; 0.5% /2.5%
ion exchange (exc. regenerate treatment)			
- biological regeneration	54,300 per column	3,850 per m^3	0.5% / 2.0%
- stripper for nh_4-n recovery			
activated carbon filter	4.5 per kg NH_4-N		
- thermal regeneration	1,580 per m^3	2,400 per m^3	0.5% / 2.0%
- activated carbon replacement	200 per m^3 AC 1.7 per kg AC		
Applied sludge handling			
sludge thickener	900 per m^2	230 per m^2	0.5% / 1.5%
cover sludge thickener	90 per m^2	-	0.5% / 1.5%
digester	180 per m^3	90 per m^3	-
total energy installation	50 per m^3	230 per m^3	0.5% / 1.5%
	biogas/day	biogas/day	0.5% / 1.5%
Tertiary treatment			
rapid sand filtration	3,600 per m^2	3,400 per m^2	0.5% / 2.0%

* For construction and electro-mechanical investments an additional 'overhead' factor of 0.7 is assumed

** Example: 0.5 % of construction cost and 1.5 % of electro/mechanical investments

APPENDIX C: OVERVIEW OVER TREATMENT TECHNIQUES

The pre-treatment techniques applied for the original DEMAS evaluation were selected from an inventory of techniques created from literature research and practical experiences. The following paragraph presents a short overview of technological details of the pre-treatment techniques. The literature references in this appendix are linked to the reference list of chapter 2.

Pre-treatment techniques

Sedimentation or settling - In a sedimentation tank, also called a settling tank, particles settle due to gravitational forces. The settling velocity of particles depends on the particle diameter and particle density, as well as the temperature (viscosity relation) of the (waste)water as presented in table C.1.

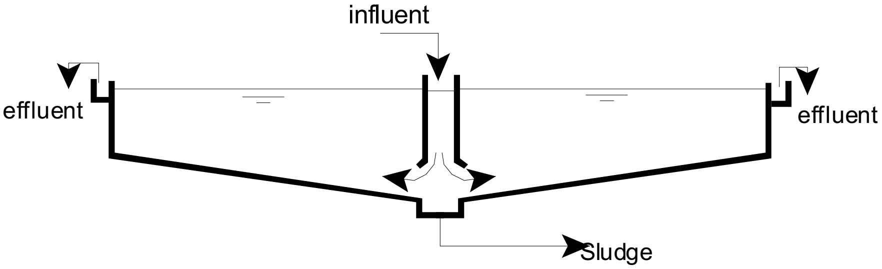


Figure C.1: Schema of a round settling tank

Table C.1: Settling velocity (m/h) of particles at T=10°C [Graaf, van der, (1995)]

particle diameter (µm)	1,000	500	200	100	50	10	5
suspended solids in wastewater (average density =1,200 kg/m^3)	122	61	18	3	0.76	0.03	0.008

The generally applied surface loading of primary sedimentation tanks varies between 1.5 to 3.0 m^3/(m^2·h) at maximum flow (swf). Retention times are minimal one to one-and-a-half hours. In general, the produced primary sludge is frequently pumped to a thickener at a dry solids content of 0.5 to 1%.

Table C.2: Removal efficiencies for simple primary sedimentation and with metal salts enhanced primary sedimentation (pre-precipitation) [Metcalf & Eddy (1991)]

parameter	primary sedimentation	pre-precipitation
suspended solids	30 – 40%	60 – 80%
COD	20 – 30%	40 – 60%
BOD	20 – 30%	40 – 60%
P_{total}	10 – 20%	60 – 80%
N	5 – 10%	20 – 30%

Flotation - Mechanical, pressured air, vacuum and electro-flotation are applicable flotation processes. Most common in water and (industrial) wastewater treatment is dissolved air flotation (DAF) with small air bubbles (diameter = 50 – 100 µm). Retention times vary between 5 - 20 minutes at surface loadings of 5 to 15 m/h [Arnold *et al.* (1995), Krofta *et al.* (1997), (1997)].

Table C.3: Removal efficiencies of DAF-applications on municipal wastewater

parameter	efficiency
suspended solids	80 - 90%
COD	50 - 60%

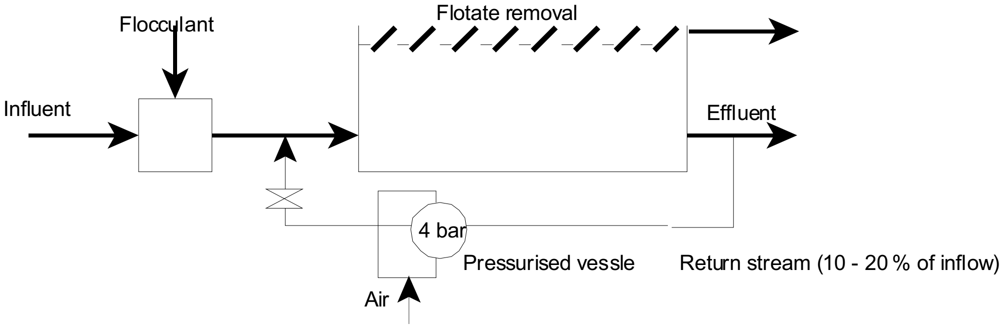


Figure C.2: Schema of a dissolved air flotation (DAF) unit

Micro screens or drum screens - Removal openings of 250 - 500 µm are generally applied for rotating drum screens in industrial (waste)water treatment. In some special occasions, very small openings of 6 to 35 µm are used [Metcalf & Eddy (1991)].

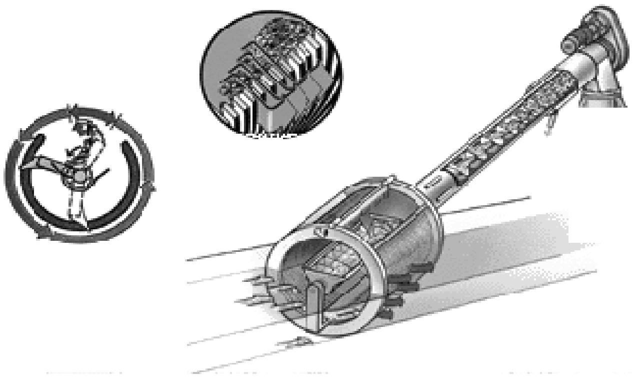


Figure C.3: Example of a rotating drum screen

Table C.4: Typical removal efficiencies for a 25 µm drum screen

parameter	simple	chemically enhanced
suspended solids	30 – 40%	60 – 80%
COD	20 – 30%	40 – 60%
BOD	20 – 30%	40 – 60%
P_{total}	10 – 20%	60 – 80%
N	5 - 10%	20 – 30%

Coarse media filtration - Research on filtration techniques for pre-treatment of wastewater is rather limited and application in practice is hardly known. Only a small number of research activities into direct influent filtration, mainly with up-flow floating filters, have been reported [Bigot *et al.* (1999), Booker *et al.* (1996), EPA (1979), Liao (1998), Mouri and Niwa (1993), Ødegaard *et al.* (1998), Tanaka *et al.* (1995)]. Filtration for the treatment of primary effluent from settling tanks has been investigated more often [e.g. Brown and Wistrom (1999)].

Table C.5: Removal efficiencies of direct filters

parameter	efficiency
suspended solids	80 - 90%
BOD	40 - 50%
[Mouri and Niwa (1993), Tanaka *et al*. (1995), Ødegaard (1997)]	

Magnetically enhanced separation techniques - Magnetic separation is based on the principle of flocculation enhanced with magnetic magnetite particles (< 20 μm) and the removal the resulting magnetic flocs by an electro magnetic field. The Australian *SIROFLOC*™ process [Booker *et al*. (1994), Booker and Brooks (1994)] is an example of this technique.

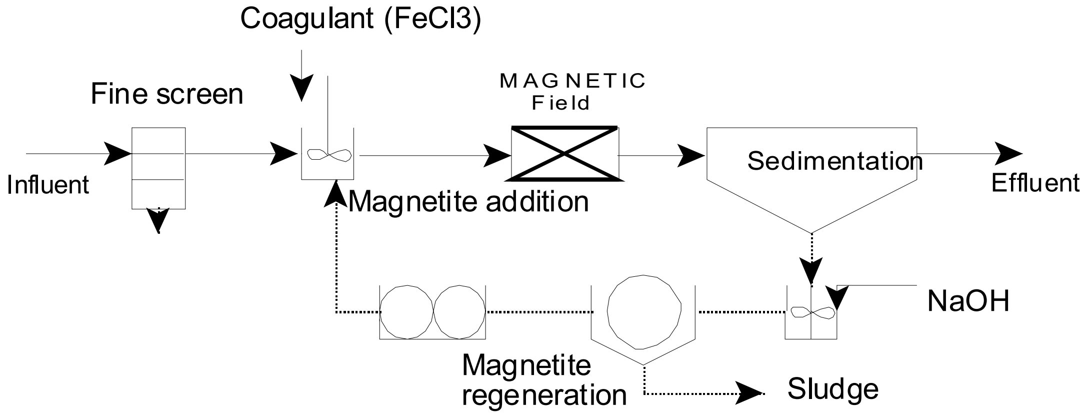


Figure C.4: Schematic *SIROFLOC* system

Table C.6: Removal efficiencies of a prototype of *SIROFLOC*™ [Booker *et al.* (1994), Booker and Brooks (1994)]

parameter	influent	effluent	efficiency
suspended solids (mg/l)	212	30	86%
BOD (mg/l)	284	129	55%
COD(mg/l)	582	255	56%
P (mg/l)	7.5	0.9	88%
dosing: 1C.3 mg/l Al-ions, 102 mg/l H_2SO_4 and 26.6 mg/l NaOH			

Micro sand enhanced settling - In the *Actiflo™* process, micro sand (100 - 150 μm) is added together with flocculants to the raw (waste)water to improve settling conditions. [Poder, le and Binot (1995), Plum *et al*. (1998), Gousailles *et al*. (1997)].

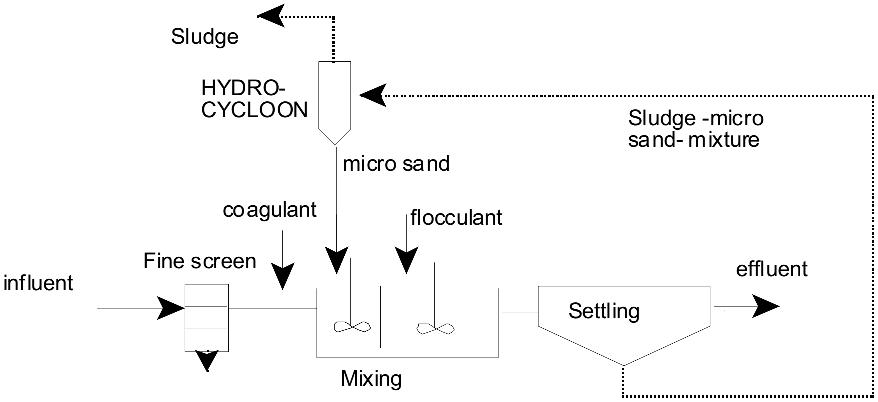


Figure C.5: Schema of the *Actiflo*^{TM} process

Table C.7: Removal efficiencies of the *Actiflo*^{TM} process

parameter	wastewater pre-treatment	storm water treatment
suspended solids	75 – 90%	80 - 98%
COD	55 – 65%	
P_{total}	50 - 95%	50 - 98%
N	10 – 20%	
[Poder, le and Binot (1995), Plum *et al.* (1998)]		

A-step (Bioflocculation Adsorption step) - The A-step is based on the German Adsorptions-Belebungs (AB) Verfahren [Böhnke (1980), Sayed (1985)] where high F/M ratios (1 - 5 kg BOD/ kg d.s. per day) are applied. Due to fast biosorption and diffusion within the bacteria, 60% to 80% of total BOD fixes to (bacteria) sludge.

Table C.8: Process parameters of the A-step of wwtp Dokhaven (Rotterdam) and a theoretical A-step with sludge regeneration

	A-step	A-step with sludge regeneration
hydraulic retention time (h)	0.25	0.5
F/M ratio [kg BOD/kg d.s. day]	1.5	4.0 - 8.5 kg COD/kg d.s. day
sludge volume index [ml/g]	73	40 – 120

At the wwtp Dokhaven in Rotterdam (The Netherlands) [Schellen (1997)] a so- called F.A.S.T.-dosage (Försedmentierungs Ausklijning av Suspenderat materia med Trippeldos) [Schellen (1997)] is used to improve floc settling. Removal efficiencies of this plant are shown in table C.9.

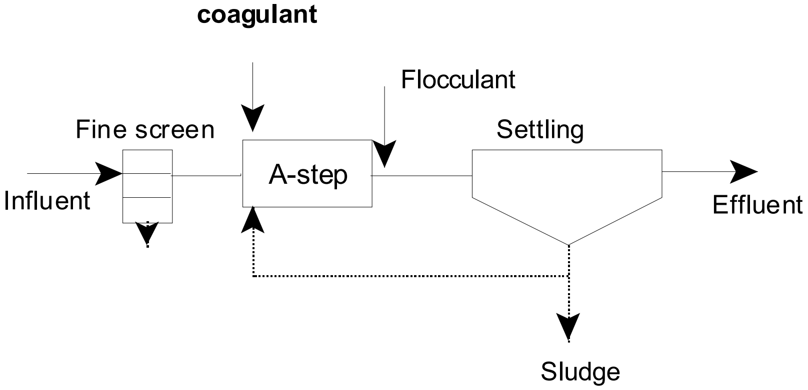


Figure C.6: Schema of the chemically enhanced A-step

Table C.9: Removal performances of the A-step at the wwtp Dokhaven - Rotterdam

parameter	no chemicals	$FeCl_3$ dosage	F.A.S.T.
dosage		13.3 g Fe/m^3 (1 mol Fe/mol P)	7.9 g Fe/m3 1 g cationic polymer /m^3 0.2 g anionic polymer/m^3
COD	65 – 70%	75 – 80%	ca. 77%
BOD	70 – 75%	75 – 85%	ca. 85%
P	ca. 50%	75 – 85%	ca. 78%
suspended solids in effluent	30 - 50 mg TSS/l	30 - 40 mg TSS/l	30 – 45 mg TSS/l
sludge production (kg/d)	22,000	27,060 (+ 23%)	25,300 (+ 15%)
[Schellen (1997)]			

Post-treatment techniques for COD, N and P removal

After the pre-treatment, one or more post-treatment unit processes will have to be applied in order to remove remaining COD, nitrogen and phosphorus. For the post-treatment, different unit processes can be applied consisting of techniques based on the removal of biological contaminants (activated sludge systems, biofilm systems, membrane-bio-reactors), or techniques based on the removal of physical-chemical components (ion exchange, activated carbon). The following paragraphs shows a review of different unit operations for post treatment.

Activated sludge systems - Activated sludge systems are the most applied treatment systems for municipal wastewater in the Netherlands. The application of (advanced) activated sludge systems aims at the degradation of organic substances and nutrient removal by suspended bacteria flocs. After treatment, the bacteria flocs are removed in a final settling tank. The major part of the active bacteria sludge is returned to the beginning of the activated sludge tank and mixed with the incoming primary effluent. The remaining part of the bacterial mass is disposed as waste sludge.

Biofilm systems - Recently many research efforts have been undertaken to develop new configurations of biofilm systems. Some operational examples are the Biofor® and the Biostyr® fixed bed systems and the patented Kaldness Moving Bed System.

The Biofor® system is an up-flow system containing expanded clay granules. These expanded clay granules are porous gravel-like media that serve as biofilm carriers.

Because of their porous structure, biofilm bacteria not only grow on the grains but also inside the gravel particles. Depending on the application, different diameters are used: 3.0 - 3.7 mm for COD removal and denitrification and C.0 - C.5 mm for nitrification [RIZA-STOWA (1994), Æsoy *et al.* (1998)].

The filter medium of Biostyr® reactors consists of polystyrene balls with diameters of 3.0 - 3.5 mm. Due to the low density of the polystyrene, the balls float in the reactor [RIZA (1994), Falkentoft *et al.* (1999)]. A typical bed height of fixed bed biofilm systems is 3 m.

The patented Kaldness process is based on the biofilm technique — a film of microorganisms which grow on carrier elements, in this case free-floating plastic carriers made of polyethylene. These have a density, without biofilm, of just under 1 g/cm^3 and are kept in motion in the reactors by aeration in aerobic units and by mechanical stirrers in anoxic units. The reactor is filled with carrier elements to a maximum of 67% giving an effective biofilm surface of approximately 350 m^2/m^3 reactor volume. The process can be used for the biological aerobic degradation of organic pollutants (BOD, COD) or for biological nitrogen removal by nitrification/denitrification. The latter process is applicable to either pre- or post-denitrification or a combination of the two. With post-denitrification alone a total hydraulic retention time in reactors of only C.5 - 3 hours is enough to obtain >70% nitrogen removal. The biofilm carriers are kept in the reactors by robust stainless steel screens fitted in the reactor outlets [Ødegaard *et al.* (2000), Rusten *et al.* (1997)].

Membrane-bio-reactor - Biomass separation membrane bio-reactors, the most common type of membrane bio-reactors mbr), are the amalgamation of a suspended growth reactor and membrane filtration device into a single unit process. The membrane unit can be configured external to, as in side stream operation, or immersed in the bio-reactor. Biomass separation mbrs should not be confused with the use of membranes for tertiary treatment after a biological process and sedimentation. In the case of an external system the membrane is independent of the bio-reactor. Feed enters the bio-reactor where it contacts biomass. This mixture is then pumped around a recirculation loop containing a membrane unit where the permeate is discharged and the retentate returned to the tank. The transmembrane pressure (TMP) and the crossflow velocity, which define the operation of the membrane, are both generated from a pump. Immersed systems differ in that there is no recirculation loop as the separation occurs within the bio-reactor itself. Under these circumstances the TMP is derived from the hydraulic head of the water above the membrane. In some systems this is supplemented by a suction pump to increase the TMP, although this remains significantly less than in side-stream operation. Fouling control is achieved by scour at the membrane surface, usually from aeration with the movement of bubbles close to the membrane surface generating the necessary liquid shear velocity [Stephenson *et al.* (200)].

The aerobic mbr process has successfully treated effluents from a range of industrial wastewaters, including cosmetics, pharmaceuticals, metal fabrication, textiles, abattoirs, dairy, food, beverage, paper and pulp, rendering and chemical manufacture. Landfill leachate treatment by the aerobic MBR has also been undertaken in Europe.

The very high quality of the treated water from an mbr process is common to all commercial aerobic systems. Complete solids removal, a significant disinfection capability, high rate and high efficiency organic and nutrient removal and a small footprint are all characteristics of the mbr, regardless of the wastewater type to be treated or the commercial process used [Stephenson *et al.* (2000)].

Ion exchange for ammonium removal - Ion exchange can be used to remove ammonium from the pre-treated influent. In this unit process ammonium ions are exchanged for sodium ions that are bound to an ion exchanger by electrical charge forces. The ion exchanger consists of non-soluble, macro-molecular material. As ion exchanging materials, various materials are available. The most important are natural zeolites and synthetically manufactured resins [STOWA (2001)].

Ammonium removal by ion exchange has some strong points in comparison with biological nitrogen removal. The ion exchanging process depends minimally on temperature and has a fast reaction rate (< 2 minutes). Additional, ion exchange creates the possibility for a complete removal of ammonium [Green *et al. (*1996)].

Ion exchangers for ammonium removal are operated in two phases. In the first phase, ammonium is removed from the wastewater, while in the second phase the ion exchanger is regenerated. For the regeneration, a salt solution removes the adsorbed ammonium.

A possible way to recover the ammonium and to avoid the production of large volumes of regenerate waste is to air strip the regenerate solution followed by precipitation of the stripped ammonium. This recovery technique can be operated in parallel with the regeneration of the ion exchanger. An operating example of this ammonium removal and recovery process was operational at the Tahoe-Truckee wastewater treatment plant in the USA [STOWA (2001)]. During the regeneration mode, the regenerate is re-circulated over the air strip column. The pH is raised by the addition of sodium hydroxide, resulting in the formation of ammonia. The ammonia is stripped at 50 ºC and washed in the gas washer while adding an acid. The final product is an ammonium salt (e.g. ammonium sulphate or struvite) or an ammonia solution. Both products can be re-used as agricultural fertiliser.

Sorption to activated carbon for dissolved COD removal - Activated carbon adsorption is frequently used in drinking water treatment to remove heavy metals and organic micro-pollutants. Furthermore, activated carbon adsorption is suggested as a method for polishing the effluent of activated sludge systems [RIZA (1995)].

Activated carbon filters in drinking water treatment are normally operated in down-flow mode with filtration rates of 5 $m^3/(m^{C\cdot}h)$ [Dijk, van and Aeyelts Averink (1995)]. The carbon adsorption capacity depends on the type of pollutant; for COD a capacity of 0.3 - 0.5 kg COD per kg activated carbon is estimated. After saturation, the activated carbon is regenerated thermally at a temperature of approximately 700 ºC.

Rapid sand filtration - Rapid sand filtration can be applied as an effluent polishing step after the post settling tank. Rapid sand filtration can be combined with the addition of coagulants and flocculants. Particle and phosphate removal efficiencies of the filter can be improved by adding chemicals (like metal salts) [Graaf, van der, and Nieuwenhuijzen, van (1998)]. Rapid sand filters for effluent polishing generally are operated as down-flow filters at filtration rates of 5 - 30 $m^3/(m^{2\cdot}h)$.

APPENDIX D: ASSUMPTIONS IN DEMAS - Calculation data

DESIGN PARAMETERS

Table D.1: Design criteria for physical-chemical pre-treatment techniques (all pre-treatment steps are designed for storm weather flow)

Physical-chemical pre-treatment techniques	**Design values**
Bar rack installation	
space between bars	3 mm
number of chambers	2
chamber width	2.20 m
Mix tank	
hydraulic retention time	15 s
height	1 m
velocity gradient stirrers	1,000 (s^{-1})
Flocculator (slow mixing)	
hydraulic retention time	1,200 s
height	2 m
velocity gradient stirrers	45 (s^{-1})
Primary Settling tank	
hydraulic surface loading	3 m^3/m$^2\cdot$h without chemicals; 6 m^3/m$^2\cdot$h with polymers
tank height	2 m
flocculant dose	4.0 g/m^3 cationic HMW polymer
Flotation tank (DAF)	
hydraulic surface loading	10 m^3/m$^2\cdot$h
tank height	2.5 m
pressure recycle flow	4 bar
recycle flow	15%
flocculant dose	4.0 g/m^3 cationic HMW polymer
Aerated A-step	
hydraulic retention time	900 s
sludge concentration	2 kg MLSS/m^3
sludge loading	6.25 kg COD / kg MLSS
sludge recycle flow	25% van Q_{SWF} (750 m^3/h)
tank height	4 m
flocculant dose (FAST principle)	4.0 g/m^3 cationic HMW polymer
Direct coarse media filtration	
filtration rate	10 m^3/m$^2\cdot$h
bed height	1.5 m
column diameter	4.1 m
backwash frequency	Dependent on suspended solids load of filter influent
backwash time	900 s
backwash volume	8% of influent flow
Direct membrane filtration	
cross-flow configuration	
flux	0.1 m^3/m$^2\cdot$h
trans-membrane pressure (TMP)	0.8 bar
recovery	75%

Table D.2: Design criteria for the post-treatment techniques

Post-treatment techniques (in brackets: aimed contaminants to be treated)	**Design values**
Low loaded activated sludge system (BOD, N,P)	
sludge load	0.06 kg BOD / kg MLSS / day
sludge production	0.4 kg TSS / kg $COD_{removed}$
sludge concentration	4 kg MLSS / m^3
High loaded activated sludge system (BOD)	
sludge load	0.8 kg BOD / kg MLSS / day
sludge production	0.6 kg TSS / kg $COD_{removed}$
sludge concentration	4 kg MLSS / m^3
Low loaded biofilm system (BOD, N)	
sludge load	0.2 kg BOD / m^3
sludge production	0.2 kg TSS / kg $COD_{removed}$
High loaded biofilm system (BOD)	
sludge load	0.6 kg BOD / m^3
sludge production	0.5 kg TSS / kg $COD_{removed}$
Membrane-bio-reactor (BOD, N)	external cross-flow driven membranes
sludge concentration	20 kg MLSS / m^3
sludge production	0.1 kg TSS / kg $COD_{removed}$ (sludge minimisation)
tubular membranes driven in cross-flow mode	
trans-membrane pressure (TMP)	0.5 bar
flux	0.03 $m^3/m^2\cdot h$ at swf
recovery	90%
backflush	depending on particle concentration in feed water
Ion exchange (N as ammonia)	
surface load	30 m/h
column capacity	5.5 kg NH_4-N per m^3 zeolite
bed height	1.3 m
column diameter	2.4 m
Regeneration combined with nitrification	
surface load	10 m/h
nitrification rate	6 kg NH_4-N / m^3 (at 30 °C)
chemical use (buffer)	6.6 kg NaOH per kg NH_4-$N_{nitrified}$
product: concentrated nitrate solution	2% of influent flow; as 1.35 kg NO_3-N / m^3
Regeneration combined with stripping	
surface load	10 m/h
regenerate concentration	1.6 eq. per l
chemical use	2.8 kg NaOH and 3.4 kg H_2SO_4 per kg NH_4-$N_{recovered}$
product	4.7 kg $(NH_4)_2SO_4$ per kg NH_4-$N_{recovered}$
Activated carbon filter (COD, heavy metals and micro pollutants)	
surface load	5 m/h
column capacity	0.45 kg COD per kg activated carbon
bed height	2 m
column diameter	4 m
bulk density	600 kg/m^3
loss of activated carbon	2.5% during each regeneration cycle
Secondary sedimentation tank	
hydraulic load	0.7 m/h

Table D.3: Design criteria for the sludge handling procedure

Sludge handling techniques	**Design values**
Sludge thickener (per m^3)	
solids loads	50 kg TSS / m^2 / day for primary sludge 30 kg TSS / m^2 / day for secondary sludge
dry solids content after thickening	5%
Digester	
solids retention time	20 days
tank height	10 m
temperature	33 °C
organic solids degradation	50% for primary sludge 30% for secondary sludge
biogas production	1.15 m^3 per kg organic solids
energy content biogas	22,000 kJ / m^3
Total energy installation	
energy conversion	32% energy conversion into electricity 68% energy conversion into heat

Table D.4: Energy calculations (for a 100,000 p.e. WTP)

Physical-chemical pre-treatment techniques	**Calculation**	**Value (kWh per year)**
Fine screen	P = 2 kW	20,000
Flocculator		
- rapid mixing	P = 14.2 kW	124,400
- flocculation mixing	P = 2.3 kW	20,200
Primary and secondary clarifiers		
- skimmers	35,000 kWh / year per clarifier for skimming	70,000
Flotation		
- pressurised recycle flow	$P = Q_{recycleflow} / p_{recycleflow} / R_{press.\ pump} = 28.9$ kW	253,000
Aerated/denitrifying A-step	20% of $COD_{influent}$ is oxidised;	
- aeration	OR =$0.2 \cdot 600$ $mgO_2/l \cdot 19{,}500$ m^3/d = 2,340 kg COD / day	688,500
Post-treatment techniques		
Activated sludge system		
- aeration energy	$OR = O_e + O_s + O_n - O_d$	Dependent on pre-treatment
- return flow	$(3 – 5) \cdot Q_{influent}$	
Biofilm system		
- aeration energy	$OR = O_e + O_s + O_n - O_d$	Dependent on pre-treatment
- return flow	$(3 - 5) \cdot Q_{influent}$	
Ion exchange	pump energy	
- biological regeneration	heating of replaced liquid (2%); aeration for nitrification; pump energy	
- stripper for NH_4-N recovery	1.35 kWh per kg $NH_4\text{-}N_{recovered}$ 0.9 kg methane kg $NH_4\text{-}N_{recovered}$	
Activated carbon filter		
- pumps	Pumping	
- reactivation		1,255,00
Sludge handling techniques		
Sludge thickener	Skimming	35,000
Digester	Heating of sludge (by heat production of TE installation)	
Total energy installation (32% efficiency)	32% of 22,000 kJ / m^3	1.95 kWh / m^3 biogas
Tertiary treatment techniques		
Rapid sand filtration		
- backwash	Pumping and backwashing	73,000 kWh

Table D.5: Energy requirement calculations for aeration

Oxygen requirement	**Oxygen and energy requirement**
$OR = O_e + O_s + O_n - O_d$ (kg O_2 / year)	$\alpha \cdot OR = OC.p.\frac{Cs}{Cs - C}$ (in kg O_2 per year)
$O_e + O_s = COD_{influent} - COD_{effluent} - COD_{sludge} =$ $((1-Y).COD_{influent} - COD_{effluent}) \cdot Q_{influent}$ $O_n = (N_{kj,i} - N_{kj,e}).Q_{influent} - N_{sludge}) \cdot 4.57$ $O_d = (N_{kj,i} - N_{kj,e}).Q_{influent} - N_{sludge}) \cdot 2.86$	2.5 kg O_2 / kWh
OR - oxygen requirement O_e – endogenous oxygen consumption O_s - oxygen consumption for biodegradable organic carbon O_n- oxygen consumption for ammonium (i.e. nitrification) O_d - reduction of oxygen consumption due to denitrification Y - biological yield (0.55 kg $COD_{biomass}$ / kg $COD_{removed}$)	OC - oxygen consumption (kg O_2/y) A - aerator efficiency (0.9 (-)) p - peak factor (1.2 (-)) C - oxygen concentration in influent (0.5 mgO_2/l)

APPENDIX E: PRELIMINARY EVALUATION RESULTS - Graphics and diagrams

In these diagrams, the first bar represents the reference scenario (ref.); the following bars are numbered identically to the scenarios in table 2.7 in chapter 2.

ENERGY BALANCE

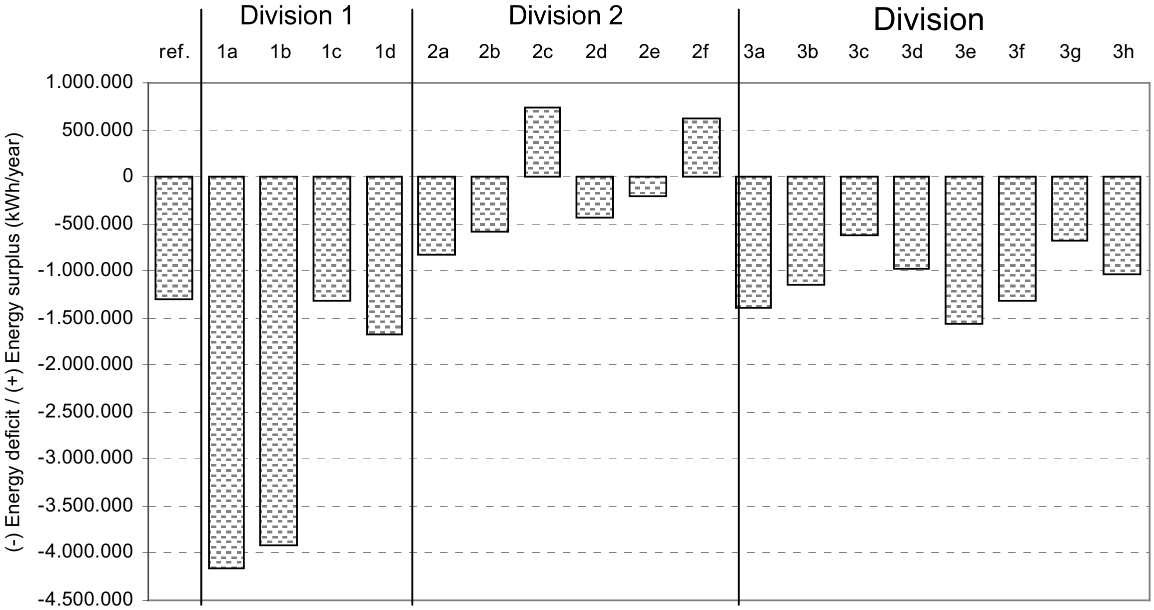


Figure E.1: Calculated energy balance of total treatment scenarios (kWh/year)

SLUDGE PRODUCTION

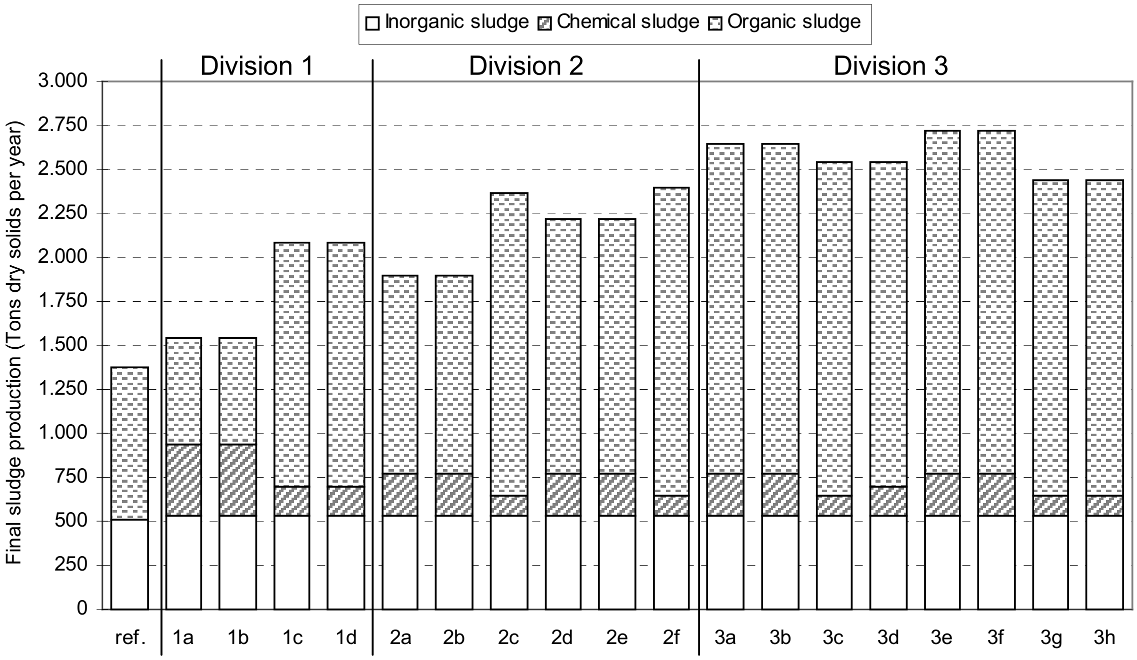


Figure E.2: Final sludge production (tons dry solids per year)

SPACE REQUIREMENT

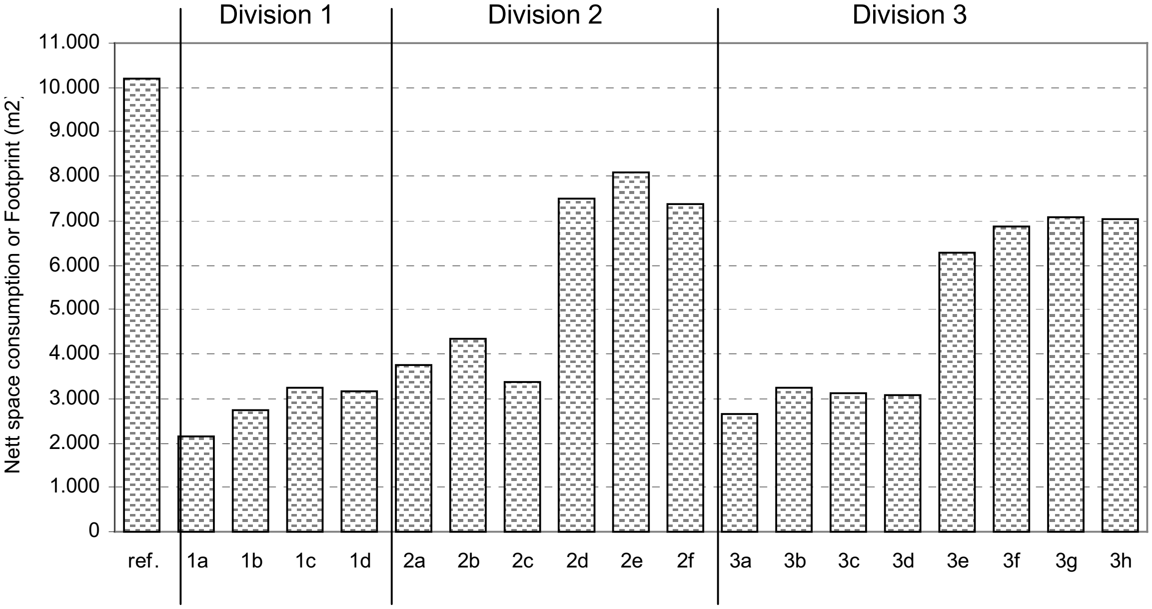


Figure E.3: Net space consumption (m^2)

COSTS AS NET PRESENT VALUES

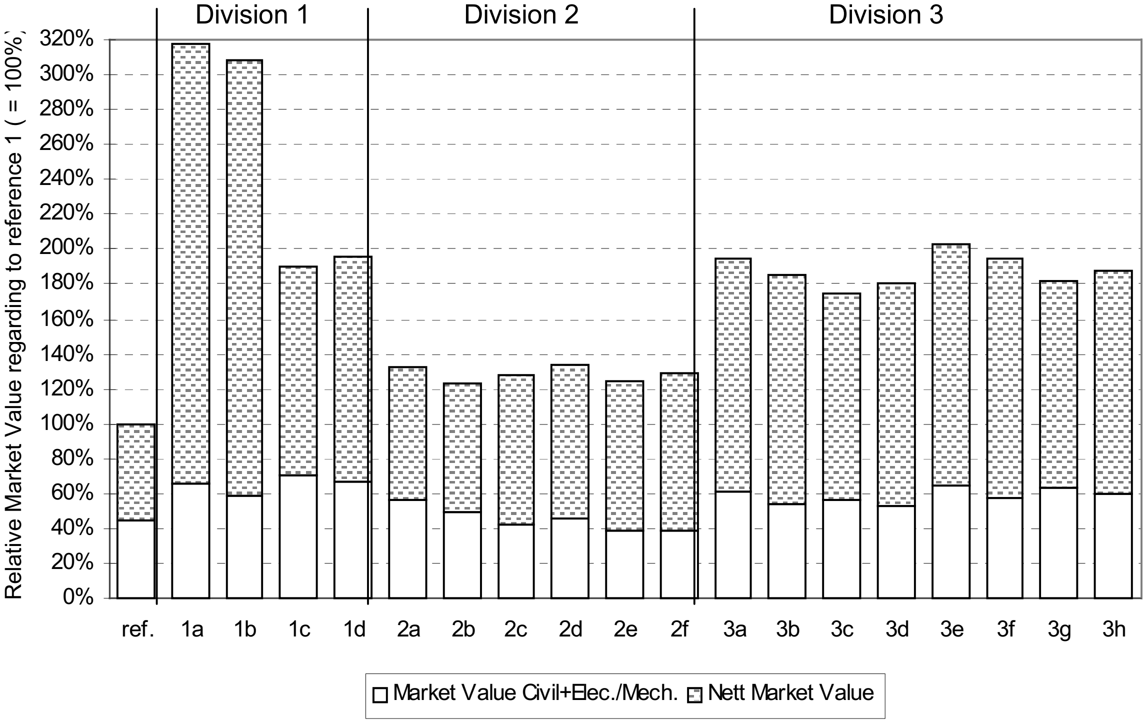


Figure E.4: Relative net present values (reference = 100%)

APPENDIX F: WASTEWATER FRACTIONATIONS - Individual influent characteristics

LOCATION OF SAMPLED TREATMENT PLANTS IN THE NETHERLANDS

WWTP APELDOORN

Table F.1: Wastewater fractions in influent of wwtp Apeldoorn

Fraction Parameter	Dissolved (< 0.1 µm)	Supra-dissolved (0.1–0.45µm)	Colloidal (0.45-1.2µm)	Supra-colloidal (1.2 – 5 µm)	Suspended (5 – 63 µm)	Settleable (> 63 µm)	Raw influent
TSS (mg/l)	n.d.	n.d.	n.d.	n.d.	30	22	52
Turbidity (NTU)	n.d.	1	3	11	36	9	60
BOD_5 (mg O_2/l)	97	n.t.	n.t.	10	6	47	160
COD (mg O_2/l)	200	30	10	40	170	30	480
N_{total} (mg N/l)	30.3	1.6	2.1	10	1,0	3,0	39.0
P_{total} (mg P/l)	3.7	0.5	0.3	0.2	1.5	0.8	7.0
TSS	-	-	-	-	58%	42%	
Turbidity	-	2%	5%	18%	60%	15%	
BOD_5	61%	-	-	6%	4%	29%	
COD	42%	6%	2%	8%	35%	6%	
N_{total}	78%	4%	5%	3%	3%	8%	
P_{total}	53%	7%	4%	3%	21%	11%	

n.d. = not detectable; n.t. = not tested

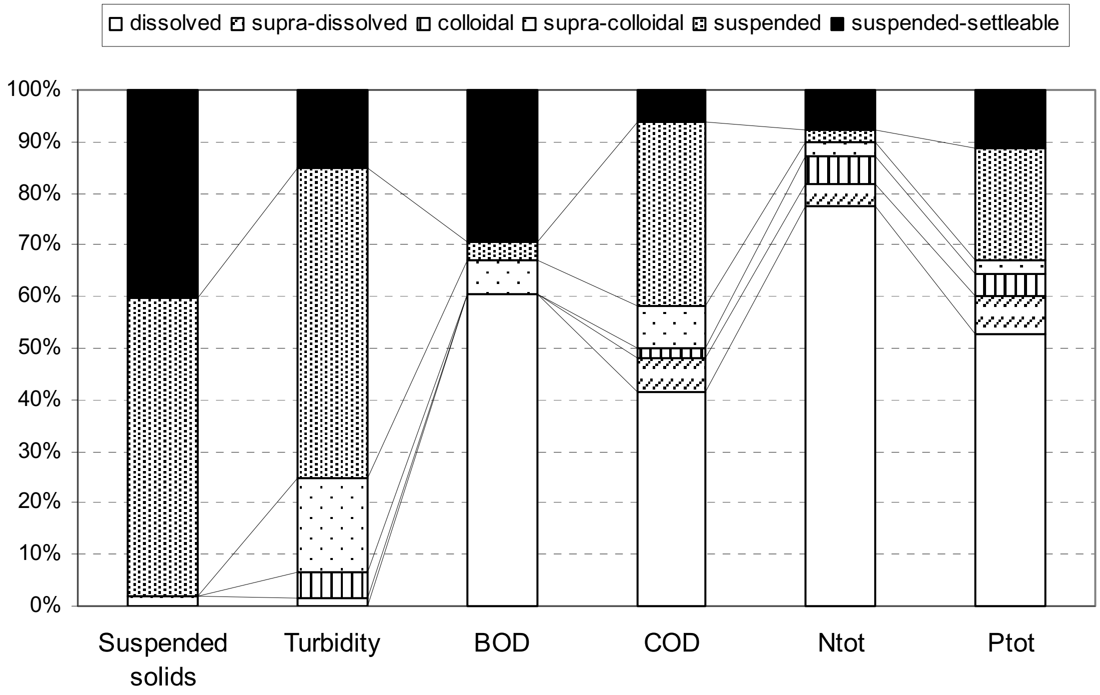


Figure F.1: Fractions in wastewater influent of wwtp Apeldoorn

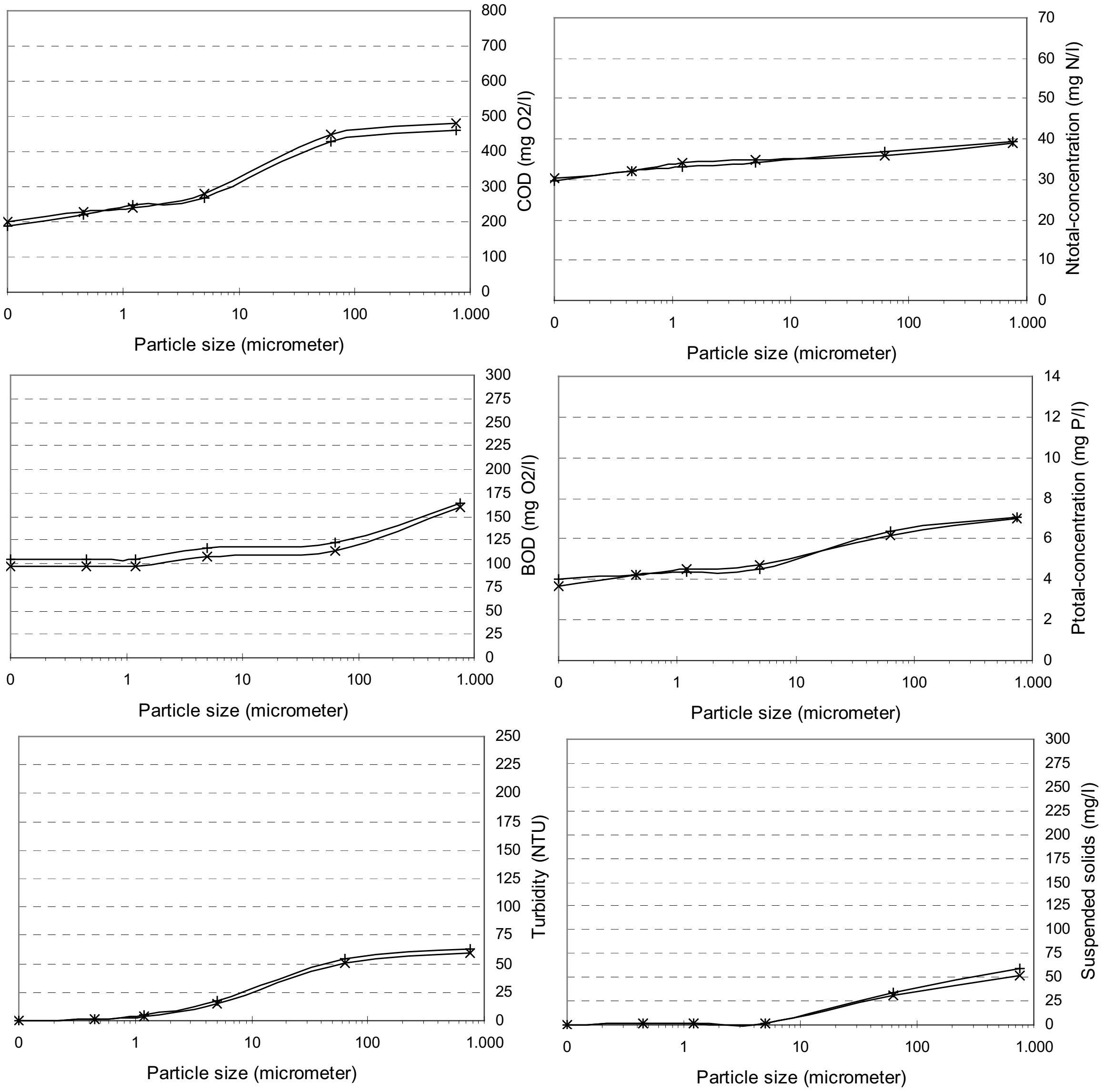


Figure F.2: Wastewater characterisation of wwtp Apeldoorn

Table F.2: Wastewater fractions in influent of wwtp Arnhem

Fraction Parameter	Dissolved (< 0.1 µm)	Supra-dissolved (0.1–0.45µm)	Colloidal (0.45-1.2µm)	Supra-colloidal (1.2 – 5 µm)	Suspended (5 – 63 µm)	Settleable (> 63 µm)	Raw influent
TSS (mg/l)	n.d.	n.d.	n.d.	n.d.	100	42	142
Turbidity (NTU)	n.d.	n.d.	8	11	93	39	151
BOD$_5$ (mg O$_2$/l)	118	n.t.	n.t.	27	95	40	280
COD (mg O$_2$/l)	190	20	60	10	220	130	630
N$_{total}$ (mg N/l)	33.0	3.5	0.5	2.0	2.0	2.0	43.0
P$_{total}$ (mg P/l)	6.5	0.4	0.6	0.3	2.4	0.3	10.5
TSS	-	-	-	-	70%	30%	
Turbidity	-	-	6%	7%	62%	26%	
BOD$_5$	42%	-	-	10%	34%	14%	
COD	30%	3%	10%	2%	35%	21%	
N$_{total}$	77%	8%	1%	5%	5%	5%	
P$_{total}$	62%	4%	6%	3%	23%	3%	

n.d. = not detectable; n.t. = not tested

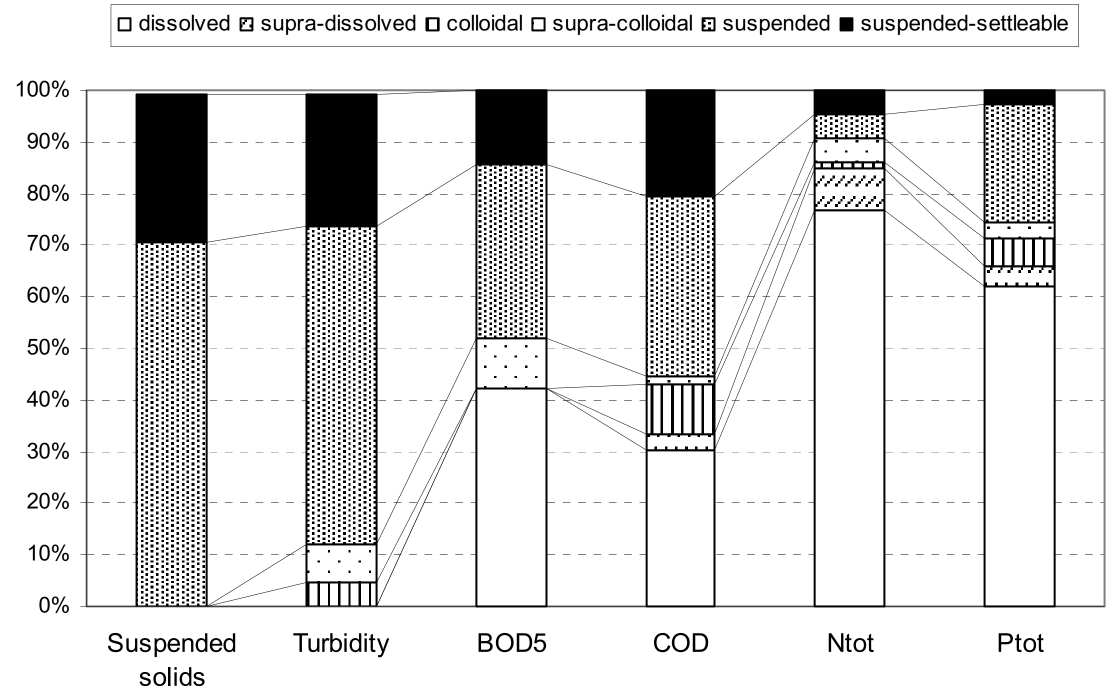


Figure F.3: Fractions in wastewater influent of wwtp Arnhem

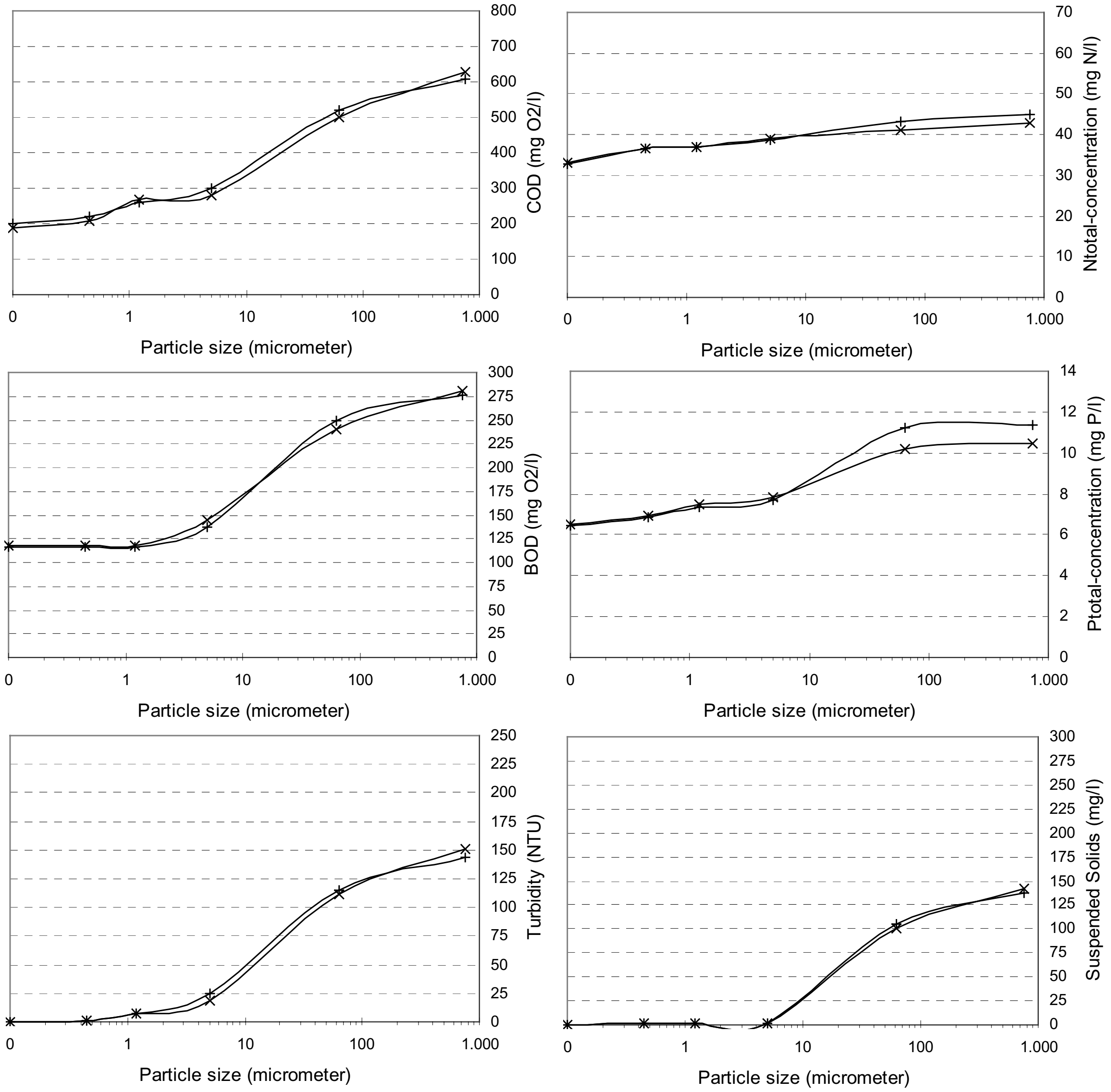


Figure F.4: Wastewater characterisation of wwtp Arnhem

Table F.3: Wastewater fractions in influent of wwtp Bennekom

Fraction Parameter	Dissolved (< 0.1 μm)	Supra-dissolved (0.1–0.45μm)	Colloidal (0.45-1.2μm)	Supra-colloidal (1.2 – 5 μm)	Suspended (5 – 63 μm)	Settleable (> 63 μm)	Raw influent
TSS (mg/l)	n.d.	n.d.	n.d.	7	125	40	172
Turbidity (NTU)	n.d.	n.d.	5	16	89	34	144
BOD$_5$ (mg O$_2$/l)	156	n.t.	n.t.	40	47	24	267
COD (mg O$_2$/l)	270	20	20	40	210	90	650
N$_{total}$ (mg N/l)	59,0	1,0	1,5	3,5	4,0	6,0	75,0
P$_{total}$ (mg P/l)	7,2	0,4	0,2	0,6	1,8	0,3	10,5
TSS	-	-	-	4%	73%	23%	
Turbidity	-	-	3%	11%	62%	24%	
BOD$_5$	58%	-	-	15%	18%	9%	
COD	42%	3%	3%	6%	32%	14%	
N$_{total}$	79%	1%	2%	5%	5%	8%	
P$_{total}$	69%	4%	2%	6%	17%	3%	

n.d. = not detectable; n.t. = not tested

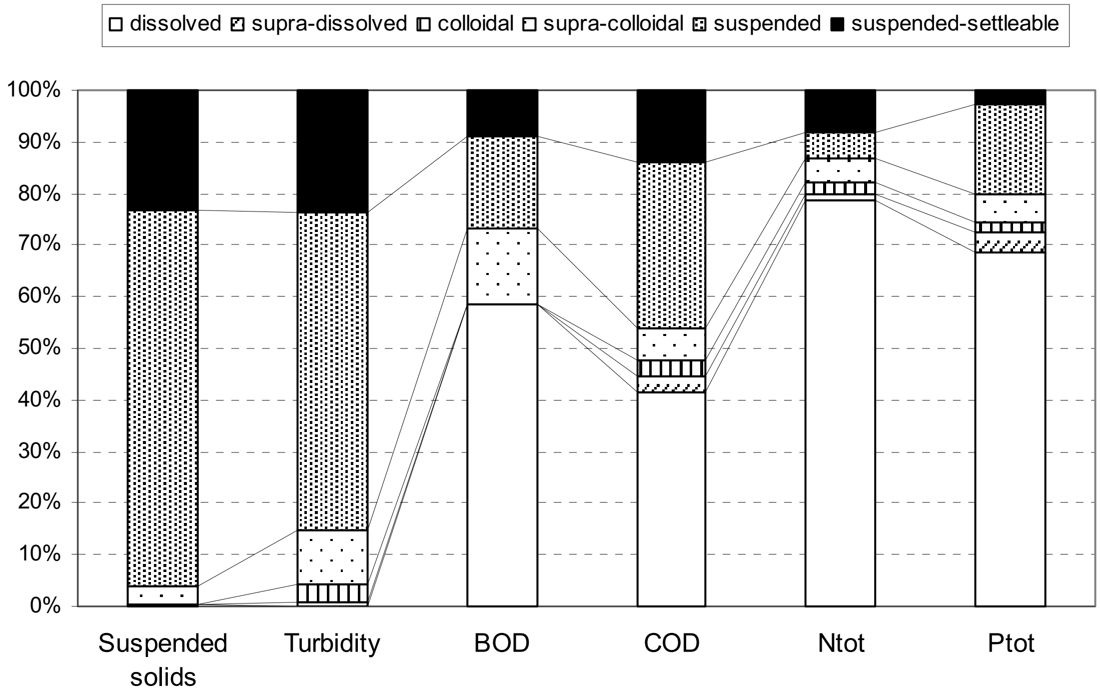


Figure F.5: Fractions in wastewater influent of wwtp Bennekom

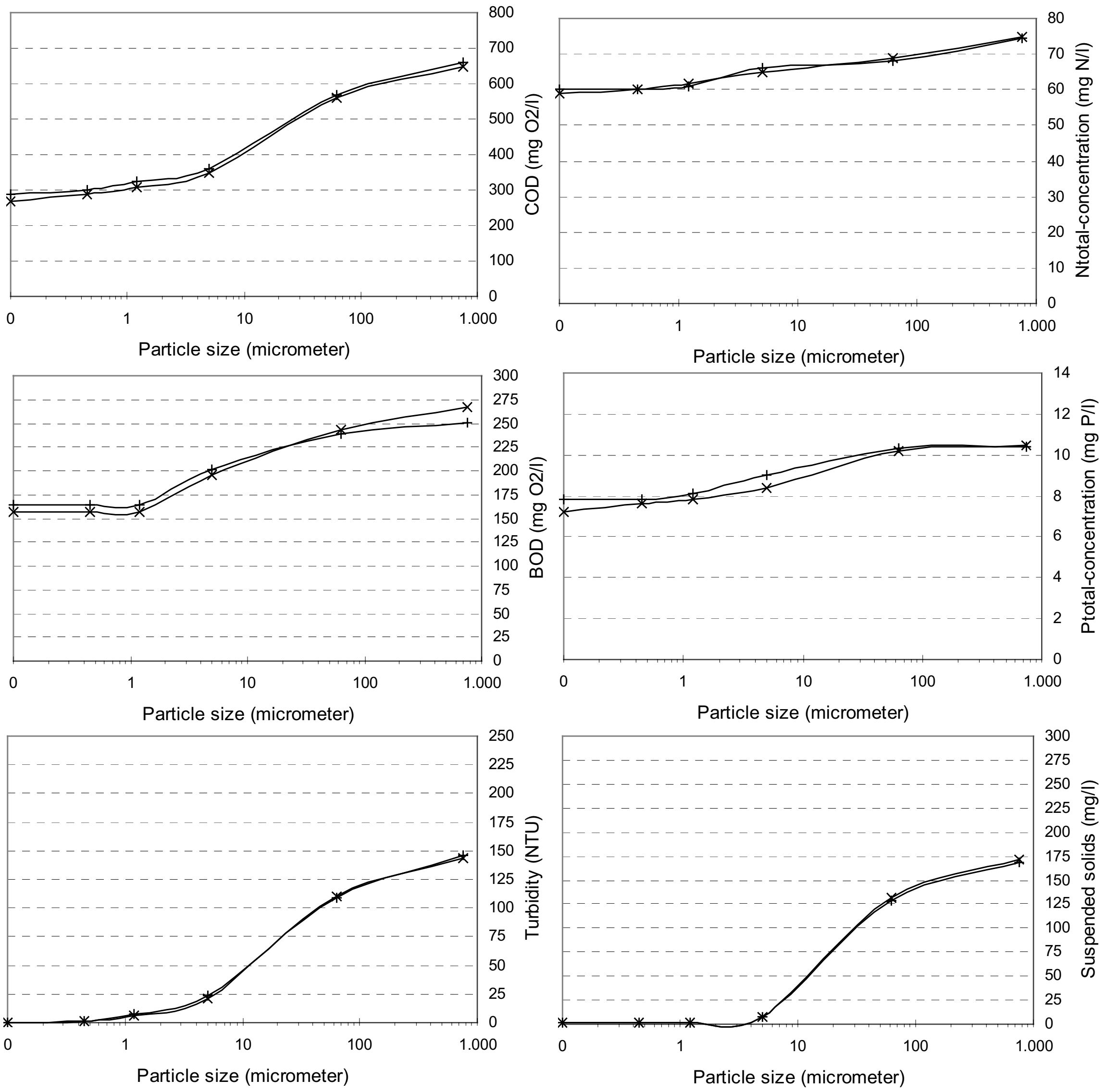


Figure F.6: Wastewater characterisation of wwtp Bennekom

Table F.4: Wastewater fractions in influent of wwtp Berkel

Fraction Parameter	Dissolved (< 0.1 µm)	Supra-dissolved (0.1–0.45µm)	Colloidal (0.45-1.2µm)	Supra-colloidal (1.2 – 5 µm)	Suspended (5 – 63 µm)	Settleable (> 63 µm)	Raw influent
TSS (mg/l)	n.d.	n.d.	n.d.	n.d.	85	150	235
Turbidity (NTU)	n.d.	n.d.	6	12	123	51	192
BOD$_5$ (mg O$_2$/l)	110	n.t.	n.t.	21	69	7	207
COD (mg O$_2$/l)	200	0	0	120	100	130	550
N$_{total}$ (mg N/l)	43.0	1.0	0	1.0	2.0	1.0	48.0
P$_{total}$ (mg P/l)	5.9	0.1	0.5	0.4	2.9	0.3	10.1
TSS	-	-	-	-	36%	64%	
Turbidity	-	-	-	5%	64%	27%	
BOD$_5$	53%	-	-	10%	33%	3%	
COD	36%	0%	0%	22%	18%	24%	
N$_{total}$	90%	2%	0%	2%	4%	2%	
P$_{total}$	58%	1%	5%	4%	29%	3%	

n.d. = not detectable; n.t. = not tested

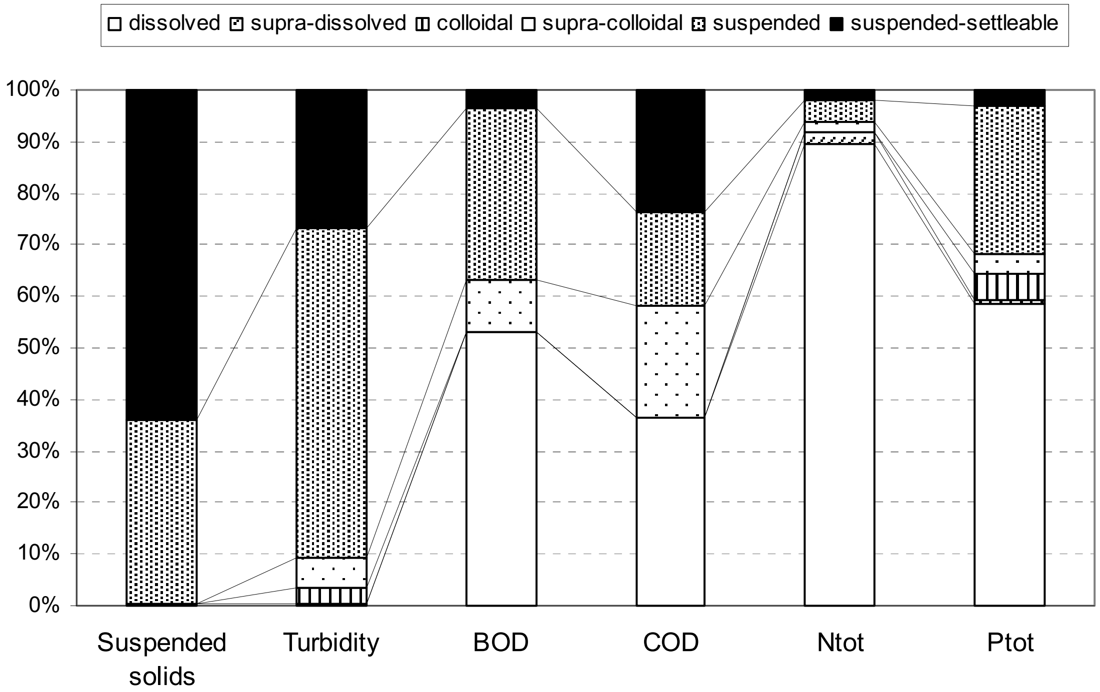


Figure F.7: Fractions in wastewater influent of wwtp Berkel

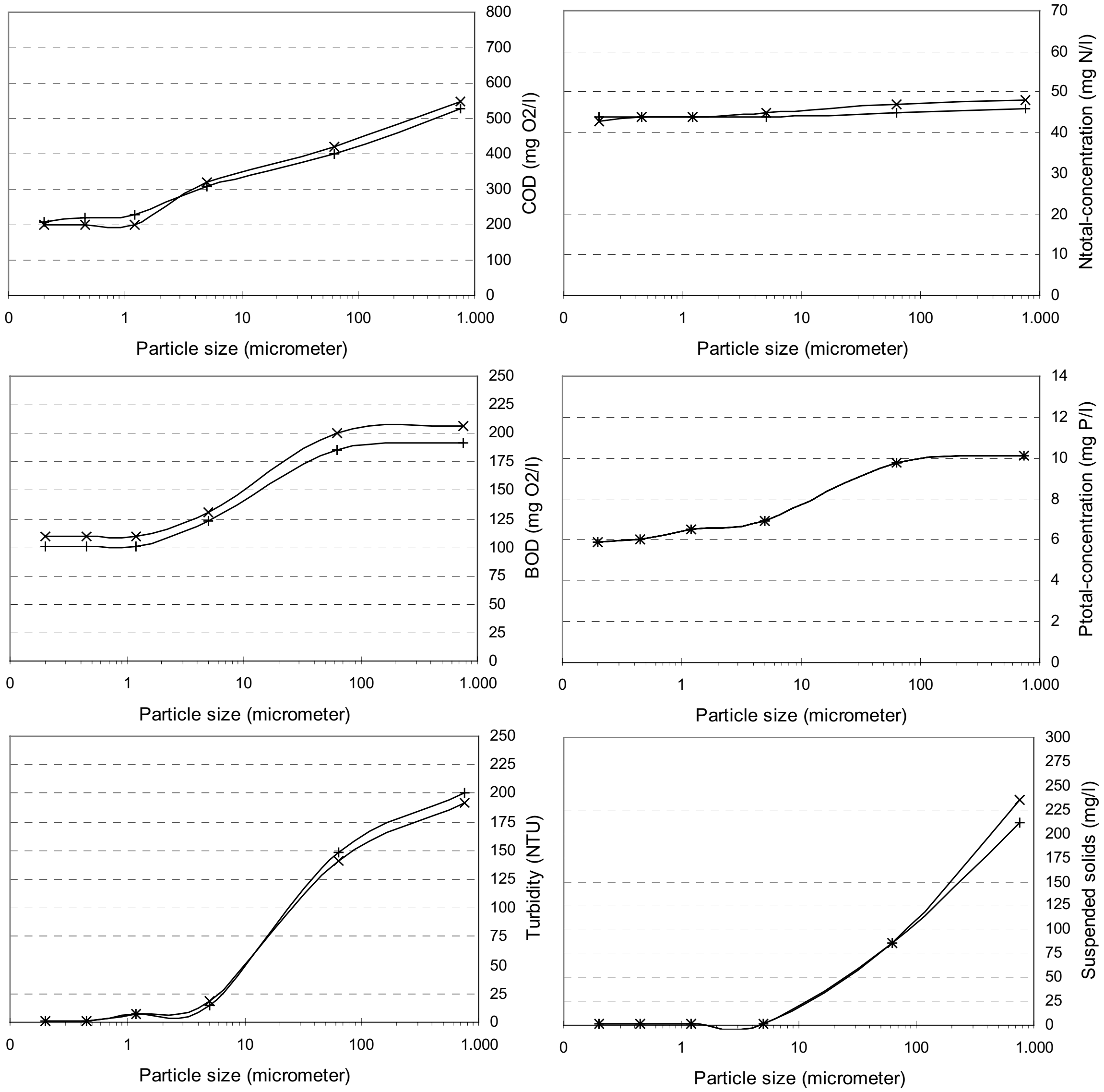


Figure F.8: Wastewater characterisation of wwtp Berkel

Table F.5: Wastewater fractions in influent of wwtp Boxtel

Fraction Parameter	Dissolved (< 0.1 μm)	Supra-dissolved (0.1–0.45μm)	Colloidal (0.45-1.2μm)	Supra-colloidal (1.2 – 5 μm)	Suspended (5 – 63 μm)	Settleable (> 63 μm)	Raw influent
TSS (mg/l)	n.d.	n.d.	n.d.	n.d.	94	26	121
Turbidity (NTU)	n.d.	n.d.	7	18	81	8	115
BOD$_5$ (mg O$_2$/l)	120	n.t.	n.t.	31	79	15	245
COD (mg O$_2$/l)	200	10	40	60	150	90	550
N$_{total}$ (mg N/l)	38.0	0.5	4.5	2.0	3.5	0.5	49.0
P$_{total}$ (mg P/l)	5.3	0.1	0.1	0.3	2.2	0.1	8.1
TSS	-	-	-	-	78%	21%	
Turbidity	-	-	6%	16%	70%	7%	
BOD$_5$	49%	-	-	13%	32%	6%	
COD	36%	2%	7%	11%	27%	16%	
N$_{total}$	78%	1%	9%	4%	7%	1%	
P$_{total}$	65%	1%	1%	4%	27%	1%	

n.d. = not detectable; n.t. = not tested

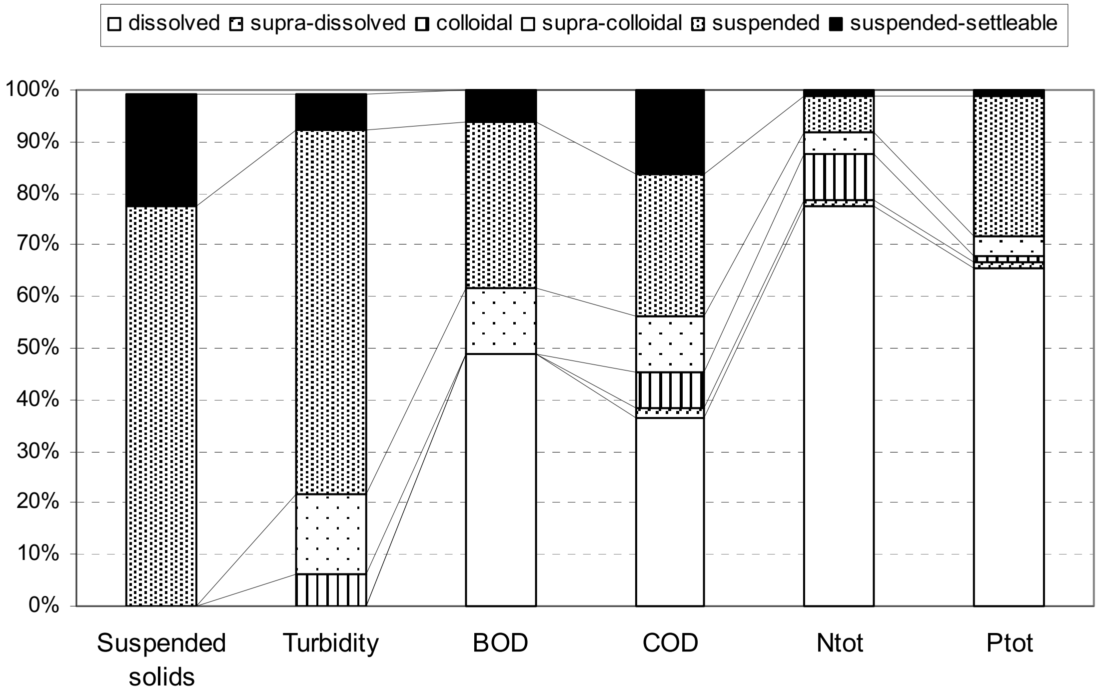


Figure F.9: Fractions in wastewater influent of wwtp Boxtel

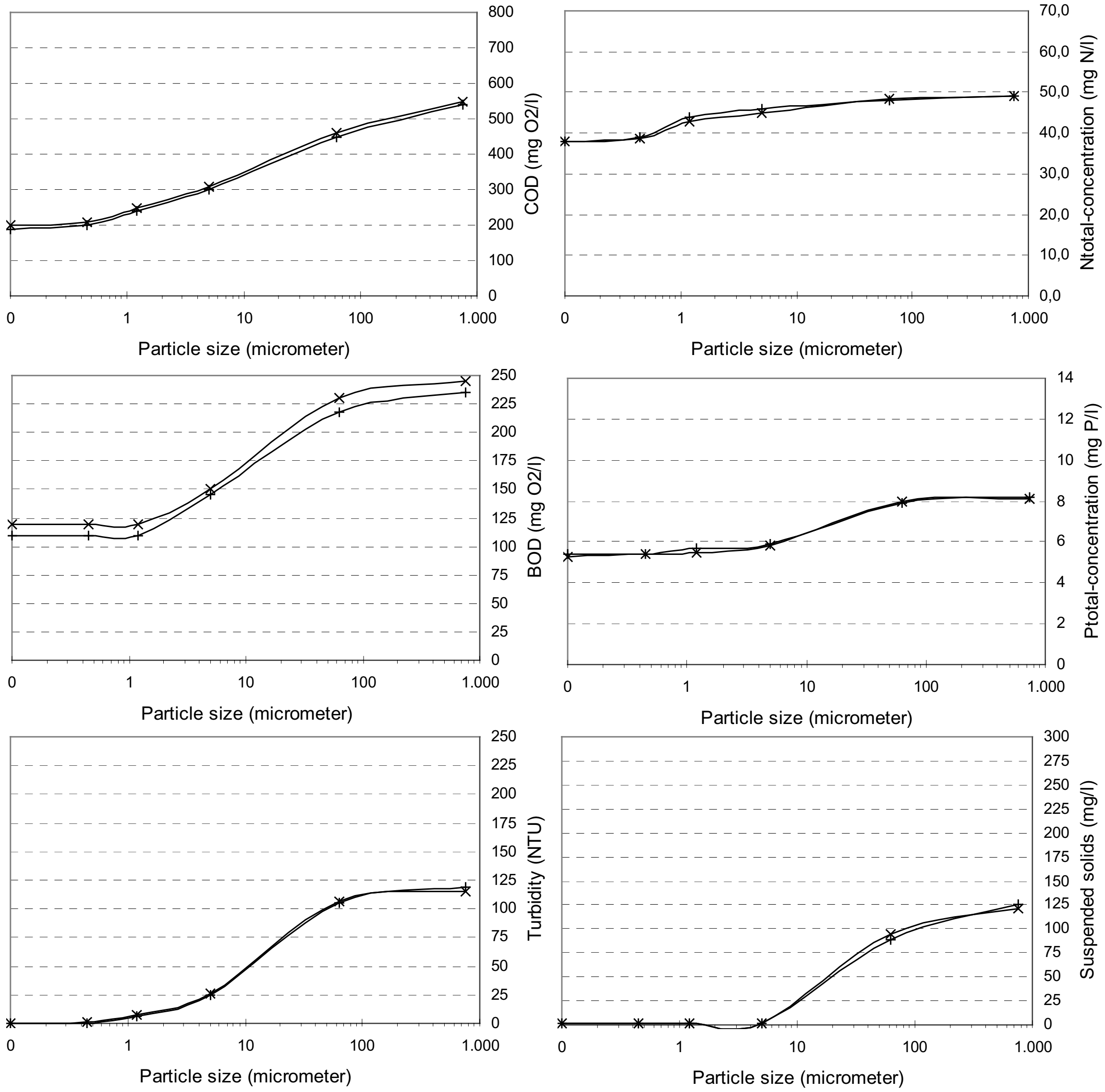


Figure F.10: Wastewater characterisation of wwtp Boxtel

Table F.6: Wastewater fractions in influent of wwtp Haarlem

Fraction Parameter	Dissolved (< 0.1 µm)	Supra-dissolved (0.1–0.45µm)	Colloidal (0.45-1.2µm)	Supra-colloidal (1.2 – 5 µm)	Suspended (5 – 63 µm)	Settleable (> 63 µm)	Raw influent
TSS (mg/l)	n.d.	n.d.	n.d.	n.d.	58	17	75
Turbidity (NTU)	n.d.	n.d.	3	11	70	11	95
BOD_5 (mg O_2/l)	71	n.t.	n.t.	8	50	33	162
COD (mg O_2/l)	130	10	0	20	130	70	360
N_{total} (mg N/l)	41.5	0.5	2.0	2.5	1.0	1.0	48.5
P_{total} (mg P/l)	5.5	0.1	0.1	0.2	1.7	0.3	7.9
TSS	-	-	-	0%	77%	23%	
Turbidity	-	-	3%	12%	74%	12%	
BOD_5	44%	-	-	5%	31%	20%	
COD	36%	3%	0%	6%	36%	19%	
N_{total}	86%	1%	4%	5%	2%	2%	
P_{total}	70%	1%	1%	3%	22%	4%	

n.d. = not detectable; n.t. = not tested

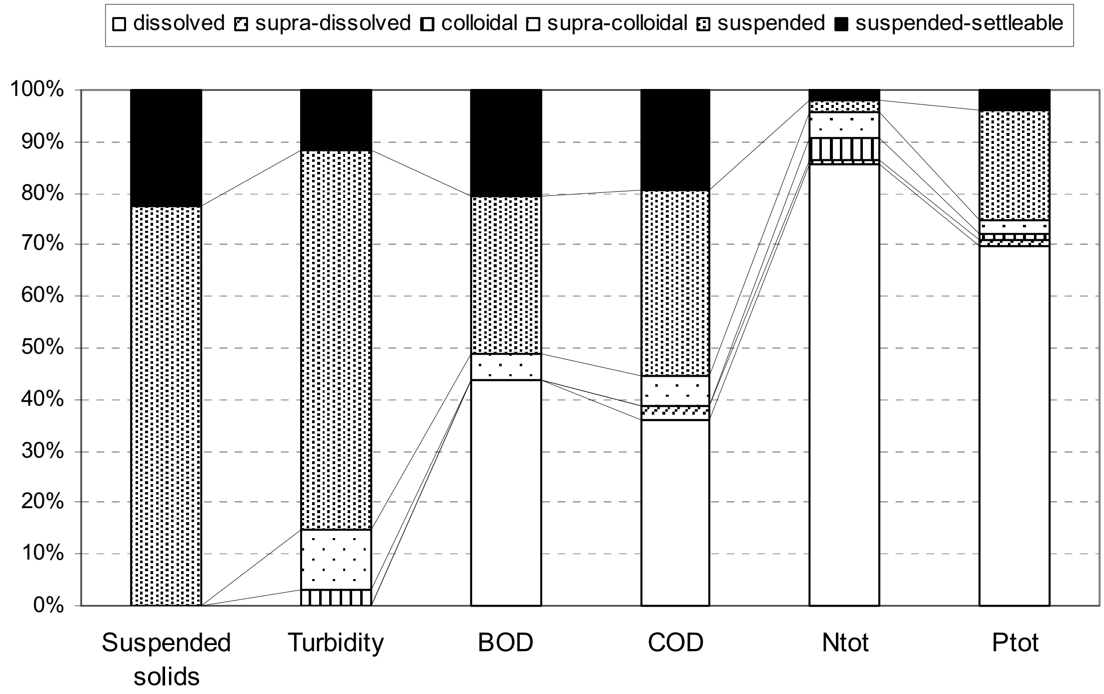


Figure F.11: Fractions in wastewater influent of wwtp Haarlem

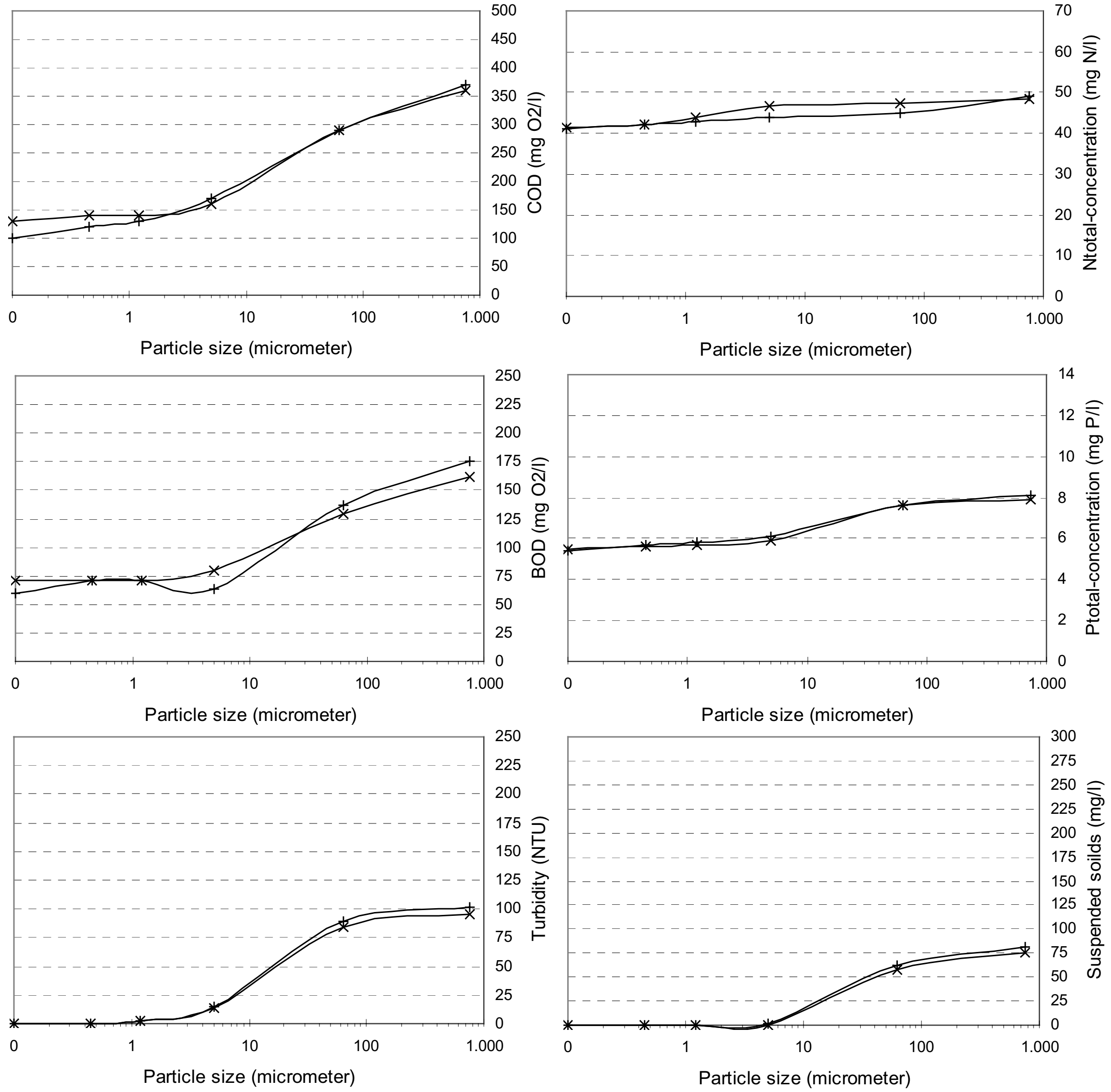


Figure F.12: Wastewater characterisation of wwtp Haarlem

Table F.7: Wastewater fractions in influent of wwtp Hoek van Holland

Fraction Parameter	Dissolved (< 0.1 µm)	Supra-dissolved (0.1–0.45µm)	Colloidal (0.45-1.2µm)	Supra-colloidal (1.2 – 5 µm)	Suspended (5 – 63 µm)	Settleable (> 63 µm)	Raw influent
TSS (mg/l)	n.d.	n.d.	n.d.	n.d.	92	39	132
Turbidity (NTU)	n.d.	n.d.	14	15	81	6	116
BOD_5 (mg O_2/l)	74	n.t.	n.t.	29	29	19	151
COD (mg O_2/l)	150	50	10	60	70	40	380
N_{total} (mg N/l)	36.0	0.5	0.5	3.0	1.0	1.0	42.0
P_{total} (mg P/l)	4.9	0.2	0.2	0.1	1.5	0.2	7.1
TSS	-	-	-	-	70%	30%	
Turbidity	-	-	12%	13%	70%	5%	
BOD_5	49%	-	-	19%	19%	13%	
COD	39%	13%	3%	16%	18%	11%	
N_{total}	86%	1%	1%	7%	2%	2%	
P_{total}	69%	3%	3%	1%	21%	3%	

n.d. = not detectable; n.t. = not tested

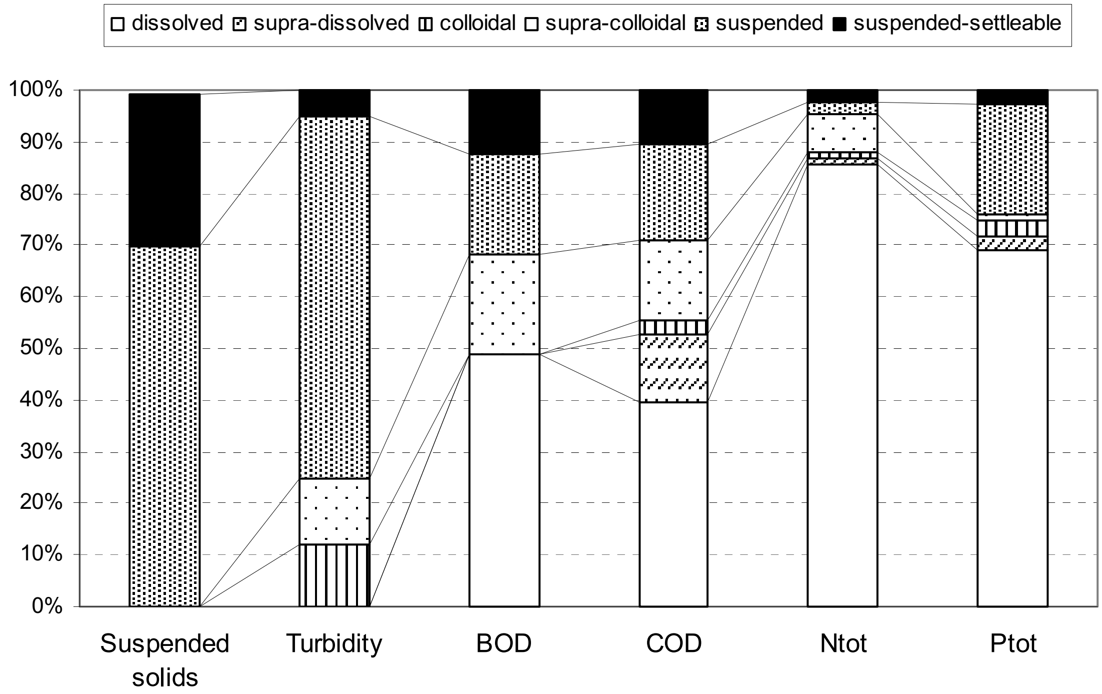


Figure F.13: Fractions in wastewater influent of wwtp Hoek van Holland

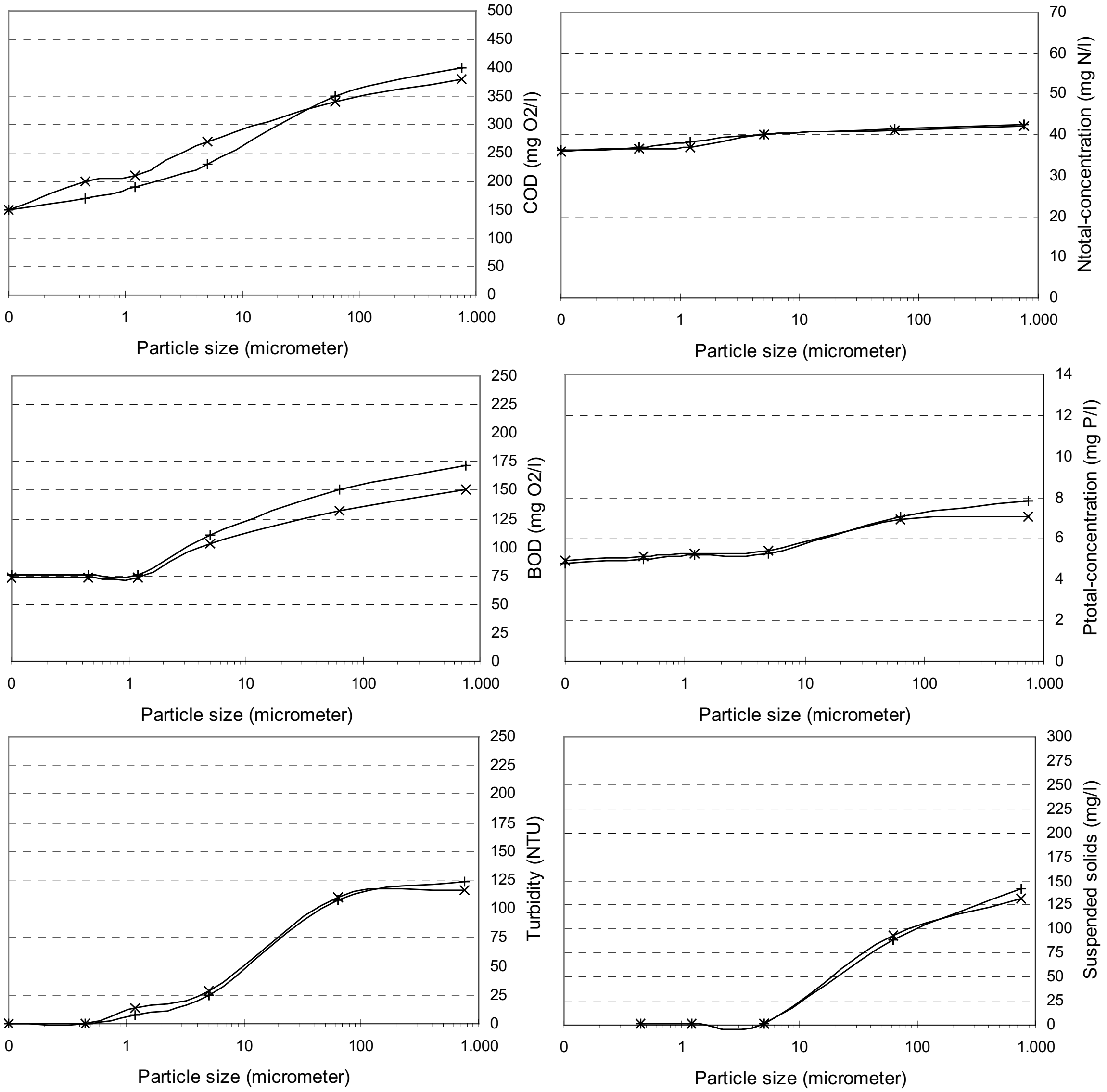


Figure F.14: Wastewater characterisation of wwtp Hoek van Holland

Table F.8: Wastewater fractions in influent of wwtp Vlaardingen

Fraction Parameter	Dissolved (< 0.1 µm)	Supra-dissolved (0.1–0.45µm)	Colloidal (0.45-1.2µm)	Supra-colloidal (1.2 – 5 µm)	Suspended (5 – 63 µm)	Settleable (> 63 µm)	Raw influent
TSS (mg/l)	n.d.	n.d.	n.d.	n.d.	118	64	183
Turbidity (NTU)	n.t.	n.t.	18	30	81	8	137
BOD_5 (mg O_2/l)	72	n.t.	n.t.	33	36	14	155
COD (mg O_2/l)	160	10	20	30	130	100	450
N_{total} (mg N/l)	29.0	0.5	1.0	1.5	2.5	4.5	39.0
P_{total} (mg P/l)	4.0	0.1	0.2	0.4	1.4	0.1	6.2
TSS	-	-	-	0%	64%	35%	
Turbidity	-	-	13%	22%	59%	6%	
BOD_5	46%	-	-	21%	23%	9%	
COD	36%	2%	4%	7%	29%	22%	
N_{total}	74%	1%	3%	4%	6%	12%	
P_{total}	65%	2%	3%	6%	23%	2%	

n.d. = not detectable; n.t. = not tested

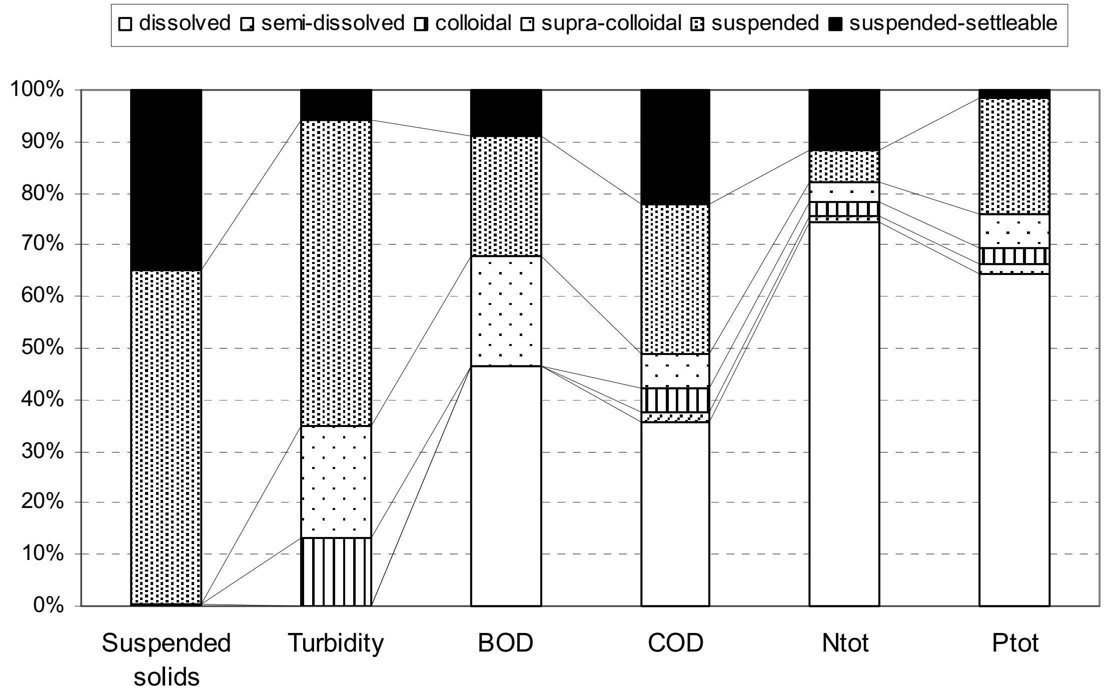


Figure F.15: Fractions in wastewater influent of wwtp Vlaardingen

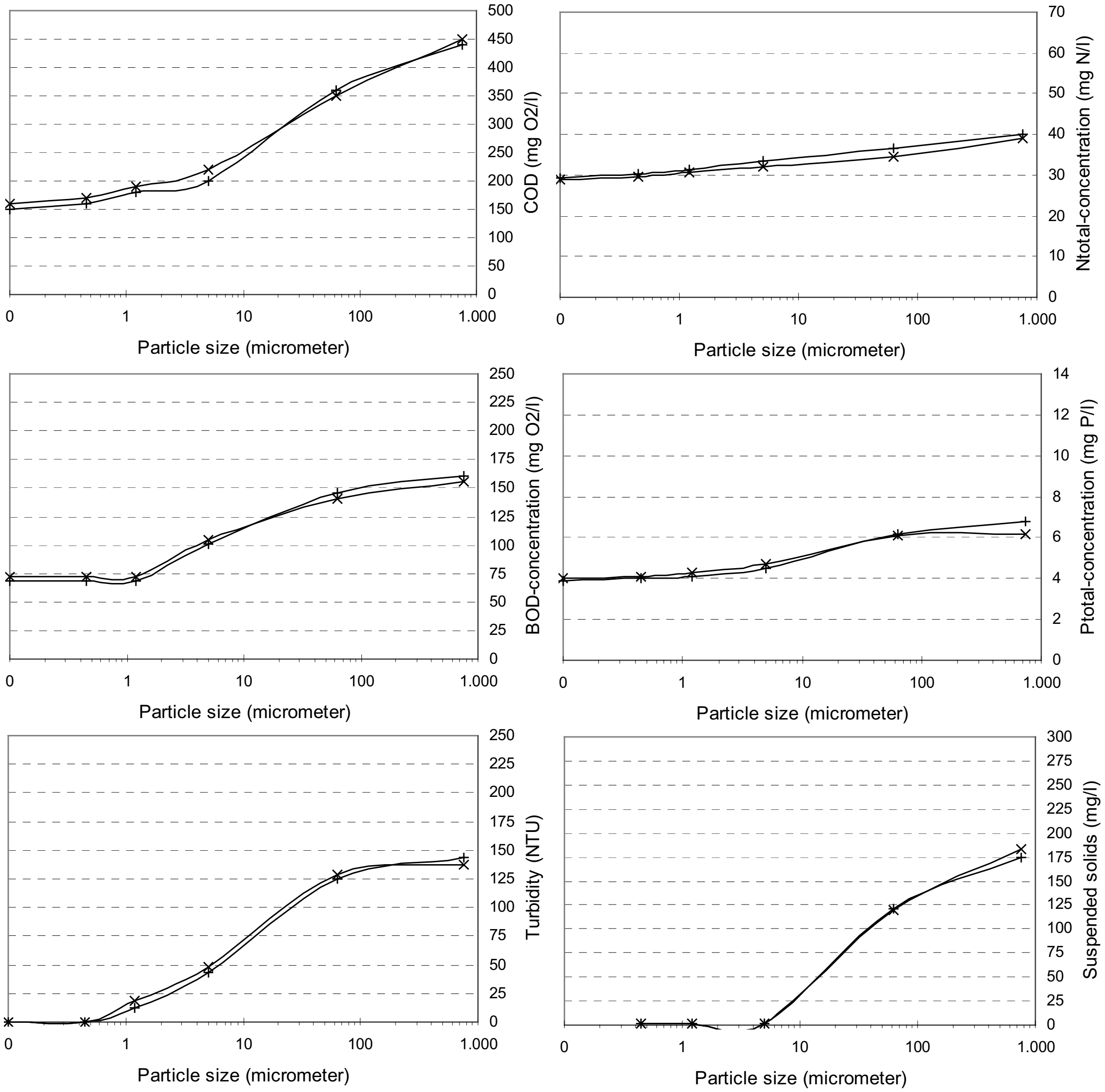


Figure F.16: Wastewater characterisation of wwtp Vlaardingen

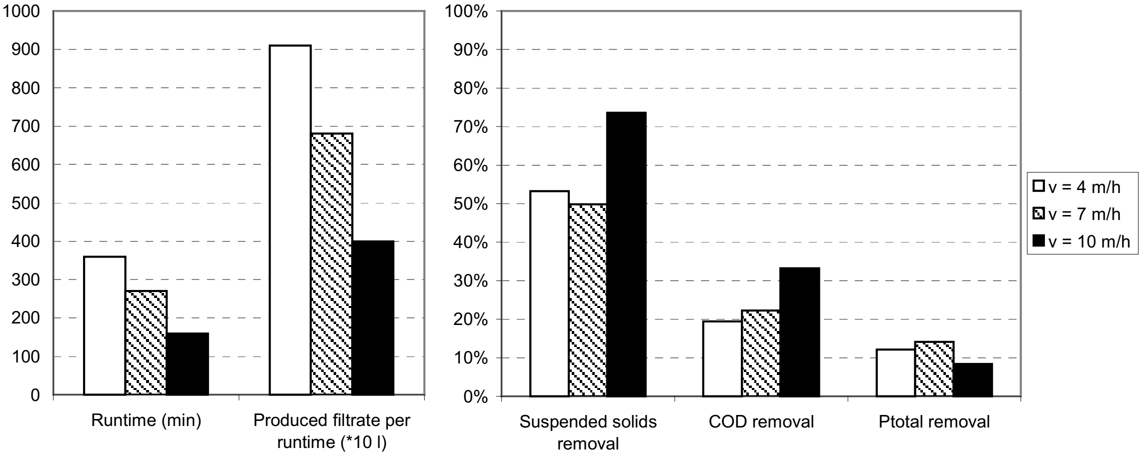
APPENDIX G: TESTED POLYMERS - Types and suppliers

Table G.1: Overview over types of polymers used in the experimental research

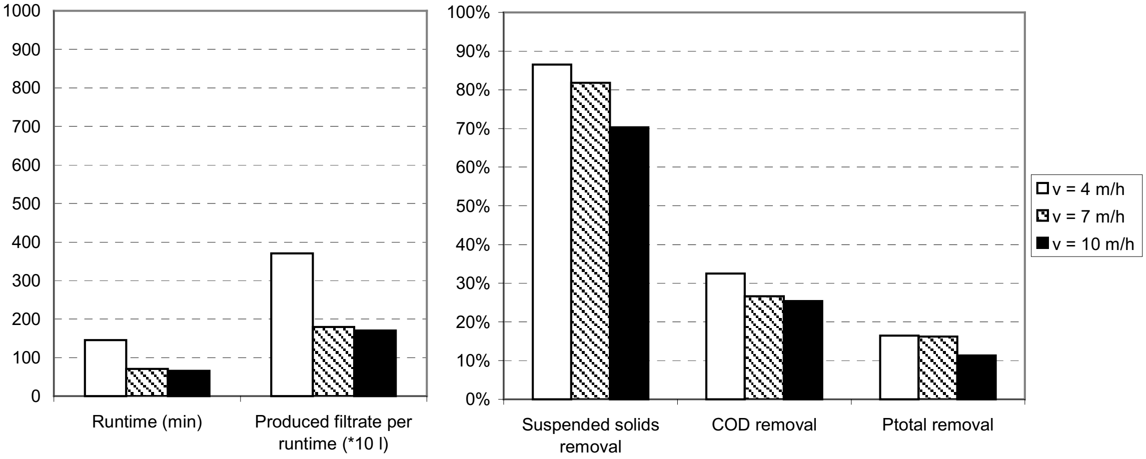
Charge	Cationic	Cationic	Cationic	Anionic	Anionic	Anionic
Mass	Charge density 10 – 30 %	Charge density 30 - 70 %	Charge density 70 – 90 %	Charge density 10 %	Charge density 20 %	Charge density 30 %
HMW	CIBA Zetag 32 CIBA Zetag 34 CIBA Zetag 7650 Cytec SF C-492 HMW ONDEO Nalco Filtrafloc 4203	CIBA Zetag 48 CIBA Zetag 7652 CIBA Zetag 7653 Cytec SF C-494 ONDEO Nalco Filtrafloc 4403 ONDEO Nalco Filtrafloc 4543	CIBA Zetag 7651 CIBA Zetag 7654 ONDEO Nalco Filtrafloc 4703 ONDEO Nalco Filtrafloc 4993	ONDEO Nalco Filtrafloc 2133	ONDEO Nalco Filtrafloc 2403	Cytec SF A130 HMW
MMW	Cytec SF C-492 MMW Cytec SF C-442	Cytec SF C-444		Cytec SF A110 MMW	Cytec SF A120 MMW	Cytec SF A130 MMW
LMW	CIBA Magnafloc 368 Cytec SF C-571 ONDEO Nalco Filtrafloc 4382					Cytec SF A130 HMW

Performance of discontinuous filter configurations

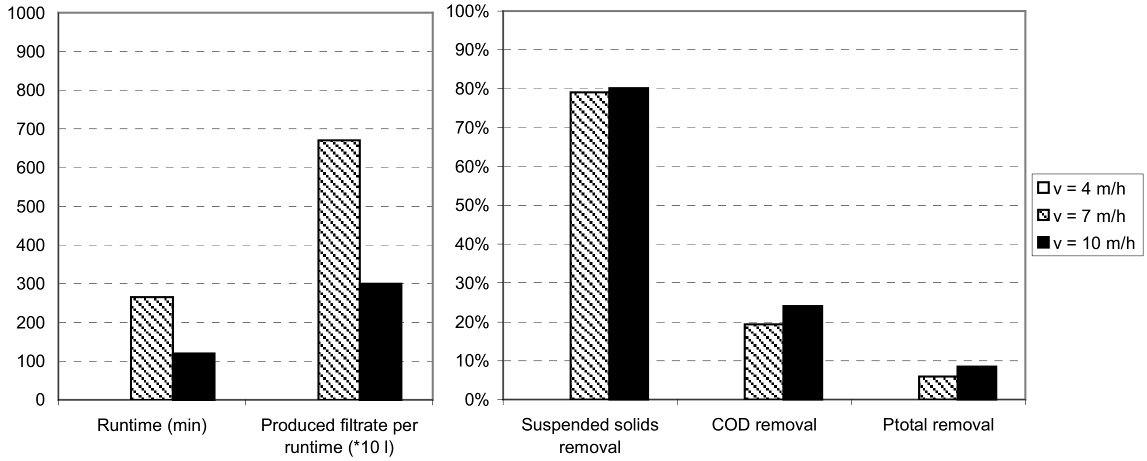
Anthracite up-flow



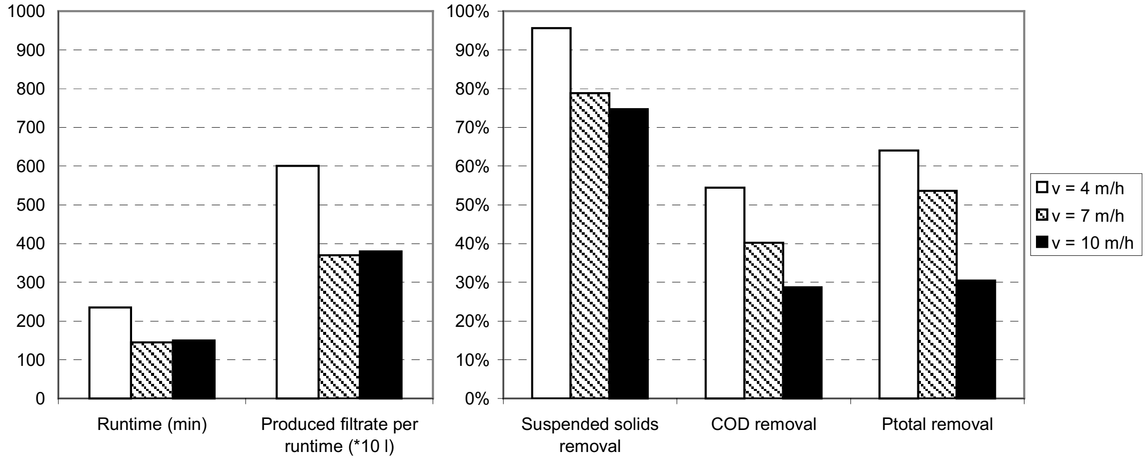
Anthracite down-flow



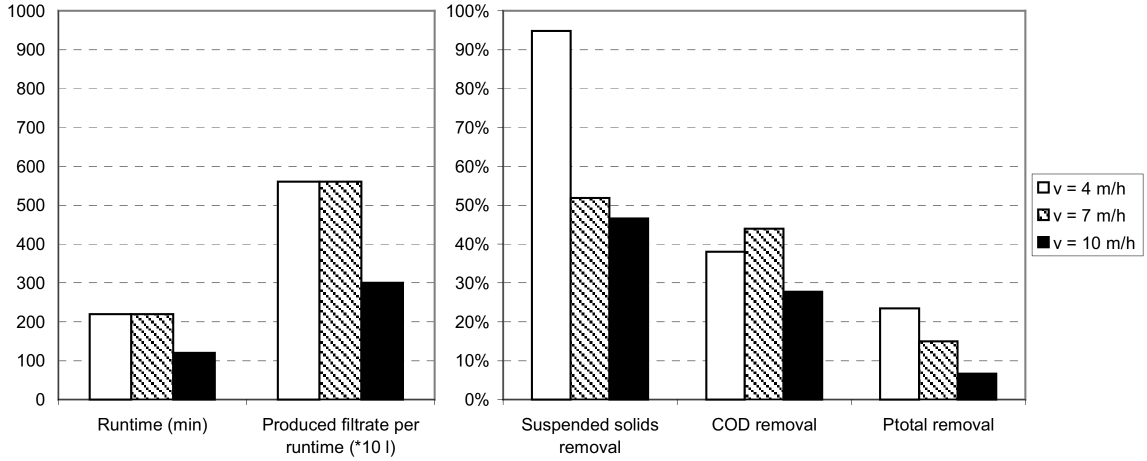
Anthracite + volcano



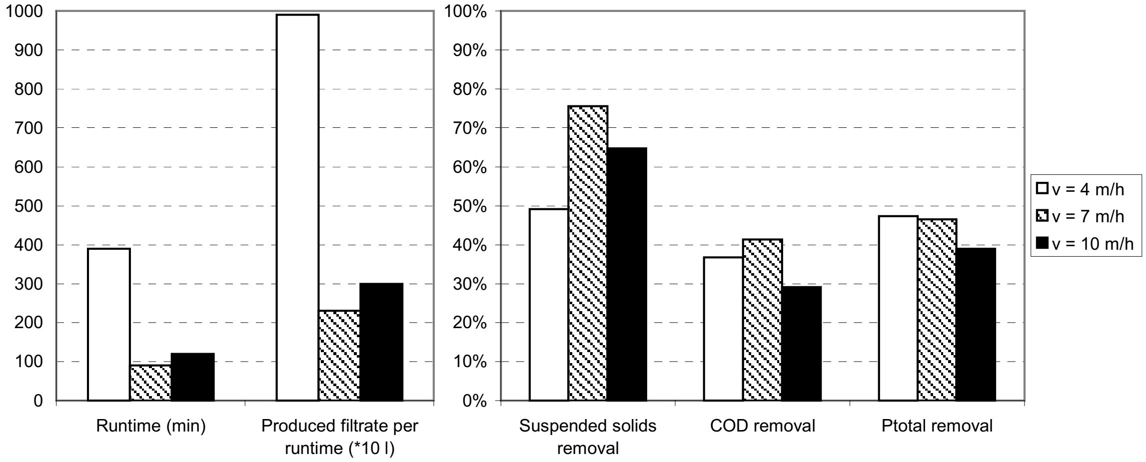
Anthracite + volcano + Fe3+



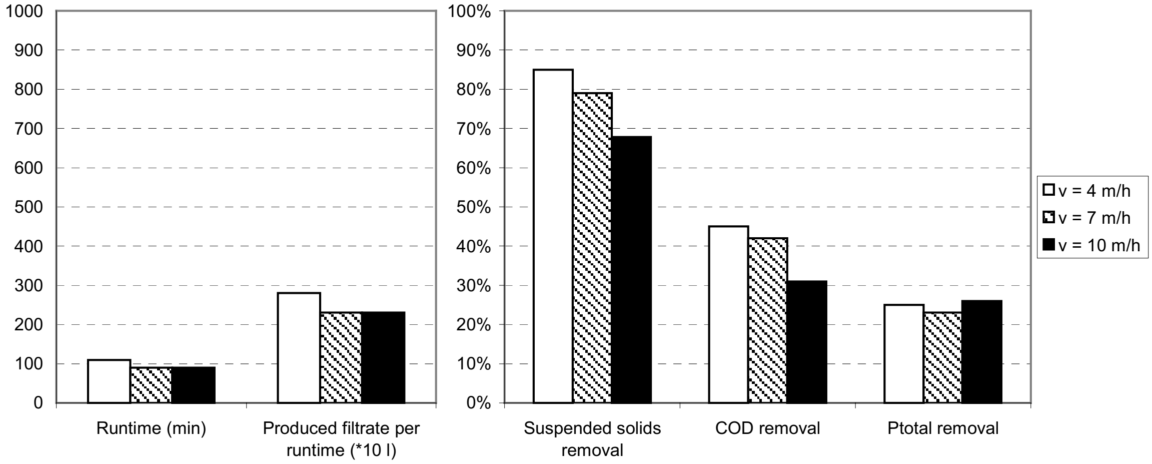
Sand + volcano



Sand + volcano + Fe3+

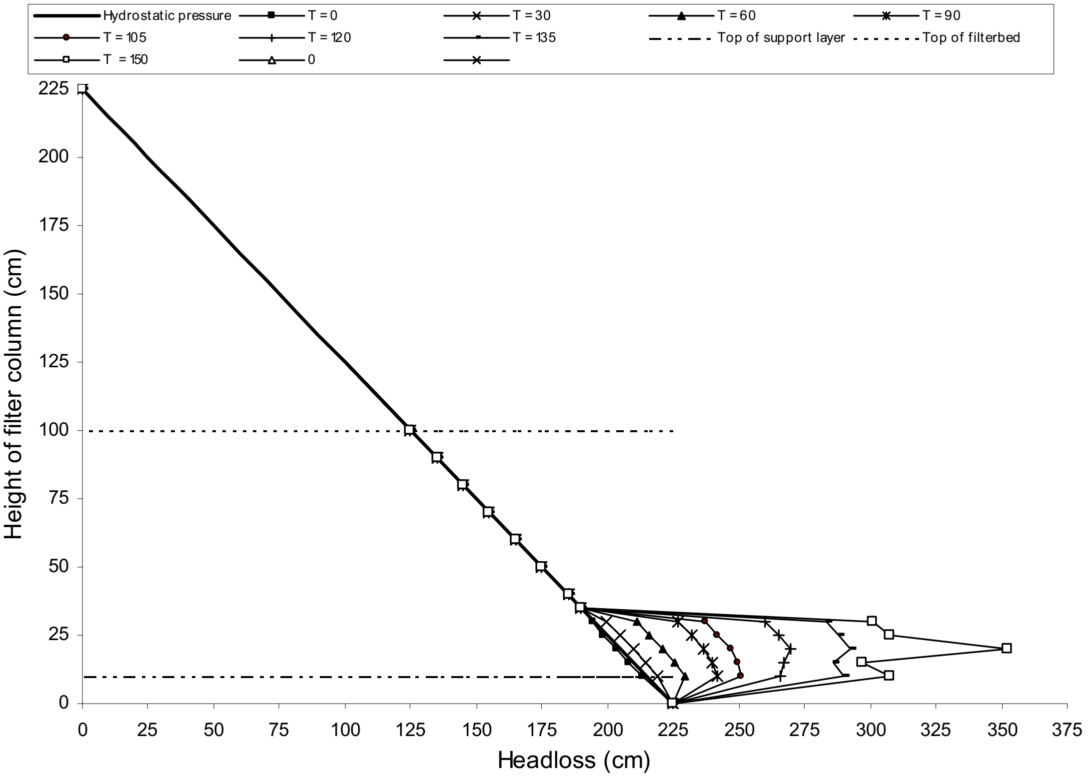


Sand + volcano + PE

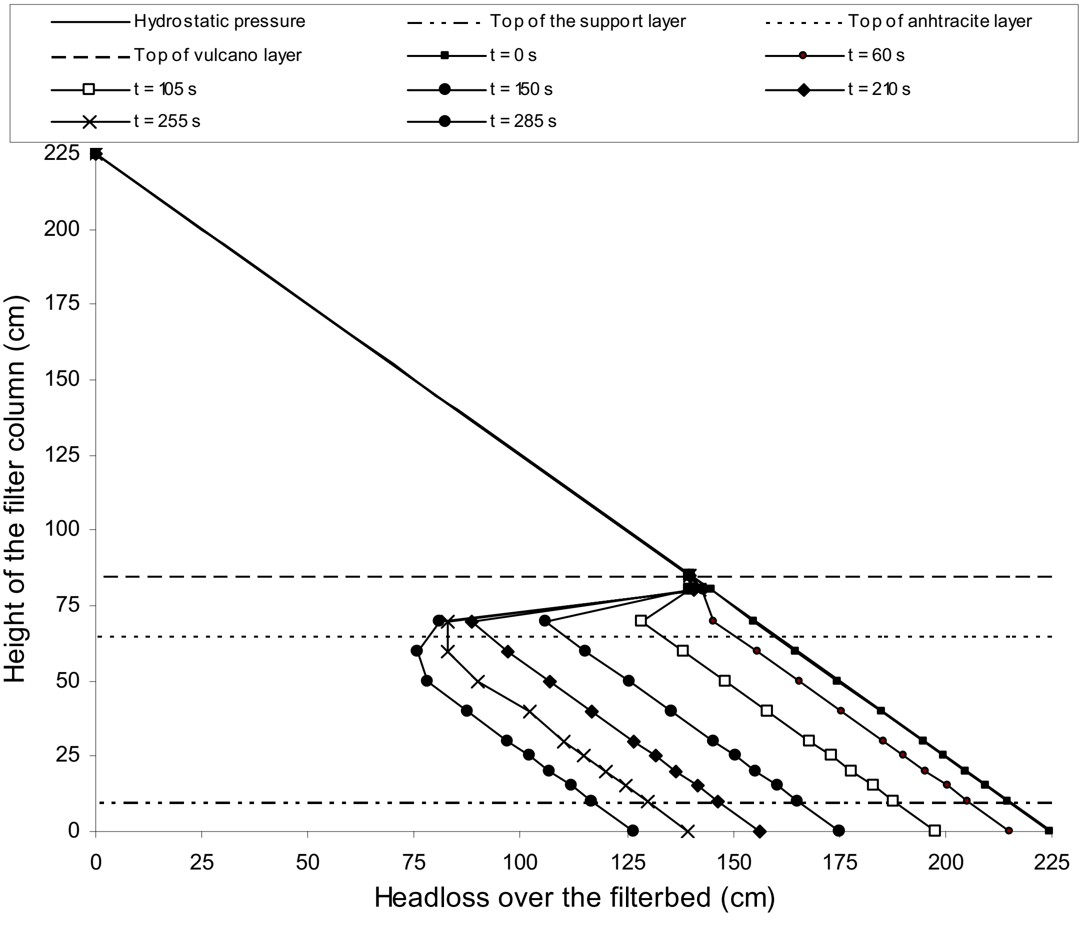


Examples of Headloss measurements over the discontinuous filters by means of Lindquist diagrams

Up-flow filtration

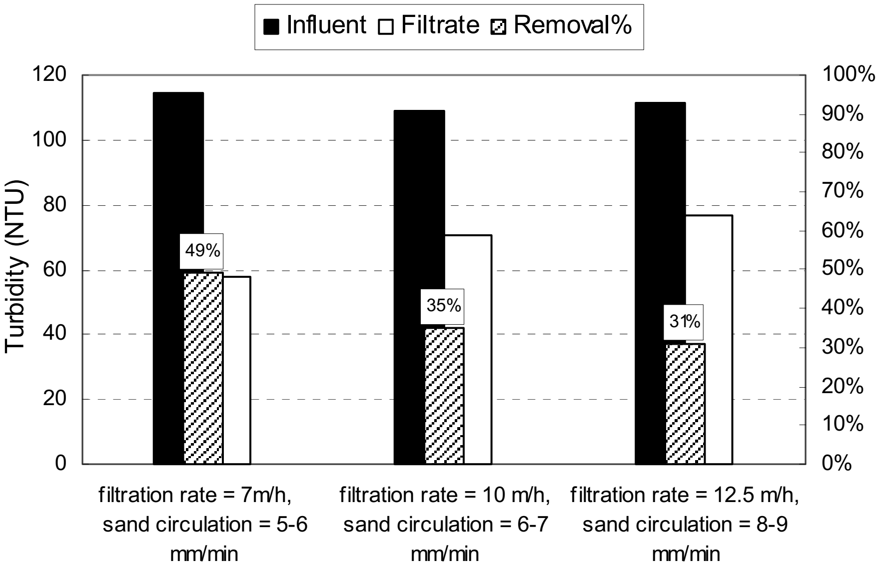


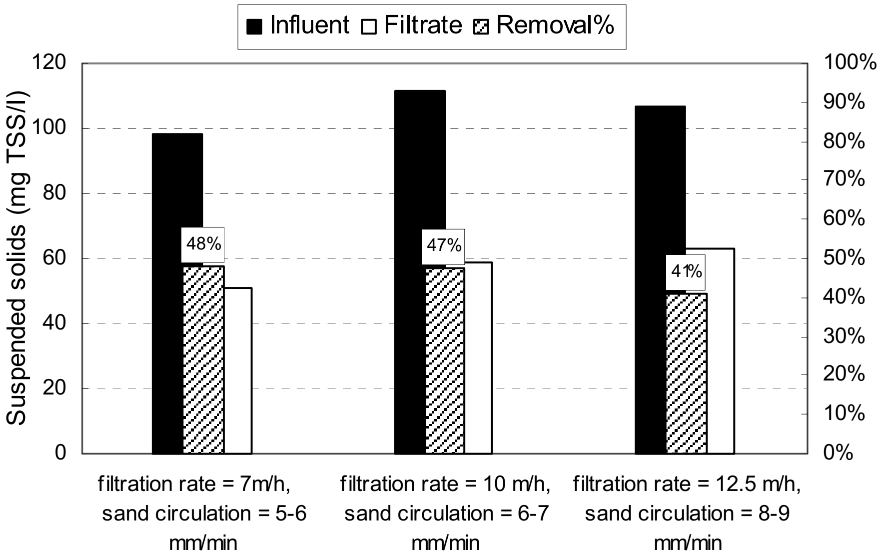
Down-flow filtration

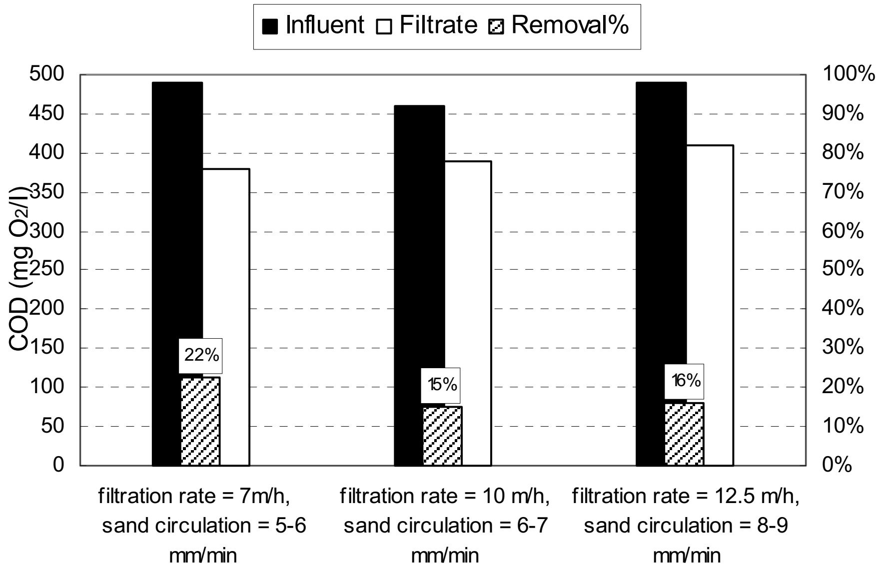


Performance of the continuous filter

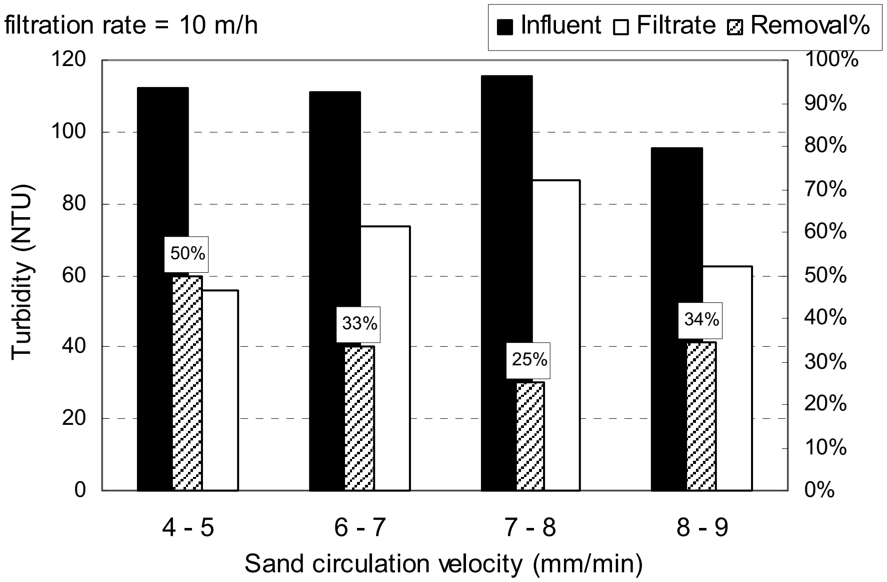
Influence of the filtration rate on the performances

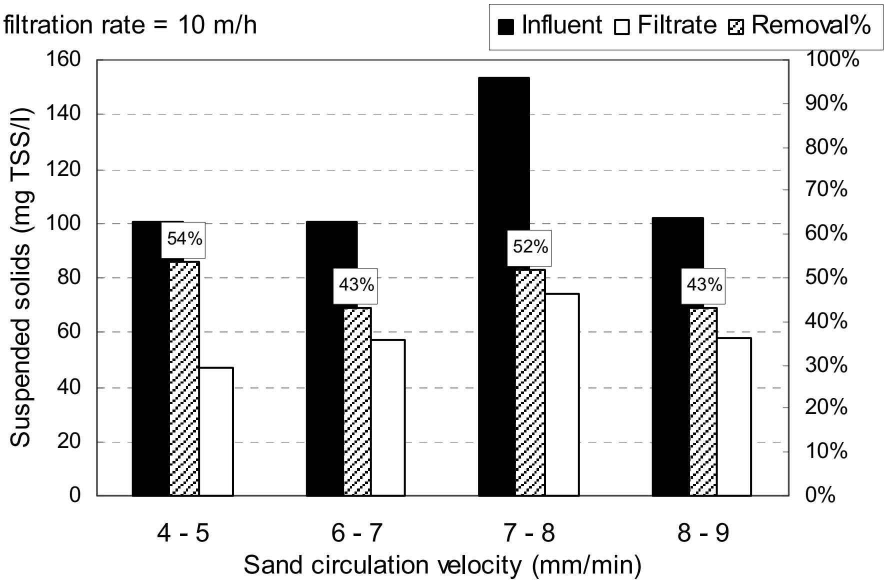


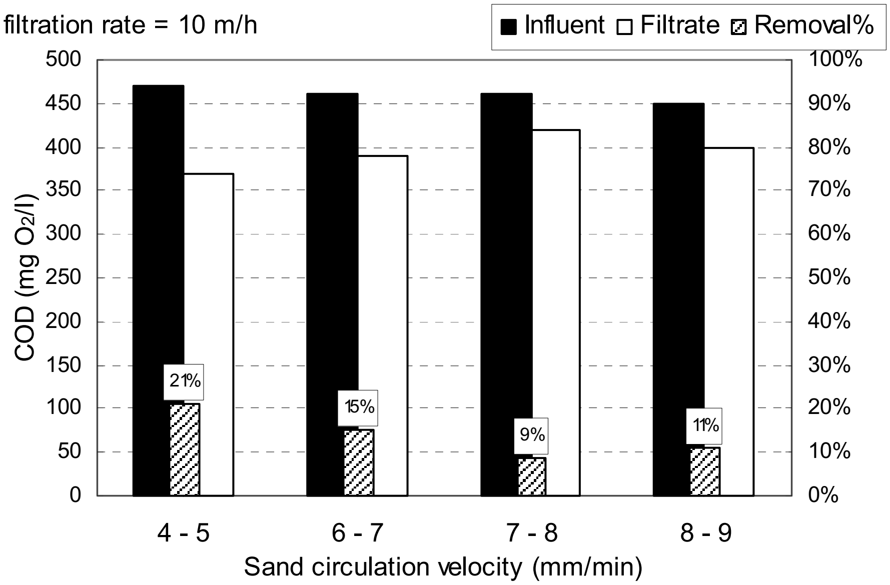




Influence of the sand circulation velocity on the performances







APPENDIX I: MEMBRANE SPECIFICATIONS - Operational data of tested membranes

STORK MEMBRANE TYPE FX4382 AND FX4385

Performances

Initial (clean water) flux (distilled water, 25 °C) at 100 kPa	> 1,000 $l/m^{2} \cdot h$
Maximum tolerable pressure	300 kPa (3 bar)
Maximum tolerable backflush pressure	100 kPa (1 bar)
Nominal pore size	30 nm
Molecular weight cut off (MWCO)	1,000,000

Physical and chemical properties

Membrane material:	hydrophilic poly-vinyli-denefluoride
Structure:	asymmetric / micro-porous
Support layer material:	composite polyester/polyolefine
Geometry:	hollow fibre tube membrane

Dimensions

hydraulic diameter:	5.2 mm
effective membrane surface:	0.17 m^2 per module

Tolerable pH-range (continuous at 25 °C):	2 - 10
Maximum tolerable chlorine concentration:	250.000 ppm.hr
Maximum tolerable temperature:	70 °C (at pH 7 and 100 kPa)

APPENDIX J: DEMAS$^+$ CALCULATION RESULTS - Evaluation data

Table J.1 Costs as net present value (NPV) in €

	scenario	pre-treatment	post-treatment	**total NPV (€ $1*10^6$)**	civil+EM (€ $1*10^6$)	variable (€ $1*10^6$)	relative NPV
	Ref	primary sedimentation	low loaded activated sludge	**37.7.-**	18.9.-	18.7.-	100%
division 1	1a	primary sedimentation + PE	low loaded activated sludge	**33.3.-**	14.6.-	18.7.-	88%
	1b	coarse media filtration	low loaded activated sludge	**41.4.-**	21.8.-	19.6.-	110%
	1c	coarse media filtration+10gFe^{3+}/m^3	low loaded activated sludge	**42.5.-**	20.9.-	21.7.-	113%
	1d	direct membrane filtration	low loaded activated sludge	**40.9.-**	18.8.-	22.1.-	109%
division 2	2a	primary sedimentation + PE + Fe	bio film reactor (post denitrifying)	**36.9.-**	10.4.-	26.5.-	98%
	2b/c	coarse media filtration+10gFe^{3+}/m^3	bio film reactor (post denitrifying)	**44.5.-**	16.1.-	28.3.-	118%
	2d	direct membrane filtration + Fe	bio film reactor (post denitrifying)	**43.2.-**	16.2.-	27.0.-	115%
division 3	3a	primary sedimentation + PE + Fe	membrane bio-reactor	**61.5.-**	31.7.-	29.8.-	163%
	3b/c	coarse media filtration+10gFe$^{3+/}$m^3	membrane bio-reactor	**66.3.-**	36.8.-	29.5.-	176%
	3d	direct membrane filtration + Fe	membrane bio-reactor	**71.3.-**	37.8.-	33.4.-	189%

Table J.2 Energy consumption and space requirement

scenario	energy consumption (MWh/year)	total net space requirement (m^2)	space use of water system (m^2)	space use of sludge system (m^2)
Ref	1,601	10,010	9,640	370
1a	548	7,552	7,160	392
1b	1,364	8,321	7,940	381
1c	1,203	7,828	7,420	408
1d	2,032	6,313	5,910	403
2a	915	2,156	1,740	416
2b/c	2,068	2,194	1,800	394
2d	2,158	1,654	1,230	424
3a	7,266	1,263	930	333
3b/c	8,425	1,307	1,030	277
3d	8,490	758	390	368

Table J.3 Sludge production

scenario	sludge production (ton d.s./year)	inorganic sludge (ton d.s./year)	chemical sludge (ton d.s./year)	organic sludge (ton d.s./year)
Ref	1,498	565	0	933
1a	1,414	461	0	953
1b	1,476	525	0	951
1c	1,638	501	161	976
1d	1,386	420	0	966
2a	1,556	486	117	954
2b/c	1,570	551	117	902
2d	1,499	414	109	976
3a	1,183	402	116	665
3b/c	1,044	424	114	506
3d	1,260	381	109	771

Table J.4 Required extra carbon and nitrogen in primary effluents

scenario	extra carbon addition (ton methanol/year)	nitrogen in primary effluent (mg N/l)
Ref	0	59.4
1a	0	54.6
1b	0	57.6
1c	0	56.5
1d	110	52.5
2a	428	54.6
2b/c	485	57.6
2d	395	52.5
3a	0	54.6
3b/c	0	57.6
3d	107	52.5