# Physical-chemical pretreatment as an option for increased sustainability of municipal wastewater treatment plants

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"Je kent het mysterie pas als je er door gegrepen wordt."

# Abstract

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Most of the currently applied municipal wastewater treatment plants in The Netherlands are based on the activated sludge process and include nitrogen and phosphorus removal. Physical-chemical pretreatment might be an option to improve the environmental sustainability of these wastewater treatment facilities. Physical-chemical pretreatment is meant to separate suspended and colloidal particles in the first step of treatment. The organic concentrate of separated particles is considered a product that can be digested to produce biogas. Since a large part of the organic pollutants in wastewater is present as not-dissolved material (65-70% of COD), the pollutant load to pursuing treatment steps is consequently reduced.

In the first part of the thesis the concept of physical-chemical pretreatment was evaluated for its potential to design more sustainable treatment systems. Based on a review of unit operations, ten different scenarios for wastewater treatment were designed and evaluated along environmental and financial criteria. The evaluation showed that these scenarios could be designed more energy efficient (savings of 60-75% of energy) and smaller compared to currently applied treatment systems. The total cost of treatment scenarios, which combine pretreatment with secondary biological treatment appeared to be in the same range as that of a reference scenario. The scenario evaluation also revealed two important bottlenecks, namely (i) the use of metal-based flocculating agents results in the production of a considerable amount of extra inorganic sludge and in an increased salinity of the effluent and (ii) the removal of particles is partially accompanied by the removal of biodegradable COD which may affect the denitrification capacity of a biological post treatment step.

The second part of the thesis describes experimental work into the identified bottlenecks. As alternatives to metal-based flocculating agents the potential application of organic polymers was investigated. Jar tests and experiments with continuous reactors showed that particle removal efficiencies of 70-90% could be achieved by applying these polymers in combination with eighter settling or dissolved air flotation. Especially cationic, high molecular weight polyelectrolytes showed interesting characteristics for practical application as they appeared effective at relatively low doses. A polymer dosing control strategy based on continuous measurement of the influent-turbidity proved effective to create different levels of particle removal, despite large concentration fluctuations in the influent. The effect of particle removal on the elimination of nitrogen was investigated by *Nitrogen Uptake Rate* (NUR) tests and experiments with a pilot scale activated sludge plant. It was found that with the removal of particulate COD, the biodegradable COD of the investigated wastewater was reduced by 50%. Calculations based on the values obtained in the experiments however indicated that well-designed plants that receive wastewater with an average COD/N ratio still contain sufficient denitrification capacity to comply with the nitrogen effluent constraint at particle removal efficiencies as high as 80-90%.

*Keywords* : municipal wastewater treatment, physical-chemical pretreatment, chemically enhanced primary treatment, organic polymers, environmental sustainability

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# Preface

The investigation described in this thesis, forms part of the outcomes of a research project initiated and funded by STOWA, the Foundation for Applied Research on Water Management. The project addressed the potential application of various scenarios of physical-chemical pretreatment of municipal wastewater and has been conducted by a research consortium of the Delft University of Technology, Department of Sanitary Engineering, and the Wageningen University, Sub-department of Environmental Technology.

Results of this project were reported by STOWA (see List of Publications) and are as well described in two Ph.D. dissertations, namely the thesis "Scenario studies into advanced particle removal in the physical-chemical pretreatment of wastewater" by Ir. Arjen van Nieuwenhuizen (Delft University of Technology, January 2002) and the present thesis. Both dissertations investigate the potential of physical-chemical pretreatment to contribute to improvement of the environmental sustainability of wastewater treatment plants. Furthermore, the thesis by Van Nieuwenhuijzen investigates the size distribution of contaminants in municipal wastewater and its relation to the performance of various unit operations for pretreatment.

The investigation that is described in this dissertation contains a study on scenarios with pretreatment, as well as experimental work that focussed on bottlenecks that were encountered in this evaluation. The experimental study deals with the potential application of organic poly electrolytes, in combination with settling and dissolved air flotation. Furthermore, experiments are described into the effect of pretreatment on the elimination of nutrients in a secondary biological treatment system.

Chapter 1 starts with a general introduction, describing the municipal wastewater treatment practice in The Netherlands. It proceeds with a description of the concept of physical-chemical pretreatment. In the final paragraph the objectives of the investigation and an outline of the thesis are given.

Adriaan Mels
Wageningen, September 2001

# 1 Introduction

## Municipal wastewater treatment in The Netherlands

Starting in the 18th century, sewerage systems were constructed in many cities in Europe to collect human wastes, wastewater and rainwater for transportation out of the city area. Previously, human waste used to be collected in barrels for reuse in agriculture or disposed off into the streets, together with other household waste. Thanks to growing hygienic awareness, health workers and sanitarians, in particular in Great Britain, started promoting the idea to provide water supply to every single household individually and to construct sewer systems in cities. The latter were meant to remove the increasing volumes of domestic waste and wastewater.

The wastewater discharged by the newly built sewer systems, was supposed to be used for land treatment as a fertilizer. A large volume of wastewater however, was ultimately discharged into rivers and lakes, which resulted in many problems of water quality. After examination of surface water, in 1873 the assembly of Dutch public health inspectors considered the quality of a lot of surface water alarming and called for legal action. By 1897 a Commission of State had been constituted with the assignment to prepare measures against pollution of surface water. In 1901 this Commission published a 'Proposal of law against the annoying and damaging water pollution', which was withdrawn by the government in 1905, because the availability of feasible wastewater treatment technologies at that time was limited (Koot, 1983). It took quite some time and several attempts to constitute a legal framework for the battle against water pollution, until in 1970 the first Pollution of Surface Waters Act was signed and enforced in The Netherlands.

The Pollution of Surface Waters Act stated that all sewerage effluent had to be treated before discharge into surface water. At first, compliance of the Act required basically the removal of oxygen consuming material (BOD and ammonium) and pathogenic bacteria. During the 1990s however, more stringent effluent standards came into effect, which also required the removal of nutrients (phosphate and nitrogen) because of their eutrofying properties. In 1992 the EU Urban Wastewater Treatment Directive EC 91/271 got into implementation. Table 1.1 shows the emission standards anno 2001.

**Table 1.1:** Emission standards in The Netherlands as of 2001 (measured as 10-days averages of flow proportional daily composite samples, except for $N_{total}$ which is based on yearly averages)
(taken from: Dirkzwager, 1997)

| | **Discharge standard** | | **Applies** | **Applies to** |
|---|---|---|---|---|
| BOD | $\leq 20$ | $mgO_2.l^{-1}$ | now | |
| N-Kjeldahl | $\leq 20$ | $mgN.l^{-1}$ | now | |
| Suspended solids | $\leq 30$ | $mgTS.l^{-1}$ | now | |
| $P_{total}$ [a] | $\leq 2$ | $mgP.l^{-1}$ | now | new plants + existing plants < 100,000 p.e.[c] |
| | $\leq 1$ | $mgP.l^{-1}$ | now | new plants + existing plants > 100,000 p.e. |
| $N_{total}$ [a] | $\leq 15$ | $mgN.l^{-1}$ | now | new plants < 20,000 p.e. |
| | $\leq 10/15$[b] | $mgN.l^{-1}$ | now | new plants > 20,000 p.e. |
| | $\leq 15$ | $mgN.l^{-1}$ | from 2005 | existing plants < 20,000 p.e. |
| | $\leq 10$ | $mgN.l^{-1}$ | from 2005 | existing plants > 20,000 p.e. |

[a] exceeding of these limits is possible under the condition that an average 75% elimination rate has been achieved within the area of a water board

[b] $\leq 10mgN.l^{-1}$; unless phosphate is removed by simultaneous precipitation; in the latter case $\leq 15mgN.l^{-1}$

[c] p.e. = population equivalent

Despite the absence of legislation, wastewater treatment plants were built since the 1920s. In 1958, some 154 - mostly mechanical or physical-chemical - plants were in operation and the municipal wastewater of 2 million people was treated before discharge (Koot, 1983). The enforcement of the Surface Water Act in 1970 induced a strong increase of the construction of municipal wastewater treatment plants. Nowadays, The Netherlands has approximately 400 treatment plants treating approximately 5 million $m^3$ of wastewater per day. This treatment covers 97% of the total volume of wastewater that is produced in households (toilet, cleaning and bathing water). Furthermore, a large part of industrial wastewater discharges are transported by the sewerage system and are treated. Additionally, during storm weather events, the sewer is used for the transport of rainwater out of urban areas. In such occasions, the rainwater is mixed with the wastewater which entails an increased volume of water to be treated at the treatment facilities.

The Central Bureau of Statistics collects and keeps data with regard to wastewater treatment plants, volumes and characteristics of municipal wastewater. Table 1.2 shows the number and design capacity of wastewater treatment plants, as well as the treated volume and the average influent and effluent concentrations of different pollutants in different years.

**Table 1.2**: Number, design capacity, influent flows and influent and effluent characteristics of municipal wastewater treatment plants in 1981, 1990 and 1998

| **Treatment plants** | **1981** | **1990** | **1998** |
|---|---|---|---|
| total number | 504 | 469 | 409 |
| design capacity | $20.0\cdot10^6$ p.e. | $23.7\cdot10^6$ p.e. | $24.8\cdot10^6$ p.e. |
| influent flow | $3.5\cdot10^6$ $m^3.day^{-1}$ | $4.6\cdot10^6$ $m^3.day^{-1}$ | $5.9\cdot10^6$ $m^3.day^{-1}$ |

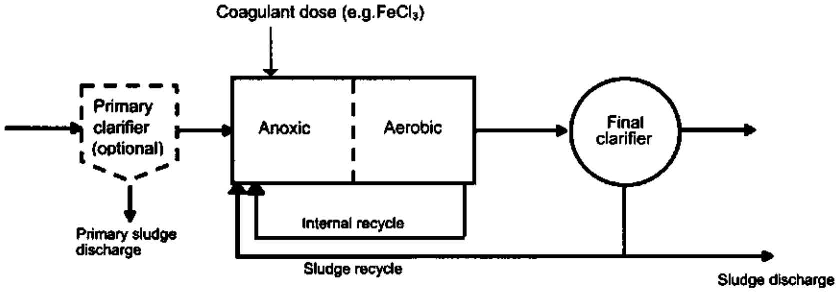
| | **Influent composition (average concentrations)** | | | **Effluent composition (average concentrations)** | | |
|---|---|---|---|---|---|---|
| | 1981 | 1990 | 1998 | 1981 | 1990 | 1998 |
| COD ($mgO_2.l^{-1}$) | 551 | 595 | 456 | 112 | 88 | 51 |
| $BOD_5$ ($mgO_2.l^{-1}$) | 222 | 222 | 173 | 29 | 18 | 7 |
| N-Kjeldahl ($mgN.l^{-1}$) | 53 | 52 | 42 | 18 | 14 | 7 |
| $N_{total}$ ($mgN.l^{-1}$) | | | | | 19 | 14 |
| $P_{total}$ ($mgP.l^{-1}$) | 15 | 8 | 7 | 10 | 4 | 2 |
| Influent flow ($m^3.day^{-1}$) | $3.5\cdot10^6$ | $4.6\cdot10^6$ | $5.9\cdot10^6$ | | | |

source: Central Bureau Statistics - The Netherlands, 2000

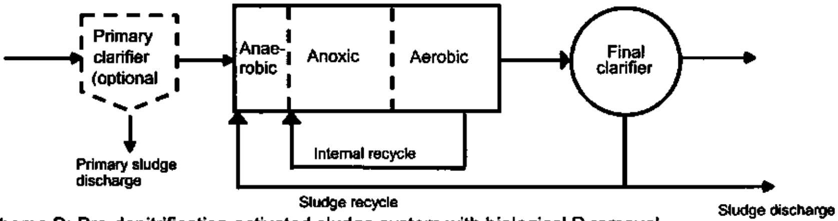
The current wastewater treatment plants in The Netherlands are generally based on the activated sludge process. In activated sludge systems, degradation of organic matter and removal of nutrients is established by the activity of suspended bacteria. Subsequent to the treatment, the bacterial flocs ('activated sludge') are removed in a final clarifier by gravitational settling. A part of this settled activated sludge is wasted because of the increase in mass as a result of growth. The remainder is returned to the beginning of the plant and mixed with untreated wastewater.

Figure 1.1 shows two frequently applied process configurations of activated sludge plants. In both systems nitrogen removal by subsequent nitrification and (pre) denitrification is applied. The conversion of ammonium to nitrate (*nitrification*) takes place in the aerobic compartment in the second part of the reactor. The nitrate that is produced during this phase, is returned to the anoxic part of the reactor where the nitrate is converted into nitrogen gas (*denitrification*) in the presence of biodegradable COD. The removal of phosphate in configuration A is established by the addition of precipitants like ferric chloride ($FeCl_3$). In configuration B, phosphate is removed through phosphate accumulating bacteria. Both configurations can be preceded by a primary clarifier for the removal of a part of the suspended particles that are present in the wastewater.

An important by-product of wastewater treatment is waste sludge. As was explained earlier, the activated sludge is to be partially removed from the final clarifier. When a primary clarifier is applied, also primary sludge is produced. The total sludge flow is in many cases partially treated on-site (i.e. at the treatment plant) by digestion and dewatering. After dewatering it is transported to an incinerator or a final disposal site.



Scheme A: Pre-denitrification activated sludge system with simultaneous P precipitation



Scheme B: Pre-denitrification activated sludge system with biological P removal

**Fig. 1.1:** Schemes of two frequently applied process configurations for wastewater treatment

Table 1.4 shows some important operational data of wastewater treatment plants. The table shows the energy consumption, the sludge production (in kg total solids – kg TS) and the costs. The table shows the total values as well as the average values per population equivalent (p.e.).

**Table 1.3:** Energy consumption, costs and sludge production of municipal wastewater treatment in The Netherlands

| Item | 1981 | 1990 | 1998 |
|---|---:|---:|---:|
| Energy consumption | | | |
| total (kWh.year$^{-1}$) | | | $520 \cdot 10^6$ |
| per p.e. (kWh.year$^{-1}$) | | | 24 |
| Final sludge disposal | | | |
| total (kg TS.year$^{-1}$) | $184 \cdot 10^6$ | $315 \cdot 10^6$ | $350 \cdot 10^6$ |
| per p.e. (kg TS.year$^{-1}$) | 9.2 | 13.3 | 14.1 |
| Average treatment costs | | | |
| total (Euro €) | $250 \cdot 10^6$ | $480 \cdot 10^6$ | $700 \cdot 10^6$ |
| per p.e. (Euro €) | 15.4 | 22.7 | 34.0 |

sources: Central Bureau Statistics - The Netherlands, 2000; van der Graaf, 2001

The 'Decision on Remaining Organic Fertilizers' that was enforced by VROM (Ministry of Housing, Land Use and Environment) in 1991, put restrictions on the maximal deposit of heavy metals on farmlands. Because of this legislation the use of municipal waste sludge as an agricultural fertilizer is no longer allowed. Incineration and discharge into a landfill have increasingly replaced the use of waste sludge on farmlands. Due to these relatively complicated procedures, the costs of sludge disposal have increased significantly (€ 400-500 per ton dry solids in 2000) and nowadays put a high burden on the total treatment of wastewater (40-50% of the total treatment cost).

## Environmental sustainability of wastewater treatment plants

The concept of "environmental sustainability" was introduced by the OECD Commission Brundlandt in 1987 (WCED, 1987). Environmental sustainability seeks to preserve the available natural resources and environmental conditions so that future generations will still be able to subsist in a sound ecosystem. Sustainable technological development is aimed at the development of technologies that decrease the input of not-renewable resources and prevent the output of emissions that are harmful to the ecosystem.

In recent years a number of studies has been published on the assessment of the environmental sustainability of municipal wastewater treatment and the urban water cycle. Regarding these studies a general distinction can be made into studies that focus on the wastewater treatment plant alone and studies that consider the urban water cycle and include the sewerage system and the drinking water supply (e.g. Otterpohl *et al.*, 1999; IWA, 2001). The review in this paragraph is limited to (Dutch) references that are dealing with the treatment plant itself.

In 1993 Rulkens and Van Starkenburg stated that, although municipal wastewater treatment plants are effective in reaching their primary goal of attaining the effluent constraints, additional attention should be paid to secondary environmental side effects. The environmental side effects they suggested were sludge production, emissions of greenhouse gases like $CO_2$ and $N_2O$ and the use of not renewable resources. They proposed general environmental starting points that should be taken into consideration for the design and development of new wastewater treatment plants (table 1.4).

**Table 1.4:** General environmental starting points that should be taken into consideration for the design and development of new wastewater treatment plants according to Rulkens and Van Starkenburg (1993)

- maximization of the use of valuable components in wastewater and sludge;
- minimization of energy demand and maximization of (re)use of energy;
- minimization of the use of not renewable resources;
- minimization of emissions to air and surface waters;
- minimization of final inert waste products.

In an exploratory study of the research program 'Sustainable Technological Development' the potential 'non-sustainability' of the urban water cycle was investigated (DTO, 1994; van der Graaf *et al.*, 1995). In this study, non-sustainability factors were defined and a ranking was introduced. With respect to wastewater treatment plants the following factors were identified and ranked:

**Table 1.5:** Non-sustainability factors of wastewater treatment plants according to DTO (1994) and Van der Graaf *et al.* (1995)

- discharge of heavy metals and organic micro-pollutants with the effluent;
- production of waste sludge;
- discharge of oxygen consuming compounds with the effluent;
- energy consumption and the subsequent $CO_2$ emission;
- chemical use;
- building materials.

In 1996 STOWA (*Foundation for Applied Research on Water Management*) published an environmental *Life Cycle Assessment* (Heijungs *et al.*, 1992) on the wastewater treatment practice in The Netherlands (STOWA, 1996; Roeleveld *et al.*, 1997). The use of resources and the emissions to the environment during the total life cycle was investigated for several treatment systems for municipal wastewater. The contribution of either treatment scenario to the total non-sustainability in The Netherlands was assessed by assuming that each system was the only type of system to treat wastewater in The Netherlands and comparing the subsequent total environmental impacts to the total national values.

The results of this assessment for the most frequently applied treatment system (configuration A of fig. 1.1) are shown in figure 1.2. The assessment showed that the main contribution was related to the discharge of oxygen consuming pollutants (COD 37%). Other contributions were related with the discharge of nutrients (nutrification potential) and micro pollutants (aquatic ecotoxicity, human toxicity), the production of waste sludge (final waste, toxic waste) and energy demand (global warming potential, fossil energy depletion, acidification potential). The researchers concluded that the depletion of raw materials due to the building and ultimate wastage of the wastewater treatment plant and the use of chemicals ($FeCl_3$ and organic polymer for sludge dewatering) are mainly connected with environmental impacts, which have a minor contribution to the non-sustainability of a wastewater treatment plant.

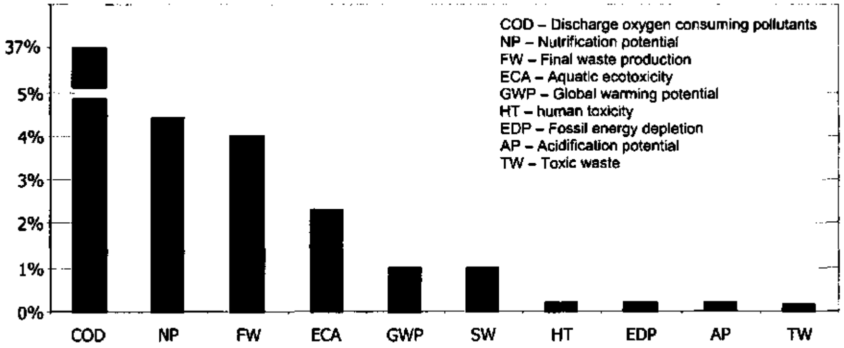


**Fig. 1.2:** Environmental impacts of currently applied wastewater treatment systems as percentage of the total contribution in the Netherlands according to STOWA, 1996 and Roeleveld *et al.*, 1997

The work that is described in this thesis contains evaluations of newly designed treatment scenarios with physical-chemical pretreatment (see next paragraph). These scenarios are evaluated along criteria that are shown in table 1.6. The first row shows the criteria that, based on the recited literature, appear most important with respect to the sustainable technological development of treatment plants. The second row shows two additional criteria that are important for the practical application.

**Table 1.6:** Evaluation criteria of newly designed wastewater treatment scenarios, within the framework of this thesis

| **Criteria that, based on the literature review, appear most important with respect to the sustainable development of wastewater treatment plants:** |
|---|
| ▪ energy demand and maximization of reuse of energy;<br>▪ minimization of final waste sludge production (especially of inert material);<br>▪ effluent quality with regard to oxygen consuming compounds, nutrients and micro pollutants;<br>▪ potential recovery of nutrients. |
| **Criteria that also will be assessed:** |
| ▪ space requirement (footprint);<br>▪ economic feasibility. |

## Physical-chemical pretreatment

In this dissertation the concept of physical-chemical pretreatment is studied for its potential to design more sustainable wastewater treatment plants. *Physical-chemical pretreatment* (or *enhanced primary treatment*) is meant to separate suspended and colloidal particles in the first step of treatment. By removing the particles a large part of the pollutants can be removed in a relatively easy way. Figure 1.3 shows a general picture of this concept (Ødegaard, 1998). As is shown in this figure, the organic concentrate of separated particles is considered to be a product. It can be digested to produce biogas, be acidified to serve as a carbon source for denitrification or can be re-used as a source of nutrients.

Due to the removal of particles, the load of pursuing unit operations is reduced so that they can be designed smaller. Pretreatment may reduce the load of organic pollutants to the post treatment considerably, because a large fraction of them (55-85%) is present in particulate form (Levine, 1985; Ødegaard, 1987). Moreover, due to the fact that the primary effluent is almost free of particles, alternative treatment techniques can be applied for secondary treatment, including physical-chemical techniques that may be feasible to recover nutrients.

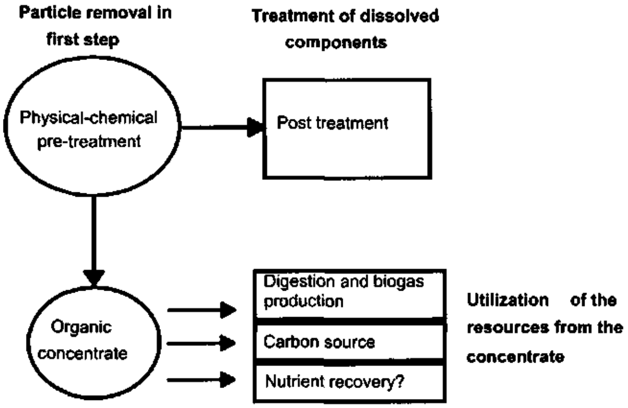


**Fig. 1.3:** Concept of physical-chemical pretreatment as a starting point for the design of potentially more sustainable treatment plants (Ødegaard, 1998)

Enhanced primary treatment in order to remove the particles from the wastewater is not certainly new. It is reported that back in 1740, chemical precipitation using lime was being applied in Paris for the partial treatment of wastewater discharged by the newly constructed sewerage system at that time (Wardle, 1893 in Cooper, 2001). In the 1890s in Great Britain, almost 200 plants employed chemical treatment of sewage discharges (Leentvaar, 1982), whereas most of the treatment plants constructed before 1970 in The Netherlands, consisted of (only) physical-chemical treatment (Koot, 1983). The purpose of the physical-chemical treatment used to be twofold (a) to remove a large part of the pollutants and (b) to produce a concentrated sludge that could be used for fertilization (instead of using wastewater directly at all its volume). However, chemical treatment had a major disadvantage: it only removed particulate pollutants. As soon as biological processes (which also removed dissolved organic matter and - dependent on the configuration - nitrogen) became available, chemical treatment gradually went out of use.

## Particles in municipal wastewater

As mentioned earlier, the contaminants are to a large extent present in municipal wastewater in the form of not-dissolved, particulate material. Although various definitions exist, the particulate material in wastewater can broadly be considered as the fraction of material that is retained by a membrane filter with a pore size of 0.1 µm (Levine *et al.*, 1991).

Figure 1.4 shows a size distribution for different contaminants in municipal wastewater. In these four graphs the relation between COD, N and P and the particle size is shown. The presented values are taken from a literature review with regard to the particle size distribution of different Dutch wastewater sources (VROM, 1986; De Man, 1990; STOWA, 1996; Hettinga, 1997).

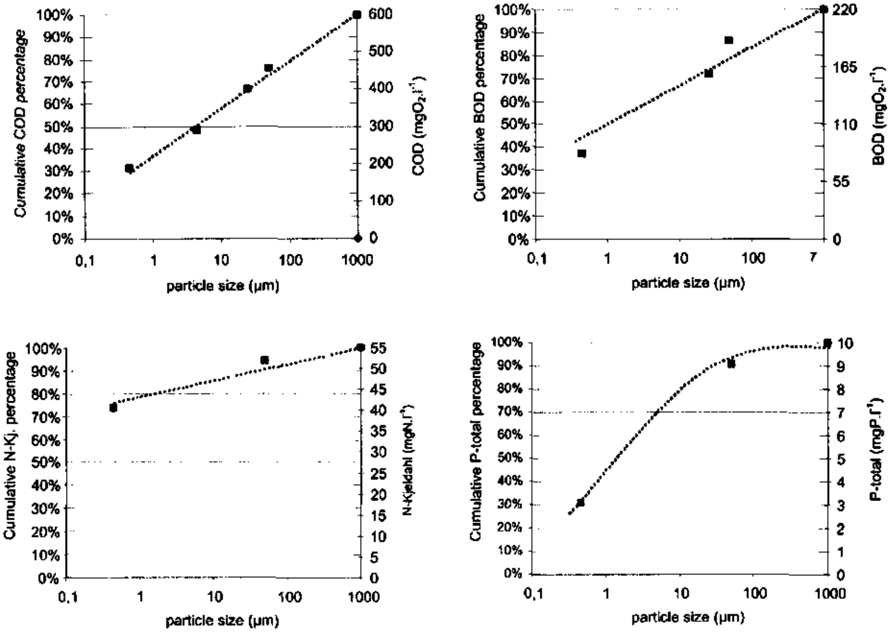


**Fig. 1.4**: Cumulative distribution for COD, BOD, N-Kjeldahl and $P_{total}$ over the different particle sizes; the shown values are average results of a review on different Dutch wastewaters (VROM, 1986; De Man, 1990; STOWA, 1996; Hettinga, 1997)

The literature data that are depicted in figure 1.4 represent the average concentrations of wastewater samples of different treatment plants. These samples were fractionated into particle sizes of around 50 µm (by settling), 25 and 4.4 µm (paper filtration) and 0.45 µm (membrane filtration) after which the different fractions were analyzed for COD, BOD, N-

Kjeldahl and $P_{total}$. The graphs indicate that approximately 70% COD, 60% BOD, 25% N-Kjeldahl and 30% $P_{total}$ are related to particles with a size > 0.45 µm.

The literature review of figure 1.4 also showed that data on the size distribution of contaminants in municipal wastewater are rare. Regarding the presented figures, not all the reviewed publications contained a full fractionation into the different sizes. In fact, only one source (VROM, 1986) reported data on samples that were filtered through a 25 µm filter. Similarly, not all the publications reported values regarding the four contaminants. A more thorough analysis of the size distribution of the four shown contaminants is presented in appendix A. This appendix contains data of an intensive measurement campaign that was conducted by Van Nieuwenhuijzen (2002) within the broader framework of the research project.

Heavy metals in municipal wastewater are mainly associated with solid material (Ødegaard, 1987). This is due to the fact that particulate material in municipal wastewater mainly consists of organic material and therefore has strong heavy metal sorbing characteristics. Table 1.7 shows the results of a study conducted by the Ministry of Housing, Planning and Environment (1986) into the speciation of metals in municipal wastewater. As shown in the table more than 90% of the metals – with the exception of nickel – are found in the particle fraction > 0.45 µm.

**Table 1.7**: Presence of heavy metals in particulate material

| **Metal** | **settleable solids (> ~ 50 µm)** | **all solids > 0,45 µm** |
|---|---|---|
| Copper | 11 - 45 % | 86 - 97 % |
| Chromium | 25 - 39 % | 53 - 96 % |
| Zinc | 18 - 30 % | 73 - 95 % |
| Lead | 25 - 50 % | 67 - 90 % |
| Nickel | 12 % | 25 - 29 % |

source: VROM, 1986

The size distribution of particles in wastewater is a determining factor for the removal efficiencies that can be achieved when a certain pretreatment method is applied. Figure 1.5 shows a number of unit operations for the (pre)treatment of wastewater and drinking water. In this figure the range of removal of these unit operations is given in relation to the size of the particles in influent (Levine *et al.*, 1985).

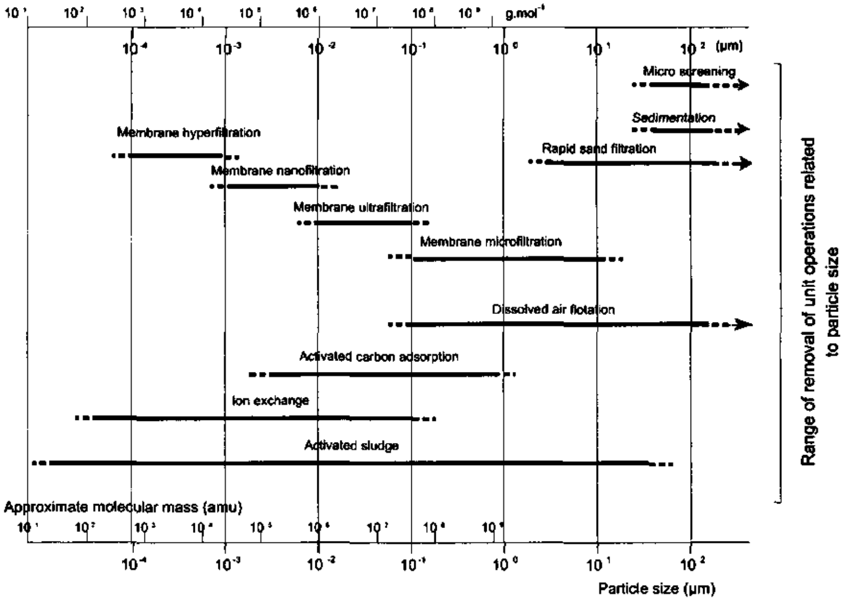


**Fig. 1.5:** Size removal range of a number of unit operations for (waste)water treatment (redrawn after Levine et al., 1985)

Based on the size removal efficiency of unit operations and knowledge on the particle size distribution of a specific wastewater, the removal efficiency of a particular unit operation can be estimated. This is illustrated by the unit operations settling and membrane micro filtration as given in graph 1.6. The graph shows that regular pre-settling results in a removal of 30-40% for COD, while membrane microfiltration results in the removal of 70%.

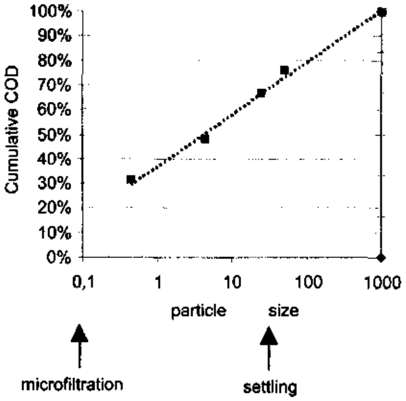


**Fig. 1.6:** The removal of COD is dependent on the particle size range of the applied unit operation for pretreatment

## Objectives and outline of the thesis

The investigation described in this dissertation was performed with the general objectives (i) to evaluate the concept of physical-chemical pretreatment for its potential to design more sustainable treatment wastewater systems as against treatment plants currently applied; and (ii) to contribute to further development of the concept of physical-chemical pretreatment by performing experimental research into encountered bottlenecks.

This thesis continues in chapter 2 with the description of a study on various treatment scenarios with physical-chemical pretreatment. The major conclusion of that study is that the application of pretreatment in combination with biological post treatment, improves the environmental sustainability of a municipal wastewater plant in terms of energy balance and also reduces the space requirement. However, two important bottlenecks were identified, being: (i) the use of metal-based flocculating agents leads to an increased sludge production and an increased salinity of the effluent; and secondly (ii) the particle removal particle is partially accompanied by the removal of biodegradable COD which is required later on at the denitrification step. Of course, without proper denitrification the nitrogen concentration of the effluent might exceed the effluent standards.

The experimental work described in chapter 3 to 6 addresses the bottlenecks as identified and described in chapter 2. The first bottleneck is dealt with in chapters 3 and 4, highlighting the use of organic poly electrolytes as alternatives for metal salts. Chapter 3 gives a description of a literature review as well as experiments exploring the application of these polymers. Then, chapter 4 deals with the combination of organic poly electrolytes and dissolved air flotation as potential pretreatment operation.

The second bottleneck (removal of biodegradable COD during pretreatment) is addressed in chapter 5 and 6. Investigations with regard to the implementation of a turbidity-related polymer dosing strategy in order to create different degrees of particle removal are described in chapter 5, which is followed in chapter 6 by a study on the effects that such different levels of particle removal might have on the elimination of nutrients in an activated sludge system.

Then, chapter 7 discusses an updated scenario study addressing the implications of the experimental research for several scenarios as described in chapter 2. Concluding this dissertation, chapter 8 summarizes the outcomes of the research and evaluates the potential of physical-chemical pretreatment for improvement of the sustainability of municipal wastewater treatment plants.

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## Appendix A Size distribution of contaminants in municipal wastewater

This appendix contains an average cumulative distribution of COD, BOD, N-Kjeldahl and $P_{total}$ over the different particle sizes of municipal wastewater samples of eight wastewater treatment plants. The presented data are the results of an investigation that was performed within the framework of the research project into physical-chemical pretreatment (see the dissertation by Van Nieuwenhuijzen, 2001 or STOWA, 2001). The data are shown to provide a more accurate picture with regard to the size distribution of contaminants in Dutch municipal wastewater.

The graph shows the average values of (grab) samples that were taken on three different days (at 9.00 h a.m) at eight different locations. The samples were mainly taken during dry weather conditions. After collection the samples were cooled and transported to the Laboratory of Sanitary Engineering of Delft University of Technology. The samples were fractionated over cleaned, pre-flushed sieves and membrane filters with five different pore sizes (63, 5, 1.2, 0.45 and 0.1 µm). The filtrates of these filtration steps were analyzed on COD, $BOD_5$, nitrogen, phosphorus and several other parameters.

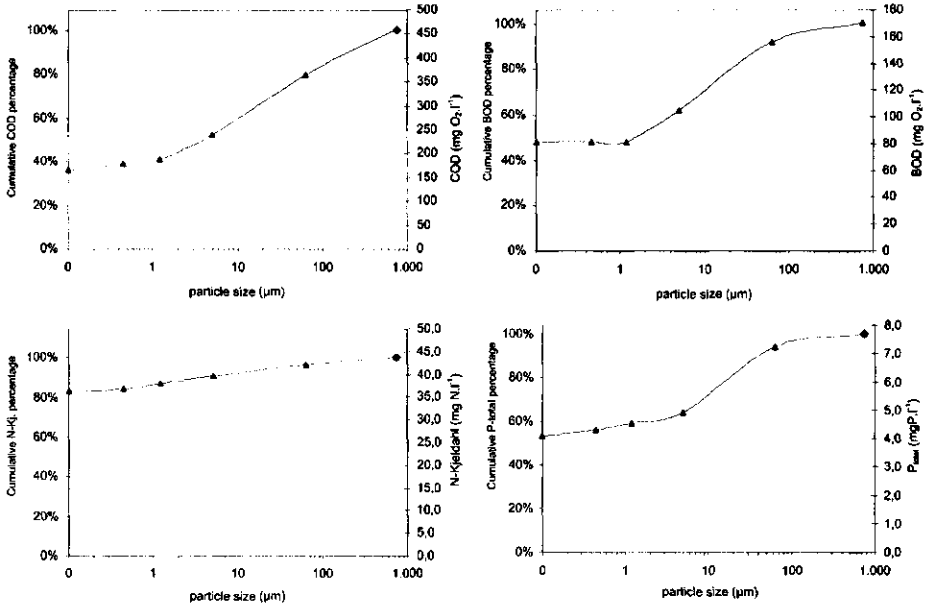

**Figure A:** Cumulative distribution of COD, BOD, N-Kjeldahl and $P_{total}$ over the different particle sizes; the shown values are the average results of wastewater samples of eight wastewater treatment plants (Van Nieuwenhuijzen, 2001; STOWA, 2001)

# 2 Identification and evaluation of wastewater treatment scenarios with physical-chemical pretreatment

**Abstract.** The objective of this chapter was to investigate the concept of physical-chemical pretreatment for its potential to design more sustainable wastewater treatment systems. Based on a review of unit operations, ten different scenarios for wastewater treatment were designed and evaluated along environmental and financial criteria. Within the evaluation special attention was paid to the energy balance, the final sludge production, the potential application of nitrogen recovery techniques, the expected effluent quality, the requirement of land and the economic feasibility of the identified scenarios.

The evaluation of scenarios shows that wastewater treatment systems with physical-chemical pretreatment can potentially be designed more energy efficient and smaller than the commonly applied treatment systems. The total cost of treatment scenarios, which combine physical-chemical pretreatment with biological post treatment, appeared to be in the same range as that of the reference scenario. The costs of scenarios that include ion-exchange and a stripper to recover nitrogen are at least 75% higher than the reference scenario (the possible profits of the re-usable product have not been part of the calculations) and result in a high energy requirement. Scenarios that include activated carbon adsorption for dissolved COD removal result in 300% higher total costs.

The scenario study revealed two important bottlenecks regarding the application of physical-chemical pretreatment, being (i) the partial removal of biodegradable carbon which is required for denitrification in a biological post treatment step and (ii) the use of metal-based flocculating agents. The use of metal salts contributes considerably to the overall environmental impact of the evaluated treatment systems, due to a significant amount of inorganic (chemical) sludge and the resulting increased salinity of the effluent.

## Introduction

Physical-chemical pretreatment is meant to separate suspended and colloidal particles from wastewater. Particle separation can form an important first step of a wastewater treatment system, since a major part of the influent COD consists of particulate material. The literature review in chapter 1 showed that approximately 70% of COD in wastewater consists of particles ≥ 0.45 µm and that many micro pollutants are adsorbed to particulate material (e.g. > 90% of most heavy metals). By applying an enhanced primary treatment step these particles can be separated. The result is the production of (more) primary sludge that can be re-used for energy recovery through digestion. The pre-treated wastewater contains low concentrations of pollutants, so a down-stream treatment step can be designed smaller. Moreover, due tot the fact that the primary effluent is almost free of particles, alternative treatment techniques can be applied, including physical-chemical techniques that are suitable to recover nutrients.

The objective of the scenario study that is described in this chapter is to investigate the concept of physical-chemical pretreatment for its potential to design more sustainable wastewater treatment systems. Based on a review of unit operations ten different scenarios for wastewater treatment are designed and evaluated along environmental and financial criteria. In the evaluation special attention is paid to the energy balance, the final sludge production, the potential application of nitrogen recovery techniques, the expected effluent quality, the requirement of land and the economic feasibility of the identified scenarios.

The present chapter consists of two parts. In the first part a review of unit operations for pre- and post treatment is made. In the second part scenarios for wastewater treatment are designed and evaluated, consisting of a selection of the reviewed unit operations.

### Review of unit operations for pre- and post treatment

Table 2.1 shows a brief overview of unit operations that were pinpointed and studied in the inventory part of the research. The identified unit processes serve as basic elements for the formation of the complete wastewater treatment scenarios, later in the chapter. The unit operations that are indicated by a dot (•) are used in the scenario study.

**Table 2.1**: Overview of reviewed wastewater treatment unit operations

| **Physical-chemical pretreatment**<br>(removal of particulate material) | **Secondary treatment**<br>(removal of soluble COD and(+) or(/) N) | **Tertiary treatment**<br>(final particle removal) |
|---|---|---|
| Primary settling ● | Activated sludge systems (COD/+N) ● | Rapid sand filtration ● |
| Pre-precipitation ● | Biofilm systems (COD/+N) ● | |
| Flotation (DAF) ● | Membrane bio-reactor (COD/+N) | |
| Micro screening | Ion-exchange (N) ● | |
| Direct wastewater filtration | Activated carbon adsorption (COD) ● | |
| Enhanced settling with magnetite | | |
| Enhanced settling with micro sand | | |
| Direct membrane filtration | | |

● - used in the scenario study

## *Unit operations for pretreatment*

**Primary settling and pre-precipitation** - Primary settling is a frequently applied unit operation for pretreatment. In primary settling, particle separation is achieved by sedimentation. The settling velocity of particles is dependant on the particle diameter and density, as well as on the temperature of the surrounding water. This is illustrated by table 2.2. A flocculating agent can be added to the wastewater prior to settling in order to enlarge the size of smaller, not-settleable particles. The foregoing application is called pre-precipitation.

In practice, for primary clarifiers hydraulic surface loads of 1.5-3.0 m.h$^{-1}$ are applied (Metcalf&Eddy, 1991; Van der Graaf, 1995; STOWA, 1996). The average dry solids concentration of sludge that is discharged by primary clarifiers is 0.5-1.0%.

**Table 2.2**: Settling velocities of particles at T = 10°C

| **Particle diameter (μm)** | **1000** | **500** | **200** | **100** | **50** | **10** | **5** | |
|---|---|---|---|---|---|---|---|---|
| Quarts sand<br>(density = 2,650 kg.m$^{-3}$) | 502 | 258 | 82 | 24 | 6.1 | 0.3 | 0.06 | m.h$^{-1}$ |
| *Particles in municipal wastewater*<br>(density = 1,200 kg.m$^{-3}$) | 122 | 61 | 18 | 3 | 0.76 | 0.03 | 0.008 | m.h$^{-1}$ |

source: Van der Graaf, 1995

**Flotation** - The flotation process is used to separate solid or liquid material by the addition of fine gas bubbles to the carrier liquid. The bubbles are meant to attach to the solids or the emulsified liquid material and push them to the surface of the carrier liquid. The addition of bubbles can be achieved in several ways, by mechanical flotation, pressurized air flotation, vacuum flotation, electroflotation and dissolved air flotation. Among these techniques dissolved air flotation (DAF) is applied most frequently .

In a DAF system a part of the effluent of the DAF unit is recirculated (10-20%) and pressurized (4-6 bar). Due to the high pressure high concentrations of air can be dissolved into the liquid. When the return flow is mixed with the influent (fig. 2.1) the pressure is released, resulting the formation of small air bubbles (diameter = 0.05 - 0.1 mm). The floated material is removed by a scraper that moves in the opposite direction of the wastewater flow. This floated sludge may have dry solids concentrations that are relatively high, being up to 2-4% according to Krofta and Burgess (1997).

Flotation is alway applied in combination with the addition of flocculating agents. These flocculants are provided to create a surface or structure that can entrap or attach gas bubbles. The hydraulic surface loadings that are applied range from 2.4 to 11.0 $m.h^{-1}$ (Haarhoff and Van Vuuren, 1993).

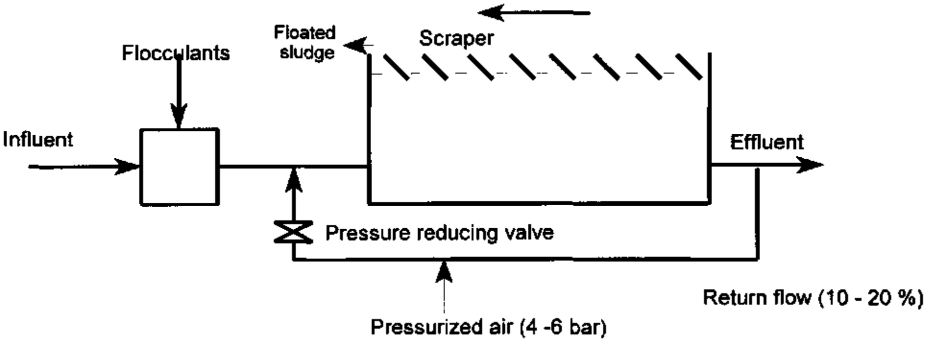

**Fig. 2.1:** Dissolved air flotation

**Micro screening** - A third potential unit operation for pretreatment is micro screening. Micro screening usually is applied in slowly rotating drum screens. The wastewater enters such a drum screen in the inner part and is forced through a perforated wall. The screened particles remain inside the drum and move towards the end of the drum by the rotation, where they are removed. The diameter of the drum screen holes is generally 25-50 µm, *although for some applications very small diameters of 6 - 35 µm are reported* (Metcalf&Eddy, 1991).

Drum screens are applied frequently for the treatment of wastewater of slaughter houses and meat processing industries. Occasionally, drum screens are used to remove suspended solids from the effluent of activated sludge systems. The volume of back wash water generally reaches 5-10% of the average daily flow and contains dry solids concentrations of 0.2-0.4%. Drum screens are normally operated without the addition of flocculating agents. Considering the reported hole diameters of 6 - 35 µm, the removal efficiencies for municipal wastewater are expected to be in the same range of those of primary settling. It is supposed that the efficiency of the system will increase by the addition of flocculants, although in literature no applications of such a configuration were found.

**Direct influent filtration** - A pretreatment process that has been under investigation in recent years is direct filtration of municipal wastewater (Mouri and Niwa, 1993; Tanaka *et al.*, 1995; Rusten *et al.*, 1997; Elmitwalli, 2000; Van Nieuwenhuijzen *et al.*, 2000). This type of filtration generally makes use of coarse, porous filter media. The filter removes particles by a combination of settling, collision to the filter medium and biological degradation by bacteria that develop on the filter medium.

The reported filtration experiments were generally conducted in columns of 2-4 m and were mainly operated in up-flow mode. The applied hydraulic surface loads were 5-40 $m.h^{-1}$. The back wash frequency varied and numbered every 5-10 h for high hydraulic loadings and 55 h to 1 week for low hydraulic loadings.

**Enhanced settling with magnetite (SIROFLOC™)** - In this type of unit operation, particles are flocculated with magnetite particles. The wastewater with the magnetite containing flocs is placed into a magnetic field. Due to the electro magnetic field forces the magnetite flocs conglomerate to large, well settling flocs. The flocs are separated in a clarifier.

The Australian SIROFLOC™ process is an example of this unit operation (Booker and Brooks, 1994; Booker *et al.*, 1994). Figure 2.2 presents a process scheme. With regard to the applied clarifier, hydraulic surface loads up to 10 $m.h^{-1}$ are reported. Subsequent to the settling, the sludge is discharged to a drumscreen in order to recover the magnetite particles. A drawback of the SIROFLOC™ process is the relative complex configuration and the use of high quantities of base.

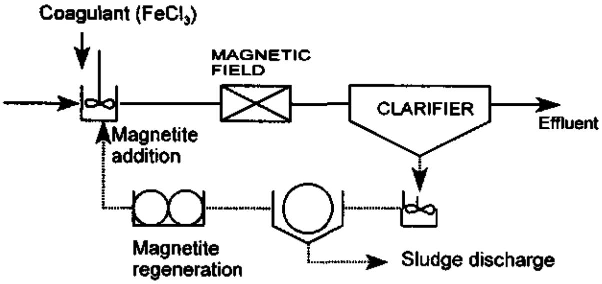

**Fig. 2.2:** Process scheme of the SIROFLOC™ system

**Enhanced settling with micro sand (Actiflo™)** - This unit operation is based on the addition of micro sand during the coagulation/flocculation stage. The sand addition is aimed to improve the settling properties of the formed sludge flocs. The process was originally applied in the preparation of drinking water, but is now increasingly implemented for the treatment of sewage water overflows and for pretreatment of municipal wastewater (the latter especially during storm water events) (Le Poder and Binot, 1995; Gousailles *et al.*, 1997; Plum *et al.*, 1998). The process is brought on the market as Actiflo™.

Figure 2.3 shows a process scheme of the process. A coagulant (e.g. $FeCl_3$) is added in a first stage. During the succeeding flocculation stage, micro sand with a diameter of 100 - 150 µm is added simultaneously with an anionic organic polymer. The incorporation of micro sand increases the settling velocity of the flocs. A clarifier is used to remove these flocs. In the Actiflo™ configuration in most cases a lamella clarifier is used. A hydrocyclone separates the micro sand from the sludge and returns it to the process.

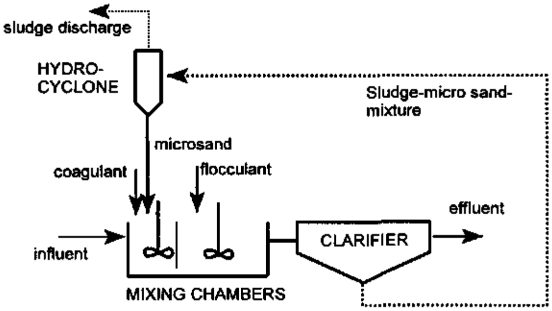

**Fig. 2.3**: Process scheme of the Actiflo™ system

**Direct membrane filtration** - Membrane filtration is used to separate pollutants from a carrier liquid by forcing the liquid through a permeable or semi permeable membrane. The process removes specific pollutants, dependent on the pore size of the membrane. A wide variety of pore sizes is applied, ranging from $10^{-3}$-$10^{-4}$ µm for reversed osmosis to $10^{-1}$-$10^{0}$ for microfiltration. The transmembrane pressure that has to be applied is dependent on pore size, salt concentration (osmotic pressure) and the fouling of the membrane. Literature sources report transmembrane pressures in a wide range of 0.1 to 80 bars, dependent on the membrane type and the application. In recent years there is a growing interest for the use of membranes. Because of the development of new membranes the process is fit for more applications, while at the same time the prices have gone down.

A possible new application is (micro- or ultra-) membrane filtration as a first step in municipal wastewater treatment. Experiments by Van Nieuwenhuijzen *et al.* (2000) showed the potential of this unit operation. During a four months trial, pilot-scale experiments with ultra filtration membranes (pore size 0.03 µm) were performed. Without backflushing it appeared possible to keep the flux at a constant level, suggesting only limited fouling of the membranes. At transmembrane pressures of 0.2-0.4 bar a normalized production flux of 300 $l.m^2.h^{-1}$ was obtained.

**Summary of removal efficiencies** - Table 2.3 presents a compilation of the removal efficiencies of the reviewed unit operations for pretreatment that were reported in the previous paragraphs. The removals are expressed as total suspended solids (TSS), COD, BOD, $P_{total}$ and N-Kjeldahl.

**Table 2.3:** Literature data on removal efficiencies of the reviewed unit operations for pretreatment

| | **Pre-settling** | **Pre-precipitation** | **Flotation** | **Micro-screen**[a] | **DWF**[b] | **SIRO-FLOC™** | **Actiflo™** | **DMF**[b] |
|---|---|---|---|---|---|---|---|---|
| TSS | 30-40% | 60-90% | 80-90% | 60-90% | 70-90% | 86% | 75-90% | 100% |
| COD | 20-30% | 40-60% | 50-60% | 40-60% | | 56% | 55-65% | 69% |
| BOD | 20-30% | 40-60% | | 40-60% | 40-50% | 55% | | |
| $P_{total}$ | 10-20% | 60-80% | | 60-80% | | 88% | 50-95% | 17% |
| N-Kj. | 5-10% | 20-30% | | 20-30% | | | 10-20% | 36% |

[a] These removal efficiencies are estimations and suppose the addition of flocculating agents

[b] DWF - direct wastewater filtration (coarse media); DMF - direct membrane filtration

**The importance of coagulation/flocculation** - The review shows that coagulation/flocculation is essential in the process of particle removal. All the identified pretreatment steps require flocculating agents to obtain a significant level of particle removal (i.e. >75% TSS; see table 2.3), except for direct influent filtration and direct membrane filtration. This requirement results from the fact that most of the reviewed pretreatment steps are only removing larger particles. Flocculating agents will have to be added to increase the average particle size. This particle enlargement shifts the average size distribution to a higher value, as is illustrated in figure 2.4. The higher percentage of larger particles increases the removal efficiency of most of the reviewed unit operations.

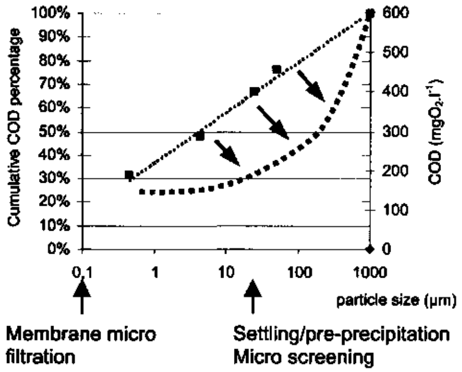

**Fig. 2.4:** Shift in the average size distribution, due to the addition of flocculating agents (the dashed curve is illustrative and shows no measured values)

Metal-based flocculating agents are dosed at many wastewater treatment plants in The Netherlands. The dosage of these chemicals has a twofold objective, (i) to precipitate phosphate and (ii) particle enlargement to enhance solids removal. The latter is relevant if the chemicals are used prior to the primary clarifier and is generally of less importance than the phosphate precipitation. At most treatment plants a ferric-based salt ($Fe^{3+}$) is used.

The STOWA report 'Variants on pre-settling' (STOWA, 1996) describes a review on the application of metal-based flocculating agents for a number of treatment plants. This review showed that the average applied dose amounted 15 $gFe.m^{-3}$, which is equal to a Fe/P ratio of 0.9 mol Fe/mol P. It should be noticed that this dosage refers to systems that were combined with biological post treatment (activated sludge) and that the pre-treated wastewater still contained 2-3 $gP.m^{-3}$ in order to provide phosphorus for bacterial growth. If phosphate has to be removed to a concentration $< 1$ $gP.m^{-3}$, which is the case if physical-chemical unit operations are applied for post treatment, approximately 1.6 mol Fe/mol P will have to be provided (Metcalf&Eddy, 1991).

The Fe-addition is occasionally complemented with the addition of an anionic polymer. This anionic polymer is applied to achieve floc enlargement by *bridging flocculation* (see chapter 3 of this dissertation). The reported values for anionic polymer dosage according to the mentioned STOWA report vary from 0.1 to 1.5 g polymer per $m^3$.

### *Unit operations for secondary and tertiary treatment*

Subsequent to pretreatment one or more additional unit operations will have to be applied in order to remove remaining BOD, nitrogen an phosphorus. With regard to secondary and tertiary treatment, different unit operations can be applied, consisting of techiques based on biological pollutant removal (activated sludge systems, biofilm systems, membrane bioreactors) or techniques based on physical-chemical pollutant removal (ion-exchange, activated carbon). The current paragraph contains a review of different unit operations for post treatment. The application of physical-chemical unit operations subsequent to enhanced primary treatment appears attractive, because the primary effluent contains low concentrations of particles.

**Activated sludge systems (BOD and (optional) N and P removal)** - As was already mentioned in chapter 1, activated sludge systems are among the most applied treatment systems for municipal wastewater. Activated sludge systems are aimed at degradation of organic matter ("BOD") and - dependent on the configuration - nutrient removal by suspended bacterial flocs. After treatment the bacterial flocs are removed in a final clarifier and for a larger part returned to the beginning of the plant. A smaller part of the bacterial mass is disposed as waste sludge.

With respect to activated sludge systems, a general distinction is made in high loaded and low loaded systems. High loaded systems are designed for BOD removal and in most cases nitrification. Low loaded activated sludge systems are aimed at nitrogen removal by nitrification/denitrification. In both types of systems biological phosphorus can be incorporated by introducing an anaerobic zone at the beginning of the installation. By introducing such a zone conditions are created that are favourable to phosphate accumulating bacteria.

A typical design value for high loaded systems is 0.8 $kgBOD.kgSS^{-1}.day^{-1}$ (SS = suspended solids). For low loaded systems 0.06 $kgBOD.kgSS^{-1}.day^{-1}$ is a common value. The required aeration capacity is determined by the BOD and the nitrogen concentration of the influent. The sludge production can be calculated based on the bacterial yield. For high loaded systems a yield of 0.6 $kgSS.kgCOD_{removed}^{-1}$ is reported. For low loaded systems a typical value is 0.4-0.5 $kgSS.kgCOD_{removed}^{-1}$ (Metcalf&Eddy, 1991; Van der Graaf, 1995; Klapwijk *et al.*, 1997).

**Biofilm systems (BOD and (optional) N removal)** - Although biofilm systems are already existing for quite some time - with as the most prominent example the trickling filter -, many research efforts have been undertaken during recent years to develop new configurations. The most important examples are the *Biofor* and the *Biostyr* fixed bed systems and the *Kaldness* moving bed system.

Biofor is an upflow system containing expanded clay granules. These expanded clay granules are porous, gravel-like media that serve as biofilm carriers. Because of their porous structure a large part of the biofilm is growing inside the gravel particles. Different gravel diameters are used, depending on application: 3.0-3.7 mm for BOD removal and denitrification and 2.0-2.5 mm for nitrification (RIZA and STOWA, 1994; Æsøy *et al.*, 1998). The filter medium of Biostyr reactors consists of polystyrene balls with a diameter of 3.0-3.5 mm. The polystyrene balls float in the reactor, due to their low density (RIZA and STOWA, 1994; Borregaard, 1997; Falkentoft *et al.*, 1999). A typical bedheight of fixed bed biofilm systems is 3 m. In Kaldness Moving Bed Biofilm Reactors (MBBR) biomass is attached to small plastic elements that move freely along with the water in the reactors (Rusten et al., 1997). The carrier elements have a specific surface of 300-350 $m^2$. MBBR's have the advantage that the system is relatively easy to handle, because it can be operated without clogging.

Similar to activated sludge systems for biofilms systems a general distinction can be made into systems that are designed for BOD removal only and systems that also include nitrogen removal by nitrification/denitrification. For low loaded biofilm systems volume loads of 0.08-0.32 $kgBOD.m^{-3}.day^{-1}$ are found. For high loaded systems volume loads of 0.3-1.4 $kgBOD.m^{-3}.day^{-1}$ are reported (Metcalf&Eddy, 1991; Henze *et al.*, 1995).

The low loaded biofilm systems that were used in the scenario study were designed with a volume load of 0.2 $kgBOD.m^{-3}.day^{-1}$. The assumed sludge yield factor is 0.2 $kgSS.kgCOD_{removed}^{-1}$ (no proper literature values were found). In this system forced aeration is applied. The energy consumption for aeration is calculated based on the BOD and nitrogen removal. For the high loaded biofilm systems a volume load of 0.2 $kgBOD.m^{-3}.day^{-1}$ and a sludge yield factor of 0.6 $kgSS.kgCOD_{removed}^{-1}$ is taken. The design is based on a trickling filter.

The effluent of both systems is treated in a rapid sand filter in order to remove the eroded biofilm particles. The application of a rapid filter is possible because the suspended solids concentrations in the effluent of biofilm systems are relatively low (50-100 $mgTSS.l^{-1}$).

**Ion exchange (ammonium removal and nitrogen recovery)** - Ion exchange can be used to remove ammonium from the pretreated influent. In this unit operation ammonium ions are exchanged for ions that are bound to an ion exchanger by electrical charge forces. Various ion exchanging 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 is hardly temperature dependant and has a fast reaction rate (< 2 minutes). Moreover, ion exchange offers the possibility of a complete ammonium removal (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. The regeneration requires a salt solution to remove the adsorbed ammonium. A typical concentration of such a salt solution is 1.6 $eq.l^{-1}$ (STOWA, 1997), which equals to 100 g $NaCl.l^{-1}$. Because the regenerate is a mixture of ammonium ions and regeneration salt, it is not re-usable as such.

To recover the ammonium and to avoid the production of large volumes of regenerate waste, the released ammonium can be stripped from the regenerate solution and subsequently be precipitated. This recovery technique can be operated in parallel with the regeneration of the ion exchanger. An example of this ammonium removal and recovery process is operated at the Tahoe-Truckee municipal wastewater treatment plant in the USA (Den Engelse and Van der Panne, 1992; STOWA, 2001). Figure 2.5 shows a scheme of this process configuration. The configuration consists of a column filled with ion exchanger (I), a mixing chamber to which NaOH is dosed (M), an air strip column (A) and a gas washer (G). During the regeneration mode, the regenerate is recirculated over the air strip column. The pH of the regenerate is raised by the addition of sodium hydroxide, resulting in the formation of ammonia. The ammonia is stripped at 50 °C. The stripper gas is washed in a gas washer in acidic conditions. The final product consists of an ammonium salt (e.g. ammonium sulphate or struvite) or an ammonia solution. Both products can be re-used as agricultural fertilizer. The scenario study later in this chapter assumes that ammonium sulphate is produced.

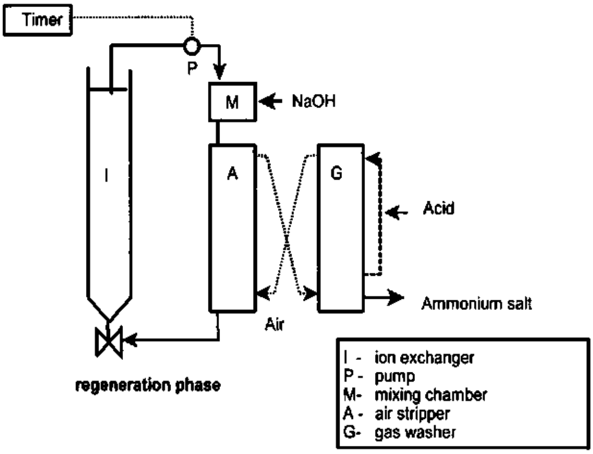

**Fig. 2.5:** Ion exchanger with regenerate treatment by air stripping at 50 °C during the regeneration phase (based on STOWA, 1995)

**Sorption to activated carbon (dissolved COD removal)** - Activated carbon adsorption is a frequently used unit operation in drinking water purification. It is applied to remove heavy metals and organic micropollutants. It is also proposed as a method to remove micro pollutants form the effluent of activated sludge systems (Meinema and Rienks, 1995).

Activated carbon filters for drinking water treatment are normally operated in downflow mode with filtration rates of 5 m.h$^{-1}$ (Van Dijk and Aeyelts Averink, 1995). The carbon adsorption capacity is dependent on the type of pollutant. No literature data were found with regard to the carbon adsorption capacity of COD (which is in fact composed of a wide variety of organic pollutants). However, a capacity of 0.3-0.5 kg COD per kg activated carbon was estimated (Norit-Netherlands, 1997). A typical density for activated carbon is 1,250 kg.m$^{-3}$. The saturated activated carbon is generally regenerated thermically, at an approximate temperature of 700 °C.

**Rapid sand filtration (final particle removal)** - Rapid sand filtration is frequently applied as an effluent polishing step subsequent to activated sludge systems in Germany, Switserland and the USA (TWFF, 1986). Rapid sand filters usually consist of double layered filter beds with a bottom layer of 0.4 m quarts sand (diameter 0.7-1.5 mm) and a top layer of 0.8 m anthracite (diameter 1.7-2.8 mm) (fig. 2.6). The application of two (or more) filter materials is advantageous, because the penetrative effect of the filter is more effectively used. Larger particles are captured in the top layer of the filterbed, while smaller particle are captured in the bottom layer. Rapid sand filtration can be combined with the addition of precipitants (ferric or aluminium salts). Due to flocculation and precipitation the particle and phosphate removal efficiency of the filter will improve (Van der Graaf and Van Nieuwenhuijzen, 1998). Rapid sand filters are generally operated in downflow mode at filtration rates of 5-30 m.h$^{-1}$.

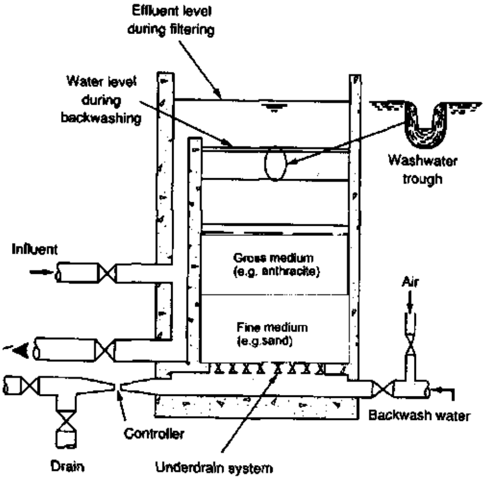

**Fig. 2.6**: Scheme of a double layered rapid sand filter (STOWA, 1998)

## Methodology applied in the scenario study

**Starting conditions** - All wastewater treatment scenarios were designed for an imaginary wastewater treatment plant (wwtp) of 100,000 population equivalents (p.e.). The hydraulic parameters are shown in table 2.4.

**Table 2.4:** Hydraulic loading rates for a 100,000 p.e. wwtp

| | | |
|---|---|---|
| Flow per p.e. | 150 l.p.e. $^{-1}$.day$^{-1}$ | |
| Dry Weather Flow ($Q_{DW}$) | 1,000 m$^3$.h$^{-1}$ | (supply during 15 hours per day) |
| Storm Weather Flow ($Q_{SW}$) | 3,000 m$^3$.h$^{-1}$ | |
| Average flow ($Q_{Av}$) | 19,500 m$^3$.day$^{-1}$ | (1.3 × DWF × 15 h) |

The influent values regarding COD, BOD$_5$, N-Kjeldahl, $P_{total}$ and suspended solids were based on average concentrations in Dutch wastewater that are yearly collected by the Central Bureau Statistics (CBS, 2000). As a prerequisite for the design, the effluent concentrations had to at least comply with the Dutch effluent constraints. The assumed influent composition as well as the effluent constraints are shown in table 2.5.

**Table 2.5:** Assumed influent concentration and effluent constraints

| | | Influent composition | Effluent constraints |
|---|---|---|---|
| COD | (mg $O_2$.l$^{-1}$) | 600 | < 50.0 |
| BOD$_5$ | (mg $O_2$.l$^{-1}$) | 220 | < 10.0 |
| N-Kjeldahl | (mg N.l$^{-1}$) | 55 | < 10.0 ($N_{total}$) |
| $P_{total}$ | (mg P.l$^{-1}$) | 9 | < 1.0 |
| Total suspended solids | (mg TSS.l$^{-1}$) | 250 | < 10.0 |

**Evaluation of the environmental sustainability** - The environmental sustainability of the identified scenarios was evaluated by means of the list of environmental interventions. The criteria of this list are based on earlier publications adressing the the environmental sustainability of wastewater treatment plants (chapter 1). Table 2.6 shows an overview of the assessed criteria. As already mentioned in chapter 1, the assessment of the space requirement is considered from a practical point of view. It can especially be of value in densely populated areas.

**Table 2.6:** List of criteria that were assessed regarding the sustainable technological development of the newly designed treatment plants (see also chapter 1)

| **Environmental interventions** | **Remarks** |
|---|---|
| Energy balance | Result of energy demand (in treatment processes) and energy recovery (from the sludge digestion). |
| Final sludge production | Sludge production after the on-site sludge handling procedure (thickening, digestion and dewatering). |
| Effluent quality | The effluent quality at least has to comply with the Dutch discharge constraints; an improvement in effluent quality regarding nutrients, micro pollutants and suspended solids is taken into consideration. |
| Use of chemicals/recovery of nitrogen | Chemicals that are used in the treatment processes, cq. nitrogen that is recovered. |
| Space requirement | The space requirement (footprint) of the designed treatment systems refers to the actual size of the installations (offices etc. are not included). |

**Cost calculations** - Supplemental to the criteria of table 2.6, the estimated costs of each of them were calculated by means of the net present value method. The net present value (NPV) reflects all aggregated costs over the total expected life time of the treatment plant (De Mol van Otterlo, 1977). The net present value therefore, includes all expected costs of investment (i.e. investment cost of civil constructions of installations and electro-mechanical works[1]) as well as all expected operational costs during the life time of the plant. Within the net present value method, the calculations are at prices at the moment of investment taking into account a constant rate of interest and a constant rate of inflation for the time of operation. Equation 2.1 shows the methodology for calculating the net present value of investment.

$$NPV_{Investment} = \sum_{t=1}^{T} \left(\frac{1}{1+i}\right)^{t-1} \bullet [D + i_t] \qquad (2.1)$$

where $NPV_{Investment}$ = net present value taking into account the annual cost of depreciation and interest over the total life span (in €)
$i$ = interest rate (%)
$t$ = number of years (-), t = 1, ..., T
$D$ = yearly depreciation (in €)
$i_t$ = interest of remaining investment (in €)
$T$ = life span of investment in number of years (-)

---

[1] Note: the cost for purchase of land is not included in the cost calculation, since it depends on the local availability; it is assessed separately as shown in table 2.6.

In case of the construction of a wastewater treatment plant, the net present value includes the investment costs for civil constructions ($I_C$) which are supposed to have a life time of 30 years, as well as costs for electro-mechanic works ($I_{EM}$) which are supposed to have a life time of 15 years. This means that the total depreciation period of the treatment plant is 30 year, whereas after 15 years a reinvestment of electro-mechanical works is due.

Assuming linear depreciation and an annual long term interest rate of 5.5%, the net present value of the civil construction investments ($NPV_C$) per scenario is calculated following the equation:

$$\mathrm{NPV_C} = \sum_{t=1}^{30} \left(\frac{1}{1.055}\right)^{t-1} \bullet \left[\frac{I_C}{30} + 0.055\left(I_C - \frac{t}{30} I_C\right)\right] \qquad \text{(in €)} \qquad (2.2)$$

with $I_C$ = total investment in civil constructions at t = 1 (in €)

The calculation of the net present value of electro-mechanical investment costs ($NPV_{EM}$) follows a similar equation, however taking into account a depreciation period of 15 years of the initial electro-mechanical investment ($I_{EM}$) as well as a reinvestment ($I_{EM}^{*}$) after 15 years. Again an annual interest rate of 5.5% and linear depreciation was assumed, while with respect to the reinvestment an annual inflation rate of 2.5% was taken into account.

$$\mathrm{NPV_{EM}} = \sum_{t=1}^{15} \left(\frac{1}{1.055}\right)^{t-1} \left[\frac{I_{EM}}{15} + 0.055\left(I_{EM} - \frac{t}{15} I_{EM}\right)\right] + \sum_{t=16}^{30} \left(\frac{1}{1.055}\right)^{t-1} \left[\frac{I_{EM}{}^{*}}{15} + 0.055\left(I_{EM}{}^{*} - \frac{t-15}{15} I_{EM}{}^{*}\right)\right]$$

with

$$I_{EM}^{*} = I_{EM} \cdot (1.025)^{15} \qquad \text{(in €)} \qquad (2.3)$$

The net present value of the operational costs reflects the total of yearly costs for maintenance ($C_M$), energy ($C_E$), chemicals ($C_C$), sludge handling ($C_S$) and labour ($C_L$) at prices of the time of plant construction. With again a life span of 30 years and the same rates of interest and inflation, the net present value of the operational costs ($NPV_{OC}$) are calculated as follows:

$$\mathrm{NPV_{OC}} = \sum_{t=1}^{30} (C_M + C_E + C_C + C_S + C_L) \cdot \frac{1.025^t}{1.055^t} \qquad \text{(in €)} \qquad (2.4)$$

See appendix A for the cost assumptions applied in the financial calculations.

**Design and evaluation model** - For the design and evaluation of the wastewater treatment scenarios a model has been developed entitled DEMAS+ (*Dimensioning and Evaluation Model for Wastewater Treatment Scenarios*) (Van Nieuwenhuijzen *et al.*, 1998; Mels *et al.*, 1999). The model combines the necessary technical and financial information concerning the various unit operations (design parameters, energy consumption, treatment efficiencies, unit operation costs, etc.). The DEMAS+ model gives calculations of the environmental interventions and the total net present value per scenario including various combinations of unit operations.

**System boundaries and standard sludge handling procedure** - The evaluation was conducted for the whole on-site treatment system. The system boundaries are shown in figure 2.7. Within DEMAS+ the identified wastewater treatment scenarios are combined with a standard sludge treatment. This standard sludge handling procedure consists of (gravitational) thickening, followed by sludge digestion. Subsequent to digestion, the sludge is dewatered by a centrifuge. After centrugation, the final sludge is transported to a central dewatering and incineration plant. See appendix B for the assumed design criteria of the standard sludge handling procedure.

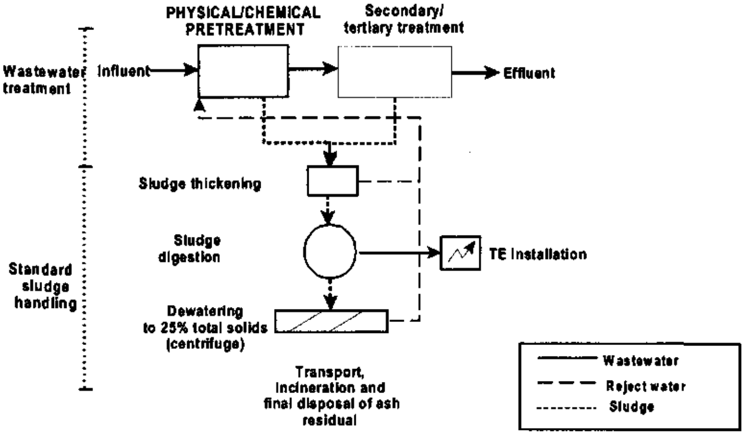

**Fig.** 2.7: System boundaries applied in DEMAS+

**Reference scenario** - A reference scenario was designed to compare the identified scenarios with existing treatment systems. This reference scenario consisted of a low loaded activated sludge system for BOD, nitrogen and phosphorus removal with a primary and a secondary clarifier (see figure 2.8). The primary clarifier was designed with a hydraulic surface loading of 3 m.h$^{-1}$ (at storm weather flow - $Q_{SW}$). The sludge loading rate of the activated sludge system was set at 0.06 kg BOD.kgSS$^{-1}$.day$^{-1}$ and a sludge concentration of 4 kgSS.m$^{-3}$. An anaerobic zone was planned to create conditions feasible for biological phosphorous removal (hydraulic retention time at dry weather flow: 2 h). The hydraulic surface loading (of $Q_{SW}$) of the secondary clarifier was 0.7 m.h$^{-1}$. The primary and secondary (biological) sludge was combined and treated by the standard sludge handling procedure.

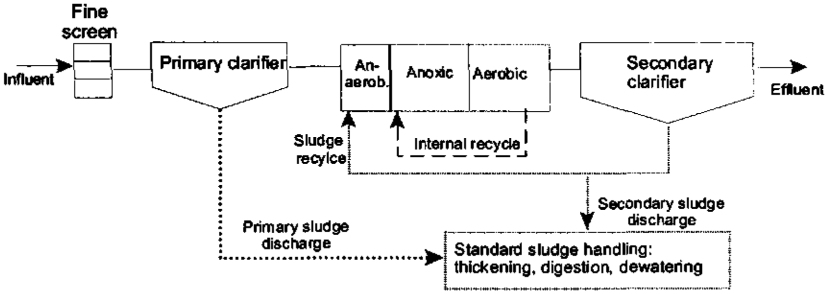

Fig. 2.8: Reference scenario: fine screen + primary clarifier + low loaded activated sludge system + secondary clarfier

**Design criteria of the unit operations -** Based on the literature review design criteria were compiled with regard to the various unit operations. Table 2.7 shows the design criteria as far as the pretreatment processes are concerned. The design criteria of the secondary treatment steps are given in appendix C.

**Table 2.7:** Design criteria of the pretreatment steps

| **Coagulation/flocculation** | |
|---|---|
| Flocculator, rapid coagulation mixing | |
| Hydraulic retention time | 15 s (at $Q_{SW}$) |
| Heigth | 1 m |
| Velocity gradient stirrers | 1,000 s$^{-1}$ |
| Flocculator, slow flocculation mixing | |
| Hydraulic retention time | 120 s (at $Q_{SW}$) |
| Heigth | 2 m |
| Velocity gradient stirrers | 45 s$^{-1}$ |
| Dose of flocculating agents | |
| combined with biological post treatment | 15 g.m$^{-3}$ $Fe^{3+}$+ 1.0 g.m$^{-3}$ anionic polymer |
| combined with physical-chemical post treatment | 25 g.m$^{-3}$ $Fe^{3+}$+ 1.0 g.m$^{-3}$ anionic polymer |

| **Physical-chemical pretreatment** | |
|---|---|
| Precipitation tank | |
| Hydraulic surface loading | 3 m.h$^{-1}$ (at $Q_{SW}$) |
| Tank heigth | 2 m |
| Flotation tank (DAF) | |
| Hydraulic surface loading | 10 m.h$^{-1}$ (at $Q_{SW}$) |
| Tank heigth | 2.5 m |
| Pressure recycle flow | 5 bar |
| Recycle flow | 15% of $Q_{DWF}$ |
| **Additional treatment** | |
| Rapid sand filtration | |
| Surface load | 15 m.h$^{-1}$ (at $Q_{SW}$) |
| Bed height | 1 m |
| Column diameter | 4.1 m |
| Back wash frequency | 1 time per day |
| Back wash time | 900 s |

**Effluent composition of the pretreatment operations** - Table 2.8 shows the influent and the assumed effluent composition after pretreatment. The effluent values of the pretreatment steps determine the pollutant load of the secondary treatment. The primary effluent characteristics are based on the particle removal efficiencies as were found in the literature review of chapter 1, taking into account the influent particulate fraction. It is assumed that 70% of the influent COD, 60% of BOD, 25% of N-Kjeldahl and 30% of $P_{total}$ occur in particulate form (> 0.45 μm). With regard to the particle removal efficiency ($R_{particles}$) of primary settling and pre-precipitation, cq. flotation 40% and 80% are assumed, respectively.

The table also includes the BOD/N ratio. As shown, the BOD/N ratio of the influent is 4.0. Enhanced pretreatment results in a decrease of the BOD/N ratio, because the particulate nitrogen fraction is relatively smaller than that of BOD. The ratio becomes 3.3 subsequent to regular pre-settling. The calculated BOD/N ratio of the primary effluent of pre-precipitation and flotation is 2.6.

**Table 2.8:** Influent and assumed effluent composition of the pretreatment operations (without extra nitrogen added by the reject water)

| **Parameter** | **Influent** | **Effluent composition after pretreatment** | |
|---|---|---|---|
| | | Primary settling ($R_{particles}$ = 40%) | Pre-precipitation Flotation ($R_{particles}$ = 80%) |
| COD total (mg $O_2.l^{-1}$) | 600 | 432 | 264 |
| particulate | 420 | 252 | 84 |
| dissolved | 180 | 180 | 180 |
| BOD (mg $O_2.l^{-1}$) | 220 | 167 | 114 |
| particulate | 132 | 79 | 26 |
| dissolved | 88 | 88 | 88 |
| N-Kjeldahl (mg $N.l^{-1}$) | 55 | 50 | 44 |
| particulate | 14 | 9 | 3 |
| dissolved | 41 [a] | 41 [a] | 41 [a] |
| $P_{total}$ (mg $P.l^{-1}$) | 9.0 | 7.9 | 1 [b] |
| particulate | 2.7 | 1.6 | |
| dissolved | 6.3 | 6.3 | |
| BOD/N | 4.0 | 3.3 | 2.6 |

[a] assuming 40 mg $N.l^{-1}$ as $NH_4^+$

[b] 2-3 $mgP.l^{-1}$ in case of secondary biological treatment

The BOD/N ratios of table 2.8 do not yet account for the extra nitrogen load that is added by the reject water flow (i.e. the water from the centrifuge). This extra nitrogen load is dependent on the type of post treatment that is applied. In the DEMAS+ model it is calculated, based on the following assumptions:

- In the digester 50% of the primary and 30% of the secondary sludge are converted (Lawler *et al.*, 1986; Gosh, 1991; and Sanders *et al.*, 1996). These conversions refer to the organic fraction of the sludge.
- The biomass yield of the anaerobic bacteria in the digester is low and is assumed negligible.
- The primary sludge conversion proceeds according to the general digestion reaction:

  $C_aH_bO_cN_dP_e \rightarrow 1/c\ CO_2 + (a-1/c)\ CH_4 + d\ NH_4\text{-}N + e\ PO_4\text{-}P$ + energy (+ cells).
- Regarding the conversion of secondary (biological) sludge, it is assumed that the COD:N:P ratio of biomass numbers 150:14 (Metcalf & Eddy, 1991). It means that for every 100 g of digested COD and 9.3 g $NH_4$-N is released.
- It is assumed that, during the dewatering stage, all the released $NH_4$-N ions end up in the reject water and are ultimately returned to the treatment plant. (The extra P load is left out of the presented calculations.)

The resulting BOD/N ratios after pre-precipitation and flotation are in the order of 2.0 to 2.2. These values are rather low for denitrification in a biological post treatment with pre-denitrification. In literature minimum ratios of 3-4 are advised (e.g. Klapwijk et al., 1997) to achieve nitrogen concentrations that comply with the effluent standard (< 10 mgN.$l^{-1}$). In the current scenario study it is assumed the mentioned BOD/N values are sufficient to reach the discharge standard. However, it should be noticed that the low BOD/N ratio is considered a possible bottleneck to the application of physical-chemical pretreatment and that it has been a point of investigation in the chapters 5 and 6 of this thesis.

## Evaluated treatment scenarios

During the research ten treatment scenarios, which are all based on physical-chemical pretreatment, were evaluated. Table 2.9 presents these scenarios dividing them into three main scenario groups according to the applied post treatment. In scenario group 1, physical-chemical post treatment is applied; in group 2, biological post treatment is applied; whereas in scenario group 3, a combination of physical-chemical and biological post treatment is applied.

It should be noticed that in scenarios 1a, 1b and 3a-3d rapid sand filtration is introduced as a second pretreatment unit, subsequent to the application of pre-precipitation and flotation. Rapid sand filtration is applied to protect the down-stream treatment steps from remaining solids.

**Table 2.9:** Evaluated treatment scenarios with pre- and post treatment steps

| Scenario | Pretreatment | Post treatment |
|---|---|---|
| Scenario 1a | Flotation (DAF) | Sand filter + ion-exchange (with air stripping[a]) + activated-carbon adsorption (thermal reactivation) |
| Scenario 1b | Pre-precipitation | Sand filter + ion-exchange (with air stripping[a]) + activated-carbon adsorption (with thermal reactivation) |
| Scenario 2a | Flotation (DAF) | Low loaded biofilm system + sand filter |
| Scenario 2b | Pre-precipitation | Low loaded biofilm system + sand filter |
| Scenario 2c | Flotation (DAF) | Low loaded activated sludge system + secondary clarifier |
| Scenario 2d | Pre-precipitation | Low loaded activated sludge system + secondary clarifier |
| Scenario 3a | Flotation (DAF) | High loaded biofilm system + sand filter + ion-exchange (with air stripping[a]) |
| Scenario 3b | Pre-precipitation | High loaded biofilm system + sand filter + ion-exchange (with air stripping[a]) |
| Scenario 3c | Flotation (DAF) | High loaded activated sludge system + secondary clarifier + sand filter + ion exchange (with air stripping[a]) |
| Scenario 3d | Pre-precipitation | High loaded activated sludge system + secondary clarifier + sand filter + ion exchange (with air stripping[a]) |

[a] product: ammonium sulphate ($(NH_4)_2SO_4$)

## Results and discussion

The results of the calculations regarding the environmental criteria and costs are summarised in the following paragraphs. An overview of the identified scenarios is presented with diagrams which enables their comparison. In these diagrams the first bar represents the reference scenario (ref.), the following bars are numbered as in table 2.9. The values of the consumption of chemicals (cq. recovery of nitrogen) and the expected effluent quality are presented in a table.

**Energy balance** - Figure 2.9 shows the results of the energy calculations. The balance includes the energy that is used for the unit operations in the water treatment and the energy that is generated from the digested primary and secondary sludge. The energy demand is expressed as the electricity requirement (in MWh.year$^{-1}$).

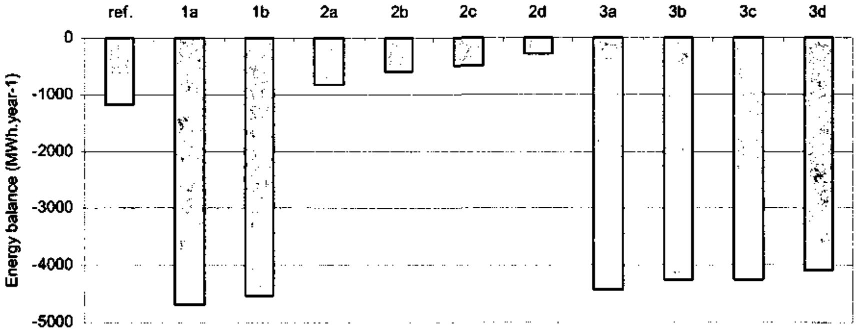

**Fig. 2.9:** Calculated energy balance of the treatment scenarios

Figure 2.9 shows that scenarios containing a secondary treatment that consists completely of physical-chemical unit operations (scenario 1a, 1b) require four times more energy than the reference scenario. The main reason for the high energy consumption is the ion exchanger + air stripper. Especially the increase in temperature of the regenerate (to 50 °C) prior to the gas washer requires a high amount of energy. The energy is supplied to the process as methane. To achieve an overall balance, the energy content of the methane was recalculated to MWh.

The scenarios that combine pretreatment (2a-2d) with biological post treatment have a high potential to save energy. As can be seen in the graph 30-85% less energy is required compared to the reference scenario. The reasons are the higher (re)use of energy through primary sludge digestion and a decreased energy demand of the post treatment.

The latter is due to a lower energy requirement of the aeration, because less COD is entering the system.

The scenarios that are combining ion exchange and biological post treatment (3a-3d) also require approximately four times more energy compared to the reference, similar to scenarios 1a and 1b. Again, the main reason is the ion exchanger/stripper combination

**Final sludge production** - The results of the calculations regarding the final sludge production after digestion and centrifugation are shown in figure 2.10. The sludge production is divided into three fractions: (i) inorganic sludge, (ii) organic sludge from (not-digested) primary and secondary sludge and (iii) chemical sludge consisting of Fe precipitates (i.e. ferric phosphate, ferric hydroxide). The final sludge production is given as the mass of total solids (TS) produced.

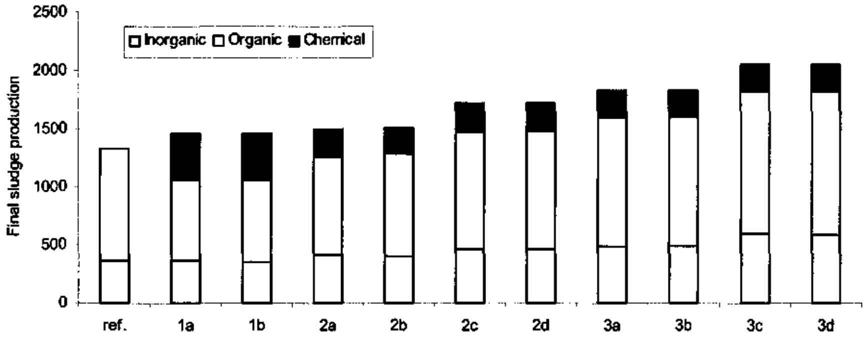

**Fig. 2.10:** Final sludge production (tonTS.year$^{-1}$)

The calculations show that the scenarios with physical-chemical pretreatment produce more final sludge after digestion than the reference. The increase of the final sludge quantity is for a large part caused by the production of chemical sludge. For a number of scenarios also a higher organic sludge production was found.

The graph shows that scenarios 1a and 1b produce more chemical sludge (approximately 400 tonTS.year$^{-1}$) than scenarios 2a to 3d (circa 240 tonTS.year$^{-1}$). The cause is the higher required Fe dosage in both scenarios. In scenario 2a to 3d a secondary biological treatment step is applied. In such a biological step a part of the phosphorus is removed by biomass uptake. Therefore, less Fe is necessary to remove phosphate up to the effluent standard (< 1 mgP.l$^{-1}$).

Differences were also found in the production of organic sludge. Scenario 1a and 1b produce less sludge, because the COD that remains after pretreatment is removed by activated carbon adsorption. In scenario 2a to 3d remaining COD is removed by biological post treatment, resulting in the production of biological waste sludge.

The low loaded biofilm systems (2a, 2b) produce a lower sludge quantity compared to the low loaded activated sludge systems (2c, 2d), because of the lower assumed bacterial yield (0.2 as against 0.4 $kgSS.kgCOD_{removed}^{-1}$). For the scenarios including a high loaded biofilm or high loaded activated sludge system (3a to 3d), the highest organic sludge production was found.

**Effluent quality** - With respect to the effluent quality a more qualitative approach was used to evaluate the scenarios. In this approach the expected effluent quality was compared to the effluent quality of the reference scenario and appreciated with +/0/- (improved, similar or declined). Table 2.10 shows the results of this appraisal.

The effluent of all scenarios complies with the effluent standards as given in table 2.5. However, due to the application of different post treatment unit operations, the effluent of some scenarios is of a higher quality. The scenarios that incorporate a rapid sand filter as a final treatment step, induce an effluent that contains less suspended solids and micro pollutants (as the latter are adsorbed to suspended solids). The application of actived carbon adsorption in scenario 1a and 1b will result in the removal of dissolved micro pollutants. The scenarios that incorporate an ion exchanger (1a, 1b; 3a - 3d) have the potential to improve the effluent quality with respect to ammonium-nitrogen. However, it should be noted that in ion exchange the ammonium is substituted by a counter ion (in most cases $Na^+$).

The addition of $FeCl_3$ for phosphate precipitation and coagulation/flocculation results for all scenarios in an increase of the $Cl^-$ concentration of the effluent. The chloride concentration of the effluent is increased by 1.9 mg Cl per mg added Fe. It results for scenarios 1a and 1b in an increase of 47.5 $mgCl.l^{-1}$ and for scenarios 2a to 3d in an increase of 28.5 $mgCl.l^{-1}$.

**Table 2.10:** Appraisal of the effluent quality of the identified treatment scenarios compared to reference (+/0/- ; improved, similar or declined)

| **Scenario** | **N** | **P** | **TSS** | **micro pollutants** | **$Cl^-$** |
|---|---|---|---|---|---|
| Scenario 1a | + | 0 | + | ++ | - |
| Scenario 1b | + | 0 | + | ++ | - |
| Scenario 2a | 0 | 0 | + | + | - |
| Scenario 2b | 0 | 0 | + | + | - |
| Scenario 2c | 0 | 0 | 0 | 0 | - |
| Scenario 2d | 0 | 0 | 0 | 0 | - |

| **Scenario** | **N** | **P** | **TSS** | **micro pollutants** | **$Cl^-$** |
|---|---|---|---|---|---|
| Scenario 3a | + | 0 | + | + | - |
| Scenario 3b | + | 0 | + | + | - |
| Scenario 3c | + | 0 | 0 | + | - |
| Scenario 3d | + | 0 | 0 | + | - |

**Use of chemicals and recovery of nitrogen** - Table 2.11 shows the quantities of required chemicals as well as the recovered ammonium sulphate (in ton per year). All the scenarios require flocculating agents ($FeCl_3$ and anionic polymer). The total polymer consumption includes the anionic polymer that is dosed in the pretreatment and the polymer that is used in the centrifuge for sludge dewatering.

Scenarios 1a and 1b require more $FeCl_3$, due to the absence of biological post treatment. The scenarios 1a to 3d show an increasing requirement for polymers. This is caused by the increasing production of sludge (fig. 2.10). The scenarios that include an ion exchanger + stripper require relatively large quantities of NaOH and $H_2SO_4$. At the same time a large quantity of $(NH_4)_2SO_4$ is produced.

**Table 2.11:** Use of chemicals and recovery of ammonium sulphate

| | **Flocculating agents** | | **Ion exchanger** | | |
|---|---|---|---|---|---|
| | | | Chemical use | | **Product** |
| Scenario | $FeCl_3$ (ton.year$^{-1}$) | Polymer (ton.year$^{-1}$) | NaOH (ton.year$^{-1}$) | $H_2SO_4$ (ton.year$^{-1}$) | $(NH_4)_2SO_4$ (ton.year$^{-1}$) |
| ref | -- | 10.0 | | | |
| Scenario 1a | 516 | 16.9 | 700 | 870 | 1170 |
| Scenario 1b | 516 | 17.0 | 700 | 870 | 1170 |
| Scenario 2a | 310 | 18.4 | | | |
| Scenario 2b | 310 | 18.6 | | | |
| Scenario 2c | 310 | 20.0 | | | |
| Scenario 2d | 310 | 20.1 | | | |
| Scenario 3a | 310 | 20.9 | 500 | 620 | 830 |
| Scenario 3b | 310 | 21.0 | 500 | 620 | 830 |
| Scenario 3c | 310 | 22.6 | 440 | 540 | 730 |
| Scenario 3d | 310 | 22.7 | 440 | 540 | 730 |

**Net space requirement** - Figure 2.11 shows that all treatment scenarios with physical-chemical pretreatment can be designed smaller compared to the reference. The footprint of scenario 1a is the smallest, with only approximately one-ninth of the space requirement of the reference. The other scenarios have a calculated space requirement of around one-third of the land use of the reference scenario, except for the scenarios that include an activated sludge system (scenario 2c-2d and 3c-3d). These systems require a relatively large area due to the secondary clarifier (4285 $m^2$).

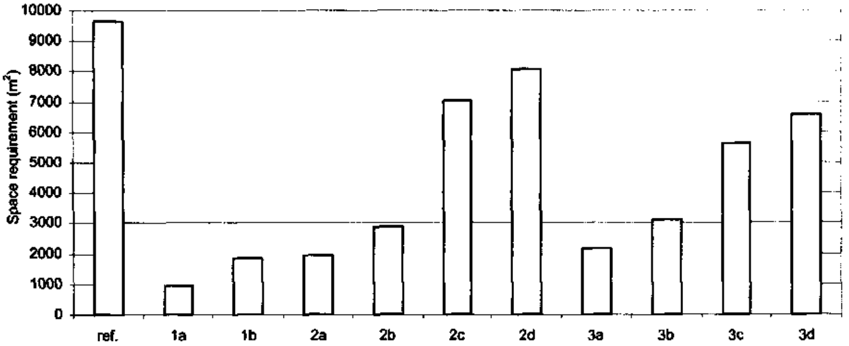

**Fig. 2.11:** Net space requirement

**Costs** - Figure 2.12 presents the results of the cost calculations. In this graph the net present values of the scenarios are compared to the reference. The graph shows the net present value of the construction/electro-mechanic (EM) works as well as the net present value of the operational costs.

The graph demonstrates that the net present value of construction/EM works is almost similar for all scenarios. Large differences, however, were found with respect to the operational costs. The scenarios that include physical-chemical post treatment steps (activated carbon, ion exchange) showed significant higher costs for operation.

The estimated net present values of scenarios 1a and 1b are approximately three times higher than those of the reference scenario. The most important reason for these high figures are the costs that are needed to regenerate the loaded activated carbon. The columns will have to be regenerated approximately 10 times per year. The costs amount around € 200 per $m^3$ regenerated activated carbon. A second major factor are the costs for ammonium recovery by the air stripper (€ 4.5 per kg recovered $NH_4$-N). The latter should however, be seen in the perspective of a potential market for the product (in this case ammonium sulphate).

The total net present values of the scenarios that combine enhanced primary treatment with biological post treatment (2a-2d) are (considering the accuracy of the calculations) in the same range of those of the reference scenario. These scenarios show a potential to be cost competitive with the currently applied systems.

The total costs of scenarios 3a-3d are approximately 75% higher than those of the reference treatment. The most important cost increasing factor in these scenarios is the ammonium recovery by the air stripper.

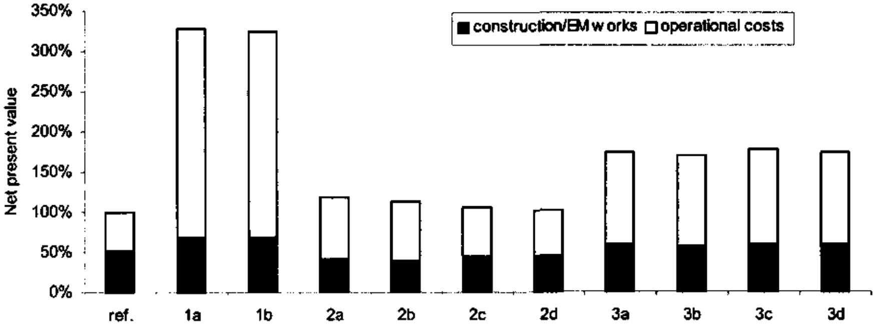

**Fig. 2.12:** Financial cost comparison based on the net present value (the reference scenario =100%)

## Conclusions and bottlenecks

The literature review showed that, for enhanced primary treatment, different unit operations are available. It appears that the possible unit operations for pretreatment include pre-precipitation, dissolved air flotation, microscreening, direct influent filtration, direct membrane filtration and enhanced settling by addition of magnetite or micro sand. The use of flocculating agents is essential to all these unit operations to obtain a significant level of particle removal, except for direct influent filtration and direct membrane filtration.

Various unit operations are available for secondary and tertiary treatment of the primary effluent. A major distinction can be made into unit operations based on biological techniques (activated sludge systems, biofilm systems, membrane bioreactors) and unit operations based on physical-chemical pollutant removal (ion-exchange, activated carbon, rapid sand filtration). The application of physical-chemical pretreatment is favourable to the application of compact post treatment steps. Unit operations like biofilm systems, activated carbon adsorption and ion exchange, preferably require wastewater containing low particle concentrations. These unit operations may also result in an improved effluent quality with regard to nitrogen, suspended solids and micro pollutants.

The scenario study confirmed that the application of enhanced particle removal enables a smaller (20-80%) and more energy efficient design of complete wastewater treatment systems in comparison with the commonly applied treatment plants.

The most important conclusions regarding the different groups of scenarios are:

- The treatment scenarios that combine enhanced primary treatment with biological post treatment for COD and nitrogen removal have a large potential to save energy. For this type of systems energy savings of 40-85% were calculated. The overall costs of these scenarios are in the same range as those of the reference.
- The costs of the scenarios that include ion-exchange are at least 75% higher than the reference. These higher costs are mainly due to the relative high costs for brine regeneration and ammonium recovery (air stripping). The regeneration consumes relatively large quantities of chemicals and energy. However, the costs and the chemical and energy consumption should be considered from the perspective that a product is generated for possible re-use as agricultural fertiliser. The possible profits have not yet been part of the calculations.
- Scenarios that include activated carbon adsorption for dissolved COD removal result in costs that are three times higher than the reference treatment. The main reasons are the costs for regeneration of the activated carbon.

The study of scenarios revealed two important bottlenecks regarding the application of physical-chemical pretreatment, being (i) the use of metal-based flocculating agents and (ii) the removal of biodegradable carbon which is required for denitrification in a biological post treatment step.

The use of metal salts contributes considerably to the overall environmental impact of the evaluated treatment systems. Due to the application of metal salts a significant amount of inorganic (chemical) sludge is produced. This chemical sludge will increase the volume of final waste after incineration. In addition, the use of metal- salts based flocculating agents will result in an increased grade of salinity of the effluent. It is recommended to perform investigations into the possible replacement of metals salts by organic polymers to avoid the mentioned environmental impacts.

A second (potential) bottleneck regarding the application of physical-chemical pretreatment of municipal wastewater is that the removal of particles is partially accompanied by the removal of biodegradable organic carbon. A decrease in BOD/N ratio through the removal of biodegradable COD may inhibit the nitrogen removal by denitrification and may result in an increased nitrogen concentration in the effluent. The review showed that the BOD/N ratio of the wastewater is decreased after enhanced primary treatment. For an average Dutch influent the BOD/N of pretreated water was estimated at 2.0-2.2 including nitrogen that is recirculated by the reject water subsequent to digestion and centrifugation. This value is lower than values that are generally considered sufficient to achieve the nitrogen discharge standard of $< 10\ mgN_{total}.l^{-1}$.

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# Appendix A Assumptions for the cost calculations

**Table A1:** Assumptions for the operational costs

| | | |
|---|---|---|
| Electricity | € | 0.068 per kWh |
| $FeCl_3$ (41 %) | | 150 per m$^3$ |
| Anionic polymer | | 5,650 per 1000 kg |
| Final sludge disposal | | 400 per 1000 kg dry solids |
| Staff * | | 110,000 |

* assuming that the wastewater treatment plant is operated by 3 employees

**Table A2:** Assumptions for the construction and electro-mechanical costs

| **Physical-chemical pretreatment** | Construction* (€) | Electro-mechanical* (€) | Maintenance construction/EM (year$^{-1}$) |
|---|---|---|---|
| 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 clarifiers | 270 per m$^2$ | 70 per m$^2$ | 0.5%/1.5% |
| Cover primary clarifier | 90 per m$^2$ | -- | -- |
| Flotation | 1,130 per m$^2$ | 2,260 per m$^2$ | 0.5%/2.0% |
| **Secondary treatment** | | | |
| 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% |
| Ion exchange (excl. regenerate treatment) | 54,300 per column | 3,850 per m$^3$ | 0.5%/2.0% |
| -stripper for $NH_4$-N recovery*** | 4.5 per kg $NH_4$-N | | |
| Activated carbon filter | 1,580 per m$^3$ | | |
| - thermic regeneration | 200 per m$^3$ AC | 2,400 per m$^3$ | 0.5%/2.0% |
| - activated carbon replacement | 1.7 per kg AC | | |
| **Sludge handling** | | | |
| Sludge thickener | 900 per m$^2$ | 230 per m$^2$ | 0.5%/1.5% |
| Cover sludge thickener | 90 per m$^2$ | -- | -- |
| Digester | 180 per m$^3$ | 90 per m$^3$ | 0.5%/1.5% |
| Total energy installation | 50 per m$^3$ biogas/day | 230 per m$^3$ biogas/day | 0.5%/1.5% |
| Centrifuge (per piece) | 135,000 | 58,000 | 0.5%/2.0% |
| **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 was assumed

** 0.5 % of contruction and 1.5 % of electro/mechanical- investments

*** based on STOWA (1995)

# Appendix B Design criteria of the standard sludge handling and calculations with regard to sludge production

**Table B1:** Design criteria of the standard sludge handling procedure

| Unit process | Design criteria |
|---|---|
| Sludge thickener (per $m^3$) | |
| Solids load | 50 $kgTS.m^{-2}.day^{-1}$ for primary sludge;<br>30 $kgTS.m^{-2}.day^{-1}$ for secondary sludge |
| Dry solids content after thickening | 5% |
| Digester | |
| Solids retention time | 20 days |
| Tank height | 10 m |
| Temperature | 33 °C[a] |
| Organic solids degradation | 50% for primary sludge; 30% for secondary sludge |
| Biogas production | 1.15 $m^3$ per kg organic solids converted |
| Energy content biogas | 22,000 $kJ.m^{-3}$ |
| Total energy installation | |
| Energy conversion | 32% energy conversion into electricity<br>68% energy conversion into heat[a] |
| Centrifuge | |
| Capacity | 16 ton TS per centrifuge per week |
| Energy consumption | 4 $kWh.m^{-3}$ sludge |
| Dry solids content after centrifuge | 25% |
| Polymer addition | 7.5 $kg.tonTS^{-1}$ |

[a] The heat that is produced during electricity generation is used to raise the temperature of the sludge to 33 °C

**Table B2:** Sludge production

| Description | Calculation according to: |
|---|---|
| Chemical sludge production (kg $TS.year^{-1}$) | 2.25 kg TS per kg Fe dosed |
| Primary sludge production (kg $TS.year^{-1}$) | 0.71 (kg $TS.kgCOD^{-1}$) * $COD_{removed}$ in pretreatment |
| Organic fraction of primary sludge | 80% |
| Secondary sludge production (kg $TS.year^{-1}$) | $COD_{removed}$ in secondary treatment * Y[b] |
| Organic fraction of secondary sludge | 82% |

[b] Y (biological yield) as given in appendix C while expressed in kg TS (total solids) instead of kg SS (suspended solids)

# Appendix C Design criteria of the post treatment steps

**Table C:** Design criteria of the unit operations for post treatment

| **Secondary treatment** | |
|---|---|
| Low loaded activated sludge system (BOD, P, N) | |
| Sludge loading rate | 0.06 kg BOD.kgSS$^{-1}$.day$^{-1}$ |
| Sludge production factor (Y) | 0.4 kgSS.kgCOD$_{removed}^{-1}$ |
| Sludge concentration | 4 kgSS.m$^{-3}$ |
| Hydraulic load secondary clarifier | 0.7 m.h$^{-1}$ (at $Q_{SW}$) |
| | |
| High loaded activated sludge system (BOD) | 0.8 kgBOD.kgSS$^{-1}$.day$^{-1}$ |
| Sludge loading rate | 0.6 kgSS.kgCOD$_{removed}^{-1}$ |
| Sludge production factor (Y) | 4 kgSS.m$^{-3}$ |
| Sludge concentration | 0.7 m.h$^{-1}$ (at $Q_{SW}$) |
| Hydraulic load secondary clarifier | |
| | |
| Low loaded biofilm system (BOD, N) | |
| Volumetric loading rate | 0.2 kgBOD.m$^{-3}$ |
| Sludge production factor (Y) | 0.2 kgSS.kgCOD$_{removed}^{-1}$ |
| | |
| High loaded biofilm system (BOD) | |
| Volumetric loading rate | 0.6 kgBOD.m$^{-3}$ |
| Sludge production factor (Y) | 0.5 kgSS.kgCOD$_{removed}^{-1}$ |
| | |
| Ion exchange (N) | |
| Effluent concentration | 5 mg $NH_4$-N.l$^{-1}$ |
| Surface load | 30 m.h$^{-1}$ (at $Q_{SW}$) |
| 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 stripping | |
| Surface load | 10 m.h$^{-1}$ (at $Q_{SW}$) |
| Regenerate concentration | 1.6 eq.l$^{-1}$ |
| Chemical use | 2.9 kg NaOH and 3.5 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) | |
| Surface load | 5 m.h$^{-1}$ (at $Q_{SW}$) |
| 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 |

# Appendix D Energy calculations

**Table D1:** Energy requirement (100,000 p.e. wwtp)

| **Physical-chemical pretreatment** | **Calculation** | **Value ($kWh.year^{-1}$)** |
|---|---|---|
| Fine screen | P = 2 kW | 17, 500 |
| Flocculator | | |
| - rapid mixing | P = 14.2 kW | 124, 500 |
| - flocculation mixing | P = 2.3 kW | 20, 000 |
| Primary/secondary clarifiers | | |
| - skimmers | P = 4 kW | 35, 000 |
| Flotation | | |
| - pressurized recycle flow | P = 27.8 kW | 243, 500 |
| - float layer removal | P = 4 kW | 35, 000 |
| **Secondary treatment** | | |
| Activated sludge system | | |
| -aeration energy | $OR = O_e + O_s + O_n - O_d$ | dependent on influent COD and N |
| -recycle (low loaded system) | P = 11.6 kW | 102, 000 |
| Biofilm system | | |
| -aeration energy | $OR = O_e + O_s + O_n - O_d$ | dependent on influent COD and N |
| -recycle (low loaded system) | P = 11.6 kW | 102, 000 |
| Ion exchange | | |
| -pumps | P=15.4 kW | 135, 000 |
| -stripper for $NH_4$-N recovery | 13.35 kWh per kg $NH_4$-$N_{recovered}$[a] | dependent on influent N |
| Activated carbon filter | | |
| - pumps | P = 10.2 kW | 89, 500 |
| - reactivation | 0.4 kWh per $m^3.year^{-1}$ | 480, 000 |
| **Sludge handling** | | |
| Sludge thickener | | |
| - skimming | P = 4 kW | 35, 000 |
| Digester | | |
| - pumps, stirring | P=4 kW | 35, 000 |
| - heating of sludge | | 0 (heated by TE installation) |
| Total energy installation (32% efficiency) | 32% of 22,000 $kJ.m^{-3}$ | Energie production: 1.95 $kWh.m^{-3}$ biogas |
| **Tertiary treatment** | | |
| Rapid sand filtration | P = 4.5 kW | 39000 |

[a] this figure mainly consists of the energy content of methane that is used for heating of the regenerate; according to STOWA (1995) 0.9 kg methane is needed per kg $NH_4$-$N_{recovered}$

**Table D.2:** Energy requirement of the aeration of the activated sludge and biofilm systems

| **Oxygen requirement** | **Oxygen and energy requirement** |
|---|---|
| $OR = O_e + O_s + O_n - O_d$ (kg $O_2$.year $^{-1}$) | $\alpha \cdot OR = OC.p.\frac{Cs}{Cs-C}$ (kg $O_2$.year $^{-1}$) |
| $O_e + O_s = COD_{influent} - COD_{effluent} - COD_{sludge} = ((1-Y).COD_{influent} - COD_{effluent}).Q_{influent}$<br>$O_n = (N_{kj,i} - N_{kj,e}).Q_{influent} - N_{sludge}).4.57$<br>$O_d = (N_{kj,i} - N_{kj,e}).Q_{influent} - N_{sludge}).2.86$ | 2.5 $kgO_2.kWh^{-1}$ |
| OR - oxygen requirement<br>$O_e$ - endogenous oxygen consumption<br>$O_s$ - oxygen consumption for oxidation of biodegradable organic carbon<br>$O_n$- oxygen consumption for ammonium oxidation (i.e. nitrification)<br>$O_d$. reduction of oxygen consumption due to denitrification<br>Y - biological yield (0.55 $kgCOD_{biomass}.kgCOD_{removed}^{-1}$) | OC - oxygen consumption (kg $O_2$.year$^{-1}$)<br>α - aerator efficiency (0.9 (-))<br>p - peak factor (1.2 (-))<br>C - oxygen concentration influent (= 0.5 $mgO_2.l^{-1}$) |

# 3 Organic polymers for flocculation of municipal wastewater

**Abstract.** The objective of this chapter was to investigate the potential application of organic poly electrolytes 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, (i) low to moderate molecular weight polyamines and poly-DADMACs and (ii) high molecular weight (HMW) polyacrylamides. Jar test experiments with a selection of these polymer types showed that especially the cationic HMW polyacrylamides appeared to be interesting for practical application, as they got effective at relatively low doses (5-10 mg.$l^{-1}$). For a low molecular weight cationic polyamine also high turbidity removals were found, however at higher required doses (20-30 mg.$l^{-1}$). For both types of polymers the experiments resulted in turbidity removals of 65-90% and a suspended solids removal of more than 90%. Furthermore, it was found that the addition of organic polymers does not affect the pH of the treated wastewater.

The addition of HWM cationic polymers tends to result in large, well settling flocs. The sludge flocs formed after addition of a low weight cationic polyamine appeared to be significantly smaller. Furthermore, filterability tests with the Filtration-Expression cell showed that the improvement in dewaterability of sludge produced by the addition of polymers is similar to that of sludge produced by the addition of ferric chloride.

It was found that for cationic HMW polyacrylamides the turbidity removal increases gradually with increasing dose, which may imply an interesting possibility to control the particle removal of unit operations for pretreatment.

## Introduction

The study of various pretreatment scenarios in chapter 2 showed that the application of enhanced primary treatment tends to improve the overall environmental sustainability of wastewater treatment plants. However, the required addition of metal-based flocculating agents like $FeCl_3$ induce some negative environmental side effects such as the production of relatively large volumes of inorganic (chemical) sludge and an increase of salt concentration of the effluent due to the release of counter ions.

The objective of this chapter is to investigate the potential application of organic poly electrolytes as an alternative to metal-based flocculants. When organic polymers are applied for coagulation/flocculation, chemical sludge production is small because no precipitates are formed. Moreover, compared to metals salts organic polymers induce a relatively small increase in salt concentration of the effluent.

This chapter describes a literature review and experimental work into the application of organic polymers. In the literature review the principles of coagulation/flocculation are studied and a description is given of poly electrolytes that are frequently applied in industrial wastewater treatment and in sludge handling. The experimental work focuses on the applicability of different types of polyacrylamine and polyacrylamide-based organic polymers for municipal wastewater treatment. For this purpose jar test experiments were conducted. Within these jar test experiments different features of such polymers were studied, like the required doses, the potential particle removal, the effect of polymer dose on pH and certain characteristics of the produced sludge like floc size and dewaterability.

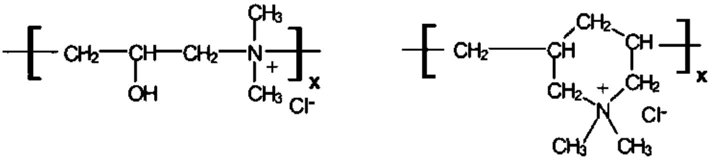
## Literature review

**Organic polymers and their characteristics**– Organic poly electrolytes are generally characterized by two main properties, their molecular weight and their amount of ionic charge. The molecular weight (or, more properly, molecular mass) is and indication of the amount of monomers and thus the length of the polymer chain. 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.

With regard to ionic charge, polymers can be cationic, anionic and non-ionic, i.e. respectively positively, negatively and neutrally charged. A polymer's charge density indicates the amount of charge available to accomplish particle destabilization. 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 percent are produced for different applications. The charge density of polymers can be measured by potentiometric, colloidal or counterion titration.

Most of the organic polymers that are currently applied in (industrial) wastewater treatment and in sludge handling are synthetically prepared polyacrylamides and polyamines. In practice two major categories are distinguished: (i) low to moderate molecular weight polyamines and poly-DADMACs and (ii) very high molecular weight polyacrylamides (Barvenik, 2000; Dentel *et al.*, 2000; Gucciardi, 2000; Will, 2000). Figure 3.1 and 3.2 show these main types of poly electrolytes.

In figure 3.1 two common chemical structures of low to moderate molecular weight cationic polymers are shown. Both polymers are homo polymers based on one type of monomer. Structure A in figure 3.1 depicts a quaternary cationic polyamine, while structure B shows a cationic polymer based on a DADMAC (diallyldimethyl ammonium chloride) 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 3.1 can also be copolymerized with acrylamide to create a variety of high weight copolymers.



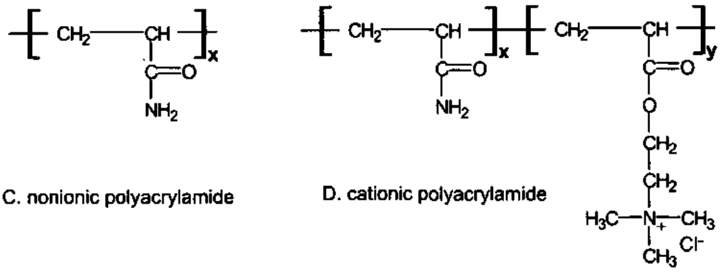
A. polyamine

B. poly-DADMAC

**Fig. 3.1:** Chemical structures of low to moderate molecular weight cationic polymers

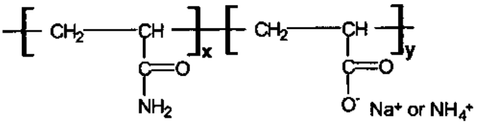
Structure C in figure 3.2 shows the basic acrylamide monomer, which polymerized, yields non-ionic polyacrylamide. Non-ionic polymers are hardly applied in wastewater treatment or sludge handling. Structure D depicts the most common acrylamide based cationic polymer, consisting of a non-ionic monomer (acrylamide) and a cationic monomer (quaternised aminomethylated acrylamide). The 'x' and 'y' indicate the ratio of nonionic and cationic monomers and determine the charge density of the polymer.

Structure E 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, because of the acidic nature of the monomer. At pH < 4 the anionic poly electrolyte is uncharged.



C. nonionic polyacrylamide

D. cationic polyacrylamide



E. anionic polyacrylamide

**Fig. 3.2:** Chemical structure of polyacrylamide based polymers

Apart from the polyacrylamides and polyamines also different organic polymers exist that are based on natural materials. Examples are quarternized starch polymers and condensed tannin (Gucciardi, 2000), cellulose and galactose products, microbial polysaccharide products, gelatine based products, seed of the Moringa Olefeira (Kalogo and Verstraete, 2000; Okuda *et al.*, 2001) and Chitine based products (Ødegaard, 1998, Pan *et al.*, 1999).

In fact, there is an increasing development of non-acrylamide based flocculants, because of the reported slow biodegradability of polyacrylamide polymers and the toxicity of acrylic acid monomers (Barvenik, 2000; Dentel, 2000; Gucciardi, 2000).

**Surface characteristics of particles in wastewater** - Particles in municipal wastewater mainly have a negative charge. This can be verified by placing the wastewater particles into an electrical field. Due to the charge, the particles move to the positive electrode. The zeta potential can be calculated based on the field potential, the mobility of the particles in the field and the viscosity. For particles in wastewater this zeta potential varies from –15 to –40 mV (see figure 3.3) (Henze *et al.*, 1995; Elmitwalli, 2000)

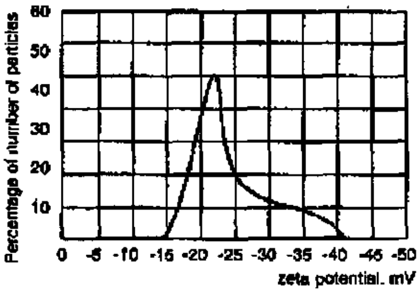


**Fig. 3.3:** Example of a zeta potential distribution curve for wastewater particles (in: Henze et al., 1995)

The negative surface charge of the particles is the result of dissociation of end groups on the surface, such as amino (-$NH_2$), hydroxyl (-OH) and acid groups (-COOH) 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 (Henze et al., 1995). It is assumed that soap 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.

Due to the negative surface charge of the particles, oppositely 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*. Because of the charge, an electrical potential difference 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. The potential at this interface is called the (earlier mentioned) zeta potential ($\xi$) or electrokinetic potential.

A diffuse layer of negative and positive ions surrounds the Stern layer. 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.

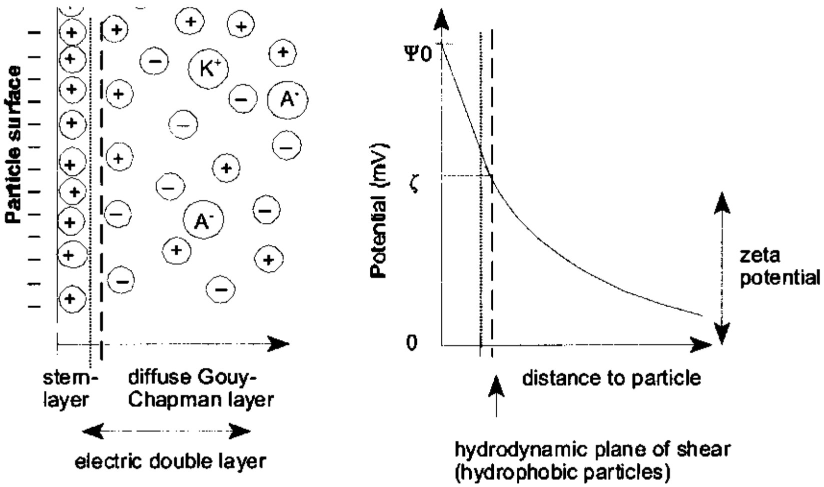


**Fig. 3.4:** Schematic presentation of the double layer of ions around a colloidal particle with a negative surface charge (left) and the decline of potential with increasing surface distance (right)

Apart from the repelling electric charge forces the colloidal particles also attract each other through the *London-Van der Waals'* forces. The attractive London-Van der Waals' forces decrease rapidly with increasing particle distance. In order to let the particles attach the repulsive forces should be neutralized, so that the attractive 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 attractive (mass force related) and the repulsive (surface force related) energies within the interparticulate sphere. Figure 3.5 shows a schematic representation of the so-called DLVO theory and describes the energy pattern in relation to the distance to the particle surface. 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.

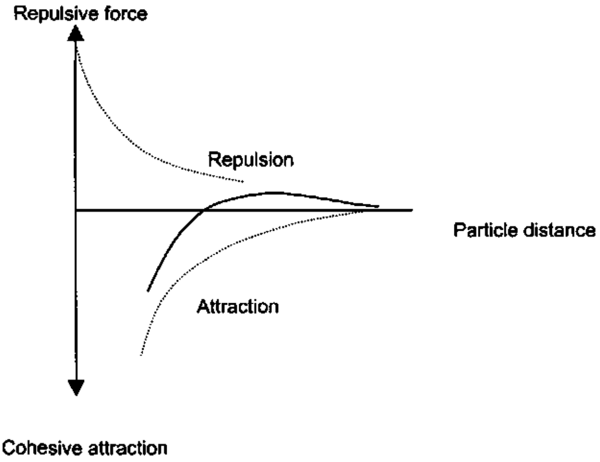


**Fig. 3.5:** Schematic presentation of the DLVO theory (redrawn after Schwoyer, 1981)

**Principles of coagulation/flocculation** - As pointed out earlier, particles in municipal wastewater are mainly negatively charged. Due to the double layer of ions and the resulting negative zeta potential particles are colloidal stabilised. In order to realise mutual attachment of particles by attracting mass forces, destabilization of the particles' repulsive surface charge is necessary. Regarding this attachment process a general distinction is made into *coagulation* and *flocculation*. The term coagulation is used to indicate the process of charge neutralization resulting in colloidal destabilization of the particles. Flocculation is generally used to indicate the process of mutual attachment that is succeeding the destabilization (Fleer, 1993).

When organic polymers are added to colloidal solutions they are adsorbed to the surface of the present particles (Stumm and O'Melia, 1968). The adsorption is caused by mutual attraction of opposite charge. In understanding coagulation/flocculation by the addition of organic polymers a distinction is generally made into two main mechanisms: electrostatic coagulation and bridging flocculation.

Electrostatic coagulation plays an important role when highly charged, low molecular weight polymers are applied. Due the adsorption of these polymers, the wastewater particles will be partially de-charged (figure 3.6). The charge neutralization results in a faster decrease of the zeta potential thus decreasing the size of the Gouy-Chapman layer. The particles are now able to draw up closer so mutual attachment can take place because of the mass forces. If high amounts of poly electrolyte are dosed, the concentration of adsorbed polymer 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.

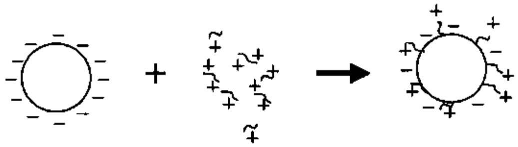


**Fig. 3.6:** Charge neutralization due to the dosage of a low molecular weight cationic polymer

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 3.7). An important characteristic of bridging flocculation is that lower electrolyte dosages can be applied to achieve particle flocculation compared to electrostatic coagulation.

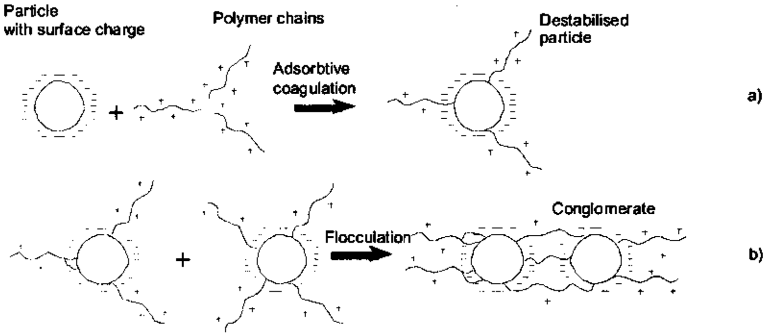


**Fig. 3.7:** Adsorptive coagulation (a) and bridging (b) due to the dosage of a high molecular weight cationic polymer

Similar to poly electrolyte overdose in electrostatic coagulation an overdose of polymer may result in restabilisation of particles. The adsorption capacity of the colloidal particle is used by several polymer chains and cannot be used for bridging with other particles (fig. 3.7).

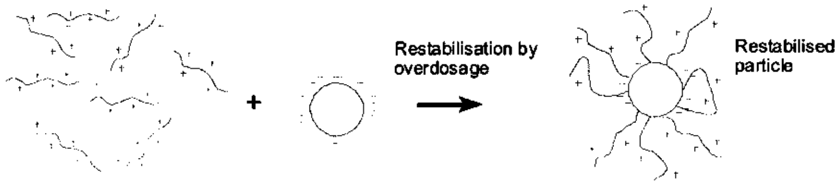


**Fig. 3.8:** Restabilisation of particles due to overdose of organic polymer

## Materials and methods

**Characterization of the used organic polymers** - For coagulation/flocculation a low molecular weight (LMW) cationic polyamine, four linear poly electrolytes based on polyacrylamides and two high molecular weight (HMW) anionic polyacrylamides (all *CYTEC*) were tested. Table 3.1 shows the characteristics of these organic polymers.

**Table 3.1:** Characterization of the organic polymers used in the experimental study

| **Type of polymer** | **Description** | **Molecular weight ($g.mol^{-1}$)** | **Degree of charge (weight%)** | **Calculated chain lenght (µm)** [1] |
|---|---|---|---|---|
| Cationic low molecular (1) | - linear polyamine<br>- water-in-oil emulsion | $0.25 \cdot 10^{6}$ | (+) 100% | 0.6 |
| Cationic high molecular (4) | - linear polyacrylamides<br>- powders | $4 \cdot 10^{6}$ | (+) 24% | 18.5 |
| | | $6 \cdot 10^{6}$ | (+) 24 / 38% | 27.8 / 25.3 |
| | | $8 \cdot 10^{6}$ | (+) 24% | 37.0 |
| Anionic high molecular (1) | - linear polyacrylamide<br>- powder | $6.5 \cdot 10^{6}$ | (-) 20% | 34.6 |

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

**Jar test set-up** - Jar tests were conducted in a set up with 6 stirred beakers of 2.5-l capacity (fig. 3.8). The beakers were filled with 2.0 l of wastewater. Portions of dissolved poly electrolyte were dosed simultaneously to all beakers under rapid coagulation mixing conditions (mixing intensity 300 rpm ($G = 800\ s^{-1}$); 120 s for LMW polymers; 20 s for HMW polymers). After coagulation mixing, slow flocculation mixing was applied (50 rpm ($G = 50\ s^{-1}$); t = 180 s). The supernatant after 900 s of settling was analyzed for turbidity, pH and total suspended solids (TSS).

Furthermore, one of the jars was left without polymer, stirred and settled to determine the settleable fraction of turbidity and TSS.

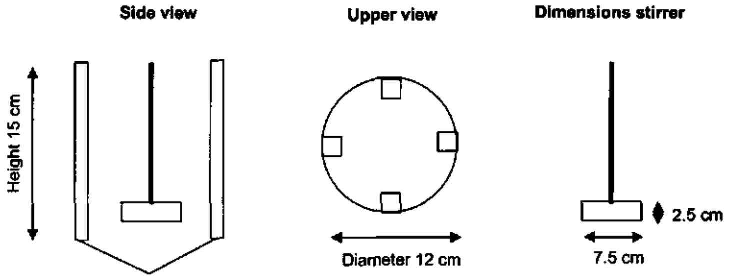


**Fig. 3.9**: Dimensions jar test set-up

**Analysis** - Turbidity of untreated wastewater and the supernatant after coagulation/flocculation and settling was measured with a laboratory turbidity meter (*HACH*). COD, total suspended solids, pH and temperature were determined according to Standard Methods (APHA, 1998).

The COD of both untreated and treated wastewater was fractionated and determined for three wastewater samples: untreated, filtered through a 4.4 µm filter (*Schleicher&Schuell, S&S*) and filtered through a 0.45 µm membrane filter (*S&S*). A fractionation was made in $COD_{ss}$, $COD_{col}$ and $COD_{dis}$, with $COD_{ss}$ = untreated –membrane filtered; $COD_{col}$ = paper filtered – membrane filtered; $COD_{dis}$ = membrane filtered ($mgO_2.l^{-1}$).

**Wastewater characteristics** - For the experiments, wastewater from the town of Bennekom (The Netherlands) was used. Table 3.2 gives an overview of relevant parameters.

**Table 3.2**: Wastewater characteristics of the influent of WTP Bennekom (June 1998 - March 1999)

| Parameter | Average concentration | Fluctuation |
|---|---|---|
| Turbidity | 130 NTU | 75 – 220 NTU |
| $COD_{total}$* | 525 $mgO_2.l^{-1}$ | 220 – 670 $mgO_2l^{-1}$ |
| Total Suspended Solids (TSS) | 160 $mgTSS.l^{-1}$ | 100–300 $mgTSS.l^{-1}$ |
| pH | 7.9 - | 7.4 - 8.5 - |

* 59 % as particulate COD (> 0.45 µm)

**Photo image analysis** – Microscopic pictures of sludge flocs were taken with a digital camera (*Olympus*) at enlargements of 10-40.

**Filtration-expression experiments** – To study the filterability of the produced sludge the average specific cake resistance (SCR) was measured with the Filtration-Expression cell (FE-cell). The average SCR is a measure for the average filtration resistance. The FE cell is an automated measuring device that is used to record the dynamic dewaterability behavior of sludges. Figure 3.10 shows a schematic set up of the device. The FE cell consists of a 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 was measured. The filtration process in monitored continuously by a computer. Based on the assumption that the sludge cake behaved incompressible, the SCR was calculated by means of the integrated Darcy equation (La Heij *et al.*, 1996).

$$t = \alpha \cdot \frac{\eta \cdot c_v}{2A^2 \cdot \Delta p} \cdot V^2 + \frac{\eta \cdot R}{A \cdot \Delta p} \cdot V$$

with: $\alpha$ - average specific cake resistance ($mg.kg^{-1}$)
$\eta$ - viscosity of the filtrate (Pa.s)
A - surface of the filter medium ($m^2$)
$c_v$ – concentration dry solids in suspension ($kg.m^{-3}$)
$\Delta p$ - pressure difference (Pa)
t - time (s)
V - filtrate volume ($m^3$)
R - resistance of the filter medium ($m^{-1}$)

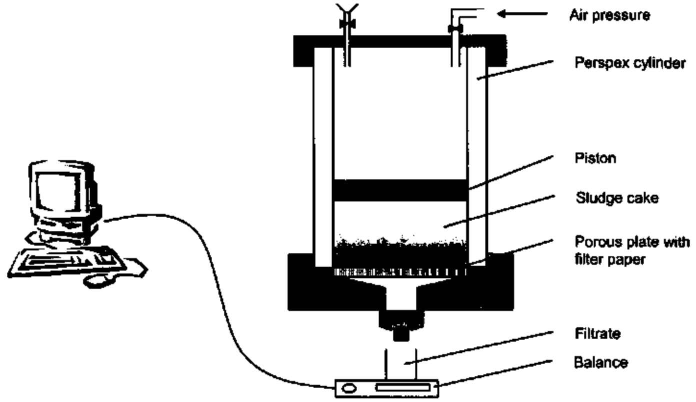


**Fig. 3.10:** Schematic presentation of the FE-cell

## Results of the experiments

Jar tests were conducted to determine the feasibility of the organic polyacrylamide and polyamine polymers mentioned in table 3.2 for coagulation/flocculation of municipal wastewater. The results are shown as the decrease in turbidity and TSS as a function of the polymer dose. During all test series a jar was left without polymer dosing in order to determine the settleable fraction of turbidity and TSS. These fractions are indicated in the graphs as gray blocks.

**Coagulation/flocculation with a LMW cationic polymer** – In figure 3.11 the turbidity and suspended solids removal after coagulation/flocculation and settling with the tested LMW cationic polymer are given. The turbidity decreased from 225 NTU to 20-40 NTU, the suspended solids concentration from 330 to < 15 mgTSS.l$^{-1}$. The settleable part of turbidity and suspended solids was 65 NTU and 225 mg TSS.l$^{-1}$, respectively. The produced sludge flocs were relatively small (see also fig. 3.19) and settled relatively slowly.

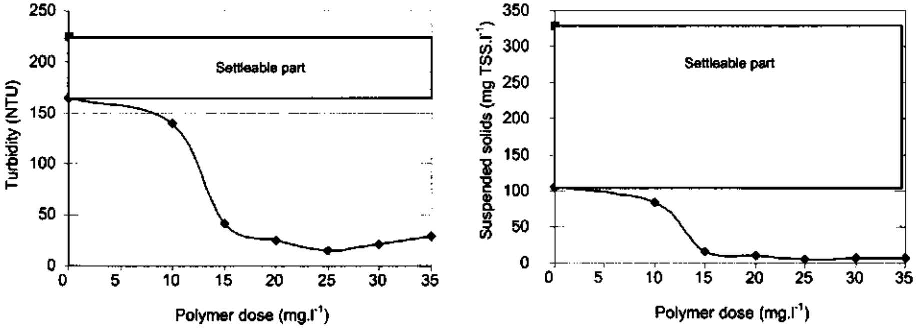


**Fig. 3.11**: Turbidity removal (left) and TSS removal (right) after coagulation/flocculation and settling with a LMW cationic polymer (0.25·10$^{6}$ g.mol$^{-1}$; charge +100%)

For the LMW cationic polymer overdosing may result in an increase of the turbidity, as is shown in figure 3.12. In this example the optimum dosage was somewhat higher than 30 mg.l$^{-1}$. A higher dose resulted in an increase of the supernatant turbidity. In a number of experiments different optimum doses were found, depending on the strength of the wastewater. The average optimum dosage of these experiments was 25 mg.l$^{-1}$.

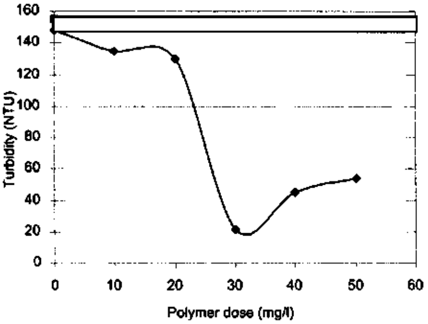


**Fig. 3.12:** The effect of overdose for the dosage of a LMW polyamine ($0.25{\cdot}10^{6}$ g.mol$^{-1}$)

**Coagulation/flocculation with HMW cationic polymers** – In figure 3.13 a jar test experiment with a HMW cationic polymer of $6{\cdot}10^{6}$ g.mol$^{-1}$ is shown. The figure exhibits that this type of polymer results in significant particle removal at relatively low dosing rates. At a dose of 5 mg.l$^{-1}$ the turbidity of the supernatant decreased to < 40 NTU, while the suspend solids concentration amounted less than 10 mgTSS.l$^{-1}$.

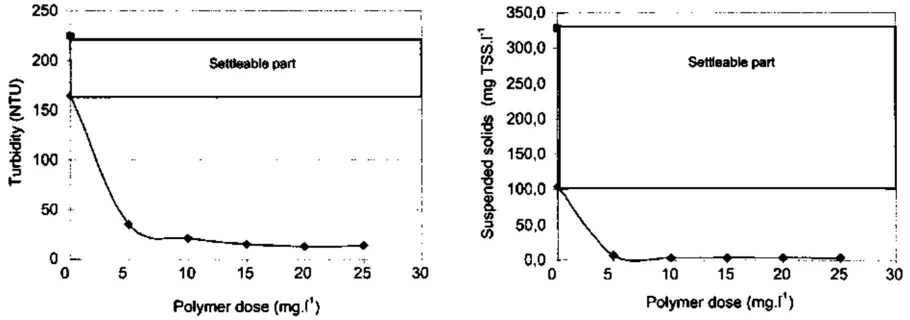


**Fig. 3.13:** Turbidity removal (left) and TSS removal (right) after coagulation/flocculation and settling with a HMW cationic polymer with a molecular weight of $6{\cdot}10^{6}$ g.mol$^{-1}$ and +24% charge

In figure 3.14 an experiment is shown in which three HMW polymers with different molecular weights were compared. 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$^{-1}$) exhibited a lower turbidity removal than both other polymers.

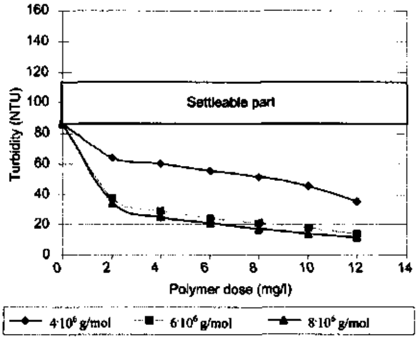


**Fig. 3.14:** Turbidity removal after coagulation/flocculation and settling with HMW cationic polymers with increasing molecular weight ($4\cdot10^6$, $6\cdot10^6$ and $8\cdot10^6$ g.mol$^{-1}$; charge 24%)

The degree of charge determines the amount 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 3.15 shows the turbidity and suspended solids removal of a jar test experiment with two HMW polymers with different charge, respectively +24 weight percent and +38 weight percent. A negligible difference in turbidity was found in favor of the 24 weight percent polymer.

An additional interesting observation from both fig. 3.14 and 3.15 is that the removal of turbidity increases gradually with increasing dosage.

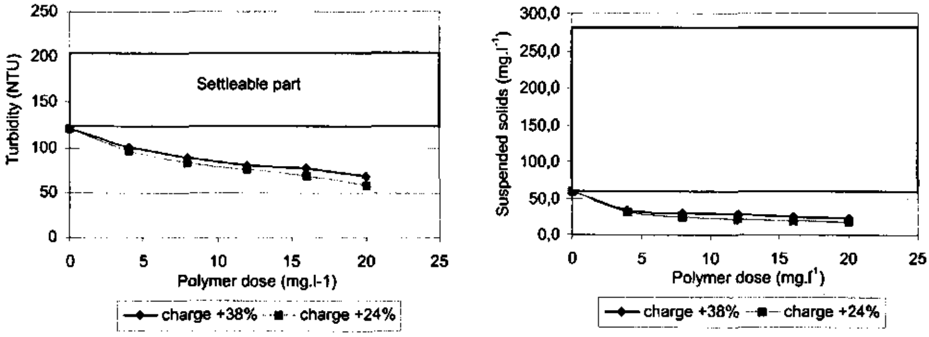


**Fig. 3.15:** Effect of charge: turbidity removal (left) and TSS removal for two cationic polymers with a different charge and similar molecular weight ($6\cdot10^6$ g.mol$^{-1}$)

**Coagulation/flocculation with HMW anionic polymers** – In figure 3.16 the turbidity and suspended solids removal after coagulation/flocculation and settling with a HMW anionic polymer of $18.5\cdot10^6$ g.mol$^{-1}$ is shown. For this polymer hardly any effect was measured. Based on the earlier mentioned fact that particles in municipal wastewater are mainly negatively charged it is obvious that anionic polymer will have hardly any effect in flocculation.

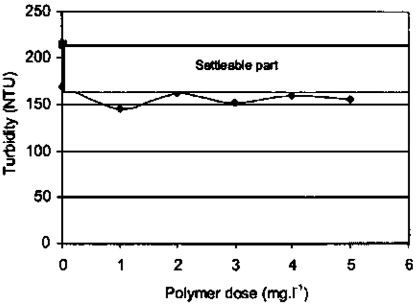


**Fig. 3.16:** Turbidity removal after coagulation/flocculation and settling with an anionic polymer (molecular weight 18.5·$10^6$ g.$mol^{-1}$; charge –30 weight%)

**COD and pH of the flocculated wastewater** – Figure 3.17 shows the result of COD measurements after coagulation/flocculation and settling with a HMW cationic polymer. Both the removal of $COD_{total}$, and the removal of the fractions $COD_{ss}$, $COD_{col}$ and $COD_{dis}$ are presented (left and right, respectively). The COD of the influent decreases from 525 $mgO_2.l^{-1}$ (settled 420 $mgO_2.l^{-1}$) to 276 $mgO_2.l^{-1}$ at a dosage of 4 mg polymer per liter influent. The fractionation shows that for the given dosage range the $COD_{ss}$ shows a removal of 42% (no dosage) to > 90% at a dosage of 4 mg.$l^{-1}$, while the $COD_{col}$ removal increases from 2% to 35%.

**Fig.3.17:** COD of the supernatant after coagulation/flocculation and settling with increasing dosages of a HMW cationic polymer (8·$10^6$ g.$mol^{-1}$, +24%); left: $COD_{total}$ removal; right: removal of COD fractions

The addition of organic polymers did not affect the pH of the supernatant, as is illustrated by figure 3.18. This figure presents the pH after coagulation/flocculation and settling with a HMW cationic polymer. The addition of a LMW polymer neither showed an effect.

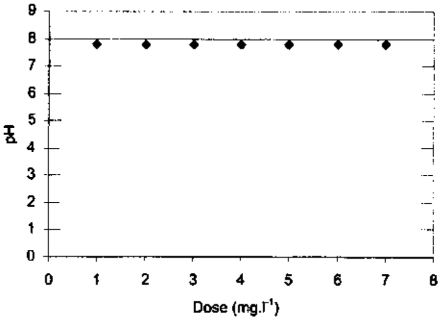


**Fig.3.18:** pH of the supernatant after coagulation/flocculation and settling with increasing dosages of a HMW cationic polymer (8·$10^6$ g.mol$^{-1}$, +24%)

**Floc size characteristics, settleability and filterability of the produced sludge** – In figure 3.19 photos of sludge flocculated with two cationic organic poly electrolytes are depicted (note: the bar in the right picture is 2 mm). The left picture shows the LMW cationic organic polymer, the right picture exhibits a HMW cationic polymer. In comparing both sludge types, the effect of chain length is demonstrated. The first polymer had a calculated chain length of 0.6 µm, the second polymer had a chain length of 37 µm. The pictures show that coagulation/flocculation with the HMW polymer resulted in relatively large flocs compared to the LMW polymer.

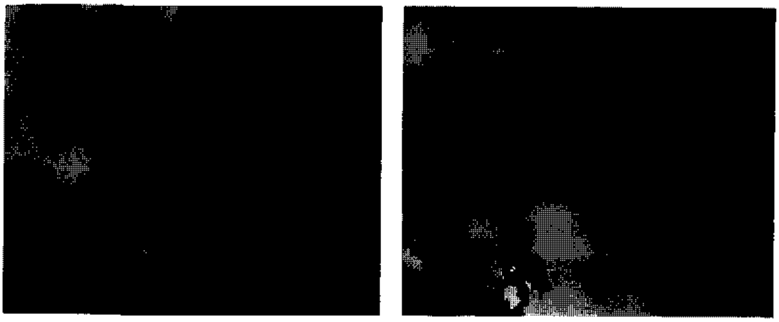


**Fig. 3.19:** Comparison of sludges flocculated by two cationic organic poly electrolytes; left: LMW cationic polymer 0.25·$10^6$ g.mol$^{-1}$, +100% (30 mg.l$^{-1}$); right: HMW cationic polymer 8·$10^6$ g.mol$^{-1}$, +24% (10 mg.l$^{-1}$). Bar = 2mm

In figure 3.20 a settling curve is given for the polymers of figure 3.19. The figure also includes untreated wastewater. In the experiment samples were taken at 10 cm beneath the water surface. The curve illustrates the effect of floc size on the settling velocity. The sludge flocs that were formed by the LMW polymer showed a lower settling velocity than the flocs formed by the HMW polymer.

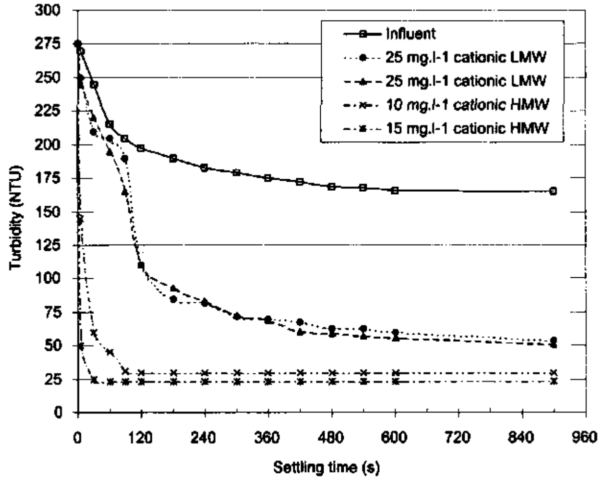


**Fig.3.20:** Settling curves of untreated wastewater and wastewater treated with LMW and HMW polymers; (LMW cationic polymer: $0.25 \cdot 10^6$ g.mol$^{-1}$, charge +100%; HMW anionic polymer $6.5 \cdot 10^6$ g.mol$^{-1}$, charge –30%, HMW cationic polymer: $8 \cdot 10^6$ g.mol$^{-1}$, +24%)

In figure 3.21 the average specific cake resistance (SCR) is given as a function of the flocculant dose. The dosage is recalculated to the flocculant dosage per gram of sludge solids (mg.gTS$^{-1}$). The figure shows the average SCR of settled sludge of untreated wastewater and settled sludge of experiments with increasing doses of HMW cationic polymer. Settled sludge of wastewater treated with ferric salt (15 mgFe.l$^{-1}$) is shown as a reference.

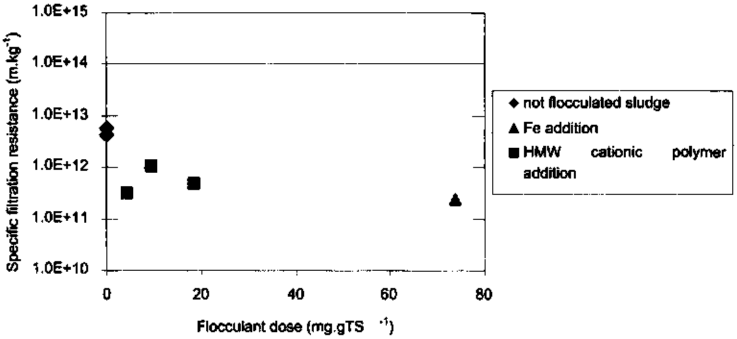


**Fig.3.21:** Average specific cake resistance of the settled sludge of flocculated and not-flocculated wastewater samples

The flocculated sludge samples showed a lower average SCR than the untreated sludge. Polymer and ferric addition improved the filterability of the produced sludge to a similar degree. The improved dewaterability characteristic can be explained by (partial) charge neutralization and by the formation of larger aggregates.

## Discussion of the experimental results

The jar test experiments demonstrate that cationic organic poly electrolytes can be applied for coagulation/flocculation of municipal wastewater. As expected the use of an anionic polymer did not show significant turbidity removal. In table 3.3 a summary of the experimental results is presented.

**Table 3.3:** Applicable organic polymers for coagulation/flocculation of municipal wastewater

| **Type of polymer** | **Assumed coagulation/flocculation mechanism** | **Dosage ($mg.l^{-1}$)[a]** | **Final value turbidity (NTU)** | **Final value suspend solids ($mg\ TSS.l^{-1}$)** |
|---|---|---|---|---|
| LMW cationic ($0.25 \cdot 10^6\ g.mol^{-1}$) | electrostatic coagulation | 15 - 30 | 15-25 | < 5 |
| HMW cationic ($> 6 \cdot 10^6\ g.mol^{-1}$) | bridging flocculation | 2 - 5 | 50-70 | 10 - 20 |
| | | 10 - 15 | 15-50 | < 10 |

[a] Indicative value: based on the wastewater used in the experiments

An overdose of HMW cationic polyamine resulted in a decrease in turbidity removal compared to lower doses (fig. 3.12). After a certain minimum concentration (25 - 35 $mg.l^{-1}$) a sudden decrease 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 becomes positive, which entails 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.

Unlike the dosage of metal-based flocculating agents, no pH effect was found for the dosage of polymer. The dosage of a metal salt may result in a strong decrease of the pH. In parallel experiments a pH drop of 7.8 to 3.8 was found when 15 $mg.l^{-1}$ poly aluminium chloride was applied. Such a pH drop will have a strong effect on the performance of a secondary biological treatment system, because especially nitrification is very pH sensitive.

**Chloride content of organic polymers** – Three out of the four HMW cationic polymers that were used in the experiments had a charge density of 24 weight percent. 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 chloride content can be calculated based on the weight of the monomers. The molecular weight of a cationic monomer is 163.4 $g.mol^{-1}$, while a non-ionic monomer weighs 71 $g.mol^{-1}$. 24 weight percent equals to $1.47 \cdot 10^{-3}$ mol of cationic monomer per gram polymer. Since per cationic monomer one chloride ion is released the chloride content per mg cationic polymer is $1.47 \cdot 10^{-3} * 35.4$ g $Cl^{-}.mol^{-1} = 0.05$ mg $Cl^{-}$.

## Conclusions

The literature review revealed that two types of organic polymers are most frequently applied in (industrial) wastewater treatment and sludge handling. These polymers are (i) low to moderate molecular weight polyamines and poly-DADMACs and (ii) high molecular weight (HMW) polyacrylamides. Jar test experiments showed that from the tested cationic polyamine- and polyacrylamide-based organic polymers, especially the cationic HMW polyacrylamides appeared to be interesting for practical application, as they got effective at relatively low doses (5-10 $mg.l^{-1}$). For a low molecular weight (LMW) cationic polyamine also high turbidity removals were found, however at higher required doses (20-30 $mg.l^{-1}$). For both types of polymers the experiments resulted in turbidity removals of 65-90% and a suspended solids removal of more than 90%. Furthermore, it was found that the addition of organic polymers does not affect the pH of the treated wastewater.

It was found that the turbidity removal increases gradually with increasing dose, when cationic HMW polyacrylamides are dosed. This may imply an interesting possibility to control the particle removal of unit operations for pretreatment.

The addition of HWM cationic polymers tends to result in large, well-settling flocs. The sludge flocs formed after addition of LMW cationic polymers appeared to be significantly smaller. Furthermore, filterability tests with the Filtration-Expression cell showed that the improvement in dewaterability of sludge produced by the addition of polymers is similar to that of sludge produced by the addition of ferric chloride.

## References

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# 4 Pretreatment of municipal wastewater by dissolved air flotation combined with the use of organic polymers

**Abstract.** This chapter describes investigations into the application of dissolved air flotation (DAF) using organic poly electrolytes as a potential pretreatment unit operation for municipal wastewater. The application of DAF may have several advantages over a system that combines flocculation and settling. DAF implies a relatively high particle removal in a relatively compact reactor configuration, while the removed particles tend to be concentrated in a float layer with a high dry solids content.

The investigations showed that DAF combined with the use of HMW cationic polymers forms an effective unit operation to remove particles from municipal wastewater. In batch flotation experiments a turbidity removal of 75% was found at poly electrolyte doses of 4-6 mg.$l^{-1}$. The results indicate that - at least for jar test experiments - the particle removal performances of dissolved air flotation and settling are the same. A continuous experiment with a DAF pilot plant showed a lower turbidity removal efficiency (maximal 70%), which was due to suboptimal mixing of the polymer with the wastewater. The dry solids concentration of the flotation sludge that was produced by the DAF unit numbered 3-5% for continuous float layer removal and 8-10% for discontinuous float layer removal. The latter value is significantly higher than the concentrations found in literature.

The experiment with the continuous DAF unit showed that the wastewater $COD_{particulate}$ forms an important parameter, to decide how much polymer should be dosed to a continuously operating pretreatment system. This parameter may vary in time, resulting in an unpredictable particle removal efficiency if a fixed polymer dose is applied. Online turbidity measurements could be a potential tool to monitor the $COD_{particulate}$ of the wastewater, because a linear relation was found between both parameters.

## Introduction

As argued in chapter 2, dissolved air flotation (DAF) has favourable characteristics as a unit operation for pretreatment of municipal wastewater. The financial calculations of treatment scenarios showed that the total costs for scenarios with DAF are in the same range as those of scenarios that include flocculation and settling. This conclusion is in line with the conclusions of Farell *et al.* (2000) who found that the costs of total investment for a DAF unit are lower than for a primary clarifier.

The application of DAF is likely to have several advantages over a system that combines flocculation and settling. DAF generally implies a relatively high particle removal in a relatively compact reactor configuration. Moreover, the removed particles tend to be concentrated in a float layer with a high dry solids content (2 - 4% according to Krofta and Burgess, 1997), which is an advantage with regard to further sludge handling.

As far as research and practical data are concerned, it seems that little work has been done on the application of DAF as a first step in municipal wastewater treatment (Krofta *et al.*, 1995; Krofta and Burgess, 1997; Cáceras and Contreras, 1995; Ødegaard, 1995; Farell et al., 2000). The main current applications of DAF in municipal wastewater treatment are as final effluent polishing step (e.g. Van Vuuren and De Wet, 1981) or as a method for sludge thickening (e.g. Van Vuuren and Van der Merwe, 1989; Bratby and Ambrose, 1995).

Usually in wastewater treatment, the flotation step is preceded by coagulation/flocculation in order to enlarge the flocs as well as change the surface characteristics of the particles (as the hydrophobic flotation bubbles only attach to hydrophobic surfaces). The reviewed research and data on the application of DAF all used inorganic metal-based flocculating agents. As pointed out in chapters 2 and 3, metal-based chemicals produce a relatively large amount of inorganic sludge and deteriorate the quality of the effluent due to release of counter ions (and thus increase in salinity). Therefore, for the DAF experiments as described in this chapter, organic poly electrolytes were used for flocculation.

The objective of the experiments was to investigate DAF using organic poly electrolytes for the removal of particles from municipal wastewater with special reference to (i) the applicability of different types of poly electrolytes; (ii) the efficiency of the flotation process for removal of total COD, particulate COD, nitrogen and phosphorous; (iii) the required amounts of poly electrolytes for optimal flotation; and (iv) the dry solids concentration of the flotation sludge. Moreover, a comparison of treatment efficiency was made between the efficiency of DAF and settling as a pretreatment step.

## Material and methods

**Batch flotation experiments** - Batch flotation experiments were conducted in a set up with 6 stirred beakers of 2.5 l. The beakers were filled with 1.5 l wastewater. Cationic poly electrolyte was dosed simultaneously to all beakers at intensive mixing conditions (mixing intensity G = 800 $s^{-1}$). After 20 s of intensive mixing slow flocculation mixing was applied (G = 50 $s^{-1}$). Tap water was saturated with air at a pressure of 5 bar in an air saturation vessel (20 l) to obtain flotation water. After 90 s of flocculation mixing 0.2 l the flotation water was dosed simultaneously to each beaker (i.e. 13% of the initial volume). After 900 s flotation time a sampling device was carefully put through the formed float layer in order to take samples from the flotate (i.e. remaining water after flotation).

**Continuous DAF pilot system** - For the continuous experiments a 100 l pilot scale DAF system was used. A process scheme of the system is given in figure 4.1. Prior to flotation, a cationic poly electrolyte was dosed to the influent in a static mixer. The static mixer (i.e. a pipe flocculator) consisted of two parts, a first part for intensive coagulation mixing and a second part for slow flocculation mixing. Air saturated water was added at the end of the static mixer and in the inlet of the flotation reactor. After the inlet the water flowed through a lamella packed plate bed which enhanced bubble-water separation. The formed float layer was removed by a chain scraper and collected in the sludge collection trough.

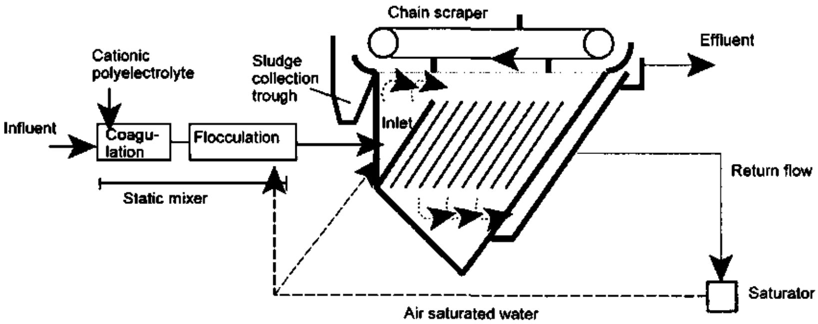

**Fig. 4.1**: Scheme of the DAF pilot plant

During the experimental period the flow was adjusted to 300 $l.h^{-1}$. This was equal to a hydraulic surface loading of 1.2 $m.h^{-1}$, based on the lamella surface (0.25 $m^2$) or a hydraulic load of 10 $m.h^{-1}$ based on the cross section of the reactor. Some 20% of the effluent was brought in again for recirculation through an air saturator at a pressure of 5 bar. The mixing conditions of the static mixer at the applied influent flow and effluent recirculation ratio are given in table 4.1.

**Table 4.1:** Mixing intensity and retention time in the static mixer during the experimental period

| | **Intensive coagulation mixing** | **Slow flocculation mixing** |
|---|---|---|
| Mixing intensity (G) | 704 $s^{-1}$ | 84 $s^{-1}$ |
| Mixing time | 0.6 s | 23.7 s |

**Wastewater characteristics** - For the batch and continuous experiments municipal wastewater of the town of Bennekom was used. Table 4.2 gives an overview of relevant parameters. For all components the particulate fraction (> 0.45 µm) was measured.

**Table 4.2:** Characteristics of the investigated wastewater (October 1998 - March 1999)

| **Parameter** | **Average concentration** | | **Fluctuation** | | **From which in particulate form *** |
|---|---|---|---|---|---|
| Turbidity | 130 | NTU | 75 - 220 | NTU | -- |
| $COD_{total}$ | 525 | mg $O_2.l^{-1}$ | 220 - 670 | mg $O_2.l^{-1}$ | 59% |
| $BOD_5$ | 230 | mg $O_2$ .$l^{-1}$ | 215 - 265 | mg $O_2$ .$l^{-1}$ | 61% |
| N-Kjeldahl | 75 | mg N.$l^{-1}$ | 60 - 90 | mg N.$l^{-1}$ | 12% |
| $P_{total}$ | 10 | mg P.$l^{-1}$ | 9.1 - 10.8 | mg P.$l^{-1}$ | 31% |
| pH | 7.9 | - | 7.4 - 8.5 | - | -- |
| Temperature | 15 | °C | 12 - 18 | °C | -- |

* particulate > 0.45 µm

**Organic poly electrolytes** - For coagulation/flocculation two different linear polyacylamide-based cationic poly electrolytes were tested (*CYTEC*). The poly electrolytes had a similar charge of (+) 24 weight percent, but different molecular weights of respectively $4*10^6$ $g.mol^{-1}$ and $8*10^6$ $g.mol^{-1}$.

**Analysis** - For the batch flotation experiments the turbidity of influent and flotate was measured (by a HACH Turbidimeter). For the continuous experiments composite samples (4 - 6 h) of both influent and effluent were taken and analysed for turbidity, COD, N-Kjeldahl and $P_{total}$. The float layer in the continuous experiment was collected from the sludge collection trough and the dry solids concentration was measured. All analyses were performed according to Standards Methods (APHA, 1998).

The COD ($COD_{total}$) of both untreated and treated wastewater was fractionated and determined for two wastewater samples: untreated and filtered through a 0.45 µm membrane filter (*Schleicher&Schuell*). A fractionation was made in $COD_{particiulate}$ and

$COD_{dissolved}$, with $COD_{particulate}$ = untreated – membrane filtered; $COD_{dissolved}$ = membrane filtered ($mgO_2.l^{-1}$).

## Flotation with organic poly electrolytes in batch experiments

For each poly electrolyte two series of batch flotation experiments were carried out to test the turbidity removal efficiency at increasing poly electrolyte dosage per liter. Figure 4.2 presents the combined results of all series:

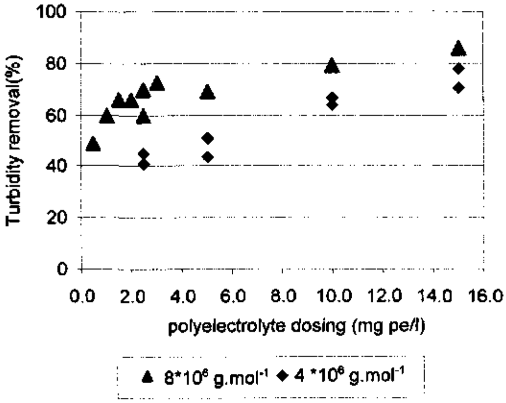

**Fig. 4.2:** Turbidity removal in batch flotation experiments after flocculation with two poly electrolytes of different molecular weight

The graph clearly shows that flocculation-flotation with the $8*10^6$ g.mol$^{-1}$ poly electrolyte resulted in a higher turbidity removal compared to the poly electrolyte with a molecular weight of $4*10^6$ g.mol$^{-1}$. This complies with the results of chapter 3, which investigated the removal performance in combination with settling. As shown, for both poly electrolytes a higher turbidity removal efficiency was found at increasing doses.

## Flotation in the continuous DAF pilot system

The DAF pilot system was operated for 6 weeks treating municipal wastewater. Based on the positive results of the batch experiments the $8*10^6$ g.mol$^{-1}$ poly electrolyte was used as coagulant/flocculant in this period. During the continuous experiments the poly electrolyte was dosed proportional to the influent volume. Dosages of 1, 2 and 4 mg pe.l$^{-1}$ were applied.

**Removal of different components** - In figure 4.3 and 4.4 the turbidity removal efficiencies during the experimental period are shown. Figure 4.3 presents the removal as a function of the polymer (pe) dose per liter wastewater. The graph shows that the deviation in removal efficiency was rather high. Moreover, the average removal at 4 mg.$l^{-1}$ was approximately the same as or lower than the average removal at 2 mg pe.$l^{-1}$. This contradicts the results that were found in the batch flotation experiments (increasing doses result in a higher removal). However, these results can be explained by the fact that the $COD_{total}$ of the influent fluctuated considerably during the experimental period. The fluctuations resulted in different polymer doses per mg of $COD_{total}$ (or per mg of $COD_{particulate}$). Therefore, in figure 4.4 the turbidity removal efficiency is plotted as a function of the dosage per gram of influent $COD_{total}$ The graph shows that by plotting the data in this way, the turbidity removal increased with increasing dose, as was found in the batch experiments.

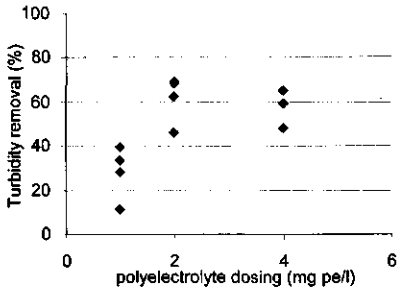

**Fig. 4.3**: Turbidity removal efficiency in the pilot experiments at different doses of pe

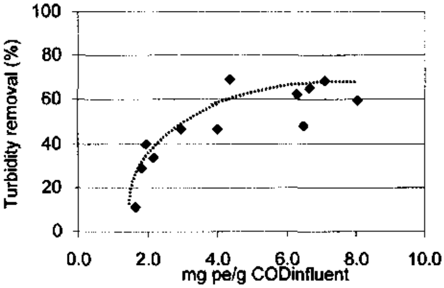

**Fig. 4.4**: Turbidity removal as function of the polymer-$COD_{influent}$-ratio (same data)

Figure 4.5 presents the absolute removal of $COD_{particulate}$ and $COD_{dissolved}$ (in mg.$l^{-1}$), plotted against the polymer-$COD_{influent}$ ratio. The graph shows that the removal of $COD_{particulate}$ increases with mounting dosage. The removal of $COD_{dissolved}$ was small and (as can be expected) independent from the polymer dosage. The average $COD_{dissolved}$ removal was 24 mg.$l^{-1}$ (range 10 - 35 mg.$l^{-1}$), i.e. 11% of $COD_{dissolved}$ of the influent.

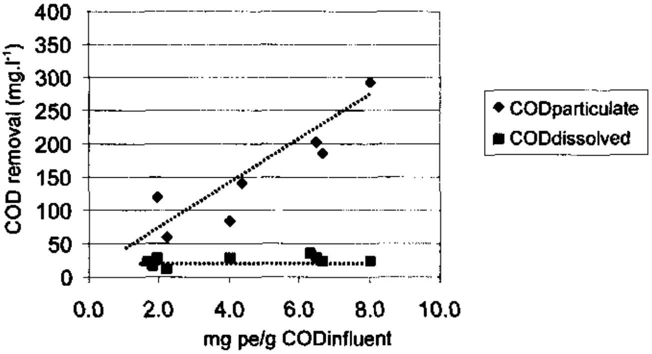

**Fig. 4.5:** The absolute removal of $COD_{particulate}$ and $COD_{dissolved}$ as a function of the polymer-$CODi_{nfluent}$ ratio

The removal efficiencies of $P_{total}$ and N-Kjeldahl were 23% and 8% respectively, at a dosage of 6.5 mg pe/g CODinfluent. This is in line with the particulate fractions given in table 4.2. The relatively low removal of N-Kjeldahl is due to the low fraction of particulate N in the investigated wastewater.

**Sludge quality** - One of the interesting features of the application of DAF as a unit operation for pretreatment is its ability to produce a relatively concentrated sludge. This is the result of the fact that the float layer, which is formed during the flotation process, is partially pushed above the water surface by the continuous flow of air bubbles. The float layer is drained resulting in an increase of the dry solids concentration. Within the context of the application of DAF as pretreatment step for wastewater this appears attractive, since smaller volumes of (relatively concentrated) sludge are being produced.

The dry solids concentration of the float layer was regularly measured during the experiments. The results are shown in figure 4.6. Sludge samples were taken from the sludge collection trough after which the dry solids concentration was determined directly. A part of the sludge was gently put on a sieve for 5 minutes and drained by gravitation. After that the dry solids content of these samples was also determined. The objective of this drainage procedure was to get an indication of the maximal achievable dry solids concentration.

During the first experimental week, the chain scraper was operated continuously. Hardly any float layer was formed during this period, because individual flocs were immediately removed. The dry solids concentration of the collected sludge ranged from 2.9 to 4.8%

(average 3.6%). The measurements after 5 minutes sieving (average 8.3%) showed that the maximum dry solids concentration had not yet been reached.

During the second period, the scraper removed the float layer every 2.5 to 4 h. The result was a coherent float layer of 3 - 6 cm. The dry solids concentration of this layer varied from 7.5 to 10.2% (average value 9.4%). After 5 minutes of drainage on the sieve the dry solids concentration ranged from 7.6 to 11.6% (average value 9.5%). The similar average values indicate that the maximal achievable dry solids concentration was reached. The achieved concentrations were significantly higher than the dry solids concentrations reported by Krofta and Burgess (1997) who performed experiments using metal-based flocculating agents.

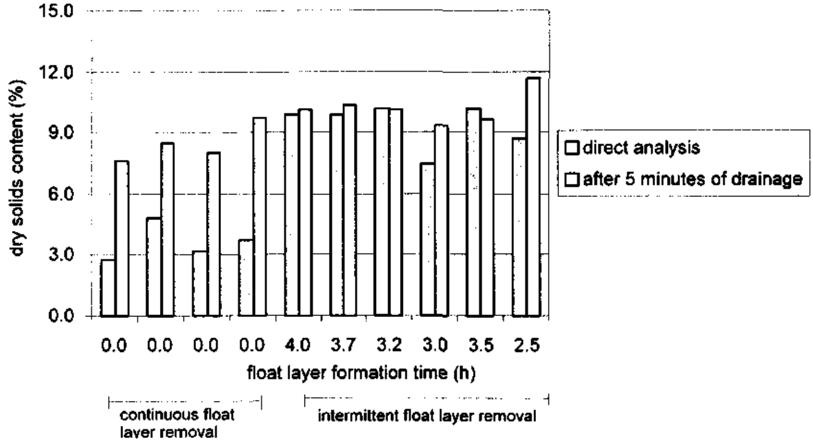

**Fig. 4.6:** Dry solids content of the sludge for continuous and intermittent float layer removal; direct analysis and analysis after 5 minutes of drainage

## Discussion

**Dosing of polymer in case of fluctuating influent $\mathrm{COD}_{particulate}$** - The continuous experiments showed that, due to fluctuations in $\mathrm{COD}_{total}$ and $\mathrm{COD}_{particulate}$ of the wastewater, the polymer dose at fixed concentration per liter resulted in a turbidity removal which was not expected. By plotting the turbidity removal efficiency as a function of the dosage per gram of influent $\mathrm{COD}_{total}$ (fig. 4.4 and 4.5) a relation was found that was more in compliance with the batch experiments, i.e. increasing turbidity removal efficiency at increasing polymer doses. As the removal of $\mathrm{COD}_{dissolved}$ was low and independent from dose a similar result may be expected if the data are plotted against the polymer dose per gram of $COD_{particulate}$.

In a data analysis, a linear relation between $COD_{particulate}$ and turbidity was found for both the influent and the DAF effluent values. This relation is shown in figure 4.7. The graph indicates that (online) turbidity measurements can be used to quantify $COD_{particulate}$ (which is a relative time consuming and costly analysis) and thus to monitor the efficiency of the flocculation-flotation process.

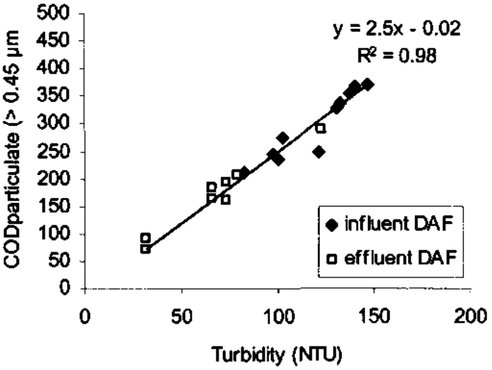

**Fig. 4.7:** Relation between particulate COD (> 0.45 µm) and turbidity of the tested wastewater

Based on the figures 4.4 and 4.7, the removal of turbidity can be plotted as a function of the polymer to influent-turbidity ratio (polymer-$NTU_{influent}$ ratio), analogous to the pe-$COD_{influent}$-ratio of figure 4.4. This method is illustrated by figure 4.8 in which the results of the batch flotation experiments of figure 4.2 are plotted. The graph gives a good picture of the difference in particle removal efficiency of the two pe's.

The method offers a potential dosing strategy for organic polymer, based on continuous monitoring of the influent turbidity. By dosing the polymer based on a fixed pe-$NTU_{influent}$-ratio a constant turbidity removal might be achieved. In figure 4.8 an imaginary set point of 5 mg/100 NTU is chosen which (for the $8*10^6$ $g.mol^{-1}$ polymer) would result in a turbidity removal of 80%.

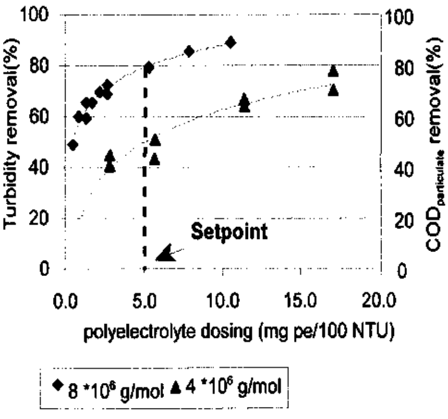

**Fig. 4.8:** Evaluation of the effectiveness of two pe's by plotting the $COD_{particulate}$-removal efficiency as a function of the pe-$NTU_{influent}$-ratio

### Comparison of the turbidity removal by settling and by dissolved air flotation -

Plotting the turbidity removal as a function of the polymer-$NTU_{influent}$ ratio can also be used to evaluate jar test experiments which have a different initial wastewater turbidity. Based on the pe-$NTU_{influent}$-ratio a comparison was made between the experiments with settling, as described in chapter 3, and the batch flotation experiments conducted within the framework of this chapter. The comparison is presented in figure 4.9. The figure shows that a similar turbidity (or $COD_{particulate}$) removal efficiency was found for both settling and dissolved air flotation.

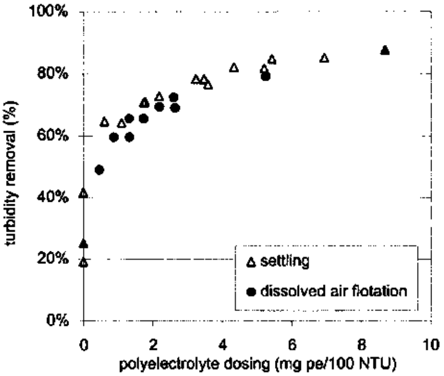

**Fig. 4.9:** Comparison of the turbidity removal for settling and dissolved air flotation in batch experiments

**Comparison of the continuous and the batch flotation experiments** - In a similar way, a comparison can be made between the removal efficiencies of the continuous and the batch flotation experiments. In figure 4.10 the turbidity removal efficiencies of both series of experiments are compared. The comparison shows that in the DAF pilot unit less turbidity is removed at lower doses. When the process conditions of the continuous and the batch experiments are compared, especially the difference in rapid mixing time becomes apparent. In the continuous experiments a mixing time of 0.6 s was applied, while in the batch experiments 20 s was used.

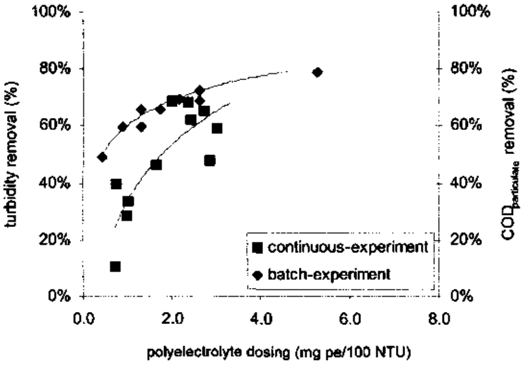

**Fig. 4.10:** Comparison of the turbidity removal efficiency of the continuous and the batch experiments

To investigate the effect of the mixing conditions on the removal efficiency, additional jar tests with settling were performed. The results are presented in figure 4.11. The figure demonstrates that decreasing the coagulation mixing time (fig. 4.11a) as well as decreasing the flocculation mixing time (fig. 4.11b) resulted in a reduced turbidity removal efficiency. The coagulation mixing time affects the performance considerably, as is indicated by the left picture. A difference of 15% in removal efficiency was found when the mixing time was reduced from 20 s to 2 s (note: the flocculation mixing time in these experiments was 30 s). It makes the hypothesis plausible that the rapid mixing time was the cause of the lower turbidity removal of the pilot unit. The flocculation time has a strong effect on the removal efficiency as well as is shown in the right picture. The removal efficiencies that were found at 0 s and 30 s flocculation time were approximately 25% lower than those found at 180 s (at velocity gradient G = 50 $s^{-1}$). Based on the performed jar tests, a rapid mixing time of 30 s and a slow mixing time of 180 s appear optimal.

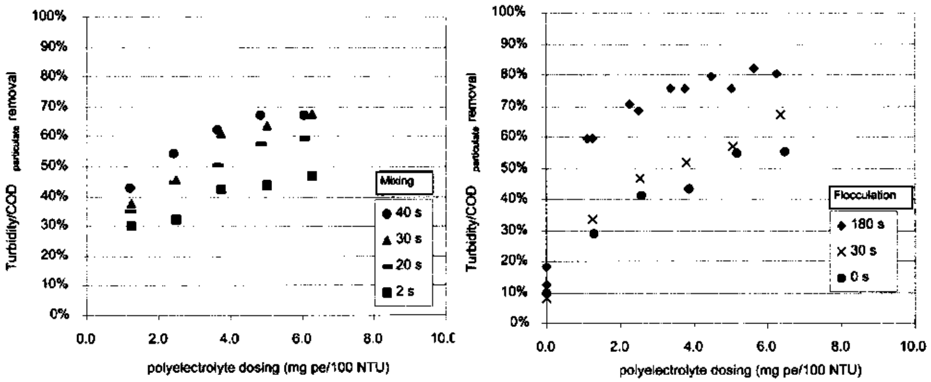

**Fig. 4.11: Left (a):** Effect of coagulation mixing time on particle removal in jar test experiments with settling ($G=800\ s^{-1}$; flocculation mixing: $t = 30\ s$, $G= 50\ s^{-1}$); **Right (b):** Effect of flocculation mixing time ($G = 50\ s^{-1}$; coagulation mixing $t=20\ s$, $G= 800\ s^{-1}$)

## Conclusions

The investigations in this chapter showed that dissolved air flotation (DAF) combined with the use of HMW cationic polymers is an effective unit operation for particle removal from primary wastewater. With regard to the two tested poly electrolytes, the polymer with the highest molecular weight (i.e. $8*10^6\ g.mol^{-1}$) showed the highest turbidity removal.

A turbidity removal of 75% was found in batch flotation experiments, at poly electrolyte doses of 4-6 $mg.l^{-1}$. The results indicate that - at least for jar test experiments - the removal performances of dissolved air flotation and settling are the same.

The continuous experiment showed a lower turbidity removal than the batch experiments (maximal 70%), which was due to suboptimal mixing of the polymer with the wastewater. Additional jar tests showed that by optimizing the mixing conditions the turbidity removal could have been increased (cq. poly electrolyte could be saved). A rapid mixing time of 30 s and a slow mixing time of 180 s appeared optimal.

The dry solids concentration of the flotation sludge amounted 3-5% for continuous float layer removal and 8-10% for discontinuous float layer removal.

The experiments with the DAF unit showed that, to decide which quantities of polymer should be dosed to a continuous system, the $COD_{particulate}$ of the influent is an important parameter. Due to daily fluctuations in $COD_{particulate}$, polymer dose at fixed concentration per liter will result in an unpredictable particle removal efficiency. Online turbidity measurements might be used to monitor the $COD_{particulate}$ of the wastewater, because a linear relation was found between both parameters.

## Acknowledgements

We would like to thank Arie Kluit and Wilbert Menkveld of Nijhuis Water Technology, Dinxperlo, The Netherlands, for providing a DAF pilot system and for giving their valuable feedback on the preliminary results.

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# 5 Testing a turbidity-related dosing strategy for organic polymers

**Abstract.** The objectives of this chapter were (i) to investigate the use of on-line turbidity measurements to monitor the particle concentration in the in- and effluent of a pretreatment step with organic polymers and (ii) to test the application of a feed forward polymer dosing strategy based on a fixed polymer to influent-turbidity ratio.

Experimental results confirm that on-line turbidity measurements can be used to quantify $COD_{particulate}$. For the investigated wastewater (both untreated and flocculated samples) a linear relation was found in a wide range of $COD_{particulate}$ (100-900 mg $O_2.l^{-1}$) and turbidity (50-450 NTU). On-line 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 per gram of polymer compared to the dosage of a fixed concentration of polymer per liter. A turbidity-related dosing strategy appeared more stable compared to dosage of a fixed polymer concentration per liter, because overdose at low influent turbidity was prevented. Turbidity-related dosing offers a tool to control the $COD_{particulate}$ removal and therefore may be interesting from the perspective of denitrification in a down-stream biological treatment system.

## Introduction

The results of chapter 4 indicated, that turbidity measurements can be used to monitor the $COD_{particulate}$ of the influent and effluent of a pretreatment system. Furthermore, the experiments of chapter 4 showed that, due to the fluctuating concentrations in the wastewater, the particle removal of a pretreatment system can be rather unpredictable when a constant polymer dose per liter wastewater is applied. The results however also indicated that a predictable $COD_{particulate}$ removal can be achieved if the polymer dose is adapted to the influent-turbidity at a fixed ratio.

The goals of this chapter are (i) to test the use of on-line turbidity measurements to monitor the particle concentration in the in- and the effluent of a pretreatment step and (ii) to investigate the application of a feed forward polymer dosing strategy based on a fixed polymer to (influent-) turbidity ratio.

Applications of controlled dosage of flocculating agents are known in municipal wastewater treatment as well as in municipal sludge handling (although in both cases rarely applied). Obviously, most of these applications refer to dosing control of metal-based flocculating agents whose flocculation mechanism is governed by colloidal destabilisation. The dosing control is generally aimed to save chemicals, especially during periods when the wastewater contains low concentrations of phosphorus and suspended solids (e.g. during rain weather). The dosing control can be based on different types of continuous analysis, like the continuous reading of suspended solids measured by monochromatic infrared light (Hansen, 1996) or streaming current detectors (Dentel *et al.*, 2000) or the measurement of the rheological characteristics or liquid-stream viscosity of the conditioned sludge (Dentel *et al.*, 2000).

The work described in this chapter consists of three parts. In the first part a further analysis is made into the relation between turbidity and $COD_{particulate}$. In the second part investigations are described into the particle concentration fluctuations in municipal wastewater. Finally, experiments are described with turbidity-related polymer dosing in a pilot scale clarifier and in a lab-scale clarifier. The experiments were conducted with municipal wastewater of the town of Bennekom.

## Material and methods

**Experimental set up and operational conditions** - For the experiments a set-up as presented in figure 5.1 was constructed. The incoming wastewater entered a buffer vessel where the turbidity of the influent was measured by an on-line turbidity meter. This meter sent a signal to a computer (software *Control EG*) that made a control action to the polymer dosing pump. The polymer was dosed to a stirred vessel (1.5 l) and was mixed intensively (300 rpm) with the wastewater. The formed flocs were settled in a clarifier. With a second turbidity meter the turbidity of the clarifier effluent was measured.

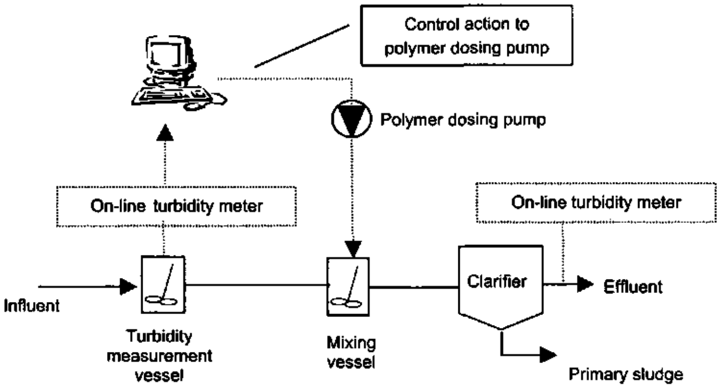

**Fig. 5.1:** Experimental set-up

Two types of settlers were used (figure 5.2): a small 9.5-l settler and a large, 350-l settler (type: Dortmund tank). The experiments with the 350-l settler were performed in parallel with the activated sludge experiments of chapter 6. Figure 5.2 gives a drawing of both settlers. The influent flows that were applied to the clarifiers were 15 $l.h^{-1}$ for the small settler and 90 $l.h^{-1}$ for the Dortmund tank, respectively.

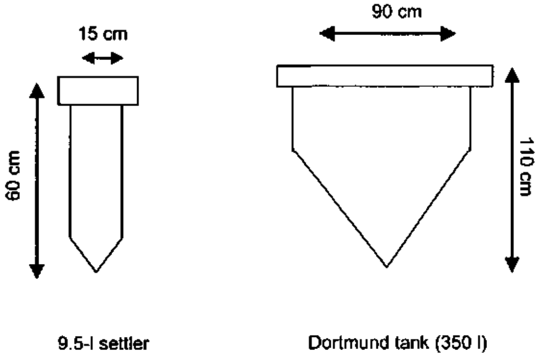

**Fig. 5.2:** Dimensions of the clarifiers used in the experiments

**Dosage control** – *In order to achieve a fixed polymer to (influent-) turbidity ratio, the flow* of the dosing pump was varied from 0 – 100% proportional to the influent turbidity , according to equation 5.1.

$$Flow\,(\%) = Turbidity_{influent}/250\,NTU \bullet 100\% \qquad (5.1)$$

At influent turbidity values > 250 NTU the dosing pump flow was at its maximal rate.

**Characteristics of the organic polymer** - For flocculation a linear cationic polyacrylamide-based polymer (*CYTEC*) was used. The molecular weight of the polymer was $8 \cdot 10^6$ g.mol$^{-1}$; the cationic charge density was 24 weight%. The polymer was available as a powder. A concentrated polymer stock solution (10 g.l$^{-1}$) was prepared in plastic beakers of 2 l. After complete solution of the polymer (24 h) the stock solution was diluted to the desired concentration and stored for use at 4°C. To avoid loss of activity a fresh polymer solution was prepared every 3 - 4 days.

**Sampling and analyses** - The turbidity of the influent and effluent of the clarifier was measured on-line. In addition, 24-h samples were taken and were analysed for turbidity and $COD_{particulate}$. $COD_{particulate}$ was calculated by substracting $COD_{dissolved}$ from $COD_{total}$, as is described in chapter 4.

Turbidity was measured on-line with two Solitax turbidity meters (*Lange Group*). The turbidity of 24-h and grab samples was measured with a laboratory turbidity meter (*WTW Turb 550*). The laboratory meter and the on-line meters were regularly calibrated and compared. All other analyses were performed according to Standard Methods (APHA, 1998).

## Results

**Relation between $COD_{particulate}$ and turbidity** - To confirm the earlier findings additional measurements on municipal wastewater were done. To obtain samples with different particle concentrations 24-h and grab wastewater samples were analysed. The samples originated from untreated samples as well as from samples that were flocculated with organic poly electrolytes. Figure 5.3 presents the results of these analyses and compares them with the data of chapter 4. The graph shows a linear relationship in a wide range of $COD_{particulate}$ (100-900 mg $O_2$.l$^{-1}$) and turbidity (50 - 450 NTU). The small difference in slopes between the data series can be attributed to seasonal variations (indicating that the relation should be updated regularly).

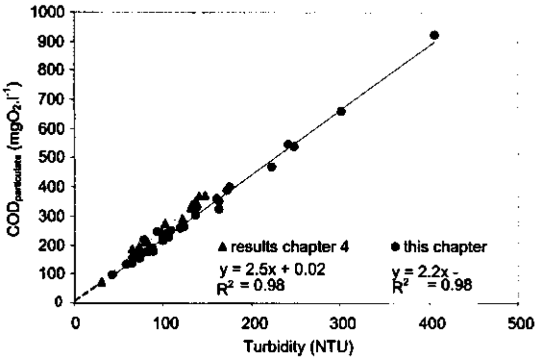

**Fig. 5.3**: Linear relation between COD$_{particulate}$ and turbidity found in chapter 4 and in this chapter

**Diurnal particle variations in untreated influent** - The composition of domestic wastewater varies strongly over the day. Figure 5.4 presents the influent turbidity (or COD$_{particulate}$) of Bennekom wastewater during two consecutive days of dry weather flow. The graph illustrates that during dry weather conditions the turbidity of this wastewater exhibits a regular, daily pattern. Around 10:00 h a first peak is measured caused by increased activity in the morning. During the day the turbidity is lower and shows peaks in the early afternoon and evening. The peaks illustrate that the hydraulic retention time of the Bennekom sewerage system is relatively small.

The average turbidity on day 1 in the figure is 140 NTU with a relative standard deviation of 35 %. The average turbidity on day 2 is 160 NTU with a relative standard deviation of 36 %.

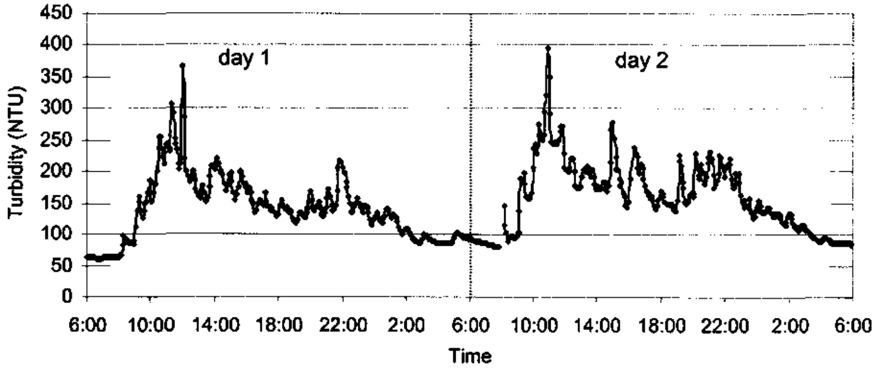

**Fig. 5.4**: Particle fluctuations in the untreated wastewater during dry weather conditions (data 10 and 11 August 2000).

Figure 5.5 presents two rain weather days. At 15:00 h on day 1 a turbidity peak of 850 NTU is visible. This peak is caused by a heavy storm weather event. The peak is followed by a period of relatively low and constant turbidity. Due to the increase in water flow, particles that were settled in the sewer as well as dust and sand from the streets were washed out to the treatment system. This sudden increase in turbidity at the start of a storm water event is known as the 'first flush'. After this first flush and the subsequent cleansing of the sewer system, the rainwater diluted the regular discharge of municipal wastewater which resulted in a low and relatively constant turbidity.

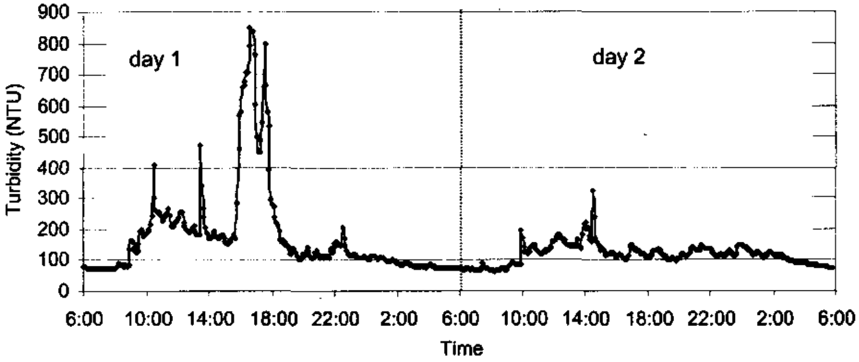

**Fig. 5.5:** Particle fluctuations in the untreated wastewater during and after a storm weather event (data 7 and 8 August 2000).

In general, during the entire monitoring period, the turbidity ranged from 100 to 400 NTU for dry weather conditions. During rain periods fluctuations of 100 to > 1000 NTU were measured. Table 5.1 shows the average turbidity and the standard deviation of the untreated influent in the experimental period.

**Table 5.1:** Average turbidity and standard deviation of the untreated influent in the experimental period (August and first half of September 2000)

| Average turbidity (based on 24-h on-line average values) | Number of investigated days |
|---|---|
| 148±35 | 31 |

**Evaluation of fixed dosing and turbidity-related dosing in the 9.5-l settler** - A comparison was made between dosing a fixed concentration of polymer per liter and dosing based on a constant polymer-turbidity ratio. The experiments were conducted with the 9.5-liter settler.

The conducted experiments consisted of two series. The first series compared the two dosing strategies at an average dose of 3.8 mg polymer/100 NTU, the second series at 5.8 mg polymer/100 NTU. The fixed doses in both series are recalculated based on the average influent turbidity. In these experiments fixed doses of 6.0 and 8.6 mg polymer per liter were applied, respectively. Table 5.2 shows the results of these experiments.

As can be seen in the table for both series the efficiency of the turbidity-related dosage was higher and resulted in lower effluent values. For the series at 3.8 mg polymer/100 NTU the fixed dosage resulted in 74 NTU, while the controlled dose exhibited 46 NTU. The series of 5.8 mg polymer/100 NTU showed turbidity values of 53 and 39 NTU, respectively.

The higher efficiency of the controlled polymer dosage can be explained by the more balanced polymer addition. If the poly electrolyte is dosed with a fixed concentration per liter, the dose is relatively high during periods with low influent turbidity (e.g. at night). At such a high dose the turbidity removal efficiency is more or less constant, despite a further increase in dose (see e.g. the graphs 4.9 and 4.10). This results in an inefficient use of the poly electrolyte. In fact an overdose is applied during these periods, which may cause a fraction of the totally added polymer to be still reactive. During periods with fixed-dose, many problems with blockage of the tube that connected the mixing vessel with the clarifier were encountered. These problems were obviously linked to the reactive polymer chains which were adsorbing to the walls of this tube.

**Table 5.2**: Comparison of fixed polymer dosage and turbidity-related polymer dosage

| Strategy | Dose | Influent (24-h av.) (NTU) | Effluent (24-h av.) (NTU) | Particle removal (%) | Number of days |
|---|---|---|---|---|---|
| fixed dosage | 3.8 mg pe/100 NTU[a] | 157±4 | 74±13 | 53% | 3 |
| turbidity-related dosage | 3.8 mg pe/100 NTU | 150±22 | 46±8 | 69% | 3 |
| fixed dosage | 5.8 mg pe/100 NTU[a] | 148±8 | 53±8 | 64% | 3 |
| turbidity-related dosage | 5.8 mg pe/100 NTU | 152±22 | 39±8 | 74% | 4 |

[a] average value: recalculated based on dose and average influent turbidity

**Turbidity-related dosing in the Dortmund clarifier** - Figure 5.6 shows the turbidity of the influent and effluent of the Dortmund-clarifier during two day without polymer dosing. The average turbidity removal during these two days was 26%. The average value of the effluent on day 1 was 104 NTU (standard deviation 14%), while on day 2 an average turbidity of 116 NTU was found (standard deviation 12%).

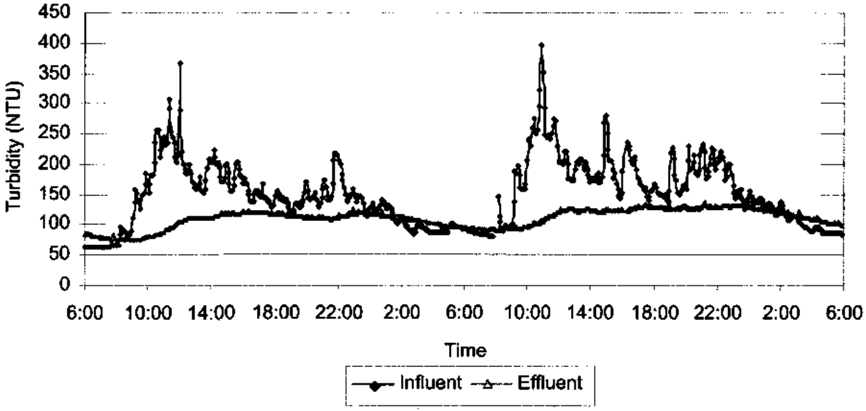

**Fig. 5.6:** Turbidity of influent and Dortmund-clarifier effluent without polymer dosing (data 10 and 11 August 2000, dry weather conditions)

In figure 5.7 an example of turbidity-related polymer dosage is given. The influent turbidity, the turbidity of the clarifier-effluent as well as the flow of the dosing pump are presented. In the shown experiment a fixed ratio of 5.5 mg polymer per 100 influent NTU was maintained. Figure 5.7 demonstrates that the control strategy proved highly effective in achieving a constant effluent quality regarding turbidity (or $COD_{particulate}$). While the influent exhibited an average turbidity of 133±48 NTU, the turbidity of the clarifier effluent had an average value of 31±3 NTU.

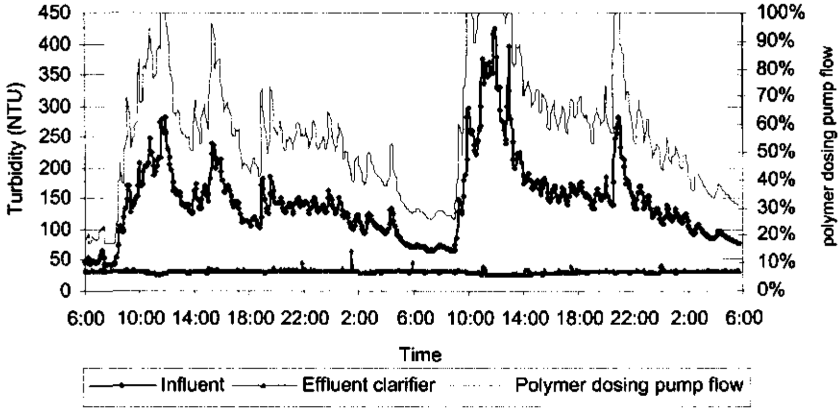

**Fig. 5.7:** Influent and Dortmund-clarifier effluent when applying turbidity-related polymer dose (data 24 and 25 August 2000, dry weather conditions)

The recalculated dosage of the experiment of figure 5.7 is presented in figure 5.8. This figure demonstrates that the control actions resulted in a very stable dosage. Only at influent turbidities exceeding 250 NTU the dosage decreased. As was explained by equation 5.1 the pump flow was maximized at this value. The effluent turbidity was not affected by this decreased dose, as can be seen in figure 5.7.

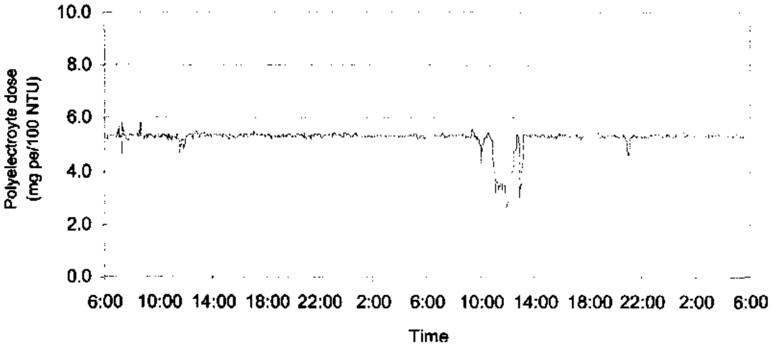

**Fig. 5.8:** Recalculated poly electrolyte dose (in mg pe/100 NTU) of the experiment shown in figure 5.7

Figure 5.9 gives the results of the turbidity-related dosage for different polymer-turbidity ratios. The figure shows 24-h average turbidity values of the influent and clarifier-effluent as a function of the polymer dosage. The figure demonstrates that by applying the controlled dosing different levels of effluent turbidity can be obtained. The standard deviation of the clarifier-effluent values proved fairly low, despite strong fluctuations in the 24-h influent averages.

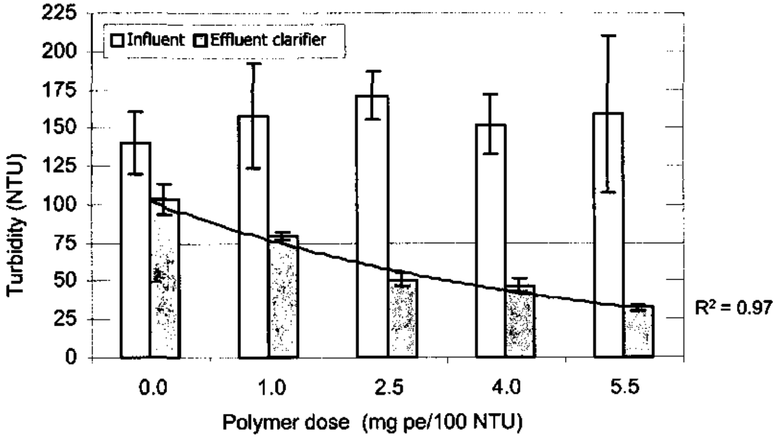

**Fig. 5.9:** Turbidity (24-h averages) of influent and Dortmund-clarifier effluent as a function of the polymer dose in case of turbidity-related dosage

In table 5.3 the actual values of graph 5.9 are given. In the third column the average particle removal efficiency of the Dortmund clarifier is calculated based on the average influent turbidity (148±35 NTU) during the experimental period. In the fourth column the standard deviation during the day is given. This can be considered as a measure for the diurnal variation of the effluent turbidity. The deviation is relatively low and this shows that the effluent turbidity values achieved with controlled polymer dose were almost constant.

**Table 5.3:** Average clarifier effluent turbidity, particle removal and daily deviation for different levels of polymer dosage

| Dosing (mg pe/100 NTU) | Effluent clarifier (24-h averages) (NTU) | Average particle removal efficiency (%) | Average deviation during day (NTU) | Number of days |
|---|---|---|---|---|
| 0 | 104 ± 10 | 30 | 17 | 11 |
| 1.0 | 80 ± 3 | 46 | 14 | 3 |
| 2.5 | 51 ± 4 | 66 | 8 | 4 |
| 4.0 | 47 ± 5 | 68[1] | 8 | 10 |
| 5.5 | 32 ± 2 | 78 | 4 | 10 |

[1] for these figures the flow was 130 $l.h^{-1}$ instead of 90 $l.h^{-1}$

## Conclusions

The investigations in this chapter confirm that, for the investigated wastewater, on-line turbidity measurements can be used to quantify $COD_{particulate}$. Both influent and flocculated clarifier effluent samples showed a linear relation in a wide range of $COD_{particulate}$ (100-900 mg $O_2.l^{-1}$) and turbidity (50 - 450 NTU) even though some variation in slope was found with regard to the data found in chapter 4.

The on-line turbidity measurements revealed that the particle concentrations in the investigated wastewater may vary significantly. During dry weather conditions the turbidity of the investigated sewage fluctuated from 100 to 400 NTU (i.e. $COD_{particulate}$ 220-880 mg $O_2.l^{-1}$), while in rainy periods fluctuations of 100 to >1000 NTU ($COD_{particulate}$ 220 to >2200 mg$O_2.l^{-1}$) were measured.

The experiments showed that the turbidity-related polymer dosing method is technically possible and results in higher removal efficiencies (or decreased polymer consumption) compared to the dosage of fixed polymer concentration per liter. 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 $COD_{particulate}$ removal and therefore may be interesting from the perspective of denitrification in a down-stream biological treatment system.

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# 6 The effect of enhanced primary treatment on nutrient removal in a down-stream biological treatment system

**Abstract.** In this chapter the effect of different levels of particle removal on the elimination of nitrogen and phosphorus in an activated sludge system was investigated. The chapter describes *Nitrogen Uptake Rate* (NUR) experiments, which were performed to evaluate the effect of particle removal on the so-called denitrification potential (DP), and experiments with a pilot scale activated sludge plant (1,000 l) that received pretreated wastewater.

The NUR experiments showed that the particulate fraction of the wastewater (~ 65% of $COD_{total}$) contained around 50% of the total DP. This finding indicates that if all the particles are removed, the biodegradable COD of the investigated wastewater is reduced by 50%. Moreover, it suggests that the effect of particle removal on the removal of biodegradable COD is smaller than is estimated following the decrease in the (frequently used) $COD_{total}/N$ ratio.

The sum of the nitrite and nitrate ($NO_{2+3}$-N) concentration in the effluent leaving the activated sludge system exceeded the nitrogen limit during the entire experiment. The N-Kjeldahl of the tested wastewater was, however, high compared to average Dutch influent values (70 mgN.$l^{-1}$ versus 50 mgN.$l^{-1}$, respectively). Calculations based on the experimental values indicate that, in well-designed plants receiving Dutch N-Kjeldahl concentration, a high level of particle removal (i.e. 80-90%) is possible without exceeding the nitrogen effluent constraint. The pilot experiment showed that pretreatment with organic polymers does not have a direct effect on the biological phosphorus removal of a down-stream activated sludge system. The removal of phosphate in the anoxic zone might even contribute to the total denitrification. In the pilot experiment an increase in denitrification of approximately 4 mgN.$l^{-1}$ was found.

## Introduction

A drawback of physical-chemical pretreatment of municipal wastewater is that the removal of particles is partially accompanied by the removal of biodegradable organic carbon (generally indicated by BOD or biodegradable COD). Because the removal efficiency of nitrogen during primary treatment generally is lower than that of biodegradable carbon, a decreased BOD/N ratio is the result. Since biodegradable carbon is necessary for denitrification, as a consequence a decreased nitrogen removal by a down-stream biological treatment system (activated sludge, biofilm or membrane bioreactor system) can be expected.

According to Van Haandel *et al.* (1982), the total denitrification of an activated sludge system with pre-denitrification is determined by the availability of biodegradable COD in the wastewater, the size of the anoxic zone (fig. 1.1) and by the recirculation flows (sludge recycle and internal recycle). As stated by different authors (a.o. Van Haandel et al., 1982; Kujawa and Klapwijk, 1999), the availability of biodegradable COD in wastewater can be characterised by the denitrification potential (DP), being the maximum amount of nitrate equivalent (per unit of volume) that can be denitrified with the present biodegradable COD. Kujawa-Roeleveld (2000) showed that, for a well-designed system, the denitrification potential is the main factor determining the total denitrification.

The DP of a specific wastewater can be estimated in *Nitrate Uptake Rate* (NUR) experiments. During NUR experiments, the nitrate consumption at anoxic conditions after a pulse dose of wastewater to activated sludge is determined. The nitrate consumption represents the DP (taking into account the dilution factor). If the biological sludge yield under anoxic conditions ($Y_{HD}$) is known, the rapidly and slowly biodegradable COD fractions ($S_s$ and $X_s$) can be estimated.

If organic poly electrolytes are applied, the phosphorus removal in a pretreatment step will be limited to the particulate P fraction. To ensure that the effluent of a down-stream activated sludge system will still meet the effluent constraints, biological phosphorus removal can be introduced. Biological phosphorus removal is implemented by introducing an anaerobic zone, thus creating conditions that are favourable to grow phosphate-accumulating bacteria (PAOs). These PAOs consume volatile fatty acids in such an anaerobic zone to produce storage products (polyhydroxy alkanoates - PHAs). The required energy is acquired from hydrolysis of polyphosphate and conversion of glycogen (Mino *et al.*, 1987). The polyphosphate hydrolysis results in the release of phosphate.

The PHAs are oxidised to $CO_2$ at aerobic or anoxic conditions. The energy that is retrieved from this oxidation is used for cell growth and for storage of phosphate and glycogen. The phosphate uptake in this stage is higher than the phosphate release during the anaerobic phase. The phosphate that is removed and stored during the aerobic or anoxic state is removed from the system with the excess sludge.

During the aerobic or anoxic stage the PHAs can either be oxidised by oxygen or by nitrate. The metabolism in both reactions is the same except that nitrate is used as final electron acceptor. The process of anoxic phosphate accumulation may increase the total denitrification in an activated sludge system, because extra nitrate is reduced (RWZI-2000, 1994; Kuba et al., 1996).

The objective of the investigation that is described in this chapter was to determine the effect of different levels of particle removal on the nitrogen and phosphorus removal of an activated sludge system. The investigation consisted of two sets of experiments. The first set of experiments dealt with the removal of nitrogen and phosphorus removal in a pilot scale activated sludge plant (1000 l). This system received wastewater that was pretreated with organic polymers. The second set of experiments consisted of NUR experiments that were used to estimate the effect of different levels of particle removal on the DP.

## Materials and methods

**Pilot plant experiments** - Figure 6.1 presents a scheme of the pilot plant. The system consisted of a primary clarifier with controlled, turbidity-based polymer dosing (see chapter 5), followed by a plug flow activated sludge system and a secondary clarifier.

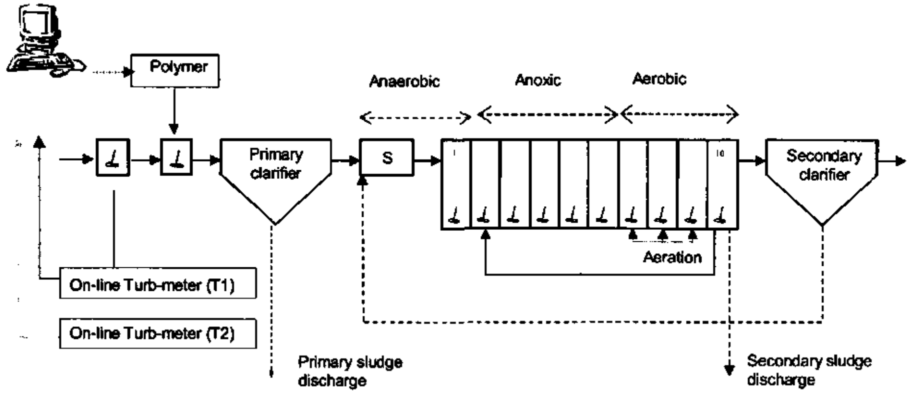

**Fig. 6.1:** Schematic presentation of the pilot plant

The pretreated wastewater entered the activated sludge system at constant flow via a selector (S, 14 l). The plug flow activated sludge reactor (total volume 1000 l) consisted of ten identical compartments (named $C_1..C_{10}$). Each compartment was completely mixed. The selector and the first compartment $C_1$ were anaerobic to provide conditions for biological P-removal. The compartments $C_2$ to $C_6$ were anoxic. In the compartments $C_7$ to $C_9$ an oxygen concentration of > 4 $mgO_2.l^{-1}$ was maintained to ensure complete nitrification. The last compartment – $C_{10}$ – was only mildly aerated ($O_2$ concentration ~ 1 $mg.l^{-1}$) to minimize anoxic conditions in the secondary clarifier (causing flotation of activated sludge by denitrification). Sludge was continuously discharged from compartment $C_{10}$ in order to maintain a constant sludge age. A nitrate rich sludge/water mixture was returned to the anoxic zone at a recycle ratio of 3. Before discharge the treated waster entered a secondary clarifier (type: Dortmund tank; 400 l) to separate the activated sludge. Settled sludge was returned to the selector. The operational characteristics of the pilot plant are presented in table 6.1.

**Table 6.1:** Average operational parameters for the pilot plant

| Parameter | Symbol | Value | Unit |
|---|---|---|---|
| Influent flow | $Q_i$ | 90 | $l.h^{-1}$ |
| Return sludge flow | $Q_{XR}$ | 70 | $l.h^{-1}$ |
| Internal recycle flow | $Q_R$ | 270 | $l.h^{-1}$ |
| Sludge concentration | $X_{SS}$ | 3.1 | $gSS.l^{-1}$ |
| Volatile suspended solids | $X_{VSS}$ | 2.3 | $gVSS.l^{-1}$ |
| Temperature | T | 21 | °C |
| Excess sludge flow | $Q_w$ | 57 | $l.day^{-1}$ |
| Sludge age | $\Theta$ | 17.6 | days |

The pilot plant was started with sludge originating from wastewater treatment plant Ede (Biodenipho system) that contained both denitrifying and phosphate-accumulating activity. The system was fed with wastewater of the town of Bennekom. After several months of start-up time the experiments were started. During the experimental period (August and September 2000) hardly any storm water events occurred, resulting in a relatively constant influent COD of 528±56 $mgO_2.l^{-1}$. The temperature in the activated sludge system in this period was 21±0.5 °C.

Three phases were distinguished in the pilot plant experiments. During phase I (day 0-14) regular pre-settling was applied. The pre-settling resulted in a particle removal of 30%. The average turbidity of the clarifier effluent in this period was 104 NTU.

Starting on day 14, a (turbidity-related) polymer dose of 5.5 mg pe/100 NTU was applied. This was the start of phase II (day 14-29). The polymer dosage resulted in an average particle removal of 78% and a turbidity of 32 NTU. On day 29, the start of phase III (day

29-40), the aeration in the compartments $C_7$ and $C_{10}$ was turned off in order to enlarge the anoxic volume to 700 l. During phase III the same polymer dose was applied as during phase II.

The denitrified fraction ($N_{denitrified}$) of the influent N-Kjeldahl ($N_{Kj,i}$) during the pilot trials was calculated with a nitrogen mass balance:

$$N_{denitrified} = N_{Kj,i} - N_X - N_e \qquad (mgN.l^{-1}) \qquad (6.1)$$

with $N_x$ = nitrogen discharge by excess sludge (mgN.l$^{-1}$) and $N_e$ = nitrogen concentration in the effluent (mgN.l$^{-1}$). The nitrogen discharge by excess sludge was calculated according to:

$$N_X = f_{XN} \bullet X_{SS} \bullet Q_w / Q_i \qquad (mgN.l^{-1}) \qquad (6.2)$$

with $f_{XN}$ = nitrogen content of sludge (gN.gSS$^{-1}$) (the other symbols are given in table 6.1). The nitrogen concentration in the effluent consisted of different components:

$$N_e = NH_4\text{-}N_e + NO_{2+3}\text{-}N_e + N_{i,e} \qquad (mgN.l^{-1}) \qquad (6.3)$$

with $N_{i,e}$ = concentration of inert nitrogen in the effluent (mgN.l$^{-1}$). Within the framework of this research $N_{i,e}$ was not measured, but estimated based on the work of previous experiments with the same wastewater. The $N_{i,e}$ for the investigated wastewater is 3-4 mgN.l$^{-1}$, according to Brouwer & Klapwijk (1997) and Kujawa-Roeleveld (2000).

**Nitrate Uptake Rate (NUR) experiments** – NUR experiments were conducted to determine the DP of wastewater samples that were subject to different levels of pretreatment. For this purpose activated sludge (2.75 or 3.00 l) was added to a 4.0-l batch vessel. In the NUR tests two types of activated sludge were used: sludge withdrawn from the last compartment ($C_{10}$) of the pilot plant and sludge collected from the secondary clarifier of wastewater treatment plant (wwtp) Nijmegen. The sludge of this wwtp was used because it contained no phosphate-accumulating activity. Wwtp Nijmegen consists of an activated sludge system (sludge loading rate 0.08 kg BOD.kgSS$^{-1}$ per day; sludge age 24 days) with pre-precipitation.

Before the start of the NUR experiments, the sludge was aerated for 1 h to ensure that no external carbon sources were present in the sludge. After the aeration phase a concentrated nitrate solution ($NaNO_3$) was added to the sludge in order to achieve a level of 20 – 40 mgNO$_3$-N per liter. A thermostat was used to keep the sludge-mixture at a constant temperature of 20°C. Nitrogen gas was purged through to ensure anoxic conditions. After an endogenous denitrification phase of 30 minutes a pulse dose of wastewater with known COD was added (0.75 or 0.50 l) and samples were taken frequently. These samples were immediately paper filtered. The samples were analysed for $NO_3$-N, $NO_2$-N and incidentally also for $PO_4$-P. At the end of the test run the suspended solids concentration (SS) and volatile suspended solids concentration (VSS) were measured.

The NUR experiments were conducted with untreated wastewater, settled wastewater and wastewater flocculated with organic polymer. To obtain settled and flocculated wastewater a pretreatment was applied to raw wastewater samples in a similar jar test apparatus, as was described in chapter 3. These wastewater samples (24-h samples) were only taken during dry weather conditions in order to obtain comparable results and had a relatively constant COD of 536 ± 80 $mgO_2.l^{-1}$

Based on the course of nitrate and nitrite the denitrification rates and the DP were estimated. Denitrification proceeds according to the generally accepted pathway:

$$NO_3^-{}_{(aq)}{}^{(+5)} \rightarrow NO_2^-{}_{(aq)}{}^{(+3)} \rightarrow NO_{(g)}{}^{(+2)} \rightarrow N_2O_{(g)}{}^{(+1)} \rightarrow N_{2(g)}{}^{(0)}$$

During NUR-tests an accumulation of nitrite not always is negligible. The reduction of 1 g $S_{NO2}$ to 1 g $N_2$-gas needs the same amount of electrons as the reduction of 0.6 g $S_{NO3}$ to 0.6 g $N_2$. Therefore, to interpret the results of the NUR-test the so called nitrite-nitrate equivalent was used, being $S_{NO} = S_{NO3} + 0.6 \cdot S_{NO2}$ (Kujawa and Klapwijk, 1999). In this way both the amount of biodegradable COD utilized for the partial process of nitrate reduction to nitrite and the COD amount for the entire denitrification process were taken into account.

Figure 6.2 presents a typical example of a NUR-test in which 0.5 l untreated wastewater with a COD of 619 $mgO_2.l^{-1}$ was added to 3.0 l activated sludge (thus the fraction of wastewater of the total wastewater-sludge mixture $f_{ww}$ was 0.15). In the course three utilization rates of nitrate consumption can be distinguished, $r_{D1}$, $r_{D2}$ and $r_{D3}$ ($mgN.l^{-1}$ per h).

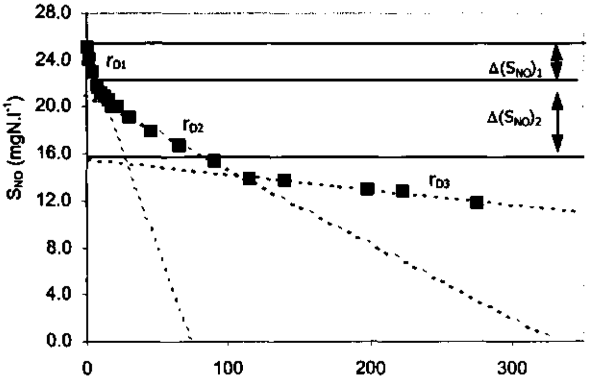

**Fig. 6.2:** NUR test: course of $S_{NO}$ (nitrite-nitrate equivalent) after a pulse dose of municipal wastewater to activated sludge ($f_{ww}$ = 0.15, COD = 619 mgO$_2$.l$^{-1}$, T= 20°C)

The three volumetric nitrate utilization rates consist of the following components:

$r_{D1} = r_{DSS} + r_{DXS} + r_{Dendogenous}$; $\quad$ mgN.l$^{-1}$.h$^{-1}$; $\quad$ (6.4)

$r_{D2} = r_{DXS} + r_{Dendogenous}$; $\quad$ mgN.l$^{-1}$.h$^{-1}$; $\quad$ (6.5)

$r_{D3} = r_{Dendogenous}$; $\quad$ mgN.l$^{-1}$.h$^{-1}$; $\quad$ (6.6)

in which $r_{DSS}$, $r_{DXS}$ and $r_{Dendogenous}$ represent the anoxic oxidation of readily biodegradable material ($S_s$), the oxidation of slowly biodegradable material ($X_s$) and the endogenous denitrification, respectively.

The DP is calculated from the nitrate consumption in the experiment and consists of $DP_{ss}$ and $DP_{Xs}$, representing $S_{NO}$ removal by the biodegradable COD fractions $S_s$ and $X_s$. The removal by these fractions is depicted in fig. 6.2 as $\Delta(S_{NO})_1$ and $\Delta(S_{NO})_2$. $DP_{ss}$ and $DP_{Xs}$ are calculated according:

$$DP_{SS} = \Delta(S_{NO})_1 * 1/f_{ww} \;; \quad \text{mgN.l}^{-1}; \quad (6.7)$$

$$DP_{XS} = \Delta(S_{NO})_2 * 1/f_{ww} \;; \quad \text{mgN.l}^{-1}; \quad (6.8)$$

$DP_{SS}$ and $DP_{XS}$ can be expressed as the biodegradable COD fractions $S_s$ and $X_s$:

$$S_s = 2.86 * DP_{SS} / (1-Y_{HD}); \quad \text{mgO}_2\text{.l}^{-1}; \quad (6.9)$$

$$X_s = 2.86 * DP_{XS} / (1-Y_{HD}) \;; \quad \text{mgO}_2\text{.l}^{-1}; \quad (6.10)$$

where $Y_{HD}$ is the anoxic heterotrophic yield. The yield was estimated with two NUR experiments with a known concentration of acetate and was equal to 0.67 (-).

**Analyses** - Pretreated 24-h samples of the primary clarifier-effluent were analysed for COD, N-Kjeldahl and $P_{total}$. The COD of these samples was fractionated into $COD_{total}$ and $COD_{particulate,}$ according to a method already described in chapter 4. The 24-h samples of the effluent of the activated sludge system were analysed for COD, ammonium, nitrate, nitrite and phosphate. The suspended solids and the volatile suspended solids concentration of the sludge were measured, as well as the nitrogen content. All analyses were performed according to Standard Methods (APHA, 1998). Ammonium, nitrate, nitrite and phosphate were determined with an auto analyser (SKALAR).

## Results

**Pilot plant results** - Table 6.2 shows the characteristics of the pretreated water, the effluent concentrations and the sludge characteristics during the three phases of the pilot experiment. The effluent concentrations are also shown in figure 6.3.

**Table 6.2:** Results of the pilot experiment: pretreated wastewater characteristics, effluent concentration and sludge characteristics during the three phases of the pilot experiment

| Phase | Particle removal | Pretreated wastewater characteristics | | | | |
|---|---|---|---|---|---|---|
| | | Turbidity (NTU) | COD ($mgO_2.l^{-1}$) | $S_s$ ($mgO_2.l^{-1}$) | $X_s$ ($mgO_2.l^{-1}$) | N-Kjeldahl ($mgN.l^{-1}$) |
| I | 30% | 104 | 420 ± 24 | 72 ± 20 | 191 ± 23 | 67 ± 4 |
| II, III | 78% | 32 | 250 ± 38 | 38 ± 36 | 161 ± 14 | 63 ± 4 |
| | | **Effluent concentrations** | | | | |
| | | | COD ($mgO_2.l^{-1}$) | $NO_{2+3}$-N ($mgN.l^{-1}$) | $NH_4$-N ($mgN.l^{-1}$) | $PO_4$-P ($mgP.l^{-1}$) |
| I | 30% | | 53 ± 10 | 11.0 ± 1.6 | 0.5 ± 0.7 | 0.6 ± 0.5 |
| II A | 78% | | 36 ± 6 | 14.7 ± 1.8 | 0.3 ± 0.2 | 0.1 ± 0.3 |
| II B | 78% | | 36 ± 6 | 18.4 ± 2.1 | 0.3 ± 0.2 | 5.6 ± 0.5 |
| III | 78% | | 36 ± 6 | 13.5 ± 0.9 | 0.2 ± 0.4 | 3.0 ± 0.9 |
| | | **Sludge characteristics** | | | | |
| | | Sludge loading rate | Sludge concentration | | | Nitrogen fraction sludge |
| | | ($kgCOD.kgSS^{-1}$ per day) | SS ($gSS.l^{-1}$) | VSS ($gVSS.l^{-1}$) | | $f_{XN}$ ($gN.gSS^{-1}$) |
| I | 30% | 0.30 | 3.1 | 2.3 | | 0.12 |
| II, III | 78% | 0.18 | 2.8 | 2.3 | | 0.12 |

A complete nitrification with effluent $NH_4$-N ($S_{NH}$) concentrations less than 1.0 $mgN.l^{-1}$ was achieved in all phases of the experiment (figure 6.3).

The average effluent concentration $NO_{2+3}$-$N_{,e}$ in phase I was 11 $mg.l^{-1}$. The average effluent concentration $PO_4$-$P_{,e}$ in this period was 0.6 $mg.l^{-1}$. Due to the increased particle removal efficiency, $NO_{2+3}$-$N_{,e}$ increased in phase II. During the first five days (indicated as phase IIA) the concentration was around 15 $mg.l^{-1}$, but after day 22 it showed a further increase to 18 $mg.l^{-1}$ (phase IIB).

Remarkably, $PO_4$-$P_{,e}$ remained < 1 $mg.l^{-1}$ during the first five days of phase II and increased to approximately 6 $mg.l^{-1}$ after day 22 (phase IIB). The probable cause of this increase was the recirculation of nitrate to the (formerly) anaerobic zone. Measurements of the $NO_{2+3}$-N concentration in this zone (i.e. compartment $C_1$) showed values of around 4 $mg.l^{-1}$ during the rest of the experiment (earlier, in Phase I it was < 0.5 $mg.l^{-1}$). Volatile fatty acids that normally were available for PAOs were now consumed by denitrifying bacteria. The fact that $PO_4$-$P_{,e}$ increased after day 22 is most likely caused by a slow decrease of the PAOs present in the sludge, finally resulting in a strong decrease in phosphate-accumulating capacity. Simultaneous to the increase in phosphate concentration, a second increase of $NO_{2+3}$-$N_{,e}$ to 18 $mg.l^{-1}$ was found. This second increase was linked to the decrease in P removal which indicates that the regular denitrifying bacteria removed less nitrate per mg biodegradable COD than the denitrifying PAOs.

During phase III, starting on day 29, the aeration in the compartments $C_7$ and $C_{10}$ was turned off in an attempt to increase the nitrate removal in the system. By turning off the aeration the anoxic volume was enlarged to 700 l. Due to this enlargement, $NO_{2+3}$-$N_{,e}$ decreased to 13.5 $mg.l^{-1}$. Despite the decrease in aerobic volume the $NH_4$-$N_{,e}$ remained < 0.5 $mg.l^{-1}$.

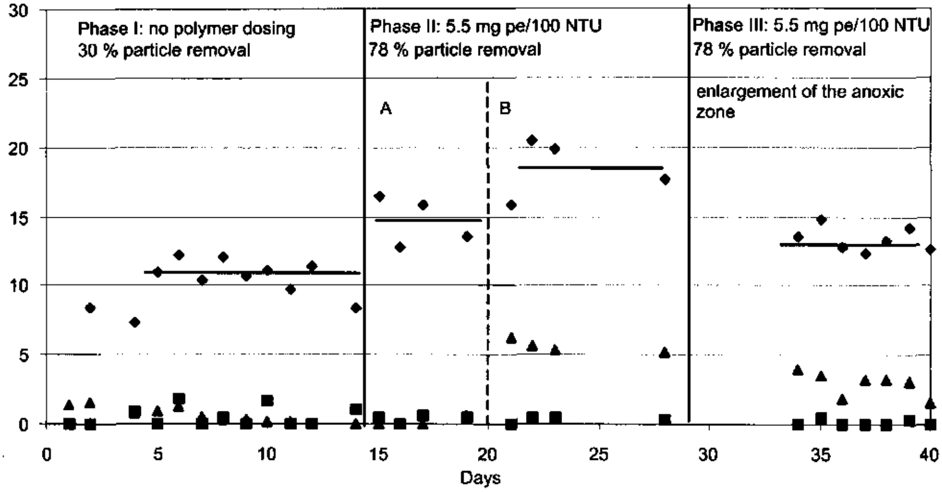

**Fig. 6.3:** $NO_{2+3}$-N (♦), $NH_4$-N (■) and $PO_4$-P (▲) concentrations (in $mg.l^{-1}$) in the effluent of the activated sludge system during the experiment

The fraction of influent N-Kjeldahl that is denitrified ($N_{denitrified}$) was calculated by using the nitrogen balance of equations 6.1 to 6.3. The result is shown in table 6.3. During phase I an average $N_{denitrified}$ of 41.6 $mg.l^{-1}$ was found. However, measurements have demonstrated that the $NO_{2+3}$-N concentration in the last compartment of the anoxic zone ($C_7$) was zero. This indicates that the total denitrification efficiency during this phase could have been increased further by increasing the internal recirculation flow.

During phase II and III, concentrations in a range of 4-10 mg $NO_{2+3}$-N per liter were found in the last compartment of the anoxic zone. It shows that $N_{denitrified}$ was maximal during both these phases and could not have been improved by increasing the internal recycle flow.

**Table 6.3:** Nitrogen balance of the system during the different phases

| Phase | Particle removal | $N_{Kj,i}$ (mgN.$l^{-1}$) | $N_X$ (mgN.$l^{-1}$) | $N_e$ (mgN.$l^{-1}$) | $N_{denitrified}$ (mgN.$l^{-1}$) |
|---|---|---|---|---|---|
| I | 30% | 67 | 9.8 | 15.6 | 41.6 |
| IIA | 78% | 63 | 8.8 | 18.0 | 36.2 |
| IIB | | 63 | 8.8 | 21.7 | 32.5 |
| III | 78% | 63 | 8.8 | 16.7 | 37.5 |

**NUR experiments** - Figure 6.4 shows the total denitrification potential (DP) that was found in the NUR tests. The DP is presented as a function of the COD of the wastewater. The COD was fractionated into $COD_{dissolved}$ and $COD_{particulate}$. The vertical line in figure 6.4 represents the average value for $COD_{dissolved}$ (180 $mgO_2.l^{-1}$). The deviation bar presents the highest and lowest observed $COD_{dissolved}$ values (150 - 200 $mgO_2.l^{-1}$).

For an untreated wastewater sample (■) with a COD of 519 $mgO_2.l^{-1}$ the DP amounted 39 $mgS_{NO}.l^{-1}$. The DP of presettled wastewater (△) was 31-33 $mgS_{NO}.l^{-1}$. Dosage of organic polymer (▲) resulted in a decreased COD and DP. To remove all particles two samples were treated with a high polymer dose and subsequently paper filtered. The DP of these samples was 18-20 $mgS_{NO}.l^{-1}$.

As is shown in the graph, the maximal reduction of DP after complete particle removal was 19-21 $mgS_{NO}.l^{-1}$. This is approximately 50% of the total DP of the considered wastewater. The other 50% were related to $COD_{dissolved}$. However, this $COD_{dissolved}$ fraction represents 35% of total COD. The DP/$COD_{dissolved}$ ratio was 0.11, which is almost twice as high as the DP/$COD_{particulate}$ ratio of 0.06. It shows that relative more DP is related to $COD_{dissolved}$ and indicates that the effect of particle removal on DP is relatively smaller than is generally assessed when the $COD_{total}$/N ratio is used as a method to quantify the denitrification potential.

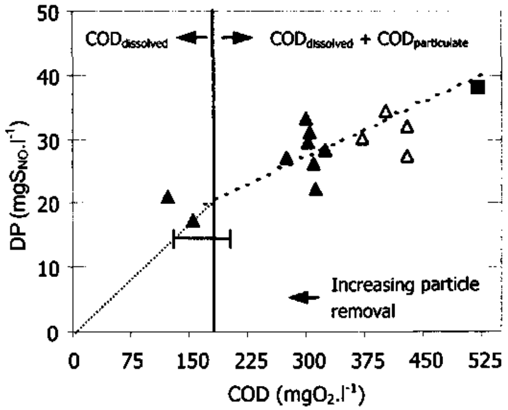

**Fig. 6.4**: Decrease of the total DP as a function of the COD of the samples; (■) untreated wastewater; (△) settled wastewater; (▲) flocculated wastewater

In table 6.4 a comparison is made between the DP and $N_{denitrified}$ that was found in the pilot plant experiments. The difference between both values can be explained by endogenous denitrification and nitrate reduction by denitrifying PAOs.

**Table 6.4**: $N_{denitrified}$ in comparison with DP

| Phase | Particle removal | DP (mgN.l$^{-1}$) | $N_{denitrified}$ (mgN.l$^{-1}$) | Difference (mgN.l$^{-1}$) |
|---|---|---|---|---|
| I | 30% | 30.6 | > 41.6 | > 11.0 |
| IIA | 78% | 22.8 | 36.2 | 13.4 |
| B |  | 22.8 | 32.5 | 9.7 |
| III | 78% | 22.8 | 37.5 | 14.7 |

As was mentioned before, two types of activated sludge were used for the determination of the DP, namely sludge of the pilot plant and sludge of wwtp Nijmegen. By comparing the total DP that is achieved by both types of sludge, it can be decided whether the presence of PAOs had an effect on the outcomes of the NUR tests. A possible effect may appear, because due to the uptake of volatile fatty acids by (strictly aerobic) PAOs, the $S_s$ can be estimated lower, while due to the oxidation of nitrate by anoxic PAOs uptake $X_s$ can be estimated higher.

Figure 6.5 presents two NUR experiments to illustrate the difference between the two sludge types. The sludge of the pilot plant (left picture) released phosphate during the first ten minutes, while later in the experiment phosphate is removed by denitrifying PAOs. The sludge of wwtp Nijmegen showed no measurable phosphate accumulating activity.

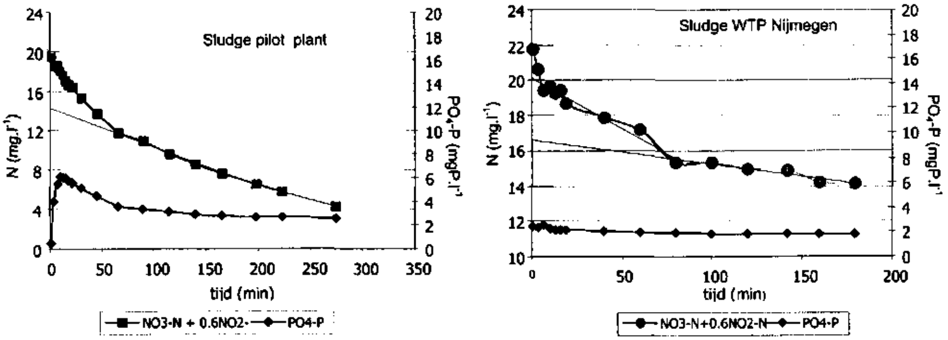

**Fig. 6.5**: NUR-test with sludge of the pilot plant (left; with PAOs) and sludge of wwtp Nijmegen (right; without PAOs)

Figure 6.6 shows the same results as were earlier presented in figure 6.4. However, in this case the results are divided into the DP achieved by the two types of sludge. As is shown, no measurable difference in the determined DP was found for the two sludges.

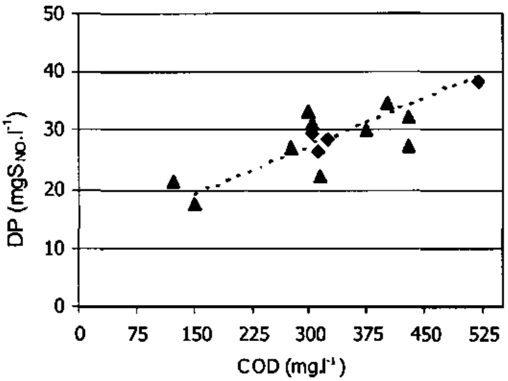

**Fig. 6.6**: Comparison of the DP estimated with sludge of the pilot plant (◆) and Nijmegen-sludge (▲)

## Discussion

**Comparison of the investigated wastewater characteristics with average Dutch wastewater** - The nitrogen concentration of the investigated wastewater is rather high compared to average concentrations in the Netherlands. The average COD and N-Kjeldahl of the used influent were 528 $mgO_2.l^{-1}$ and 70 $mgN.l^{-1}$, respectively (COD/N = 8:1). The average COD and N-Kjeldahl for Dutch municipal wastewater during 1994-1997 were 552 $mgO_2.l^{-1}$ and 51 $mgN.l^{-1}$ (COD/N = 11:1) (CBS, 2000). This should be taken into account in interpreting the results.

**Comparison of the pilot plant experiments with previous research** - A point of consideration regarding the described pilot experiments is the relatively short experimental period. The polymer dosage was only applied for 41 days, which is equal to 2.3 times the sludge retention time. Generally, a biological system is considered to be in 'steady state' after at least three times the sludge retention time. Therefore, it may be questioned if the sludge had fully adapted to the new situation. In order to check the reliability of the pilot plant results, a comparison of process performance was made between this study and earlier investigations with the same pilot plant and wastewater of the same source. Table 6.6 presents the result of this comparison and demonstrates that the experimental results are in agreement with former research.

The table shows two investigations, one described by Brouwer and Klapwijk (1997), the second by Kujawa-Roeleveld (2000). In the first investigation settled wastewater was used, in the second wastewater precipitated with $FeCl_3$. The investigations are compared to phase I and phase IIB of the pilot-experiment of this chapter. The results of phase IIB were used, since then no PAOs were present (as was in the case of the investigation with $FeCl_3$). As is shown in table 6.6, both investigations report similar influent values with regard to biodegradable COD and N-Kjeldahl as were found in the experimental study. Likewise, the sum of $NO_{2+3}$ and $NH_4$ in the effluent are in the same range. In the study of Brouwer and Klapwijk a concentration of 10.0 $mgN.l^{-1}$ is reported while 11.1 $mgN.l^{-1}$ was found in this chapter. The investigation of Kujawa-Roeleveld showed an average concentration of 17.4 $mgN.l^{-1}$, while the experimental work with the pilot plant showed a concentration of 18.6 $mgN.l^{-1}$.

**Table 6.6**: Comparison of the pilot experiments with previous research in the same pilot plant and with wastewater of the same source

| **Parameter** | **Unit** | **Settled waste-water** | **Settled waste-water** | **Precipitated wastewater ($FeCl_3.6H_2O$)** | **Pretreated wastewater (polymer)** |
|---|---|---|---|---|---|
| | | Brouwer and Klapwijk, 1997 | This research, phase I | Kujawa-Roeleveld, 2000 | This research, phase IIB |
| **Wastewater characteristics** | | | | | |
| COD | $mg.l^{-1}$ | 491 | 420 | 281 | 250 |
| $N_{Kj,i}$ | $mgN.l^{-1}$ | 65 | 67 | 60 | 63 |
| Ss+Xs | $mgO_2.l^{-1}$ | 260 | 263 | 201 | 199 |
| DP | $mgN.l^{-1}$ | 30.0 | 30.6 | 23.7 | 22.8 |
| $P_{total}$ | $mgP.l^{-1}$ | 9.2 | 9.8 | 3.8 | 7.9 |
| **Activated sluge process conditions** | | | | | |
| Sludge loading rate | $gCOD.gSS^{-1}.d^{-1}$ | 0.20 | 0.30 | 0.15 | 0.18 |
| Temperature | °C | | 21 | 18 | 21 |
| **Effluent quality** | | | | | |
| $NH_4$-N | $mgN.l^{-1}$ | 2.9 | 0.7 | 1.1 | 0.2 |
| $NO_{2+3}$-N | $mgN.l^{-1}$ | 7.1 | 10.4 | 16.3 | 18.4 |
| $N_{total}$ | $mgN.l^{-1}$ | 11 | | 19.5 | |
| $P_{total}$ | $mgP.l^{-1}$ | 0.5 | 0.6 | 0.5 | 3.0 |

**Estimation of the effluent $NO_{2+3}$-N concentration for different levels of particle removal** - As was pointed out in chapter 5, turbidity-related polymer may be applied to create different levels of particle removal. In this paragraph an estimation of the expected effluent quality of the pilot plant for different particle removals is made, based on the results of this chapter. The estimation takes into account the different COD/N ratio of 'average' Dutch wastewater (11:1).

The calculations were done under the assumption that no PAOs are present in the activated sludge (therefore no anoxic P removal was accounted for and possible effects on the denitrification capacity were left out). Thus, the assumptions reflect phase IIB of the pilot plant experiments (tables 6.2 and 6.6). The methodology followed, the assumptions and results are discussed in the following pages.

Figure 6.7 shows a scheme of the pilot plant, assuming only pre-denitrification. As there is not biological phosphorus, no anaerobic zone is incorporated.

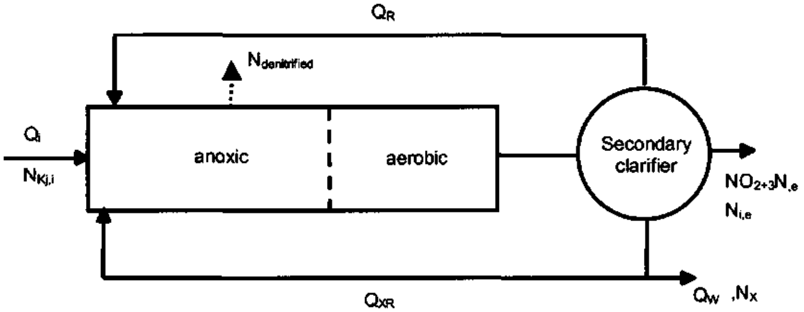

**Fig. 6.7:** Scheme of the pilot plant system (without anaerobic zone)

In the calculations the same distribution between the anoxic and aerobic volume was used as during phase IIB of the pilot experiments, i.e. an anoxic volume ($V_{anoxic}$) of 500 l and an aerobic volume ($V_{aerobic}$) of 400 l. Furthermore, the operational parameters as shown in table 6.1 were used in the calculations.

The $NO_{2+3}$-N concentration in the effluent is dependent on both the recirculation ratio and the maximal amount of nitrogen that can be removed by denitrification in the anoxic zone ($N_{denitrified,\ max}$ in mgN.l$^{-1}$), given the available biodegradable COD and the endogenous denitrification (mgN.l$^{-1}$). When biodegradable COD is sufficiently available, the $NO_{2+3}$-N concentration of the effluent can be calculated following:

$$NO_{2+3}\text{-}N_{,e} = \frac{Q_i}{Q_i + Q_R + Q_{XR}} \bullet N_{nitrified} \text{ if } NO_{2+3}\text{-}N_{,e} \le \frac{Q_i}{Q_i + Q_R + Q_{XR}} \bullet N_{dentrified,max} \quad (mgN.l^{-1}, 6.11)$$

When $N_{denitrified,\ max}$ becomes limiting, $NO_{2+3}$-$N_{,e}$ is calculated according to:

$$NO_{2+3}\text{-}N_{,e} = N_{nitrified} - N_{dentrified,max} \qquad (mgN.l^{-1}) \qquad (6.12)$$

with $N_{nitrified}$ = the nitrified fraction of the influent N-Kjeldahl (mgN.l$^{-1}$). If $NH_4$-N is assumed to be absent in the effluent, $N_{nitrified}$, can be calculated following:

$$N_{nitrified} = N_{kj,i} - N_x - N_{i,e} \qquad (mgN.l^{-1}) \qquad (6.13)$$

with $N_X$ = nitrogen discharge by excess sludge (mgN.l$^{-1}$) and $N_{i,e}$ = inert nitrogen concentration in the effluent (mgN.l$^{-1}$). The nitrogen discharge by excess sludge was calculated according:

$$N_X = f_{XN} \bullet Y_{obs} \bullet (COD_{influent} - COD_{effluent}) \qquad (mgN.l^{-1}) \qquad (6.14)$$

with $f_{XN}$ = nitrogen concentration of the sludge ($gN.gSS^{-1}$) and $Y_{OBS}$ = the observed yield ($gSS.gCOD_{removed}^{-1}$).

As mentioned, $N_{denitrified,\ max}$ is the sum of DP (estimated in the NUR experiments) and the nitrogen removal by endogenous denitrification ($N_{endogenous}$):

$$N_{denitrified,max} = DP + N_{endogenous} \qquad (mgN.l^{-1}) \qquad (6.15)$$

$$\text{with } N_{endogenous} = \frac{X_{SS} \bullet V_{anoxic} \bullet k_{endogenous}}{Q_i} \qquad (mgN.l^{-1}) \qquad (6.16)$$

with $k_{endogenous}$ = specific endogenous denitrification rate. For $k_{endogenous}$ and $Y_{OBS}$ the values found in phase IIB were used, being 18.7 $mgN.gSS^{-1}.day^{-1}$ and 0.34 $gSS.gCOD_{removed}^{-1}$, respectively. For $N_{i,e}$ and $COD_{effluent}$ values of 4 $mgN.l^{-1}$ and 50 $mgO_2.l^{-1}$ were taken. For $F_{XN}$ the reported value of 0.12 $gN.gSS^{-1}$ was used.

Table 6.7 shows $COD_{influent}$, $COD_{effluent}$, $N_{kj,i}$, $X_{SS}$, $N_{denitrified,max}$ for different levels of particle removal. The values for $COD_{influent}$ represent measurements that were done within the research work of chapter 5. The given values for $N_{kj,i}$ reflect the concentrations of 'average' Dutch wastewater taking into account the COD/N ratio of 11:1. Taking the average $COD_{influent}$ during the experiments (528 $mgO_2.l^{-1}$), N-Kjeldahl becomes 48 $mgN.l^{-1}$. In practice an additional N load coming from the reject water of the sludge digester can be expected. As indicated in chapter 7 the extra N load amounts around 15% compared to the influent N-Kjeldahl. Therefore, in the calculations an average $N_{kj,i}$ of 54 $mgN.l^{-1}$ was assumed.

**Table 6.7**: Influent characteristics, and calculated system characteristics and maximal denitrification

| | **Influent characteristics** | | | **System characterics** | | **Maximal denitrification** |
|---|---|---|---|---|---|---|
| **particle removal** | **$COD_{influent}$ ($mgO_2.l^{-1}$)** | **DP ($mgN.l^{-1}$)** | **Nkj,i ($mgN.l^{-1}$)** | **$X_{SS}$ ($mgSS.l^{-1}$)** | **$N_{endogenous}$ ($mgN.l^{-1}$)** | **$N_{denitrified,\ max}$ ($mgN.l^{-1}$)** |
| 0% | 528 | 38 | 54 | 6.2 | 21 | 59 |
| 30% | 420 | 32 | 51 | 4.8 | 17 | 49 |
| 46% | 387 | 30 | 50 | 4.3 | 15 | 45 |
| 68% | 317 | 26 | 48 | 3.4 | 12 | 38 |
| 78% | 250 | 23 | 47 | 2.6 | 9 | 32 |
| 90% | 219 | 21 | 47 | 2.2 | 8 | 29 |
| 100% | 190 | 19 | 45 | 1.7 | 6 | 25 |

The results of the calculations are presented in figure 6.8, showing the $NO_{2+3}$-$N_{,e}$ concentration for different levels of particle removal and for three different recycle ratios (3, 5 and 7). The graph reveals that the $NO_{2+3}$-$N_{,e}$ concentration is determined by the recycle ratio, for particle removals < 90%. To ensure an effluent quality < 10 $mgN.l^{-1}$ a recirculation ratio > 3 will have to be applied, as the inert nitrogen ($N_{i,e}$ ~ 4 $mgN.l^{-1}$) should also be taken into account. For particle removals > 90% the denitrification capacity of the system becomes limited, resulting in increasing $NO_{2+3}$-$N_{,e}$ concentrations.

The calculations have an indicative character - the DP was determined only for the investigated wastewater and the activated sludge parameters may vary upon temperature - but they provide a good indication of the effluent nitrogen values that can be expected when physical-chemical pretreatment is applied in an activated sludge system. Moreover, they demonstrate that the maximal denitrification in an activated sludge plant is partially dependent on the particle removal. Therefore, it can be concluded that the turbidity-related dosing strategy of chapter 5 can be used to adjust the effluent nitrogen concentration.

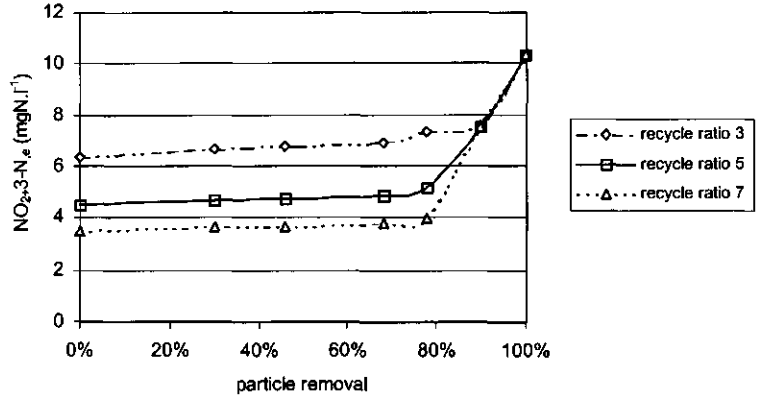

**Fig. 6.8:** Calculated values for the effluent $NO_{2+3}$-N concentration of an activated sludge system; the calculations are made for wastewater with an 'average' Dutch N-Kjeldahl concentration and different levels of pretreatment

**The effect of biological phosphorus removal on denitrification** – In phase II of the pilot experiment the particle removal was increased from 30% to 78%. The effluent $NO_{2+3}$-N concentration directly showed an increase of 3.7 $mg.l^{-1}$, due to the decrease in the biodegradable COD of the wastewater. The $PO_4$-P concentration remained at 0.6 $mg.l^{-1}$, indicating that the biological P removal was not affected by the higher level of pretreatment. After seven days a second increase in the $NO_{2+3}$-N concentration of approximately 4 $mg.l^{-1}$and a simultaneous increase in the P concentration 5 $mg.l^{-1}$ were found. Both theses increases were apparently related to the decreased presence of PAOs, caused by the increased nitrate concentration in the anaerobic zone. Because of the presence of nitrate, volatile fatty acids were for a larger part consumed by denitrifying heterotrophic bacteria. The simultaneous increase of the $NO_{2+3}$-N and P however, indicates that denitrifying PAOs had a contribution to the total denitrification during the first days of increased particle removal.

It has been shown (RWZI-2000, 1994; Kuba, 1996) that anoxic phosphate accumulation can contribute to the total fraction of nitrogen that is denitrified in an activated sludge system. This is advantageous for a system receiving wastewater that is treated with organic poly electrolytes. However, to achieve anoxic phosphate accumulation, recirculation of nitrate through return sludge should be avoided. To minimize this nitrate recirculation, process modifications like are applied in the UCT (*University of Cape Town*) process and the BCFS® (*Biological and Chemical Phosphate and Nitrogen removing reactor*; van Loosdrecht et al., 1998) process have shown to be effective. In these configurations, sludge is returned from the secondary clarifier to the beginning of the anoxic zone. A second recycle flow is used to transport sludge from the end of the anoxic compartment to the anaerobic zone.

An illustrative example of the contribution of denitrifying PAOs to the total nitrogen removal is a model study performed by Hao *et al.* (2000). For a planned wastewater treatment plant based on the BCFS® process, they performed a study based on a metabolically structured model for bio-P removal and Activated Sludge Model 2/2d for COD and N removal. In their calculations they assumed that 80-90% of $COD_{particulate}$ would be removed in an enhanced primary treatment step, resulting in a influent COD of 217 $mgO_2.l^{-1}$ and a N-Kjeldahl of 45 $mgN.l^{-1}$. Their calculations indicated that for such a situation the contribution of anoxic P removal to the total denitrification would be circa 4 $mgN.l^{-1}$.

## Conclusions

The Nitrate Uptake Rate (NUR) experiments showed that the $COD_{particulate}$ fraction (i.e. 65% of $COD_{total}$) of the wastewater that was used in the experiments contributes circa 50% to the total denitrification potential (DP). The other 50% is related to $COD_{dissolved}$. This finding indicates that if all the particles are removed, the biodegradable COD of the investigated wastewater is reduced by 50%. Moreover, it shows that the effect of particle removal on the removal of biodegradable COD is smaller than is estimated following the decrease in the (frequently used) $COD_{total}/N$ ratio.

The sum of the nitrite and nitrate ($NO_{2+3}$-N) concentration in the effluent leaving the activated sludge system exceeded the nitrogen limit during the entire experiment. The N-Kjeldahl of the tested wastewater was, however, high compared to average Dutch influent values (70 $mgN.l^{-1}$ versus 50 $mgN.l^{-1}$, respectively). Calculations based on the experimental values indicate that, in well-designed plants receiving an 'average' Dutch N-Kjeldahl concentration, a high level of particle removal (i.e. 80-90%) is possible without exceeding the nitrogen effluent constraint. The experiment also showed that pretreatment with organic polymers does not have a direct effect on the biological phosphorus removal of a down-stream activated sludge system. The removal of phosphate in the anoxic zone might even contribute to the total denitrification. In the experiment an increase in denitrification capacity of approximately 4 $mgN.l^{-1}$ was found. Biological phosphorus removal can indirectly be affected by nitrate that is recirculated to the anaerobic zone through the recycling of sludge.

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# 7 Final scenario evaluation

**Abstract**. This chapter describes an updated scenario study, evaluating the implications of the application of cationic organic polymers instead of $FeCl_3$. With DEMAS+, the evaluation tool that was described in chapter 2, the environmental impacts and costs of four scenarios were assessed. All scenarios included an activated sludge system for secondary treatment, but varied with regard to the pretreatment. The reference scenario contained a primary clarifier without dosing, while scenario A and B accommodated a clarifier complemented with dosage of $FeCl_3$ and cationic, high molecular weight polymer, respectively. Scenario C was based on a dissolved air flotation (DAF) unit using organic polymers.

The scenario study revealed that the application of organic poly electrolytes combined with either settling or flotation results in a 60-75% lower energy requirement, in 10-25% saving of space (footprint) and in a somewhat higher final sludge production (~ 10%) compared to the reference scenario. The assessment showed that polymer dosing combined with either settling or flotation is a cost effective option for the treatment of municipal wastewater. The overall costs of both scenarios were similar or potentially lower, compared to those of the scenario using $FeCl_3$ and the reference scenario. The higher chemical costs in a scenario with poly electrolytes are compensated by the lower costs for final sludge disposal and construction/electronic-mechanic works.

## Introduction

In this chapter the implications of coagulation/flocculation with cationic organic polymers instead of $FeCl_3$ for a complete wastewater treatment system are evaluated. Based on these experimental results an updated scenario study is made for a small group of scenarios. By means of this scenario study the consequences of the application of polymers for the environmental impacts and costs of wastewater treatment scenarios with physical-chemical pretreatment are investigated.

## Methodology

The evaluation was performed with DEMAS+, the evaluation tool that was described in chapter 2, using the same methodology and assumptions, except for the aspects that will be described in the following paragraphs. Similar to chapter 2, the evaluated systems were designed for a wastewater treatment plant of 100,000 population equivalents with system boundaries that include the wastewater treatment and the on-site sludge handling (digestion and dewatering, see figure 2.7).

Table 7.1 presents the four scenarios that were evaluated in this chapter. All scenarios accommodate an activated sludge system for secondary treatment. The reference scenario is the same as in chapter 2 (figure 2.8). Scenarios A and B include a primary clarifier with dosage of $FeCl_3$ and cationic organic poly electrolytes, respectively. Scenario C consists of a dissolved air flotation (DAF) system combined with coagulation/flocculation by cationic organic polymers.

**Table 7.1:** Scenarios evaluated in this chapter

| Scenario | Pretreatment | Post treatment |
|---|---|---|
| Reference | clarifier | Low loaded activated sludge system + secondary clarifier (BOD, N, P.) |
| Scenario A | clarifier with $Fe^{3+}$ dosing | Low loaded activated sludge system + secondary clarifier (BOD, N) |
| Scenario B | clarifier with cationic organic polymer | Low loaded activated sludge system + secondary clarifier (BOD, N, P) |
| Scenario C | DAF with cationic organic polymer | Low loaded activated sludge system + secondary clarifier (BOD, N, P) |

**Design criteria** – The design criteria for the scenarios are shown in table 7.2. In scenarios B and C the applied dose of cationic, high molecular weight (HMW) organic poly electrolytes was 8 mg.l$^{-1}$ (i.e. 5.5 mg polymer per 100 NTU, see chapter 5). The primary clarifiers were designed with a surface load of 3 m.h$^{-1}$ (although the experiments of chapter 3 indicate that if organic polymers are applied, the load may be increased, because of the formation of relatively large sludge flocs). The DAF unit was designed with a surface load of 10 m.h$^{-1}$. The assumed dry solids concentration of the primary sludge produced by the clarifiers and the DAF was 1.0% and 10% (chapter 4), respectively. Scenario A, B and C include a mixing unit for coagulation/flocculation.

To design activated sludge systems with biological nitrogen removal, various methodologies can be applied. The (low loaded) activated sludge systems in chapter 2 were designed with a sludge loading rate of 0.06 kgBOD. kgSS$^{-1}$·day$^{-1}$. However, as pointed out by different authors (a.o. Ødegaard and Karlsson (1994)Kujawa-Roeleveld, 2000) the biomass activity of an activated sludge system after enhanced primary treatment may increase, because less inorganic material is entering the system. This, in its turn allows a higher sludge loading rate and therefore a more compact design. In the framework of this evaluation a more detailed design is made with the frequently applied *Hochschull Gruppe Ansatz* (HGA) method. The HGA method is a design method developed in an exchange of different German universities (ATV-A131, 1991) and was proposed as a design method for The Netherlands by STOWA in 1993.

The most important design criteria of the HGA method are the characteristics of the influent, the required effluent concentrations and the minimum process temperature for which these effluent concentrations must be achieved. A peek factor has to be introduced in the calculations to deal with variations in BOD loading. Furthermore, the model requires values for a number of system parameters, like the growth rates and yield of the autotrophic and heterotrophic biomass, the inorganic fraction of the influent TSS and the inert fraction of the sludge.

The sludge loading rates that are shown in table 7.2 were calculated based on the wastewater characteristics as given in table 7.3. The minimum process temperature was set at 10 °C. With regard to the system parameters the proposed default values were used as proposed by STOWA (STOWA, 1993; STOWA, 1995).

**Table 7.2:** Design parameters of the evaluated treatment scenarios

| | **Surface load pre-treatment** **(m·h$^{-1}$)** | **Dosing** | **Activated sludge loading rate** **(kgBOD. kgSS$^{-1}$·day$^{-1}$)** | **Surface load secondary clarifier** **(m·h$^{-1}$)** |
|---|---|---|---|---|
| Reference | primary clarifier (3 m·h$^{-1}$) | -- | 0.06 | 0.7 |
| Scenario A | primary clarifier (3 m·h$^{-1}$) | 15 mg Fe$^{3+}$.l$^{-1}$ + 0.5 mg.l$^{-1}$ anionic polymer | 0.08 | 0.7 |
| Scenario B | primary clarifier (3 m·h$^{-1}$) | 8 mg.l$^{-1}$ cationic HMW polymer | 0.08 | 0.7 |
| Scenario C | DAF unit (10 m·h$^{-1}$) | 8 mg.l$^{-1}$ cationic HMW polymer | 0.08 | 0.7 |

**Influent and effluent composition of the pretreatment** - Table 7.3 shows the assumed influent and effluent composition of COD, BOD, N, P and TSS. Similar as in chapter 2, all pollutants were divided into a particulate and a dissolved fraction. The influent composition reflects the wastewater that was used in the experiments with the exception of the value taken for N-Kjeldahl. The N-Kjeldahl value was calculated based on the COD/N ratio of 11:1, taking into account the average N-Kjeldahl concentration of Dutch wastewater (chapters 1 and 6).

The composition of the primary effluent is based on the particles removal efficiencies ($R_{particles}$) that were found in chapter 4 and 5. With regard to the reference scenario a particle removal efficiency of 30% was assumed. Chapter 4 indicated that the particle removal performances of DAF and settling combined with the organic polymers are the same. Therefore, for scenarios A and B as well as for scenario C efficiencies of 80% were presumed. The calculations of chapter 6 indicate that at this level of particle removal, still sufficient biodegradable COD is available to be able to comply with the nitrogen effluent constraint. The addition of $FeCl_3$ in scenario A results in a low concentration of phosphate in the primary effluent.

**Table 7.3**: Composition of influent and effluent of the pretreatment unit operations

| | **Influent** | **Reference** ($R_{particles}$ 30%) | **Scenario A** ($R_{particles}$ 80%) | **Scenario B** ($R_{particles}$ 80%) | **Scenario C** ($R_{particles}$ 80%) |
|---|---|---|---|---|---|
| **$COD_{total}$** ($mg\ O_2.l^{-1}$) | **525** | **422** | **250** | **250** | **250** |
| $COD_{particulate}$ | 343 | 240 | 69 | 69 | 69 |
| $COD_{dissolved}$ | 182 | 182 | 182 | 182 | 182 |
| **BOD** ($mg\ O_2.l^{-1}$) | **200** | **168** | **115** | **115** | **115** |
| $BOD_{particulate}$ | 107 | 75 | 21 | 21 | 21 |
| $BOD_{dissolved}$ | 93 | 93 | 93 | 93 | 93 |
| **N-Kjeldahl** ($mg\ N.l^{-1}$) | **48** | **52** | **48** | **48** | **48** |
| $N_{particulate}$ | 8 | 6 | 2 | 2 | 2 |
| $N_{dissolved}$ | 40[a] | 46[b] | 46[b] | 46[b] | 46[b] |
| **$P_{total}$** ($mg\ P.l^{-1}$) | **9.0** | **9.9** | **2 - 3** | **7.8** | **7.8** |
| $P_{particulate}$ | 4.2 | 3.0 | | 0.8 | 0.8 |
| $P_{dissolved}$ | 4.8[a] | 6.9[b] | | 6.9[b] | 6.9[b] |

[a] Exclusive reject water [b] Including N and P from reject water ($NH_4$-N: +6 $mgN.l^{-1}$; $PO_4$-P: +2 $mgP.l^{-1}$)

**Cost calculations** – The cost calculations were performed with the net present value method, as explained in chapter 2. The cost assumptions of the flocculating agents are shown in table 7.4. For the other cost assumptions, please see appendix A of chapter 2.

**Table 7.4**: Cost assumptions of flocculating agents

| **Item** | **Cost** |
|---|---|
| Cationic HMW polyacrylamide | € 3,400 per 1,000 kg polymer |
| $FeCl_3$ | € 455 per 1,000 kg Fe |

## The application of organic polymers instead of $FeCl_3$

Figure 7.1 presents the results of the energy calculations. The assessed energy requirement of the reference scenario is 1,200 $MWh.year^{-1}$, as was already shown before in chapter 2. The DEMAS+ calculations indicate that scenarios A, B and C require 280, 275 and 485 $MWh.year^{-1}$, respectively, which are reductions of 59 to 77% compared to the reference. The decreased energy demand is the result of the production of more biogas and the reduced necessity for aeration in the activated sludge system. The difference between scenario B and C is caused by the extra electricity requirement of the DAF system which is mainly related to the pressurized recycle flow.

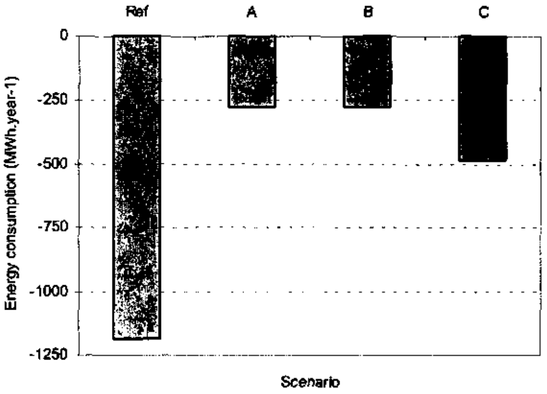

**Fig. 7.1:** Energy balance of the evaluated scenarios

Figure 7.2 presents the calculated final sludge production after digestion and centrifugation. As is shown, scenario A (with $FeCl_3$ addition) produces the largest amount of final sludge because of the production of chemical sludge. Scenarios B and C also show a small increase (~ 10%) in sludge production compared to the reference. It should be noticed that this finding is the result of the assumptions made with regard to the sludge production in the biological system (e.g. biological yield of 0.4 $kgSS.kgCOD_{removed}^{-1}$ used for the activated sludge system) and the digestibility of primary (50%) and the secondary sludge (30%). In fact, these values proved to have significant determining power with respect to the overall outcome of the scenario evaluation and should be subject to critical study.

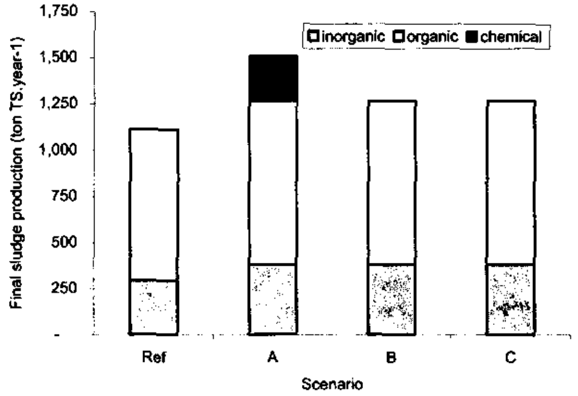

**Fig. 7.2:** Sludge production of the evaluated scenarios

In figure 7.3 the net space requirement of the evaluated scenarios is shown. The calculated footprint of the reference scenario is 9,500 $m^2$. Due to the enhanced pretreatment of scenarios A, B and C, the contaminant load to the activated sludge systems can be reduced, which in turn also induces a higher possible sludge loading rate (table 7.2). Both these factors, and the higher hydraulic load of the DAF system, result in space savings. Scenarios A and B require approximately 8,000 $m^2$, while scenario C (with DAF) needs around 7,500 $m^2$.

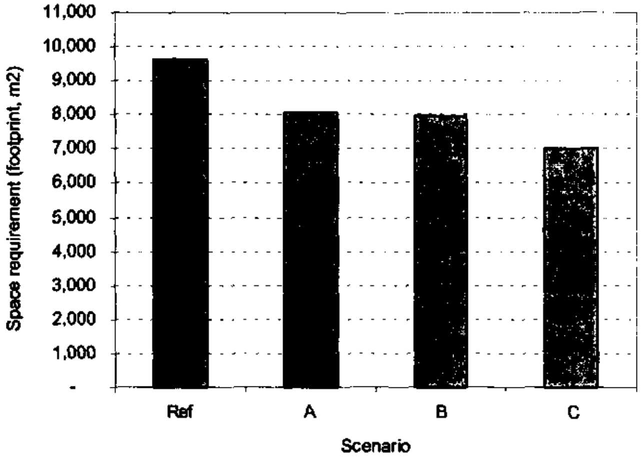

**Fig. 7.3:** Net space requirement of the evaluated scenarios

Table 7.5 presents the results regarding the use of flocculating agents ($FeCl_3$ and organic poly electrolytes) as well as the increase in salinity of the effluent due to the addition of chloride. The values of the increase of salinity are based on the chloride that is released through the applied flocculating agents. As was discussed in chapter 3, the cationic polyacrylamide-based polymers release approximately 0.05 mg $Cl^-$ per gram, while by ferric chloride addition 1.9 mg $Cl^-$ per mg $Fe^{3+}$ is added.

**Table 7.5:** Use of flocculating agents and increase in salinity of the effluent

| | Reference | Scenario A | Scenario B | Scenario C |
|---|---|---|---|---|
| Chemicals (ton.$year^{-1}$) | | | | |
| - $FeCl_3$ | -- | -- | 315 | -- |
| - Organic poly electrolyte* | 10.0 | 20.2 | 66.8 | 66.8 |
| Effluent quality ($mg.l^{-1}$) | | | | |
| - $Cl^-$ | + 0 | + 29 | + 0.4 | + 0.4 |

* Including poly electrolytes that are used for dewatering

Figure 7.4 shows the calculated net present values of the four evaluated scenarios. The net present values are given relative to the reference scenario. The calculations distinguish among the total costs for civil construction and electro-mechanical (EM) works and the costs for energy demand, the use of chemicals, final sludge disposal and personnel/maintenance.

As is shown in the graph, the total net present value of all four scenarios is approximately the same. A small difference is found between scenario A and B, indicating that the application of organic poly electrolytes might potentially save costs in comparison with the application of $FeCl_3$. However, this difference should be reckoned negligible, considering the uncertainties that are of the cost calculations. Still, the graph clearly illustrates that the higher cost of chemicals in scenario A (due to the application of polymers) are compensated by the lower cost of final sludge disposal and construction/EM works. In fact, all three scenarios with physical-chemical pretreatment show a reduction in costs of construction/EM works as the result of the smaller design. They compensate the higher operational costs that are related to the addition of flocculating chemicals.

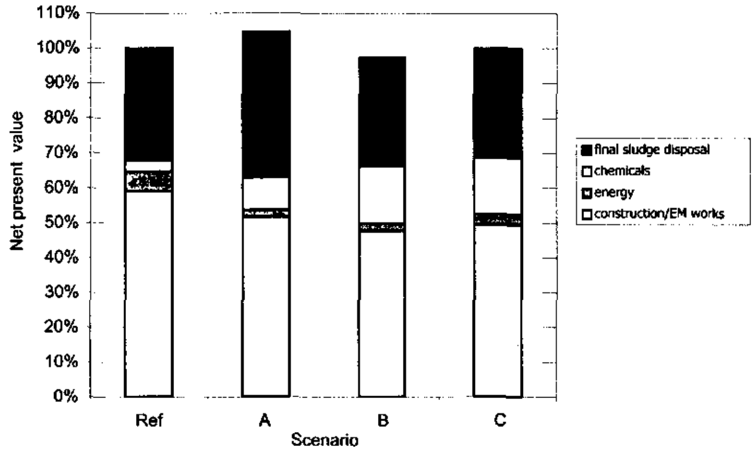

**Fig. 7.4:** Financial cost comparison based on the net present value (the reference scenario =100%)

## Conclusions

The updated scenario study that is described in this chapter reveals that the application of organic poly electrolytes combined with either settling or flotation results in a 60-75% lower energy requirement, in 10-25% saving of space (footprint) and in a somewhat lower final sludge production (5-10%) compared to a reference scenario. The study also showed that polymer dosing combined with either settling or flotation is a cost effective option for the treatment of municipal wastewater. The overall costs of scenarios that include organic poly electrolytes in the pretreatment followed by an activated sludge system for secondary treatment are similar or potentially lower, compared to those of a scenario using $FeCl_3$ and the reference scenario. The higher chemical costs in a scenario with poly electrolytes are compensated by the lower costs for final sludge disposal and construction/EM works.

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# 8 Summary and concluding remarks

## Summary

This dissertation describes a study including experimental work on the concept of physical-chemical pretreatment with the aim to evaluate its potential to design more sustainable wastewater treatment plants. *Physical-chemical pretreatment* is meant to separate suspended and colloidal particles in the first step of treatment. By removing the particles a large part of the pollutants can be removed in a relatively easy way. The organic concentrate of separated particles does not go waste, but is considered a product that can be used for the production of biogas through digestion. As a result of the removal of particles, the contaminant load of pursuing unit operations is reduced, so that they may be designed smaller. Moreover, since the primary effluent contains low concentrations of particles, alternative treatment techniques might be applied for secondary treatment, including physical-chemical techniques that are feasible to recover nutrients or improve the effluent quality.

The investigation described in this dissertation was performed with the general objectives (i) to evaluate the concept of physical-chemical pretreatment for its potential to design more sustainable treatment wastewater systems as against treatment plants currently applied; and (ii) to contribute to further development of the concept of physical-chemical pretreatment by performing experimental research into encountered bottlenecks.

The literature review in **Chapter 1** illustrates that the contaminants in municipal wastewater are to a large extent present in the form of not-dissolved, particulate material. The average values of several Dutch wastewater sources showed that approximately 70% COD, 60% BOD, 25% N-Kjeldahl and 30% $P_{total}$ can be removed by the separation of all particles with a size > 0.45 µm.

**Chapter 2** describes a study on various treatment scenarios with physical-chemical pretreatment. Based on a literature review of unit operations, ten different scenarios for wastewater treatment were designed and evaluated along environmental and financial criteria. This evaluation paid particular attention to the energy balance, final sludge production, the possibility to apply nitrogen recovery techniques, expected effluent quality, requirement of land as well as the financial feasibility of the identified scenarios. These criteria were selected after the review as given in chapter 1 of publications addressing the environmental 'non-sustainability' of wastewater treatment plants.

The literature review revealed that a variety of unit operations can be applied for enhanced primary treatment, such as pre-precipitation, dissolved air flotation, microscreening, direct influent filtration, direct membrane filtration and enhanced settling by addition of magnetite or micro sand. In order to obtain a significant level of particle removal, the use of flocculating agents is essential to all these unit operations, except for direct influent filtration and direct membrane filtration.

Various unit operations are available for secondary and tertiary treatment of the primary effluent. A major distinction can be made into operations based on biological techniques like activated sludge systems, biofilm systems and membrane bioreactors; and unit operations based on physical-chemical pollutant removal like ion-exchange, activated carbon and rapid sand filtration. The latter unit operations may induce an improved effluent quality with regard to suspended solids, micro pollutants and nitrogen, respectively.

The evaluation of scenarios showed that wastewater treatment systems with physical-chemical pretreatment can potentially be designed more energy efficient and smaller than the reference scenario. Furthermore, it was shown that the total costs (investment as well as operational) of treatment scenarios that combine physical-chemical pretreatment with biological post treatment are in the same range as the reference scenario. The costs of scenarios that include ion-exchange and a stripper to recover nitrogen are at least 75% higher than the reference (the possible profits of the re-usable product have not been taken into the calculations) and in a high energy requirement. The scenarios that include activated carbon adsorption result in 300% higher total costs.

The scenario study revealed two important bottlenecks regarding the application of physical-chemical pre-treatment: (i) the use of metal-based flocculating agents leads to an increased sludge production and an increased salinity of the effluent; and secondly (ii) the particle removal is partially accompanied by the removal of biodegradable COD which is required later on at the denitrification step (of a biological post treatment system). Of course, without proper denitrification the nitrogen concentration of the effluent might exceed the effluent standards. The experimental work described in chapter 3 to 6 addressed these two bottlenecks.

The first bottleneck is dealt with in chapters 3 and 4, investigating the use of organic poly electrolytes as alternatives to metal-based flocculating agents. When organic polymers are applied for flocculation, chemical sludge production is absent because no precipitates are formed. Moreover, organic poly electrolytes induce only a minor increase in salt concentration.

**Chapter 3** gives a description of a literature review as well as of experiments exploring the application of polymers. From the review it was found that two types of organic poly electrolytes are most frequently applied in (industrial) wastewater treatment and sludge handling, namely (i) low to moderate molecular weight polyamines and poly-DADMACs and (ii) high molecular weight (HMW) polyacrylamides. Jar test experiments with a selection of these polymer types showed that especially the cationic HMW polyacrylamides have interesting characteristics for practical application, as they are effective at relatively low doses (5-10 $mg.l^{-1}$). For a low molecular weight cationic polyamine also high turbidity removals were found, however at higher required doses (20-30 $mg.l^{-1}$). The application of both types of polymers to municipal wastewater resulted in turbidity removals of 65-90% and a suspended solids removal of more than 90%. Furthermore, it was found that the addition of organic polymers does not affect the pH of the treated wastewater.

The addition of HWM cationic polymers tends to result in large, well-settling flocs. The sludge flocs formed after addition of LMW cationic polymers appeared to be significantly smaller. Filterability tests with the Filtration-Expression cell showed that the improvement in dewaterability of sludge produced by the addition of polymers is similar to that of sludge produced by coagulation/flocculation with $FeCl_3$.

**Chapter 4** deals with pretreatment of municipal wastewater by dissolved air flotation (DAF). It describes batch flotation experiments as well as experiments with a DAF pilot unit. The investigations showed that DAF combined with the use of HMW cationic polymers is an effective unit operation for particle removal from primary wastewater. In the batch flotation experiments a turbidity removal of 75% was found at poly electrolyte doses of 4-6 $mg.l^{-1}$. The results indicate that - at least in the case of jar test experiments - dissolved air flotation and settling have the same particle removal performance.

The continuous flotation experiment showed a lower turbidity removal than the batch experiments (maximal 70%), which was due to suboptimal mixing of the polymer with the wastewater. Additional jar tests were performed and showed that the turbidity removal could be increased (cq. poly electrolyte could be saved) by optimizing the mixing conditions. The dry solids concentration of the flotation sludge amounted 3-5% for continuous float layer removal and 8-10% for discontinuous float layer removal.

The experiments with the continuous DAF unit combined with a fixed dose of polymer concentration per liter wastewater resulted in an unpredictable particle removal efficiency, due to fluctuations in the particle concentration of the used wastewater. Data analysis showed that, in order to decide which quantities of organic polymer should be dosed to a continuous system, the $COD_{particulate}$ of the influent should be known at forehand. The data analysis also demonstrated that online turbidity measurements could be a potential tool to monitor $COD_{particulate}$ of the wastewater as a linear relation was found between both parameters.

The second bottleneck as found in chapter 2 (removal of biodegradable COD during pretreatment) is addressed in chapter 5 and 6. **Chapter 5** describes investigations with regard to the implementation of a turbidity-related polymer dosing strategy in order to create different degrees of particle removal. The experiments of this chapter confirmed the results of chapter 4, showing that on-line turbidity measurements could be used to quantify $COD_{particulate}$ of influent as well as of primary effluent. The application of on-line turbidity measurements revealed that the particle concentrations in the tested municipal wastewater vary significantly. Experiments with turbidity-related polymer dosing, using a cationic HMW polyacrylamide, confirmed that this method could be used to create different, constant levels of particle removal. Moreover, the experiments with turbidity-related dosing resulted in higher removal efficiencies and a more stable operation compared to the dosing of a fixed polymer dose, as an overdose at low influent turbidity was avoided.

The objective of **chapter 6** was to determine the effect of different levels of particle removal on the elimination of nitrogen and phosphorus of an activated sludge system. The chapter describes *Nitrogen Uptake Rate* (NUR) experiments, which were performed to evaluate the effect of particle removal on the so-called denitrification potential (DP), and experiments with a pilot scale activated sludge plant (1,000 l) that received pre-treated wastewater.

The NUR experiments showed that the particulate COD fraction of the investigated wastewater (~ 65% of $COD_{total}$) contained around 50% of the total DP. This finding indicates that if all the particles are removed, the biodegradable COD of the investigated wastewater is reduced by 50%.

The sum of the nitrite and nitrate ($NO_{2+3}$-N) concentration in the effluent leaving the activated sludge system exceeded the nitrogen limit during the entire experiment. The N-Kjeldahl of the tested wastewater was, however, high compared to average Dutch influent values (70 $mgN.l^{-1}$ versus 50 $mgN.l^{-1}$, respectively). Calculations based on the experimental values indicate that, in well-designed plants receiving an 'average' Dutch N-Kjeldahl concentration, a high level of particle removal (i.e. 80-90%) is possible without exceeding the nitrogen effluent constraint. The experiment also showed that pretreatment with organic polymers is not direct affecting biological phosphorus removal of a downstream activated sludge system. The removal of phosphate in the anoxic zone might even contribute to the total denitrification. In the pilot experiment an increase in denitrification capacity of approximately 4 $mgN.l^{-1}$ was found.

**Chapter** 7 describes an updated scenario evaluation based on the conclusions of the experimental work with organic polymers. This evaluation showed that the use of polymers for coagulation/flocculation in combination with either settling or flotation results in a 60-75% lower energy demand and a 10-25% lower land requirement (considering the footprint of the reactor) compared to the reference system. The calculated final sludge production was ~10% higher. It was found that the overall costs of scenarios that apply organic poly electrolytes in the pretreatment and (in the case of the updated evaluation) an activated sludge system for secondary treatment, are similar and might be even lower than a scenario $FeCl_3$ or the reference scenario. The costs of chemicals are higher in a system with poly electrolytes, but they are compensated by the lower costs of final sludge disposal and less expensive construction and electro-mechanic works.

## Concluding remarks

The overall objectives of the investigation that is described in this dissertation were (a) to evaluate the concept of physical-chemical pretreatment as an option to improve the environmental sustainability of wastewater treatment plants and (b) to contribute to its further development by performing experimental research into encountered bottlenecks. It was shown in chapters 2 and 7 that physical-chemical pretreatment in combination with secondary biological treatment (i.e. activated sludge or biofilm) can contribute significantly to an improved sustainability with regard to the energy demand. This type of scenarios requires approximately 60-75% less energy compared to commonly applied configurations, because of the production of (more) biogas and the reduced necessity for aeration in the post treatment. Moreover, the application of enhanced pretreatment allows the design of smaller treatment plants. In terms of footprint, 20% (combined with activated sludge) to 80% (combined with high rate biofilm process) less space is required. The evaluation also showed that these scenarios are economically feasible.

With regard to the other criteria that were formulated to evaluate the environmental sustainability of scenarios (see table 1.6) it appeared that (i) the total final sludge disposal of these systems is approximately the same or a little higher (approximately 10%) compared to the reference scenario, (ii) the effluent quality is mainly dependent on the applied secondary, cq. tertiary treatment steps; and (iii) recovery of nitrogen is possible by applying an ion exchanger combined with air stripping; this last option however results in 75% higher treatment costs and a very high energy requirement.

The experimental work into the encountered bottlenecks showed that cationic organic polymers can be applied instead of metal-based flocculating agents without increasing the cost, while preventing the formation of chemical sludge and an increased salinity of the effluent. They therefore provide a solution to the first identified bottleneck. For practical application more attention could be paid to the application of organic polymers that are not acrylamide based, like for example the biosynthesized polymers mentioned in chapter 3.

With respect to the second bottleneck (removal of biodegradable COD during pretreatment, possibly affecting the nutrient elimination in secondary treatment), chapter 6 indicated that particle removals as high as 80-90% appear to be possible without exceeding the effluent nitrogen constraint. Moreover, as shown in chapter 5 and 6, the removal of particulate COD can be controlled by turbidity-related polymer dosing and thus be adjusted to the desired nitrogen effluent concentration.

The activated sludge pilot trials of chapter 6 also indicated that biological phosphorus removal in activated sludge systems is not directly affected by pretreatment with organic polymers. The removal of phosphate in the anoxic zone might even contribute to the total denitrification capacity. The activated sludge models that incorporate anoxic phosphorus removal (e.g. Kuba *et al.*, 1996; Hao *et al.*, 2000) indicate the same. To prevent the loss of the phosphate accumulating activity in general and the anoxic phosphate removal in particular, the recirculation of nitrate through return sludge should be minimized. The activated sludge scheme according to the UCT design (*University of Cape Town*) and the modification of UCT, the BCFS® process, offer a process configuration in which this can be achieved. These configurations therefore appear interesting systems in combination with pretreatment using polymers. Taking into the account the experimental results and the considerations mentioned, it appears that also the second bottleneck can be overcome.

**Final sludge production, effluent quality and nutrient recovery** - A point of attention concerns the final sludge production of the scenarios with physical-chemical pretreatment. The scenario study indicates that enhanced primary pretreatment (even with organic poly electrolytes) in combination with an activated sludge system, tends to have a final sludge production that is slightly higher (approximately 10%) in comparison with the reference. Yet, this finding might be the result of the assumptions made with regard to the final sludge production, i.e. the biological yield of 0.4 $kgSS.kgCOD_{removed}^{-1}$ used for the activated sludge system and the digestibility of primary (50%) and the secondary sludge (30%). In fact, these values proved to have significant determining power with respect to the overall outcome of the scenario evaluation and should be subject to critical study. The results of the scenario evaluation in chapter 2, suggests that the application of a biofilm system for secondary treatment entails a somewhat lower sludge production (due to the assumed value of 0.2 $kgSS.kgCOD_{removed}^{-1}$). Still, as there seem to be hardly any data on the biological yield of biofilm systems this aspect of biofilm systems calls for further investigation.

As was mentioned, the effluent quality of the composed treatment scenarios proved mainly dependent on the applied secondary/tertiary treatment steps. Enhanced particle removal may be beneficial since it accommodates the application of post treatment steps like activated carbon adsorption, ion exchange and rapid sand filtration which may induce an improved effluent quality. The scenario evaluation however shows that the application of activated carbon adsorption and ion exchange result in a significant increase in energy demand and treatment costs. Moreover, the application of ion exchange will result in an increased salinity of the effluent.

When biofilm systems are applied after pretreatment with polymers, the removal of phosphate will be a problem. These systems are normally applied in combination with pre-precipitation with metal salts. Some lab-scale investigations have recently been reported on biological phosphorus removal by biofilm systems (Falkentoft *et al.*, 2001; Helness and Ødegaard, 2001). Although there is not yet a conclusive answer, the publications indicate that it may be applicable.

With respect to reuse of nutrients from municipal wastewater, this investigation has only considered nitrogen recovery. Lately, there is a growing interest of the phosphate producing industry in recovery of phosphorus from municipal wastewater (CEEP, 2000, 2001). Some reported suggestions to recover phosphorus are to retrieve it from activated sludge systems with biological phosphorus removal. One method would be to withdraw phosphate-rich water from the anaerobic zone and precipitate the present P (e.g. Brandse, 2000; Klapwijk, 2001). A second suggestion is to process P containing waste sludge (e.g. Hultman, 2001). Both measures appear well applicable in combination with pretreatment using polymers.

**Increased flexibility of treatment plants** - A benefit of pretreatment that not has been mentioned so far, is that it can improve the flexibility of municipal wastewater treatment plants. Especially in configurations that consist of a primary clarifier followed by an activated sludge plant, the addition of organic polymers is a potential tool to extend the capacity in case of a structural increase of load. The addition of polymers increases the particle removal efficiency of the primary clarifier, which implies a lowered load to the activated sludge system. The decreased particle load to this system entails an increase of activated sludge system. The decreased particle load to this system entails an increase of the sludge activity, which in its turn allows a higher sludge loading rate. In this way the addition of polymers can prevent considerable (and costly) physical restructuring of the plant. Such a measure can also be used at to treatment plants in areas with a seasonal increase in the contaminant load, like in tourist areas.

At treatment plants that do not include a primary clarifier, dissolved air flotation appears a feasible option to introduce prior to an activated sludge system. This unit operation is relatively compact, showed almost similar investment costs as a primary clarifier and is relatively easy to install (it is even available on 'turn-key' basis). Moreover, it produces a concentrated flow of primary sludge which implies that there is no need to extend existing sludge handling equipment.

**Further development of the concept of physical-chemical pretreatment** – Based on the results of this investigation and the parallel research into particle characteristics (Van Nieuwenhuijzen, 2001; STOWA, 2001) some recommendations can be made concerning the further development of the concept of physical-chemical pretreatment as an option for increased sustainability of wastewater treatment plants.

- As mentioned, pretreatment with polymers seems very well applicable in combination with activated sludge systems of the UCT or BCFS® design. It is recommended to perform pilot experiments with such a treatment scenario in order to confirm the findings of this thesis. The turbidity-related polymer dosing can be used to optimize the level of particle removal with regard to the effluent $N_{total}$ constraint. The trials should preferably be performed with a wastewater that contains 'average' N-Kjeldahl concentrations.
- The application of biofilm systems entails a compact treatment system and might induce a lower sludge production. However, if biofilm systems are used in combination with pretreatment using polymers, the effluent will presumably not comply with the phosphorus standard. The question is raised if biological phosphorus removal can be applied in biofilm systems and some positive results are already reported. Fundamental research could address this question and could also provide data on the biological sludge production.
- The parallel investigation into the size distribution of contaminants in municipal wastewater proved to be valuable with regard to the choice and design of pretreatment operations. Further investigation into a methodology to characterize the wastewater with regard to particles, as well as investigations into the particle size removal of pretreatment operations is recommended, so that size characterization can be applied as a practical method for choosing a technology and for design.

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# 9 Samenvatting, conclusies en discussie

## Samenvatting

Dit proefschrift beschrijft onderzoek naar de toepassing van fysisch-chemische voorzuivering als eerste stap bij de zuivering van stedelijk afvalwater. Fysisch-chemische voorzuivering is gericht op de afscheiding van gesuspendeerde en colloidale deeltjes in de eerste stap van het zuiveringssysteem. Door het niet-opgelost materiaal af te scheiden kan een groot deel van de vervuilende stoffen relatief eenvoudig uit het afvalwater worden verwijderd. Het organisch concentraat dat ontstaat wordt beschouwd als een produkt dat kan worden gebruikt voor de productie van biogas. Omdat het primair effluent lage concentraties niet-opgelost materiaal bevat, kunnen mogelijk alternatieve zuiveringstechnieken worden toegepast voor secundaire behandeling, inclusief fysisch-chemische technieken die geschikt zijn voor de terugwinning van grondstoffen of voor de verbetering van de effluentkwaliteit.

Het onderzoek dat in dit proefschrift beschreven is had de volgende algemene doelen (i) het evalueren van het concept fysisch-chemische voorzuivering als een optie voor het vergroten van de duurzaamheid van zuiveringssystemen voor stedelijk afvalwater (in vergelijking met huidig toegepaste systemen) en (ii) bijdragen aan de verdere ontwikkeling van dit concept door experimenteel onderzoek gericht op specifieke knelpunten, uit te voeren.

Een literatuurstudie in **hoofdstuk 1** liet zien dat de verontreinigingen in stedelijk afvalwater voor een groot deel aanwezig zijn in de vorm van deeltjes. Uit de gemiddelde waarden van onbehandeld afvalwater afkomstig van verschillende Nederlandse rioolwaterzuiveringsinstallaties bleek dat ongeveer 70% CZV, 60% BZV, 25% N-Kjeldahl en 30% $P_{totaal}$ kan worden verwijderd door afscheiding van het deeltjesmateriaal met een diameter > 0,45 µm.

**Hoofdstuk 2** beschrijft een studie naar verschillende waterzuiveringsscenario's met fysisch-chemische voorzuivering. Gebaseerd op een literatuurstudie naar voor- en nazuiveringsstappen werden tien verschillende scenario's gedimensioneerd en geëvalueerd aan de hand van een aantal milieu- en kostencriteria. In deze evaluatie werd speciale aandacht besteed aan de energiebalans, de slibproductie na gisting en ontwatering, de mogelijkheid om stikstofterugwinningstechnieken toe te passen, de te verwachten effluentkwaliteit, de benodigde ruimte en de financiële haalbaarheid van de geïdentificeerde zuiveringsscenario's. De genoemde criteria zijn geselecteerd op basis van een aantal publicaties waarin de milieugerichte duurzaamheid van rioolwaterzuiveringsinstallaties bestudeerd werd (zie hoofdstuk 1).

Uit de literatuurstudie bleek dat voor fysisch-chemische voorzuivering verschillende processen beschikbaar zijn, waaronder préprecipitatie, dissolved air flotation, microzeven, directe influentfiltratie, directe membraanfiltratie en verbeterde bezinking onder toevoeging van magnetiet of microzand. Om een redelijke mate van deeltjesverwijdering te verkrijgen (> 70-80%) dienen aan de meeste voorzuiveringssystemen vlokmiddelen te worden toegevoegd. Uitzondering hierop vormen de voorzuiveringsstappen directe grovemediafiltratie en directe membraanfiltratie.

Voor de verdere behandeling van het primair effluent (ook wel *secundaire* en (evt. aanvullend) *tertiare* behandeling genoemd) kunnen verschillende zuiveringssystemen worden toegepast. Een algemeen onderscheid kan gemaakt worden naar systemen die gebaseerd zijn op biologische technieken (waaronder actief-slibsystemen, biofilmsystemen en membraanbioreactoren) en systemen gebaseerd op fysisch-chemische technieken (ionenwisseling, actief-koolbehandeling en snelle zandfiltratie). Door de toepassing van ionenwisseling in combinatie met luchtstrippen kan stikstof teruggewonnen worden.

De evaluatie van de opgestelde scenario's toonde aan dat waterzuiveringsinstallaties gebaseerd op fysisch-chemische voorzuivering kleiner en potentieel energiezuiniger ontworpen kunnen worden in vergelijking met huidig toegepaste systemen. De totale kosten (zowel de investerings- als de operationele kosten) van systemen die een fysisch-chemische voorzuiveringsstap combineren met biologische nabehandeling (actief-slib, biofilm) liggen in dezelfde orde van grootte als die van het referentiescenario. Scenario's waarin een ionenwisselaar gecombineerd met luchtstripper wordt toegepast ten behoeve van stikstofterugwinning resulteerden in minimaal 75% hogere totale kosten vergeleken met het referentiescenario (hierbij zijn de mogelijke financiële voordelen van het herbruikbaar product niet betrokken) en in een zeer hoog energiegebruik. Scenario's met actief-kooladsorptie resulteerden in 300% hogere zuiveringskosten.

Uit de scenariostudie kwamen twee belangrijke knelpunten wat betreft de toepassing van fysisch-chemische voorzuivering naar voren, namelijk (i) het gebruik van metaalgebaseerde vlokmiddelen leidt tot een sterke verhoging van de (anorganische) slibproductie en een verhoogd zoutgehalte van het effluent, (ii) de verwijdering van deeltjes leidt tot gedeeltelijke verwijdering van biodegradeerbaar CZV; dit laatste kan potentieel leiden tot verhoogde stikstofconcentraties in het effluent van een secundair biologisch behandelingssysteem vanwege verminderde denitrificatie. Het experimenteel werk dat beschreven wordt in de hoofdstukken 3 tot en met 6 van dit proefschrift gaat nader in op de beide knelpunten.

Een mogelijke oplossing voor het eerstgenoemde knelpunt wordt beschreven in de hoofdstukken 3 en 4. Deze hoofdstukken gaan in op de toepassing van organische polymeren als alternatief voor metaalgebaseerde vlokmiddelen. Bij toepassing van organische polymeren voor coagulatie/flocculatie wordt geen extra slib geproduceerd. Bovendien is de bijkomende verzouting van het effluent zeer klein.

**Hoofdstuk 3** bevat verkennend literatuuronderzoek en experimenteel werk naar het gebruik van polymeren. Uit het literatuuronderzoek bleek dat er twee typen organische polymeren zijn die veruit het meest frequent worden toegepast bij de behandeling van (industrieel) afvalwater en slib, namelijk (i) polyamines en poly-DADMACs met een laag tot gemiddeld moleculair gewicht en (ii) hoog moleculaire (HMW) polyacrylamiden. Beide polymeergroepen zijn kationisch (positief geladen). Uit bekerglasexperimenten met een selectie van deze beide polymeertypen bleek dat vooral de laatstgenoemde groep HMW polyacrylamiden interessante eigenschappen heeft voor toepassing bij de zuivering van stedelijk afvalwater, omdat ze effectief zijn bij relatief lage doseringen (5-10 $mg.l^{-1}$). Ook de dosering van een laagmoleculair polyamine resulteerde in hoge troebelheidsverwijderingen, echter bij hogere doseringen (20-30 $mg.l^{-1}$). De toevoeging van beide polymeertypen aan afvalwater uit Bennekom resulteerde in troebelheidsverwijderingen van 65-90% en een zwevendestofverwijdering van meer dan 90%. Daarnaast bleek dat de toevoeging van polymeren geen effect had op de pH van het behandelde water.

Na toevoeging van de HMW polymeren werden grote, goed bezinkende vlokken gevormd. De slibvlokken die gevormd worden na dosering van het polyamine bleken veel kleiner. Filtreerbaarheidstesten met de Filtratie-Expressiecel lieten zien dat de ontwaterbaarheid van dit slib beter was in vergelijking met slib afkomstig van een voorbezinker. De verhoging van de ontwaterbaarheid was even groot als na dosering van $FeCl_3$.

**Hoofdstuk 4** beschrijft experimenteel werk aangaande de voorbehandeling van stedelijk afvalwater met *dissolved air flotation* (DAF). Het hoofdstuk beschrijft batchflotatie-experimenten en experimenten met een DAF pilot unit. Het onderzoek gaf aan dat DAF, gecombineerd met coagulatie/flcocculatie door HMW kationische polymeren, een effectieve voorzuiveringsstap vormt voor de verwijdering van deeltjes uit stedelijk afvalwater. In de batchflotatie-experimenten werden bij polymeerdoseringen van 4-6 $mg.l^{-1}$ troebelheidsverwijderingen van 75% gevonden. Uit een vergelijking met de bekerglasexperimenten van hoofdstuk 3, waarin bezinking werd toegepast, bleek dat er geen verschil in verwijderingsrendementen gevonden werd tussen bezinking en flotatie.

In de DAF pilot unit werden lagere troebelheidsverwijderingen gevonden in vergelijking met de batchflotatie-experimenten (maximaal 70%). Dit werd veroorzaakt door suboptimale menging van het polymeer met het afvalwater. Aanvullende bekerglasexperimenten toonden aan dat de troebelheidsverwijdering verhoogd kon worden (cq. dat polymeer bespaard kan worden) door het optimalizeren van de mengcondities. De drogestofconcentratie van het flotatieslib bedroeg 3-5% bij continu schrapen en 8-10% bij discontinuë verwijdering van de flotatielaag.

De experimenten met de DAF pilot-unit toonden ook aan dat fluctuaties in de deeltjesconcentratie van het influent een grote invloed hebben op het verwijderingsrendement en dat dit rendement bij vaste dosering in feite onvoorspelbaar is. Uit een data-analyse bleek dat om te bepalen welke dosering organisch polymeer dient te worden toegepast in een continu-systeem, het deeltjes-CZV van het influent bekend dient te zijn. Uit deze data-analyse kwam ook naar voren dat continuë troebelheidsmetingen een mogelijkheid bieden om dit $CZV_{deeltjes}$ te bepalen, omdat er een lineaire relatie werd gevonden tussen beide parameters.

Het tweede knelpunt dat gevonden werd in hoofdstuk 2 (de verwijdering van biodegradeerbaar CZV in de voorzuivering) is onderzocht in hoofdstuk 5 en 6. **Hoofdstuk 5** beschrijft experimenten betreffende de toepassing van troebelheids-gerelateerde polymeerdosering met het doel verschillende niveaus van deeltjesverwijdering te creëren. Bij nieuwe metingen aan influent en primair effluent werd eenzelfde lineaire relatie tussen $CZV_{deeltjes}$ en troebelheid gevonden als in hoofdstuk 4. Uit de toepassing van continuë troebelheidsmetingen bleek dat de deeltjesconcentratie in het onderzochte afvalwater aanzienlijke dagelijkse variaties vertoonde. Experimenten met troebelheidsgerelateerde dosering waarbij een kationisch HMW polyacrylamide werd gebruikt, lieten zien dat deze manier van dosering kan worden gebruikt voor het creëren van verschillende niveaus van deeltjesverwijdering. Voorts bleek dat troebelheidsgerelateerde dosering resulteerde in hogere verwijderingsrendementen en in een stabielere procesvoering vergeleken met de dosering van een vaste concentratie polymeer per liter. Beide bevindingen kunnen verklaard worden uit het feit dat door troebelheidsgerelateerde dosering overdosering bij lage influenttroebelheid wordt vermeden.

Het doel van **hoofdstuk 6** was vast te stellen hoe de verwijdering van stikstof en fosfaat in een actief-slibsysteem beïnvloed wordt door verschillende niveaus van deeltjesverwijdering. Het hoofdstuk beschrijft (i) zgn. *Nitrogen Uptake Rate* (NUR) experimenten die werden uitgevoerd om het effect van deeltjesverwijdering op de zgn. *denitrificatiepotentiaal* (DP) te bepalen en (ii) experimenten met een actief-slibsysteem (1000 l) waarin voorgezuiverd afvalwater werd behandeld.

Uit de NUR-experimenten bleek dat de fractie $CZV_{deeltjes}$ van het onderzochte afvalwater (ongeveer 65% van het totale CZV) ongeveer 50% van de DP bevat. Dit resultaat geeft aan dat met de afscheiding van het niet-opgelost materiaal ongeveer 50% van het biodegradeerbaar CZV wordt verwijderd.

De som van de nitriet- en nitraatconcentratie in het effluent van het actief-slibsysteem was in alle gevallen hoger dan de stikstoflimiet. De N-Kjeldahl van het onderzochte afvalwater was echter hoog vergeleken met gemiddelde Nederlandse waarden (respectievelijk 70 $mg.l^{-1}$ versus 50 $mg.l^{-1}$). Berekeningen gebaseerd op de waarden die verkregen werden in de experimenten geven aan dat, voor goed gedimensioneerde actief-slibsystemen die afvalwater met een gemiddelde stikstofconcentratie behandelen, een hoog niveau van deeltjesverwijdering mogelijk is (80-90%) zonder dat de effluentstikstofeis wordt overschreden. Het experiment toonde ook aan dat voorbehandeling met organische polymeren geen direct effect heeft op de biologische fosfaatverwijdering van een actief-slibsysteem. De verwijdering van fosfaat in de anoxische zone kan zelfs bijdragen aan de totale denitrificatiecapaciteit. In het pilot experiment werd een bijdrage aan de dentrificatie van ongeveer 4 $mg.l^{-1}$ gevonden. Biologische fosfaatverwijdering kan indirect beïnvloed worden door recirculatie van nitraat naar de anaërobe zone door de terugvoer van retourslib. Deze nitraatrecirculatie kan echter voorkomen worden door procesaanpassingen.

**Hoofdstuk 7** beschrijft een bijgestelde scenariostudie gebaseerd op de conclusies van het experimenteel werk. Deze bijgewerkte scenario-evaluatie liet zien dat het gebruik van organische polymeren voor coagulatie/flocculatie in combinatie met bezinking of flotatie kan leiden tot een 60-75% lager energiegebruik en 10-25% minder ruimtebehoefte. De berekende slibproductie na gisting en ontwatering is circa 10% hoger. De totale kosten van scenario's waarin organische polymeren worden toegepast in de voorzuivering gecombineerd met (in dit geval) een actief-slibsysteem voor nabehandeling zijn gelijk of zelfs iets lager dan de kosten van een scenario met $FeCl_3$ of dan het referentiescenario. Hoewel de kosten voor chemicaliën van een systeem met polymeren hoger zijn, wordt dit gecompenseerd door de lagere kosten voor slibafzet en lagere civiele en electromechanische investeringskosten.

## Conclusies en discussie

De doelen van het onderzoek in dit proefschrift waren (a) het evalueren van het concept fysisch-chemische voorzuivering als een optie voor het verbeteren van de milieuduurzaamheid van rioolwaterzuiveringsinstallaties en (b) het bijdragen aan de verdere ontwikkeling van dit concept door experimenteel onderzoek te doen aan gevonden knelpunten. Hoofdstukken 2 en 7 toonden aan dat fysisch-chemische voorzuivering in combinatie met secundaire biologische behandeling (actief-slib of biofilm) aanzienlijk kan bijdragen aan een verhoogde duurzaamheid door de vermindering van het energieverbruik. De toepassing van dit type scenario's resulteert in een energiebesparing van 60% tot 75% vergeleken met momenteel toegepaste zuiveringssystemen, vanwege de productie van (meer) biogas en verminderde beluchting in de nabehandeling. Voorts maakt de toepassing van vergaande voorzuivering het ontwerp van kleinere installaties mogelijk en zijn netto ruimtebesparingen van 20% (in combinatie met een actief-slibsysteem) tot 80% (in combinatie met een biofilmsysteem) mogelijk. Uit de evaluatie bleek ook dat dergelijke scenario's economisch haalbaar zijn.

Wat betreft de andere criteria die werden geformuleerd om de milieuduurzaamheid van scenario's te evalueren (zie tabel 1.6) bleek dat (i) de finale slibafzet van deze systemen gelijk of iets hoger (circa 10%) ligt in vergelijking met referentiesystemen, (ii) de effluentkwaliteit voornamelijk afhankelijk is van de toegepaste secundaire, cq. tertiaire behandelingsstappen; en (iii) de terugwinning van stikstof technisch mogelijk is door toepassing van een ionenwisselaar in combinatie een luchtstripper; deze laatste optie resulteerde echter in 75% hoger zuiveringskosten en in een erg hoog energieverbruik.

Het onderzoek wat betreft de knelpunten liet zien dat organische polymeren kunnen worden toegepast in plaats van metaalgebaseerde vlokmiddelen zonder dat dit leidt tot een kostenverhoging. De toepassing van polymeren vormt daarmee een oplossing voor het eerste knelpunt zoals beschreven in hoofdstuk 2. Voor praktische toepassing zal meer aandacht besteed moeten worden aan de toepassing van organische polymeren die niet acrylamidegebaseerd zijn, zoals bijvoorbeeld de biogesynthetiseerde polymeren die genoemd worden in hoofdstuk 3.

Wat betreft het tweede knelpunt (de verwijdering van biodegradeerbaar CZV in de voorzuivering) bleek uit hoofdstuk 6 dat deeltjesverwijderingen van 80 tot 90% mogelijk lijken zonder dat de effluentstikstofeis overschreden wordt. Voorts werd uit de hoofdstukken 5 en 6 duidelijk dat de verwijdering van deeltjes-CZV gestuurd kan worden door troebelheidsgerelateerde polymeerdosering. Hiermee kan de verwijdering van biodegradeerbaar CZV afgestemd worden op de gewenste denitrificatiebehoefte.

De experimenten met de actief-slibinstallatie in hoofdstuk 6 gaven ook aan dat biologische fosfaatverwijdering niet direct beïnvloed wordt door voorzuivering met organische polymeren. De verwijdering van fosfaat in de anoxische zone kan zelfs bijdragen aan de totale denitrificatiecapaciteit. De actief-slibmodellen waarin ook de biologische fosfaatverwijdering onder anoxische omstandigheden wordt gemodelleerd (zie bijvoorbeeld Kuba *et al.*, 1996; Hao *et al.*, 2000) geven hetzelfde aan. Om het verlies van fosfaataccumulerende activiteit in het algemeen en anoxische fosfaatverwijdering in het bijzonder te voorkomen, dient de terugvoer van nitraat met retourslib geminimaliseerd te worden. De actief-slibschema's zoals toegepast in het UCT-ontwerp (*University of Cape Town*) en de modificatie van het UCT-proces, het BCFS®-ontwerp, bieden procesconfiguraties waarin dit kan worden bereikt. Deze configuraties lijken daarom interessant in combinatie met toepassing van organische polymeren. Wanneer de resultaten van de experimenten en de bovengenoemde overwegingen in ogenschouw worden genomen leidt dit tot de conclusie dat ook het tweede knelpunt ondervangen kan worden.

***Slibproductie, effluentkwaliteit en nutriëntenterugwinning*** – Een punt van aandacht vormt de slibproductie na gisting en ontwatering. De scenariostudie geeft aan dat vergaande voorzuivering (zelfs met toepassing van organische polymeren) in combinatie met een actief-slibsysteem leidt tot een iets hogere slibproductie (circa 10%) in vergelijking met het referentiescenario. Deze bevinding is echter het resultaat van de aannames die gemaakt zijn met betrekking tot de finale slibproductie (te weten de biologische yield van 0,4 $\text{kgSS.kgCZV}_{\text{verwijderd}}^{-1}$ die werd aangenomen voor het actief-slibsysteem en de vergistbaarheid van primair (50%) en secundair (30%) slib). Deze waarden hebben een sterk bepalend karakter voor de uitkomsten van de scenario-evaluatie en dienen kritisch bekeken worden. De resultaten van de scenario-evaluatie in hoofdstuk 2 geven aan dat de toepassing van biofilmsystemen kan leiden tot een wat lagere slibproductie (vanwege de aangenomen slibproductiefactor van 0,2 $\text{kgSS.kgCZV}_{\text{verwijderd}}^{-1}$). Echter, omdat er weinig gegevens zijn wat betreft de biologische yield van biofilmsystemen vormt ook dit een aandachtspunt waarmee rekening gehouden dient te worden.

Zoals als eerder werd genoemd bleek de effluentkwaliteit van de opgestelde scenario's voornamelijk afhankelijk van de toepaste secundaire en tertiaire behandelingsstappen. Vergaande deeltjesverwijdering kan gunstig zijn omdat het de toepassing van nabehandelingsstappen zoals actief-kooladsorptie, ionenwisseling en snelle zandfiltratie mogelijk maakt. De toepassing van deze stappen kan leiden tot een verbeterde effluentkwaliteit. De scenario-evaluatie toonde echter ook aan dat de toepassing van actief-kooladsorptie en ionenwisseling resulteert in een aanzienlijke verhoging van het energieverbruik en van de zuiveringskosten. Daarnaast leidt de toepassing van ionenwisseling tot een verhoging van het zoutgehalte van het effluent.

Wanneer biofilmsystemen worden toegepast na voorzuivering met polymeren zal de verwijdering van fosfaat een probleem vormen. Normaal gesproken wordt dit type systemen toegepast in combinatie met fosfaatprecipitatie met metaalzouten. Er zijn enkele recente publicaties waarin onderzoek naar biologische fosfaatverwijdering in biofilmsystemen wordt beschreven (Falkentoft *et al.*, 2001; Helness en Ødegaard, 2001). Hoewel er nog geen eensluidende conclusie getrokken kan worden, lijkt dit mogelijk toepasbaar.

In dit onderzoek is, wat betreft hergebruik van nutriënten uit stedelijk afvalwater, alleen gekeken naar de optie van stikstofhergebruik. Recentelijk is er echter een groeiende belangstelling vanuit de fosfaatproducerende industrie voor de terugwinning van fosfaat uit stedelijk afvalwater (CEEP, 2000, 2001). Een aantal gerapporteerde terugwinningsopties richt zich op de terugwinning van fosfaat in actief-slibinstallaties met biologische P-verwijdering. De terugwinning van P door precipitatie uit water dat ontrokken wordt van de anaërobe zone vormt hierbij een mogelijke techniek (zie hiervoor bijvoorbeeld Brandse, 2000 en Klapwijk, 2001). Een tweede optie vormt de terugwinning van P uit secundair spuislib (zie bijvoorbeeld Hultman, 2001). Beide methoden lijken goed toepasbaar in combinatie met voorzuivering met polymeren.

**Vergroting van de flexibiliteit van zuiveringsinstallaties -** Een voordeel van voorzuivering dat tot dusver nog niet genoemd is, is dat het de flexibiliteit van rioolwaterzuiveringsinstallaties kan vergroten. Vooral in configuraties die bestaan uit een primaire bezinker gevolgd door een actief-slibinstallatie vormt het toevoegen van organische polymeren een mogelijk middel om de capaciteit te vergroten bij structurele verhoging van de belasting. De dosering van polymeren verhoogt de deeltjesverwijdering in de voorbezinker, wat leidt tot een lagere belasting van het actief-slibsysteem. De verlaagde deeltjesbelasting van het systeem leidt tot een verhoging van de slibactiviteit, wat op zijn beurt leidt tot de mogelijkheid om een hogere slibbelasting toe te passen. Hierdoor kan het gebruik van polymeren een ingrijpende (en kostbare) verbouwing van de zuiveringsinstallatie voorkomen. Een dergelijke methode kan ook toegepast worden in gebieden met een seizoensafhankelijke verhoging van de vervuilingslast, zoals voor kan komen in toeristische gebieden.

Bij waterzuiveringsinstallaties zonder voorbezinker vormt de toepassing van *dissolved air flotation* een interessante optie. Deze voorzuiveringsstap is relatief compact, blijkt vergelijkbare investeringskosten met zich mee te brengen als een voorbezinker en is relatief gemakkelijk te installeren (zelfs beschikbaar op 'turn-key' basis). Daarnaast wordt een relatief geconcentreerde slibstroom geproduceerd waardoor de capaciteit van bestaande slibbehandelingsapparatuur niet uitgebreid hoeft te worden.

**Verdere ontwikkeling van fysisch-chemische voorzuivering** – Op basis van de resultaten van dit onderzoek en het parallel uitgevoerde onderzoek ten aanzien van deeltjeskarakteristieken (Van Nieuwenhuijzen, 2001; STOWA, 2001) kan een aantal aanbevelingen gedaan worden voor de verdere ontwikkeling van het concept fysisch-chemische voorzuivering als optie voor een verdere verduurzaming van rioolwaterzuiveringsinstallaties.

- Zoals eerder genoemd lijkt fysisch-chemische voorzuivering met polymeren zeer goed toepasbaar in combinatie met actief-slibsystemen volgens het UCT- of het BCFS®-ontwerp. Aanbevolen wordt om pilot experimenten te doen met een dergelijk zuiveringsscenario om de resultaten van dit proefschrift te bevestigen. De troebelheidsgerelateerde polymeerdosering kan worden gebruikt om het niveau van deeltjesverwijdering te optimaliseren met het oog op de effluentstikstofeis. De experimenten dienen bij voorkeur uitgevoerd te worden met afvalwater dat een 'gemiddelde' N-Kjeldahlconcentratie bevat.
- De toepassing van biofilmsystemen leidt tot een compact zuiveringssysteem en kan leiden tot een lagere slibproductie. Wanneer dit type systemen echter wordt toegepast in combinatie met voorzuivering met polymeren zal het effluent waarschijnlijk niet aan de effluentfosfaateis kunnen voldoen. Het is de vraag of biologische fosfaatverwijdering toegepast kan worden in biofilmsystemen. Enkele positieve resultaten zijn wat dit betreft al gerapporteerd. Fundamenteel onderzoek kan deze vraag verder beantwoorden en kan mogelijk ook gegevens opleveren over de biologische slibproductie.
- Het parallele onderzoek naar de grootteverdeling van verontreinigingen in stedelijk afvalwater bleek waardevol met betrekking tot de keuze en het ontwerp van voorzuiveringsstappen. Meer onderzoek naar een methodiek om afvalwater te karakteriseren ten aanzien van deeltjes wordt aanbevolen, zodat deze techniek praktisch toegepast kan worden bij de keuze voor en het ontwerp van een specifieke voorzuiveringsstap.

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# Curriculum Vitae

The author of this thesis, Adriaan Reinier Mels, was born on 17 June 1971 in Schoonhoven. After finishing secondary school (VWO) he started his study Environmental Hygiene at the Wageningen University in 1989 . Two M.Sc. theses were done at the Sub-department of Environmental Technology, partly in cooperation with the Sub-department of Sociology. During a training period he performed research at the Technion in Haifa, Israel. After finishing his study in 1995 he worked during six months as a private consultant in wastewater technology. In 1996 he came to work with the Sub-department of Environmental Technology on the research project 'physical-chemical pretreatment of municipal wastewater'. The results of this research are partially published in this thesis. Since 1 April 2001 he is working with the Lettinga Associates Foundation (LeAF) for environmental protection and resource conservation.

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