KOMPETENZZENTRUM
WasserBerlin

REPORT

Cicerostr. 24
D-10709 Berlin
Germany
Tel +49 (0)30 536 53 800
Fax +49 (0)30 536 53 888
www.kompetenz-wasser.de

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Feasibility of the microsieve technology for advanced phosphorus removal

Project acronym: OXERAM 2

by
Margarethe Langer
Andrea Schermann

Kompetenzzentrum Wasser Berlin gGmbH
Cicerostr. 24, 10709 Berlin, Germany
for
Kompetenzzentrum Wasser Berlin gGmbH

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Feasibility of the microsieve technology for advanced phosphorus removal

Authors
Margarethe Langer, Kompetenzzentrum Wasser Berlin gGmbH

Andrea Schermann, Kompetenzzentrum Wasser Berlin gGmbH

Quality Assurance
Ulf Miehe, Kompetenzzentrum Wasser Berlin gGmbH

Cécile Bourdon, Direction Technique Veolia Eau

Rune Strube, Hydrotech AB

Publication / Dissemination approved by technical committee members:
C. Bourdon, Veolia

A. Tazi-Pain, Veolia

C. Bartholomäus, Berliner Wasserbetriebe

R. Gnirß, Berliner Wasserbetriebe

A. Peter-Fröhlich, Berliner Wasserbetriebe

M. Jekel, FG Wasserreinhaltung, TU Berlin

A. Hartmann, Kompetenzzentrum Wasser Berlin gGmbH

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Abstract (English)

The pilot trials at the Ruhleben wastewater treatment plant proved that the microsieve technology combined with chemical pre-treatment achieves good and reliable phosphorus removal with effluent values < 80 µg/L TP. The first three months of pilot operation confirmed the general process performance observed during the pre-trials in 2009 but also revealed a need for process optimization with regard to the removal of suspended solids and the reduction of coagulant breakthrough. An improved performance was achieved through change from ferric chloride ($FeCl_3$) to polyaluminum chloride (PACl). In the presented case, PACl gave clearly better results for the removal of phosphorus and suspended solids than $FeCl_3$. Additionally, the occurrence of coagulant residues could be noticeably reduced. In contrast to $FeCl_3$, dosing PACl led to an improvement of the water transmittance simplifying disinfection with UV irradiation. Load proportional dosing of PACl and polymer was introduced in order to avoid under as well as over dosing of the chemicals. The dose of cationic polymer had a significant impact on water quality and backwash time: With the initial process configuration 1.5 to 2 mg/L cationic polymer were recommended for a safe and stable operation with adequate backwash time resulting in an average polymer dose of 1.7 mg/L. However, latest results showed that a polymer dose of only 0.6 mg/L is possible without losses in water quality and filtration performance when mixing conditions were optimized. During the constructional modifications the hydraulic retention time of the coagulation was reduced from 4 to 1 min at peak flow. Due to the installation of a Turbomix^{TM} short-circuiting could be avoided. Furthermore, the turbulence in the flocculation tank was increased. Despite the noticeable reduction of the hydraulic retention time and the polymer dose the rebuild resulted in improved reduction of suspended solids (2.2 mg/L) and coagulant residues in the microsieve effluent. The operation regime of the chemical treatment prior to the microsieve filtration showed to be a trade-off between the energy demand for mixing and the polymer consumption. Due to the continuous operation over more than 20 months important operational experience was gained with regard to backwash behavior and cleaning intervals. The backwash time mainly correlates with the influent flow (10-30 m^3/h), the influent water characteristics and the properties of the formed flocs. Due to progressing fouling of the filter panels chemical cleaning was necessary every 4 to 7 weeks. A shorter cleaning interval (e.g. every 4 weeks) might be beneficial as the backwash time and thus the energy demand could be kept on a lower level. In this application the microsieve produced on average 1.8 % of backwash water. The backwash water showed excellent settling properties (SVI << 50 mL/g) and might be easily treated via returning to the primary clarifiers. The UV disinfection plant behind the microsieve was operated with a fluence of 730 J/m^2. Good disinfection could be provided for a continuous operation of 7 months. During this period there were always less than 100 MPN/100 mL of *E. coli* and *Enterococci* in the effluent of the UV disinfection.

Overall, the microsieve in combination with dosing of coagulant and polymer is a robust technology with low phosphorus effluent values (< 80 µg/L) and a low energy demand of about 21 Wh/m^3 (+ site-specific energy demand for water lifting). Microsieving, together with UV disinfection, can be an option for applications targeting phosphorus removal and disinfection, e.g. effluent polishing for sensitive areas or landscape irrigation.

Abstract (German)

Die Pilotversuche auf dem Klärwerk Ruhleben haben gezeigt, dass die Mikrosiebung mit vorgeschalteter zweistufiger Flockung eine verlässliche Phosphorentfernung mit Ablaufwerten < 80 µg/L ermöglicht. In den ersten drei Monaten Versuchsbetrieb konnte die in den Vorversuchen in 2009 beobachtete allgemeine Leistungsfähigkeit des Verfahrens bestätigt werden. Allerdings wurde in Bezug auf die Entfernung suspendierter Stoffe (SS) und den Durchbruch von Flockungsmittel (FM) ein Optimierungsbedarf festgestellt. Durch den Wechsel von Eisen(III)chlorid ($FeCl_3$) zu Polyaluminiumchlorid (PACl) konnte die Reinigungsleistung des Prozesses deutlich verbessert werden. Mit PACl wurde eine verbesserte Feststoff- und damit auch Phosphorentfernung erzielt. Darüber hinaus wurden ein deutlich verminderter Durchbruch des Flockungsmittels und eine Erhöhung der Transmission erreicht. Dies vereinfacht den Betrieb einer nachgeschalteten UV-Desinfektion. Durch die Einführung der frachtproportionalen Dosierung des FM und Flockungshilfsmittels (FHM) konnte unter Beibehaltung der Ablaufqualität sowohl Über- als auch Unterdosierung unterbunden und der Bedarf an Flockungschemikalien reduziert werden. Mit der ursprünglichen Prozesskonfiguration wurde ein sicherer und stabiler Betrieb bei einer FHM-Dosierung von 1,5-2,0 mg/L erreicht (im Mittel 1,7 mg/L). Die neusten Ergebnisse haben allerdings gezeigt, dass bei optimierten hydraulischen Bedingungen eine mittlere Dosierung von 0,6 mg/L FHM ausreichend sein kann. Während des Anlagenumbaus zur zweiten Versuchsphase wurde die minimale Aufenthaltszeit im Koagulationstank von 4 auf 1 min reduziert. Durch die Installation eines Turbomix^{TM} konnten Kurzschlussströmungen vermieden werden. Außerdem wurde die Turbulenz im Flockulationstank erhöht. Trotz der deutlichen Reduktion der Aufenthaltszeit zur Koagulation und der FHM-Dosierung führte der Umbau der Pilotanlage zu einer Verbesserung der Feststoffentfernung und einer Reduktion des FM-Durchbruchs. Im Mittel waren nur noch 2,2 mg/L SS anstelle von 3,4 mg/L im Ablauf des Mikrosiebes. Die Pilotversuche haben gezeigt, dass die optimale Betriebsweise der Flockung als Vorbehandlung zur Mikrosiebung ein Kompromiss zwischen dem FHM-Verbrauchs und dem Energiebedarf für die Einmischung ist. Während des 20-monatigen Dauerbetriebs hat sich gezeigt, dass die Rückspülungszeit vor allem durch den Zulaufvolumenstrom (10-30 m^3/h), die Wasserqualität und die Eigenschaften der erzeugten Flocken beeinflusst wird. Aufgrund von Fouling der Filterpanele und damit zunehmender Rückspülungszeiten war eine chemische Reinigung alle 4 bis 7 Wochen notwendig. Kürzere Reinigungsintervalle (z.B. 4 Wochen) können möglicherweise zu reduzierten Rückspülungszeiten und damit zu einem reduzierten Energiebedarf der Mikrosiebung führen. Durchschnittlich wurden 1,8 % Spülwasser produziert. Das Spülwasser zeigte sehr gutes Absetzverhalten (SVI << 50 mL/g) und könnte daher zur weiteren Behandlung leicht in die Vorklärung zurückgeführt werden. Die nachgeschaltete UV-Desinfektion wurde mit einer Fluenz von 730 J/m^2 betrieben und erzielte während des 7-monatigen Dauerbetriebs eine sichere Desinfektion mit Ablaufwerten < 100 MPN/100 mL für *E. coli* und *Enterokokken*.

Zusammengefasst ist die Mikrosiebung mit vorgeschalteter Flockung und ein robustes Verfahren mit ein geringen Phosphorablaufwerten (< 80 µg/L) und einem geringen Energiebedarf von 21 Wh/m^3 (+ lokalbedingte Wasserhebung). Kombiniert mit einer UV-Desinfektion eignet sie sich zur weitergehenden Abwasserreinigung in sensiblen Gebieten oder als Vorbehandlung zur Wiederverwendung (z.B. Bewässerung).

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Chapter 1

Introduction

Phosphorus is often a limiting nutrient for the biological productivity in a water body. High concentrations can lead to eutrophication. The high nutrient loads in the rivers Havel and Spree lead to excessive algae growth. Hence the ecological status of the Berlin surface water is assessed as “moderate” to “poor” according to the European water framework directive. As the directive demands a “good status” in all surface waters until 2027 there is a need for reduction of point and non-point nutrient emissions. A “good” ecological status e.g. requires total phosphorus (TP) concentrations between 60 and 80 µg/L. (SENGUV 2009, SENSTADT 2001)

Waste water treatment plants with biological phosphorus removal achieve phosphorus effluent values of about 0.5 mg/L (Bratby 2006). Thus, the situation in Berlin is very particular, because the rivers Spree and Havel have very low water flows. 15 to 50 % of the flow in the rivers originates from the Berlin wastewater treatment plants, which have strong influence on the water quality in Berlin (Reddersen 2004, Ziegler 2001). To meet the demands of the European water framework directive tertiary wastewater treatment for advanced phosphorus removal is necessary.

The process combining coagulation, flocculation and microsieve filtration presented in this study is part of the OXERAM project. The project deals with the comparison of different technologies according to their efficiency and applicability as a tertiary wastewater treatment step for advanced phosphorus removal and disinfection. The competing technologies are:

- Membrane filtration vs. pre-ozonation + membrane filtration
- Dual media filtration + UV vs. microsieve filtration + UV.

During the project a microsieve, a polymeric membrane, a ceramic membrane and an ozonation unit are installed at the wastewatertreatment plant (WWTP) Ruhleben. Detailed investigations concerning dual media filtration were already conducted in 2006/2007 by Miehe (2010).

The microsieve filtration designed as disc filtration is said to be a cost and energy efficient technology with high filtration velocities and a small footprint. Because of the central location and the expansion of the Ruhleben WWTP, a tertiary treatment step for advanced phosphorus removal with a compact design is desirable. Initially microsieve filtration combined with coagulation and flocculation was expected to achieve total phosphorus concentrations (TP) below 120 µg/L.

Preliminary trials in May 2009 at the Ruhleben WWTP revealed that even effluent values below 80 µg/L are possible. Applied were ferric chloride (2.5 or 5.0 mg Fe/L) and cationic polymers (1.0 or 2.0 mg/L). The tested pilot plant (see Figure 1) was more simply designed than the finally in OXERAM operated high-end pilot unit.

Figure 1: Pilot plant for preliminary tests (May 2009)

The preliminary tests gave very good results concerning the phosphorus removal (see Figure 2, for an explanation of the box plot diagram see Appendix A). 24 out of 28 samples contained less than 80 µg /L TP. The average reductions were 64 % for total phosphorus and 14 % for the chemical oxygen demand (COD). The apparent suspended solids (SS) removal averaged only 11 % (0.4 mg/L). The iron concentrations amounted to 0.1 - 2.0 mg/L with a mean value of 0.9 mg/L.

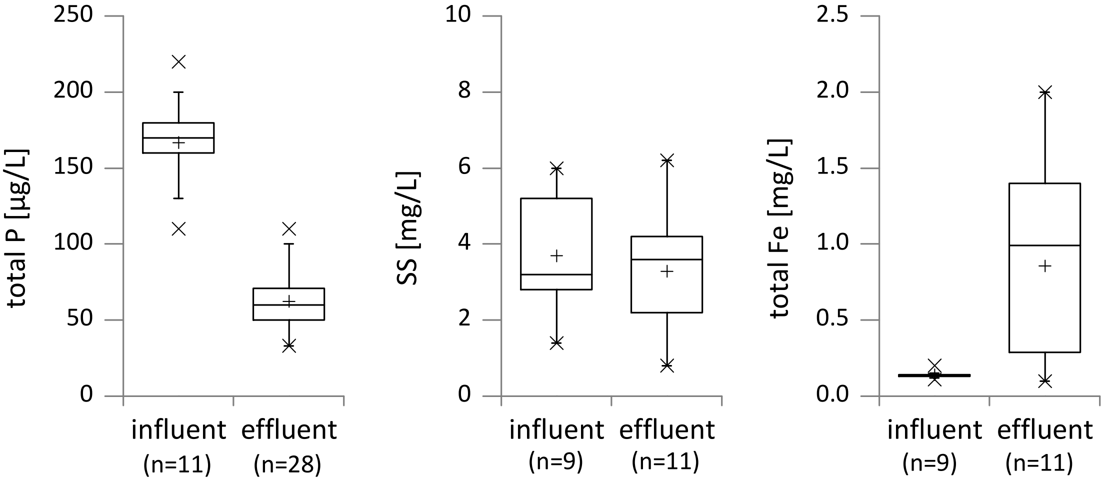


Figure 2: Preliminary tests (13.05.2009 to 20.05.2009)

The preliminary tests showed that TP concentrations below 80 µg/L can be reliably achieved with the microsieve technology. However, the total iron and the SS concentrations in the effluent were high due to coagulant breakthrough. The flocculation needs to be optimized to reduce the iron concentration in the effluent especially for ensuring the operation of a downstream UV disinfection.

The objectives of OXERAM 2 work package 5 are:

- Testing of microsieve technology (mesh size 10 µm) for low TP effluent values (< 50-120 µg/L)
- Evaluation of continuous operation
- Optimization of coagulant and polymer demand
- Reduction of amount and amplitude of backwash water
- Evaluation of effluent quality (e.g. TP, suspended solids, COD) and operational issues
- Jar tests double stage filtration

The overall objective of OXERAM is the development of sustainable technologies for tertiary wastewater treatment, targeting advanced phosphorus removal and disinfection.

The objectives of OXERAM 2b are:

- Evaluation of the microsieve technology for advanced phosphorus removal (TP <50-120µg/L)
- Extend long-term trials over the next winter period including storm weather conditions to assess the process reliability and the effluent quality (6 months)
- Test new design proposed by DT with reduced hydraulic retention time (HRT) on process reliability (to reduce invest cost)
- Estimation of cost, energy and footprint savings compared to membrane filtration/dual media filtration (including new design)
- Assess design specifications for UV disinfection

Chapter 2

State of the art

2.1 Phosphorus in surface waters and eutrophication

Sources of phosphorus (P) in municipal wastewaters are mainly from domestic and industrial wastewater flows. In domestic wastewaters human waste makes up about 30 to 50 % of the P content. The remaining 50 to 70 % of P derive from detergents and cleaning agents. Industrial P sources are e.g. fertilizer manufacturers and commercial laundries (Jiang and Graham 1998). Depending on the composition of the wastewater the P concentration can vary between 5 mg/L and 20 mg/L (ATV-DVWK-A 2004).

The presence of excess P and nitrogen in effluent waters discharged into natural water bodies is known to be the cause of eutrophication. Eutrophication describes a natural process in aquatic systems where an increase of the nutrient content leads to an excessive growth of aquatic plants, e.g. algae and phytoplankton. The process of eutrophication is accelerated by anthropogenic influences. The excessive algae bloom can be followed by oxygen depletion in deeper levels, advancing sludge formation and reduced light transparency of the water. This has a negative impact on the ecosystem of the water body, e.g. fish mortality. Moreover, massive algae growth and the formation of toxic algae products have hygienic and visual impacts on the bathing water quality. In algal protoplasm the molar ratio of nitrogen, phosphorus and carbon is about 15:1:105. Thus, phosphorus can be considered as the limiting factor and point of actions for lowering eutrophication. (Jiang and Graham 1998; SENSTADT 2001;Li and Brett 2012).

In the period 1995 to 1997 188 t/a TP entered the Berlin surface water system through the rivers Spree, Dahme, Havel, and other small streams. 60 % of this amount derived from non-point sources (groundwater, erosion, surface runoff). The Berlin wastewatertreatment plants emitted 109 t/a of total phosphorus. 38 t/a of total phosphorus was discharged through sewer overflows. Just before flowing into river Havel the total phosphorus concentration in river Spree amounted to 0.197 mg/L. 2001 there were still 0.195 mg/L total phosphorus in the inner-city Spree. (Behrendt and Opitz 1998; SENSTADT 2001; Rehfeld-Klein 2011)

Table 1: Quality classification based on chlorophyll-a contents and expected overall phosphorus concentrations and view depths (classes not related to classes from water framework directive) (LAWA 1996)

Class	Chlorophyll-a mean average value [µg/L]	Overall phosphorus mean average value [mg/L]	View depth mean average value [m]
I	1 - 4	-	4.6 - 6.0
I - II	3 - 8	0.006 - 0.018	3.5 - 5.0
II (target)	**7 - 30**	**0.016 - 0.082**	**1.5 - 3.7**
II - III	25 - 50	0.067 - 0.150	1.0 - 1.7
III	50 - 100	0.150 - 0.320	0.5 - 1.0
III - IV	> 100	> 0.320	< 0.5
IV	Not defined	-	-

According to the LAWA assessment for surface waters the Spree in Berlin meets the quality standards of class III (see Table 1). To meet the goals of the European water framework directive, a “good” ecological status in all surface waters, the quality standards of class II have to be achieved. For class II a mean TP value between 16 and 82 µg/L is estimated. (LAWA 1996)

According to the Berlin senate the precise goals for the Lower Havel, where the rivers Havel and Spree are united, are (Rehfeld-Klein 2011):

- < 32 µg/L chlorophyll-a
- 1.5 m view depth
- < 60 µg/L TP.

The geogenic background concentrations for Spree and Havel are in the range of 60 to 90 µg/L total phosphorus (SENSTADT 2004). Thus, the discharge of phosphorus into the receiving waters has to be strongly restricted in the future in order to achieve the demanded water quality standards.

Due to the different sources of P in municipal wastewater there are different species of P. There are soluble and particulate fractions, which furthermore can be differentiated into organic and inorganic P. Particulate P can be part of solid matter or adsorbed on the surfaces of particles. The largest part of P is present as inorganic dissolved P, e.g. especially orthophosphate. Only a small fraction occurs as organic P either dissolved or particulate (ATV-DVWK-A 2004; Barjenbruch and Exner 2009).

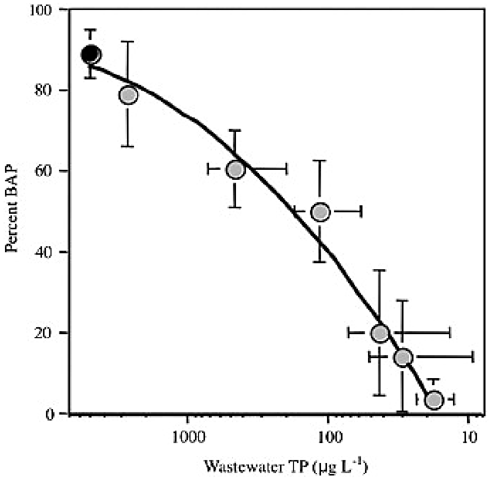


Figure 3: Relationship between the %BAP of TP and TP concentrations in the effluents. Error bars represent standard deviations (Li and Brett 2012).

The refractory dissolved organic P is defined as the sum of polyphosphate and dissolved organic P. The refractory dissolved organic P concentration in WWTP is about 20 to 40 µg/L and will only slightly be eliminated by precipitation. However, this refractory dissolved organic P is assumed to be non-bioavailable (Thomasius 2011; Li and Brett 2012). The bioavailable phosphorus (BAP) is the fraction of TP which assists algae growth and represents the best indication for measurements of water quality regarding P. The BAP percentage of secondary WWTP effluents is about 80 to 90 %, whereas urban and agricultural runoffs have only about 20 to 40 % BAP (Li and Brett 2012). Li and Brett (2012) have tested how different P removal levels in various alum based processes for P removal affect the BAP. They have shown that with increasing P removal the percentage of BAP in the effluent decreases. At increasing P removal levels the BAP content in the effluent is not equivalent to the TP concentration in the effluent (see Figure 3).

2.2 Ruhleben wastewatertreatment plant (WWTP)

The Ruhleben WWTP was commissioned in 1963. After three expansion steps the plant has now a capacity in dry weather of 247.500 m^3/d and a maximum capacity in stormy weather of 600.000 m^3/d. This equates to a total number of inhabitants and population equivalents of 1.6 million. The Ruhleben WWTP is centrally located in Berlin and treating wastewater from households, commerce and industry. The treatment technique combines mechanical and biological processes (see Figure 4). The WWTP is equipped with racks, grit chambers and primary sedimentation. The activated sludge treatment includes nitrification-denitrification and biological phosphorus removal. In order to stabilize the phosphorus a simultaneous precipitation with a low dose of $FeCl_2$ is conducted. The wastewatertreatment in Ruhleben closes with secondary sedimentation. (BWB 2007, BWB 2009)

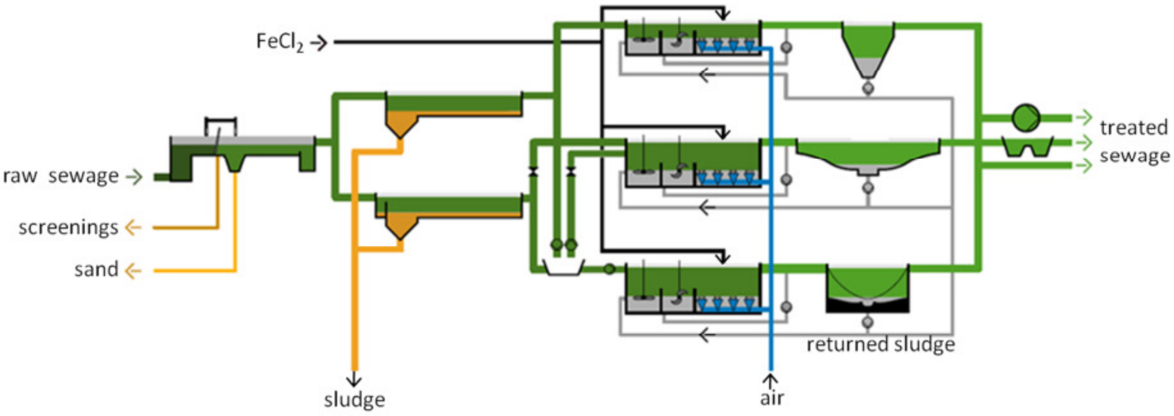


Figure 4: Ruhleben WWTP - process flow (BWB 2007)

In Table 2 the main parameters describing the effluent water quality of the Ruhleben WWTP are summarized. Aside from the turbidity all parameters were determined out of grab samples during the test operation period in the pilot plant influent. Characteristic are the low total phosphorus, and ortho phosphate concentrations and high concentrations of dissolved organic carbon (DOC). Due to co-precipitation with $FeCl_2$ the water already holds 0.13 to 0.51 mg/L of iron.

Table 2: Ruhleben WWTP effluent water quality (grab samples 29.09.10 to 15.09.12)

Parameter		mean value	min	max	n
Total phosphorus (TP)	[µg/L]	323	110	1400	382
PO_4-P	[µg/L]	67	15	978	338
Suspended solid (SS)	[mg/L]	5.6	1.4	22	359
COD	[mg/L]	42	25	65	341
DOC	[mg/L]	12.8	7.5	17.0	154
pH-value	[-]	7.2	6.9	7.6	157
UV absorbance	[1/m]	29.9	18.5	44.1	168
Transmittance	[%]	47	29	63	232
Turbidity (online)	[NTU]	3.5	1.4	15.0*	382
Iron (Fe)	[mg/L]	0.23	0.13	0.51	89
Aluminum (Al)	[mg/L]	0.03	0.01	0.16	292

* Measurement range ≤ 15 NTU

For the chemical oxygen demand (COD) and total phosphorus at the Ruhleben WWTP the water authority defined monitoring values of 60 mg/L and 0.5 mg/L (BWB 2007). In all grab samples the WWTP met the monitoring value for the COD. 120 out of 129 grab samples contained less than 0.5 mg/L total phosphorus.

2.3 Physico-chemical phosphorus removal

There are five main steps of the physico-chemical phosphorus removal:

- Dosage and dispersion of coagulant
- Precipitation: Formation of particulate compounds out of coagulant cations (Fe^{3+}, Al^{3+}) and phosphate anions (PO_4^{3-}) or other anions
- Coagulation: Destabilization of colloids and agglomeration to micro flocs
- Flocculation: Floc growth, formation of good separable macro flocs
- Floc separation.

Figure 5 shows the three main possibilities of integrating physico-chemical phosphorus removal in a common biological wastewater treatment process: pre-precipitation, co-precipitation and post-precipitation. For the pre-precipitation the coagulant is dosed before the primary sedimentation. The floc removal takes place in the sedimentation tank. There are three different possible coagulant dosing spots for the co-precipitation. The coagulant can be dosed into the activated sludge tank, before the secondary sedimentation or into the return sludge. When dosing into the activated sludge tank bivalent coagulants can also be applied. Due to the aeration Me^{2+} will be oxidized to Me^{3+}. (ATV-DVWK-A 2004)

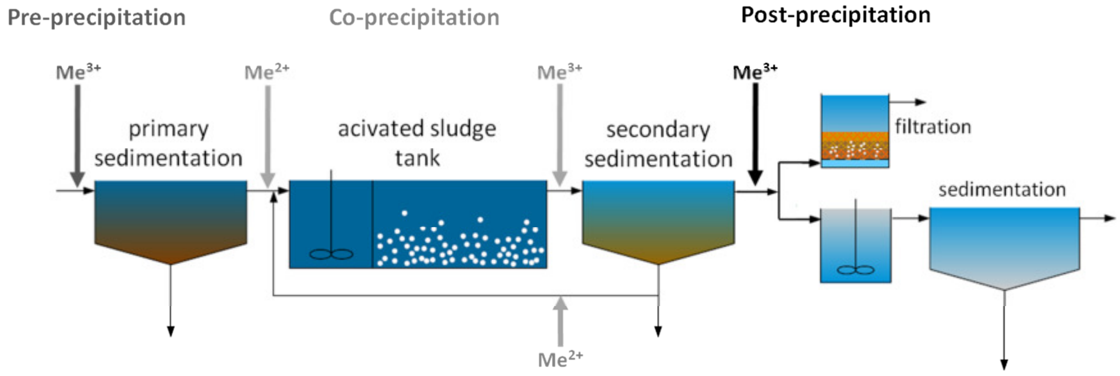


Figure 5: Phosphorus precipitation in wastewater treatment plants (Pinnekamp et al. 2007)

Whereas pre- and co-precipitation can be integrated into existent processes the post-precipitation is a separate treatment step placed at the end. Furthermore, an extra separation step for the floc removal is necessary, e.g. a sedimentation or flotation. For very strict water quality parameters a filtration step is required. Many wastewater treatment plants conduct the activated sludge process including nitrification-denitrification and biological phosphorus removal. With biological phosphorus removal alone the discharge values can often not be met. In such cases a supporting co-precipitation can be applied like it is done in the Ruhleben WWTP. (ATV-DVWK 2004, Bratby 2006)

In this study a post-precipitation is applied. The flocculation is enhanced by dosing polymers and the separation step is carried out through microsieve filtration. The importance and characteristics of the chemicals and the separation step are enlarged in the following chapters.

2.3.1 Precipitation and coagulation

Dispersed particles in water have the ambition to accumulate due to van der Waals forces. However, this is hampered by an electrostatic repulsive force as dispersed material often possesses a negative surface charge. Figure 6 shows schematically the potential of the two forces depending on the distance of the particle surface. If particles coagulate, they have to overcome repellent electrostatic forces. The goal of the destabilization is to minimize or even prevent the electrostatic repulsive force. Consequently particles reach the van der Waals dominated area with less kinetic energy. (Gimbel et al. 2004)

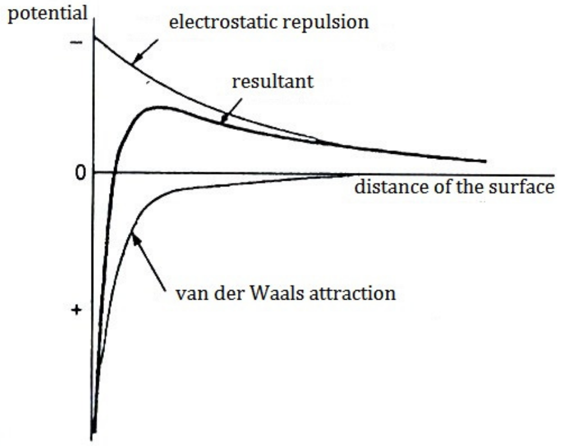


Figure 6: Colloid surface charge (Gimbel et al. 2004)

Coagulants are added to destabilize dispersed water compounds. Aluminum (Al) and iron (Fe) salts are conventionally considered as coagulants for P removal, because of their relatively low costs and constant availability (Bratby 2006). For many applications pre-polymerized coagulants like polyaluminum chloride (PACl) and polyferric chloride are selected as coagulant. It was shown that they have high treatment efficiency for turbidity, heavy metals and chemical oxygen demand (COD), as well as an improved performance at low water temperatures (Jiang and Graham 1998).

The easiest phosphorus species removed by chemical precipitation are orthophosphate and particulate phosphorus. However organic phosphorus and polyphosphates are not as readily available for chemical precipitation but may take part in some precipitation or adsorption reactions. Of orthophosphate, a number of ionic forms exist, depending on the pH of the solution. In the pH range of 5 to 9 $H_2PO_4^-$ and HPO_4^{2-} are the predominant dissolved orthophosphate species, as displayed in Figure 7 (Bratby 2006).

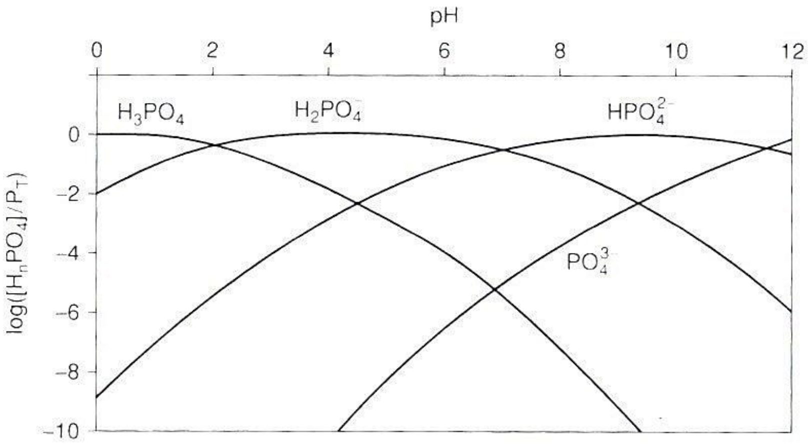


Figure 7: Distribution of phosphorus species with pH (Bratby 2006)

The P removal is based on the precipitation of phosphate anions with metal ions (Me) (Sigg and Stumm 1979; ATV-DVWK-A 2004):

$$Me^{3+} + PO_4^{3-} \leftrightarrow MePO_4 \qquad (1)$$

$$Me^{3+} + HPO_4^{2-} \leftrightarrow MePO_4 + H^+ . \qquad (2)$$

Due to the application of metal salt an exchange of anions in the water is initiated. Phosphate anions and metal ions create insoluble compounds, which can be removed by physical treatment. The minimum solubility of $AlPO_4$ is at pH 6 to 7 and of $FePO_4$ at pH 5 to 5.5. In these pH regions the best precipitation results can be theoretically expected (ATV-DVWK-A 2004). However, in this pH range the required floc formation can be unsatisfying. Due to this reason, efficient P removal is carried out practically at pH values from 6.5 to 8.5 (Barjenbruch and Exner 2009).

The pH plays an essential role in coagulation. If the water is not well buffered and has a low alkalinity, the pH of the treated wastewater decreases with increasing metal coagulant dose. If the pH value drops under the optimal pH, the residual P concentration can progressively increase despite an increasing coagulant dose. This is due to the formation of competitive complexes (Bratby 2006). The pH range influences the removal of suspended solids (SS) and natural organic matter (NOM), too. PH-values around neutral show advantages for efficient SS and NOM removal. Additionally, the residual aluminum concentration is strongly depending on the pH value. A pH between 6.5 and 7 is favorable to achieve low residual aluminum concentrations (Zouboulis and Traskas 2005; Mortula et al. 2009).

Simultaneously with the phosphate precipitation competitive reactions occur, which results in a higher consumption of coagulant and increases the amount of sewage sludge. The most important competitive reaction for Al^{3+} and Fe^{3+} is the formation of hydroxide:

$$Me^{3+} + 3OH^- \rightarrow Me(OH)_3 . \qquad (3)$$

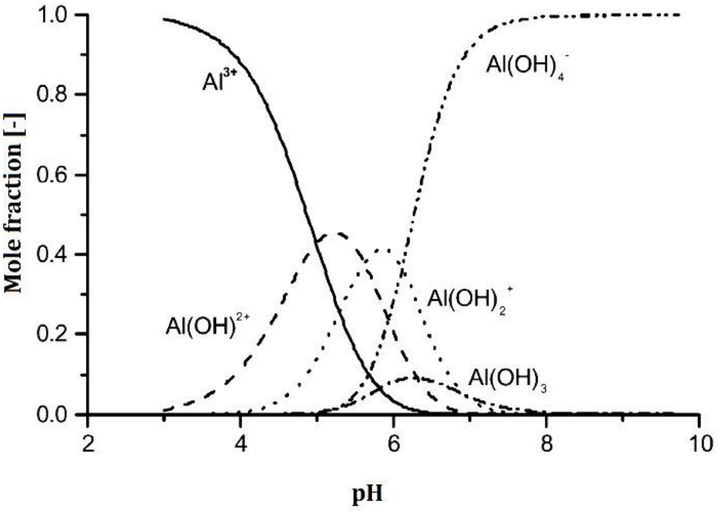


Figure 8: Mole fraction of dissolved hydrolysis products in equilibrium with amorphous hydroxides (Duan and Gregory 2003)

Figure 8 shows which hydrolysis products are formed in the dependence of pH, when using aluminum as coagulant. Efficient rapid mixing can avoid competing reactions with hydroxide, as reactions between metal ions and phosphate anions complete rapidly in less than one second. Furthermore, rapid mixing is advantageous, as the destabilization process demands an efficient dispersion of the coagulant. The applied coagulant should reach as many particles in the dispersion as possible. Especially when PACl is used, to achieve an effective dispersion is often a problem. Besides enhancing the dispersion of coagulant, rapid mixing speeds up the initial growth stage during floc formation. Additionally it influences the structure of the primarily formed flocs. Due to the high shear forces during rapid mixing, the surviving flocs are formed to dense and small flocs and serve as floc core (Bache and Gregory 2007). The average shear rate is expressed with the *G* value (see equation (4)). The G value is defined as average of the energy dissipation rate per unit volume of suspension. Often, the *G* value associated with the mixing time *t* is utilized as an index to design and operate large scale plants. In literature *G* values ranging from 40 to 1000 s^{-1} and mixing times from 0.5 s to 4 min have been reported as optimal (Sheng et al. 2006).

$$G = \sqrt{\frac{P_{Water}}{V \cdot \eta}} \tag{4}$$

P_{Water}	power input into the water [W]
V	volume of the water [m^3]
η	dynamic viscosity [kg/(m·s)]

The required coagulant dose can be estimated with the aid of the β factor. The β factor is the quotient of the coagulant dose (Me) and the orthophosphate phosphorus (PO_4-P) in the influent, which can be removed via precipitation (see equation (5)).

$$\beta = \frac{Me^{3+}\left[\frac{mmol}{L}\right]}{PO_4 - P\left[\frac{mmol}{L}\right]} \tag{5}$$

For conventional flocculation filtration in tertiary wastewater treatment a β factor of 2.5 is recommended (ATV-DVWK-A 2004). It has to be noticed that the β factor increases with decreasing P concentrations. Furthermore, high turbidities of the treated water and a high content of dissolved organic matter will increase the coagulant demand.

2.3.2 Flocculation with the aid of polymers

In this application polymer is used as flocculant aid. The polymer is added after the destabilization with metal coagulants to promote the formation of larger, stronger and denser flocs for better separation behavior during filtration (Bratby 2006).

Flocculation describes the particle aggregation and growth into millimeter sized flocs. The flocculation process is divided into two stages. The first stage is the perikinetic flocculation. It is based on the Brownian movement and occurs naturally. The perikinetic flocculation is completed immediately after coagulation and the floc size is limited. The second stage is the orthokinetic flocculation which is caused by an induced velocity gradient in the liquid. Due to the velocity gradient, particles within the liquid possess different relative velocities and the opportunity for particle contact is created. The higher the velocity gradient in the liquid, the higher the probability of particle contact within a given time. Though, the higher the applied velocity gradient, the smaller the final floc size due to the breakup of larger flocs. As a result, the *G* value and the time of flocculation are the main parameters influencing the flocculation (Chen at al. 1998).

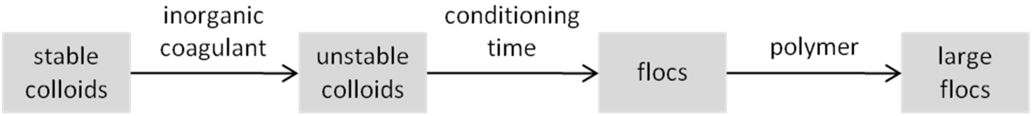


Figure 9: Floc formation with the aid of polymer (Kitchener 1972)

In many applications it is essential to produce strong flocs, which are resistant to high shear forces. A polymer is often added to supplement the orthokinetic flocculation process and to alter floc characteristics. The floc size and strength will improve likely due to the formation of polymer bridges between particles. Usually the best results are obtained when the polymer is added to the suspension after the coagulant addition. The primary formed flocs already could grow to a reasonable size, thus resulting in a lower polymer demand. Moreover, this enhances complete utilization of the coagulant and hence there are fewer residues in the treated water. The process is displayed schematically in Figure 9 (Kitchener 1972; Chen et al. 1998; Bratby 2006).

There is a multitude of products and a wide variety of parameters influencing the choice of a polymer. The applicability of polymers is dependent on raw water quality, type of primary coagulation, polymer characteristics, costs and the separation technique. Available polymer products differ in e.g. charge (anionic, nonionic or cationic), charge density, molecular weight (10^6 to 10^7 g/mol), form (powdered, liquid) and the chemical composition. Furthermore, there are natural and synthetic polymers. Natural polymers are based on e.g. starch, chitosan or alginates. They are known to be less effective than synthetic polymers but hold the benefit of less or no toxicity and biodegradability. (Bratby 2006; Bache and Gregory 2007)

Synthetic polymers are much more commonly used. They have a better performance due to the possibility of controlling polymer characteristics e.g. molecular weight, number and type of charged polymer units. The toxicity of synthetic polymers lays in residual unreacted monomers (e.g. acrylamide and ethylenimine), unreacted chemicals used to produce the monomers and reaction by-products of polymers in water. Especially cationic polymers are considered being highly toxic. They are 100 times more toxic to aquatic organism than anionic polymers (Harford et al. 2011; Bolto and Gregory 2007). After the coagulation with metal coagulants the application of anionic polymers is often very effective. This is due to the fact that the metal hydroxides form positively charged particles on which the anionic polymers can easily adsorb. (Bratby 2006; Bolto and Gregory 2007; Gregory 1977)

The polymer demand is depending on the type, concentration, size distribution and surface charge of the particles to be removed and the formed metal hydroxo-complexes. Furthermore, influencing parameters are the pH, the amounts of organic and other substances and the floc separation technology. Typical polymer doses are 0.5 to 1.5 mg/L, occasionally even up to 10 mg/L (Bratby 2006). In drinking water treatment with flocculation and filtration polyacrylamide doses up to 0.2 mg/L are common (DVGW 1990).

2.3.3 Floc separation via filtration

There are several technologies available for floc separation via filtration. Conventionally dual media filtration is applied for floc separation after flocculation (without polymer). Apart from the dual media filtration and microsieve filtration the OXERAM project also considers ultrafiltration and microfiltration (see Table 3).

Table 3: The floc separation technologies compared in OXERAM

Process	Dual media filtration	Ultrafiltration*	Microfiltration*	Microsieve filtration
Separation mechanism	depth filtration	surface filtration	surface filtration	surface filtration and straining
Filter opening	-	0.01-0.1 µm	0.01-0.1 µm	10-100 µm
Driven Force	gravity	pressure	pressure	gravity
Transport mechanism	dp >1 µm: diffusion, sedimentation dp < 1 µm: diffusion	diffusion	diffusion	convective
Filter material	sand, anthracite, pumice, coke	polymer	ceramic	polyester
Advantages	robust technology	disinfection	disinfection	small headloss small footprint
Disadvantages	large filter area	energy demand	energy demand	chemical demand less SS removal

*(Melin and Rautenbach 2007)

Dual media filtration is classified as depth filtration. Depth filtration means particle separation in the pores between the filter grains. Depending on their size the particles reach the surface of the filter material due to sedimentation, diffusion or interception. Through Van-der-Waals forces the particles adhere to the filter granulate. In most cases the filter material consists of sand, pumice or anthracite. Depth filtration is often designed as a non-pressurized dual media filter. When there is cake or surface filtration the particles are primarily retained on the filter media or in the filter cake. (Seyfried 1991; Bever et al 2002; Gimbel et al. 2004)

Microsieve filtration or rather microstraining is associated with surface filtration but dominated by straining effects. This means only particles larger than the mesh size (e.g. 10 µm) are retained. However, at filtering processes like depth or cake filtration particles are retained due to physico-chemical effects. Thus, also particles smaller than the filter openings are retained. Often there is no sharp boundary between straining and filtering. In a microsieve light physico-chemical retention processes can occur due to the deposition of a thin particle layer. (Seyfried 1991; Bever et al. 2002)

Microfiltration (MF) and ultrafiltration (UF) membranes are known to be porous membranes with pore sizes of 0.1 to 10 µm (MF) and 0.01 to 0.1 µm (UF). Thus, the particle removal is influenced by straining effects as long as there is no cake layer. Dead-end filtration accelerates the formation of a cake layer which determines the separating cut. MF and UF are pressure driven filter systems. They work in the ranges of 0.3 to 3 bar (MF) and 0.5 to 10 bar (UF). (Melin and Rautenbach 2007)

Table 4 gives the results of coagulation in combination with membrane and dual media filtration. The pilot plants treated the secondary effluent of the WWTP Ruhleben. All results are given as 80 % percentiles. Both membrane pilot plants achieved total phosphorus values in the effluent of about 30 µg/L. In the effluent of the dual media filter there were about 70 µg/L TP. Effluent contents of 0.5 mg/L SS were achieved by the dual media filter and 0.2 mg/L SS by the MF. The SS contents in the effluent of the UF were below the detection limit (0.1 mg/L). The membrane pilot plants achieved effluent iron contents of 0.001 mmol/L. In the effluent of the dual media filtration there were 0.002 mmol/L Fe.

Table 4: Pilot plant results from membrane and dual media filtration at Ruhleben WWTP. Membrane results from the project OXERAM. Results from dual media filtration see (Miehe 2010).

		Depth filtration Dual media filter 0.034 mmol/L Fe	**MF (0.1 µm)** ceramic membrane 0.07 to 0.2 mmol/L Fe		**UF (0.02 µm)** polymeric membrane 0.036 mmol/L Fe	
		80th percentile	80th percentile	n	80th percentile	n
TP [µg/L]	Influent	350	460	24	350	10
	Effluent	70	31	24	30	10
SS [mg/L]	Influent	5.6	7.7	17	8.6	8
	Effluent	0.5	0.2	17	< 0.1	8
Fe [mmol/L]	Influent	0.004	0.005	23	0.006	9
	Effluent	0.002	0.001	23	0.001	9

2.4 Microsieve filtration

2.4.1 The history of microsieve filtration in tertiary wastewater treatment

In the beginning of applying microsieve filtration for tertiary wastewater treatment usually drum filters with mesh sizes of 23 or 35 µm were used. In the 1940's the first rotating drumfilter was developed by Glenfield & Kennedy Ltd. It was used for effluent polishing of biologically treated wastewater. (Ljunggren 2006)

The first installation was established in Luton, England in 1950 and soon installations could be found in Australia, South Africa and North America (Diaper 1969). In the beginning of the 1960's microsieve filtration was tested at the WWTP Harpenden. The drumfilter was equipped with a steel mesh (35 µm, 1 m/h). An average reduction of SS in the effluent of a thickling filter from 46.9 to 12.8 mg/L could be achieved. In comparison after a secondary clarification the SS amounted to 22.3 mg/L. (Truesdale et al.1964)

Soon afterwards chemical pre-treatment to microsieve filtration was tested in order to improve the performance of microsieves and enlarge their applicability. The results showed that adding coagulant did not improve the process because the flocs were not strong enough to endure the shear forces appearing in a microsieve (Truesdale and Birkbeck 1968; Lynam et al. 1969). The flocs can be strengthened by applying polymers. Good results were achieved by pre-treating the water with trivalent metal salts and anionic polymer (SS removal of 85 %, effluent SS < 4 mg/L) (Ewing 1976). Nowadays many microsieves are equipped with filter cloths of 10 µm or similar. Besides more and more disc filters are applied instead of drum filters (Ljunggren 2006).

2.4.2 Characteristics and operation principle of a disc filter

Microsieves are surface filtration systems equipped with synthetic or steel filter fabrics offering mesh sizes between 10 and 200 µm. The two main designs are the drum and the disc filter. In both designs the water enters the microsieve through a central drum. Whereas the central drum of a drum filter is directly covered with the filter fabric a disc filter (see Figure 10) features discs that are covered on both sides with the filter fabric and are vertically mounted on the central drum. Both systems are gravity driven and mainly designed for an inside-out filtration. Approximately 50 to 65 % of the filter area are submerged in the filtrate depending on the filter model. During the filtration with a disc filter particles are retained inside the discs and block the filter. Due to a decreasing permeability the water level in the disc filter rises until a level sensor initiates the backwash. The maximum operational differential pressure is typically in the range of 250 to 300 mm. The discs start rotating and the spray nozzles clean the filter panels outside-in with filtrate water at approximately 8 bar. The removed solids are collected and discharged by a trough. The backwash requires 0.5 to 3 % of the total flow. During backwash the filtration process continues. The filtrate flow increases and the water level decreases. As soon as the level sensor loses touch with the water a timer is activated which stops the backwash after 30 sec. The major advantages of a disc filter are the low head loss and the compact design. Thus, the disc filter could be an alternative to the conventional filtration technologies with a low energy demand and a small footprint. (Bever et al. 2002; HYDROTECH 2010a)

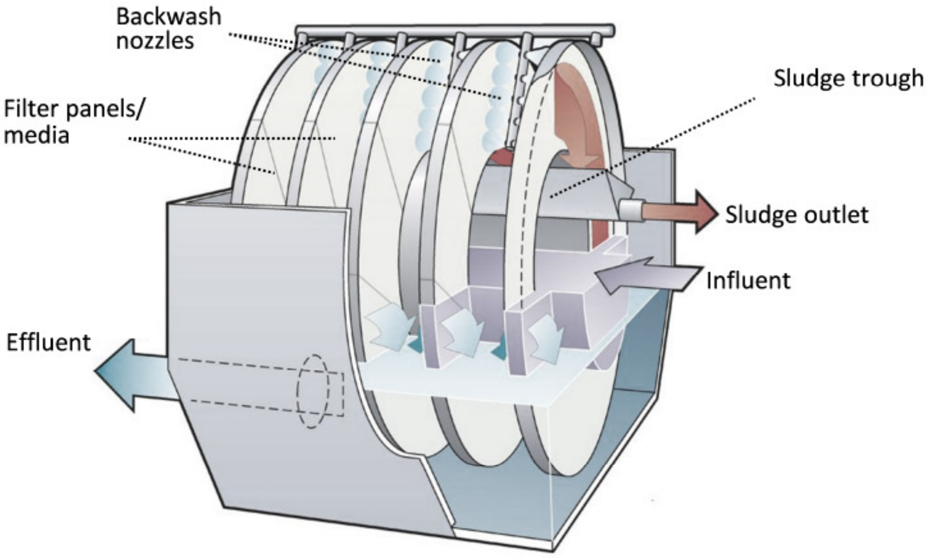


Figure 10: Principle of a disc filter (HYDROTECH 2010a)

2.4.3 Application of disc filters in tertiary treatment

Disc filters in large-scale applications for tertiary wastewater treatment are usually simple microsieve filtration processes without chemical pre-treatment. The world largest disc filter application has been installed in Gothenburg, Sweden. The WWTP was expanded by 32 Discfilter^{TM} units (June 2010) in order to meet the new phosphorus removal goals (< 0.3 mg/L TP). The maximum capacity amounts to 36.000 m^3/h (see Table 5). Since June 2006, there is a large-scale plant operating in Barcelona with the ACTIDisc® process, combining the Actiflo® process with microsieve filtration. This is the hereto first large-scale disc filter application in tertiary treatment with chemical pre-treatment. The Actiflo® process comprises coagulation, flocculation, maturation and lamella separation. The formation of good settleable flocs is enhanced by the addition of microsand. Achieved effluent values for turbdity and SS are < 2 NTU and ≤ 5 mg/L. (HYDROTECH 2010b; HYDROTECH 2010c; Sanz et al. 2007)

Table 5: Examples for application of disc filters in tertiary wastewater treatment (HUBER 2009; HUBER 2010; HYDROTECH 2010b; HYDROTECH 2010c; SIEMENS 2009; SIEMENS 2010; SIEMENS 2011)

Location	objective	Filter type	Number of units	Mesh size [µm]	Capacity [m^3/h]	Supplier
Gothenburg, Sweden	P removal (< 0.3 mg/L TP) via filtration	HSF 2200	32	15	36.000	Hydrotech
Barcelona, Spain	Treatment for water reuse via ACTIDisc®	HSF 2220	10	10	14.400	Hydrotech
Kuwait City, Kuwait	Treatment for water reuse	Forty-X	8	10	7.500	Siemens
Ohio, USA	Replacement of sand filtration	Forty-X	4	10	2.840	Siemens
Lie De, China	Pre-treatment to UV disinfection	RoDisc	28	-	30.350	Huber

By replacing the sand filtration system in the Sycamore Creek WWTP with 4 disc filters the area required for filtration was reduced by 50 %. Furthermore, the capacity was increased by 470 m^3/h. The Forty-X disc filters (10 µm mesh size) from Siemens are supposed to achieve a SS reduction of 10-30 mg/L down to < 5 mg/L when placed behind a secondary clarifier. (SIEMENS 2009; SIEMENS 2011)

In the course of the modernization and extension of infrastructures for the Asia Olympics 2010 the WWTP of Lie De in China was equipped with 28 RoDisc® units with a maximum capacity of 30,350 m^3/h. (HUBER 2009; HUBER 2010)

2.5 Microbiological water quality

2.5.1 Pathogen removal in wastewater treatment

Many pathogenic microorganisms (MO) are added to water bodies through effluents of WWTPs, as the reduction of pathogens through the primary and secondary cleaning stages of WWTPs is often not sufficient. The concentration of fecal coliforms in the influent of WWTPs is located at around 10^6 MPN/100 mL and is reduced to 10^5 after secondary treatment and 10^4 after sand filtration, as shown in Figure 11. A further reduction of pathogens in WWTPs is necessary, because at a concentration of 10^4 MPN/100 mL fecal coliforms the water is still considered as infectious. (Schöler 2002)

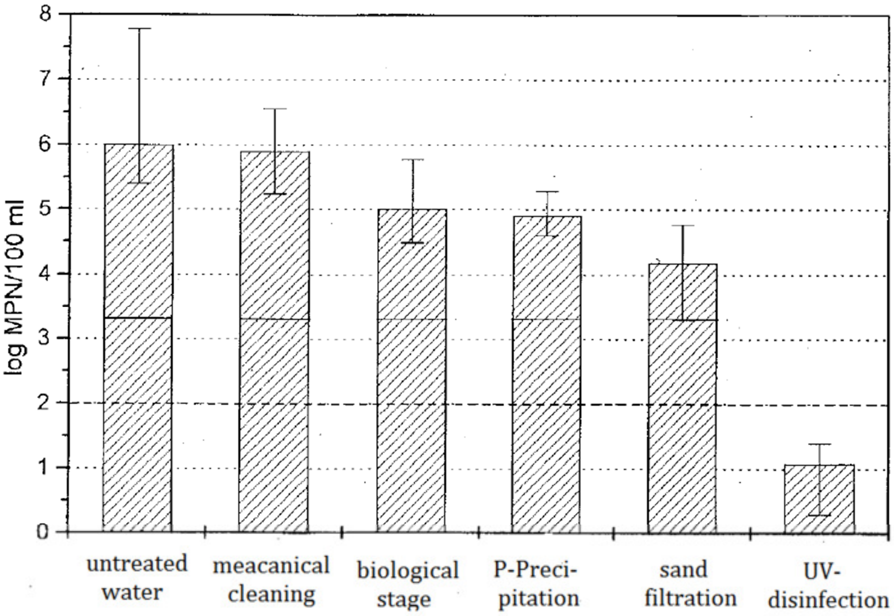


Figure 11: Content of fecal coliform bacteria of effluents of different cleaning stages (Schöler 2002)

Without a specific treatment for pathogenic-germs removal a sufficient pathogenic reduction usually cannot be achieved in a WWTP. Furthermore, the assimilative capacity of water bodies is often not satisfactory (Schöler 2002).

Until now no requirements for WWTPs discharges exist that specify bacterial parameters. To estimate and set the bacterial standards the microbial requirements of the EG bathing water quality Framework Directive often is referred to (see Table 6). In

2011 the bathing water quality Directive divided the bathing water quality into four categories: excellent quality, good quality, adequate quality and poor quality. The parameter *E. Coli* represents *Fecal coliform* bacteria and represents germs, which are located in the feces of humans. The parameter *Intestinal Enterococcus* represent the *Fecal Streptococci* and covers also bacteria, which are present in the feces of animals (Müller et al. 2009).

The effectiveness of wastewater disinfection is detectable only via biological analysis. In general for WWTP effluents fecal indicators are used to determine the effectiveness of disinfection. They serve as verification of fecal impurities of the tested water. To evaluate the effectiveness of disinfection, the concentration of fecal indicators is determined before and after the disinfection process. The reduction rate is given in log levels.

As fecal indicators are present in large quantities in the intestine of endotherms, they are easier to detect than other pathogens. Their number indicates how many other pathogenic germs also remain. Fecal indicators are *Fecal* Coliform bacteria, *Fecal Enterococcus* and *Coliform bacteria.*

Table 6: Microbial requirements according to EG bathing water quality Frameworks Directive (2006/7/EG)

Parameter	Excellent quality	Good quality	Adequate quality
Intestinal Enterococcus [cfu/100 mL]	200 (*)	400 (*)	330 (**)
Escherichia Coli [cfu/100 mL]	500 (*)	1000 (*)	900 (**)

(*) Based on a 95- percentile validation

(**) Based on a 90- percentile validation

Coliform bacteria are not only present in the intestine of endotherms. Their presence only gives evidence of possible fecal impurities, but no verification ATV-M-205 1998. Therefore the parameter coliform bacteria is not considered in the EG bathing water quality Frame Directive 2006/7/EG .

For wastewater disinfection many processes are available, which are based on different physical and chemical principles. The most important ones are listed in Table 7. However, due to economic and environmental reasons as well as practical experience just a few of them are suited for the disinfection after secondary treatment.

Table 7: Disinfection processes (ATV-M-205 1998)

Processes
Ultraviolet radiation
Microfiltration
Ozonation
Chlorination
Peracetic acid or hydrogen peroxide applications

Chlorination is a common process for wastewater disinfection. However, the disinfection effectiveness depends on the type of MO. Chlorine is very effective against enteric bacteria, but it is less effective in the inactivation of viruses. Furthermore, possibly occurring chlorine forms have different oxidation strength. The dominant form of chlorine is depending on pH, temperature and ammonia concentration of the water. One main disadvantage of chlorine is the occurrence of undesirable side reactions with chlorine and organic or inorganic water substances, which can be toxic (EPA Victoria 2002; Schöler 2002).

Ozone is a very strong oxidant. Ozone achieves disinfection through free radicals, which are serving as oxidizing agents. Against viruses and bacteria ozonation can be more effective than chlorine. But the wastewater quality influences the effective bacterial action of ozone significantly. As ozone decays rapidly it has to be produced onsite. The high costs make ozonation usually not competitive with available alternatives (USEPA 1999; EPA Victoria 2002).

Peracetic acid is a strong disinfectant. In recent years, it has gained more attention because of its wide and strong antimicrobial activity. Furthermore, it does not produce toxic byproducts; and it has a small dependence of the water’s pH. The disadvantage of the application of peracetic is an increase of the organic matter content, which can cause microbial regrowth. Furthermore, a necessary contact time at least 15 min was recommended for secondary effluents (Mehmet 2004).

Membrane filtration is another save disinfection method. Particles, bacteria, many viruses and nutrients are removed efficiently. An addition of chemicals is not needed, neither toxic byproducts are formed. Although membrane prices are decreasing, the technology is still costly. Moreover, the highly microbial contaminated concentrate has to be depolluted and chemicals may be needed for the backwash (EPA Victoria 2002).

UV-disinfection represents a low-priced method. The efficiency of the UV-disinfection strongly depends on the water quality. For the applied fluence range in wastewater disinfection no chemical changes of water substances have been detected and no addition of toxicity of the water has been observed (Schöler 2002). In this study UV radiation is used to disinfect the tertiary wastewater effluent.

2.5.2 UV disinfection for advanced wastewater treatment

Until now no microbial limit values exist for effluents of WWTPs in Germany. However for discharges of WWTP effluents into bathing water, the EG BWQ Frame Directive (2000/60/EG 2000) has to be fulfilled. As a result, several installations of WWTP effluent UV-disinfection exist at the North and Baltic Sea. Furthermore, many local communities and communes have launched programs to improve the microbiological quality of rivers e.g. the program “Badewasserqualität Obere Isar”. Therefore ten UV-disinfection plants are located in the north and south of Munich at the Isar. Currently in Germany there are approximately 20 installations for UV-disinfection operating (Blume 2002; Bleisteiner and Pfeiffer 2006). In the Ruhleben WWTP a UV-disinfection unit is installed, too. It is designed to treat 1 m^3/s of wastewater (UV transmittance of 45 %) with a fluence of 1000 J/m^2. The disinfection stage operates only during the summer months, treating the excess effluent water that cannot be pumped into the Teltowkanal through the high pressure pipeline. The energy demand of the installed UV-disinfection in Ruhleben is 50 Wh/m^3. The typical range of the specific energy demand for the UV-disinfection is 30 to 60 Wh/m^3 depending on the water quality (Müller et al. 2009).

The essential parts of an UV system are gas-discharge lamps, mostly mercury vapor lamps. The two main types of mercury vapor lamps operate with low pressure (LP) or middle pressure (MP). LP lamps emit monochromatic light at 254 nm, which is close to the absorbance maxima of MOs (Figure 12). MP lamps emit UV light in the rage of 200 – 400 nm. LP lamps have a higher conversion efficiency of electrical input to UV output then MP lamps. But, because a higher number of lamps and therefore also a larger apparatus are needed, the investment costs for LP lamps are also higher. MP lamps have a higher energy demand (DVGW 2006).

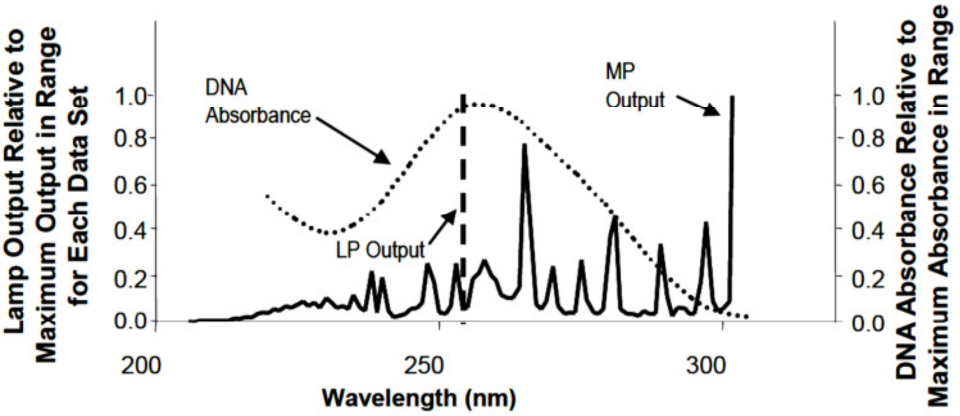


Figure 12: UV lamp output and its relationship to the UV absorbance of DNA (USEPA 2006)

To ensure the inactivation of MOs the fluence is an important parameter to determine. It is due to the product of constant fluence rate E_0 and time of exposure T (DVGW 2006). Because the fluence cannot be measured online yet, bioassay tests, calculations or numerical simulations are used. The bioassay tests determine the reduction equivalent fluence. Numerical simulations determine the physically applied fluence.

The inactivation rate of different MOs depends on the wavelength of the UV light applied, the microbe population and the species. The UV light disinfects at a wavelength between 240 and 290 nm, because DNA of MOs absorbs UV light in those ranges. After absorbing the light, it disables MOs to replicate and induces their cell death. As can be seen in Figure 12 the peak of absorption coincides with the emitted wavelength of LP lamps (254 nm). But depending on the species the resistance to this wavelength varies. Bacteria are less resistant than viruses and bacterial spores. Highly resistant are protozoan cysts and oocysts (Wrigth 1998).

A parameter influencing the fluence and therefore the UV efficiency is the transmittance. It describes a ratio between the transmitted light intensity and the incident light intensity.

The transmittance is dependent on the wavelength of the radiation, the radiation path length and the specific characteristics of the solved and unsolved waste-water contents to absorb or scatter light. Organic and inorganic solved substances can possess the ability to absorb UV light causing the radiation intensity to increase. This absorbed light is not available for the disinfection. Chemicals, added to cause coagulation and flocculation can intensify this effect (Schöler 2002). Typical UV absorbers are humic and fulvic acids, other aromatic organics, metals and anions (USEPA 2006). Suspended solids can also absorb or scatter the UV light. Additional they can shadow MOs or enclose them in the particles and make them inaccessible for UV disinfection.

An effect decreasing the transmittance is fouling outside of the glass lamps. Fouling material, composed of calcium, iron, magnesium, aluminum, sodium, phosphorus and other metals, absorbs UV light and decreases the fluence. The fouling process depends on the water quality and therefore is site specific. Fouling at the outside of the quartz sleeves of the UV reactor is one of the limitations of the UV system. Fouling material absorbs UV radiation and therefore causes the delivered fluence to decrease. Inorganic fouling layers on quartz sleeves are mainly composed of calcium, iron, magnesium, aluminum, sodium, other metals and phosphorus.

Three mechanisms have been detected to explain the deposition of foulants on quartz sleeves, which are:

- thermally induced precipitation of metals, which solubility decreases as temperature increases
- settling of particles due to gravitation and impaction of particles
- flocculation.

The fouling process is site specific and varies with composition of the treated water. Until now no general correlation could be drawn. The water quality influences the fouling process. Additionally, the flow rate in the UV reactor plays an important role, as it affects to which extent deposits remain on the sleeves. Furthermore, calcium seems to stand out as a major foulant next to iron. Calcite precipitation causing inorganic fouling is induced at calcium concentration above 115 mg/L. Studies also have shown that when aluminum was used as coagulant, it made up a significant portion of the fouling layer. The addition of aluminum leads to a floc formation, which sediment onto the quartz surface (Lin et al. 1999; Sehnaoui and Gehr 2000; USEPA 2006).

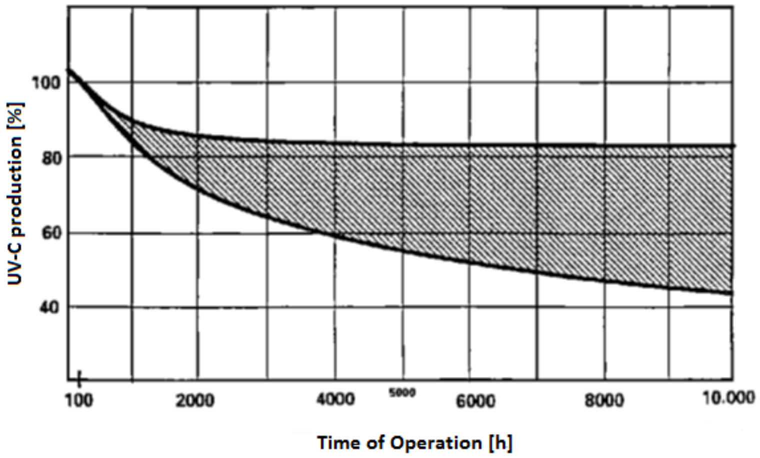


Figure 13: Lamp aging (DVGW 2006)

Another influencing parameter which all LP and MP lamps have in common is lamp aging. Lamp aging causes a decrease of the lamp output, which is shown schematically in Figure 13. The lamp degradation is a function of the time of operation, number of on/off turns, water temperature, power applied per lamp length and heat transfer from lamp (EPA 2006).

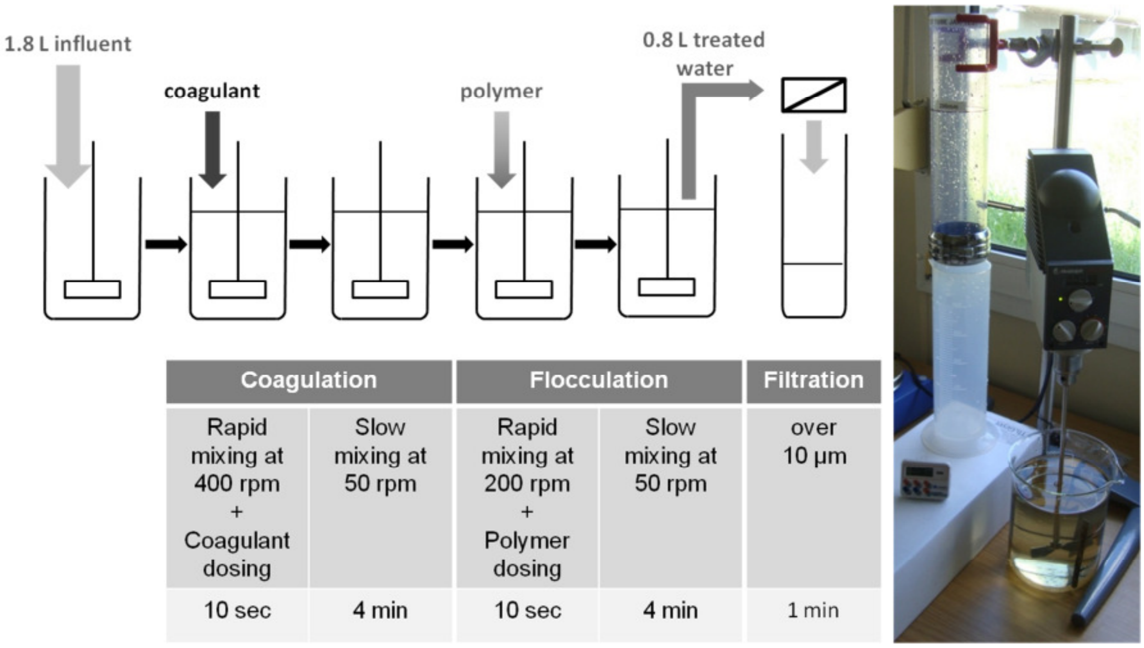
Chapter 3

Materials and methods

3.1 Laboratory tests

3.1.1 Jar test

Different operating conditions were tested with the aid of jar tests. All jar tests were conducted according to the DVGW worksheet W 218 (1998). To simulate the filtration step the test setup was complemented with a HYDROTECH test tube. The end of the test tube was covered with the filter cloth (mesh size 10 µm), which was also used in the pilot plant. For the jar test 1.8 L of pilot plant influent are filled into a 2 liter beaker equipped with a stator and a stirrer (see Figure 14). The coagulant is added at 400 rpm. The rapid mixing lasts 10 sec. Afterwards the mixing velocity is reduced to 50 rpm. After for 4 minutes of slow mixing the polymer is added at 200 rpm. Again the rapid mixing lasts 10 sec and is followed by 4 min of slow mixing at 50 rpm. Afterwards 0.8 liters of the treated water are poured into the test tube and pass the mini microsieve. Within the first 10 sec of filtration the filterability is determined (see equation (6).The filtration is closed after 1 min.



Coagulation		Flocculation		Filtration
Rapid mixing at 400 rpm + Coagulant dosing	Slow mixing at 50 rpm	Rapid mixing at 200 rpm + Polymer dosing	Slow mixing at 50 rpm	over 10 µm
10 sec	4 min	10 sec	4 min	1 min

Figure 14: Jar test set-up and procedure

Besides the coagulant and polymer dosing also the pH was varied and different types of suspended solids were added. To change the hydraulic conditions mixing velocities and retention times were varied. To characterize the filtrate water quality several parameters were measured. On-site total phosphorus concentration and the turbidity were measured. For selected samples the contents of total phosphorus, aluminum and iron were determined at the laboratories of BWB. Besides, the filtrate was tested for its post-flocculation behavior and the flocculated water for its filterability.

3.1.2 Fractionation test

In order to identify the main size fractions of total phosphorus and total aluminum/ iron in the effluent of the pilot plant the filtrate was filtered over a cellulose nitrate filter with different pore sizes. The pore sizes used were 5.0, 1.0, 0.45 and 0.1 µm. Each filter was fed with 300 mL of filtrate. Thus prepared water samples were sent to the BWB laboratories to be analyzed.

3.1.3 Post-flocculation test

The post-flocculation test was conducted to check the floc formation in the filtrate due to coagulant breakthrough. For the test 0.15 to 1.0 L of filtrate were stirred at a medium stirring rate. After 24 h the water was tested for floc formation by a visual control. For an example see Figure 15. The evaluation was carried out with the aid of the following scheme:

0	no post-flocculation: no flocs visible
1	post-flocculation: few small flocs
2	strong post-flocculation: many large flocs

Figure 15: Post-flocculation test (15.11.2010)

3.2 Microsieve pilot unit

3.2.1 Initial design

The microsieve pilot unit was planned and built by Hydrotech AB, Sweden. It was installed at the Ruhleben WWTP in September 2010. In Table 8 the design parameters of the pilot plant are listed. It is dimensioned for 10 to 30 m^3/h and a maximum suspended solids concentration in the influent water of 20 mg/L.

Table 8: Pilot plant design parameters

Parameter		Maximum	Average
Flow	[m^3/h]	30	10
Suspended solids	[mg/L]	20	5 - 10
Total phosphorus	[mg/L]		0.4
Total filtrated phosphorus	[mg/L]		0.3
Ortho-phosphate (as P)	[mg/L]		0.2
Coagulant dose (as metal)	[mg/L]		3 - 8
Polymer dose	[mg/L]		1 - 3

Figure 16 shows the flow chart of the pilot plant at its initial configuration. The pilot unit is fed with the effluent of the secondary clarifier. The coagulant is conveyed by a membrane pump and injected into the influent water pipe in front of a static mixer. Only seconds after passing the static mixer the water enters the coagulation tank at the bottom of the tank. The coagulation tank is fitted with a stirrer and has a volume of 2.2 m^3 which can be reduced with the aid of a movable wall to 1.4 m^3 (66 %). At peak flow (30 m^3/h) the hydraulic retention time (HRT) amounts to 4.3 min. The water exits the tank through an overflow. In order to benefit from the turbulence the polymer is dosed via another membrane pump into the overflow. Optionally, the polymer can be post-diluted with tap water by a factor 2 to 20. For the flocculation and maturation a second stirred tank is provided. The tank has a volume of 2.0 m^3 (4.1 min HRT at peak flow) which can also be reduced to 1.4 m^3. Through another overflow pipe the water gets to the microsieve. Due to the operation principle of a disc filter the water level on the feed side varies and thus also the submerged filter area varies. Nonetheless it can be assumed that on average the submergence amounts to 50 %. The filter media applied in this study had a mesh size of 10 µm.

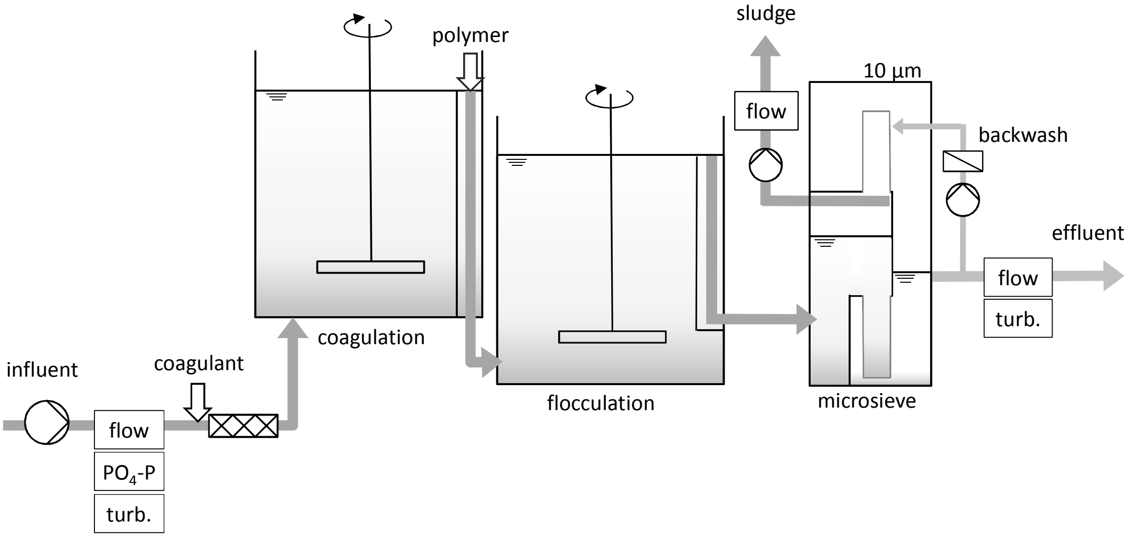


Figure 16: Flow chart of the microsieve pilot plant

The polymer is prepared in an automated polymer station enabling continuous operation 24/7. Each make-up has a volume of 110 to 200 liters and a concentration of active substance between 0.5 and 4.0 g/L. The cationic polymer was made up with 1 g/L. At a dose of 2.0 mg/L and a flow of 20 m^3/h one make-up lasts for 5 h.

The pilot plant is equipped with several online measurement devices. Turbidity is measured in the influent and effluent with Ultraturb plus sc probes from Hach Lange. Ortho phosphate was determined in the influent with a Phosphax sc probe in combination with a Filtrax filtration unit also from Hach Lange. As the ortho phosphate content proofed to be lower than expected and the detection limit of the Phosphax sc too high the orthophosphate signal of the WWTP effluent (Phosphax sigma) was used. Furthermore, the influent, effluent, and backwash flow and the backwash time were quantified.

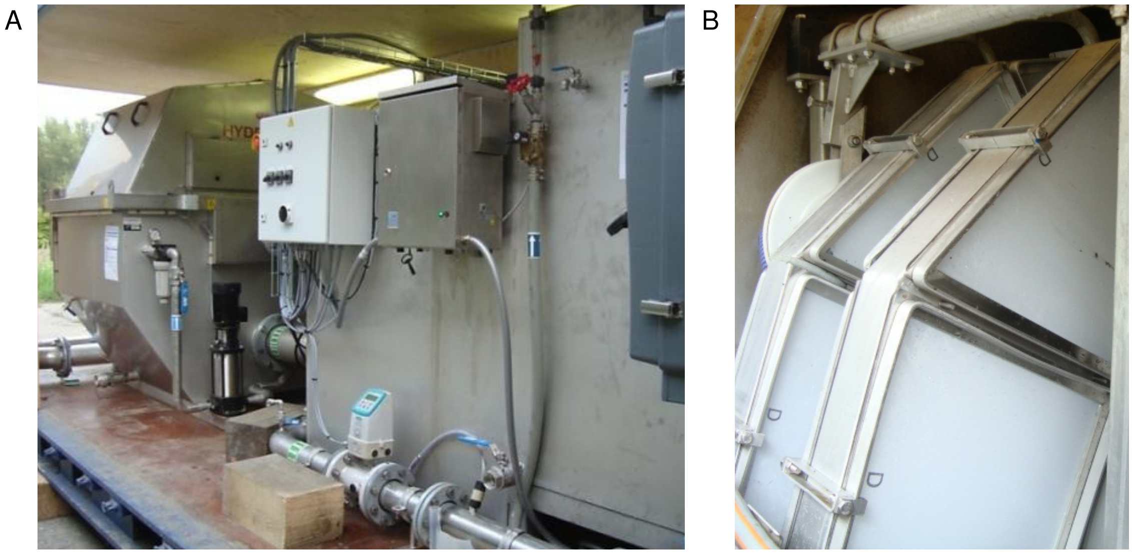


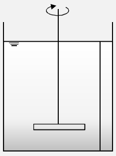
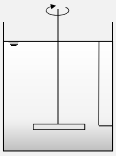
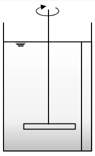
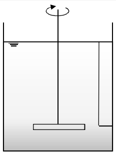
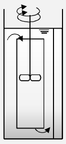
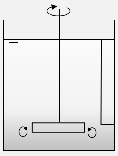
Figure 17: Microsieve pilot plant at Ruhleben WWTP (A) and microsieve filter discs (B)

Figure 17 gives some impressions of the microsieve pilot plant. The picture on the left shows the disc filter and the flocculation and coagulation tanks.

3.2.2 Configuration of coagulation and flocculation

Initially both tanks for coagulation and flocculation were equipped with 2 blade propeller mixers. The configuration of the coagulation and flocculation tanks was adjusted twice. From October 2010 to June 2011 the pilot unit was operated like described above. In June 2011 the coagulation tank was reduced by 33 % (see Table 9). With a volume of only 1.4 m^3 the HRT is reduced to 3 min at peak flow and the G value rises from 70 to 90 s^{-1}. During reconstruction works in October/ November 2011 another partition wall was integrated into the coagulation tank reducing the tank volume to 0.6 m^3. In order to avoid short circuiting a Turbomix™ was installed consisting of a 3 bladed propeller mixer, a cylindrical baffle plate and anti-vortex baffles. Furthermore, the angle of the stirrer blades in the flocculation tank was adjusted to increase the turbulence (G value 150 s^{-1}).

Table 9: Volumes, hydraulic retention times and G values of the coagulation and flocculation tank at the different pilot plant configurations

Time scale	Coagulation			Flocculation		
October 2010		100 %	2.2 m^3 4-13 min 70 s^{-1}		100 %	2.0 m^3 4-12 min 80 s^{-1}
June 2011		66 %	1.4 m^3 3-9 min 90 s^{-1}		100 %	2.0 m^3 4-12 min 80 s^{-1}
November 2011		Turbomix™ 26 %	0.6 m^3 1-3 min 120-380 s^{-1}		100 %	2.0 m^3 4-12 min 150 s^{-1}

3.2.3 Process control

For the first 8 months the microsieve pilot unit was operated at a constant flow rate of 10 or 20 m^3/h. In May 2011 a fixed daily flow pattern was introduced (see Figure 18) to test the process stability during changing hydraulic conditions. A flow pattern for storm weather was also defined.

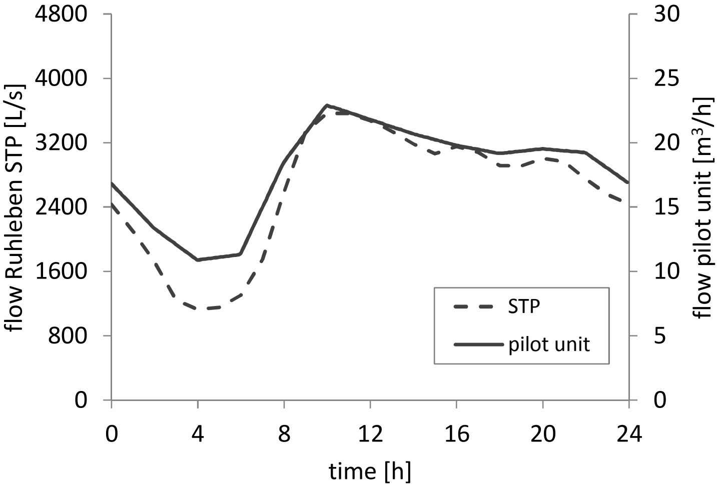


Figure 18: Averaged daily flow pattern of the microsieve pilot trials for dry weather. The averaged WWTP flow pattern was calculated out of the 1h values from 12 days of the year 2010 (one day per month).

The process control of the pilot unit receives the flow signals of the three lines of the WWTP. Thus, in September 2011 it was possible to link the flow of the pilot unit directly to the flow of the WWTP (see Figure 19). Hydraulic conditions of dry and rain weather were likewise reproduced in real time. Due to different process management of the WWTP in winter and summer (bathing season) the pilot unit was either fed with the water from line B or A+C. As the minimum flow of the microsieve pilot unit is 10 m^3/h WWTP flows ≤ 1,050 L/s in winter and ≤ 1,180 L/s in summer were translated with 10 m^3/h.

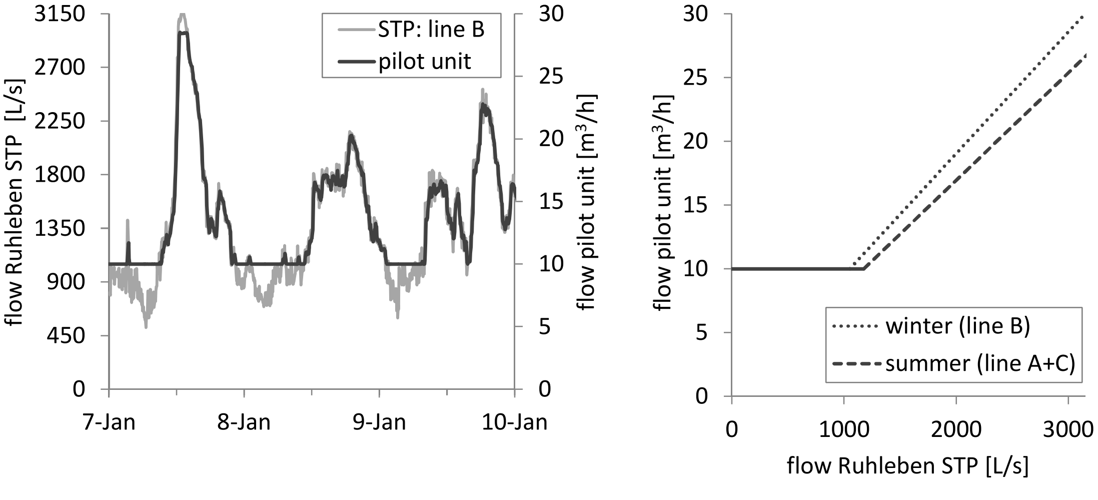


Figure 19: Dynamic operation of the pilot unit and the translation of the WWTP flow to the flow of the pilot unit

From October 2010 to August 2011, the coagulant and the polymer were solely dosed proportionally to the water flow. Afterwards the load proportional dosing of the chemicals was introduced. In a certain range the coagulant was dosed in relation to the ortho phosphate signal. Therefore, a minimum and a maximum dose, a lower ortho phosphate limit and a β factor had to be defined (see Figure 20).

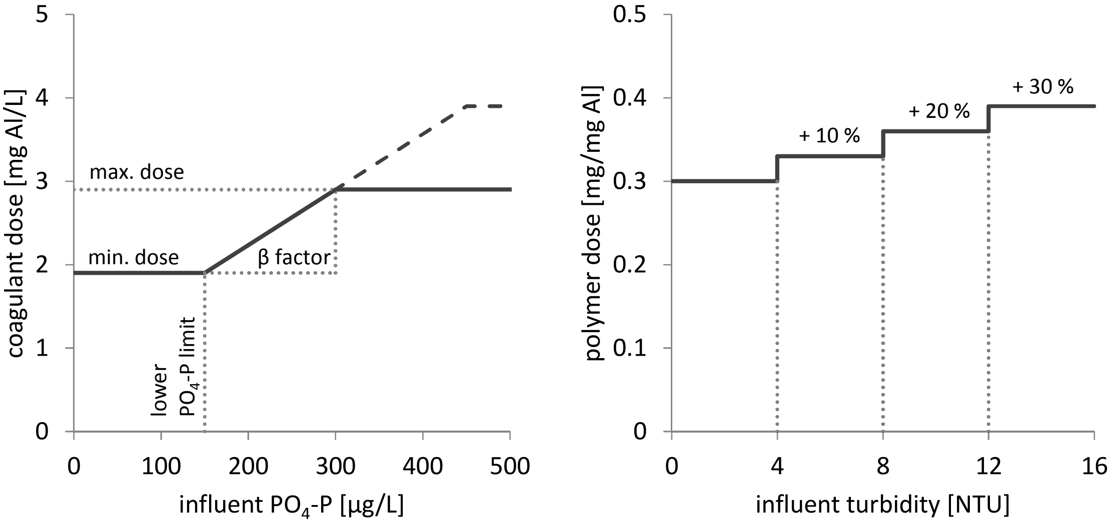


Figure 20: Scheme for load proportional dosing of coagulant and polymer

Because the coagulant dose influences the amount of produced suspended solids the polymer dose was linked to the coagulant dose. E.g. in August and September 2011 0.79 mg polymer per dosed mg of aluminum were applied. After the reconstruction works the factor could be reduced to 0.3. Furthermore, extra 10 - 30 % of polymer was dosed in dependence on the influent turbidity.

Table 10 gives an overview of the different applied process control regimes with regard to the flow and the dosing of chemicals.

Table 10: Overview of the different process control regimes

Time scale	Hydraulics		Chemical dosing		
	Control	Flow [m^3/h]	Control	Coagulant	Polymer
October 2010	Constant flow	10.0 or 20.0	Volume proportional	4 or 5 mg Fe/L 1.9 or 2.4 mg Al/L	2 mg/L (predominant)
May 2011	Fixed daily flow pattern	10.9 - 22.9	Volume proportional	1.9 or 2.4 mg Al/L	2 mg/L (predominant)
August 2011	Fixed daily flow pattern	10.9 - 22.9	Load proportional	1.9 - 2.9 mg Al/L	1.5 - 2.0 mg/L (0.79 mg/mg Al)
September 2011	Dynamic operation	10.0 - 28.6	Load proportional	1.9 - 2.9 mg Al/L	1.5 - 2.0 mg/L (0.79 mg/mg Al)
November 2011	Dynamic operation	10.0 - 28.6	Load proportional	1.9 - 2.9 mg Al/L	0.76 - 1.5 mg/L (0.40 mg/mg Al)
January 2012	Dynamic operation	10.0 - 28.6	Load proportional	1.9 - 2.9 mg Al/L	0.57 – 1.13 (0.30 mg/mg Al)

3.3 UV disinfection unit

During the reconstruction works for the project extension OXERAM 2b a UV disinfection unit was added to the process. A partial flow (6.0 m^3/h) of the microsieve filtrate is conveyed by a radial pump to a closed channel UV reactor (LBX 10, WEDECO). The UV reactor contains three low pressure (LP) lamps which are placed in parallel to the flow direction of the water. The UV system is equipped with an automated wiping system which cleans the quartz sleeves once an hour. In the UV reactor the irradiance is measured online with a sensor and recorded in the microsieve process control. After the disinfection the water is returned to the filtrate tank to avoid running dry (see Figure 21) as the UV disinfection is controlled manually. The flow is regulated with the aid of a valve.

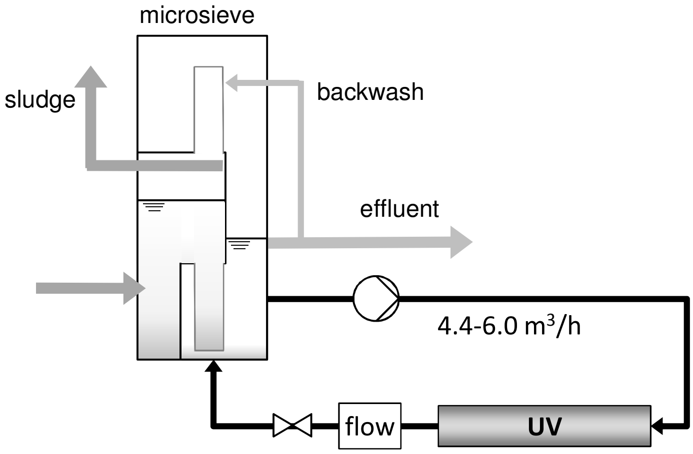


Figure 21: Scheme of the UV disinfection

The UV reactor is designed for a fluence of 1000 J/m^2 at a UV transmittance of 45 % and a flow rate of 2.6 m^3/h. The power output of the lamps cannot be regulated. Hence, the applied fluence can only be changed by varying the flow rate. The flow rate can be measured with a flow meter (range: 2 to 6 m^3/h).

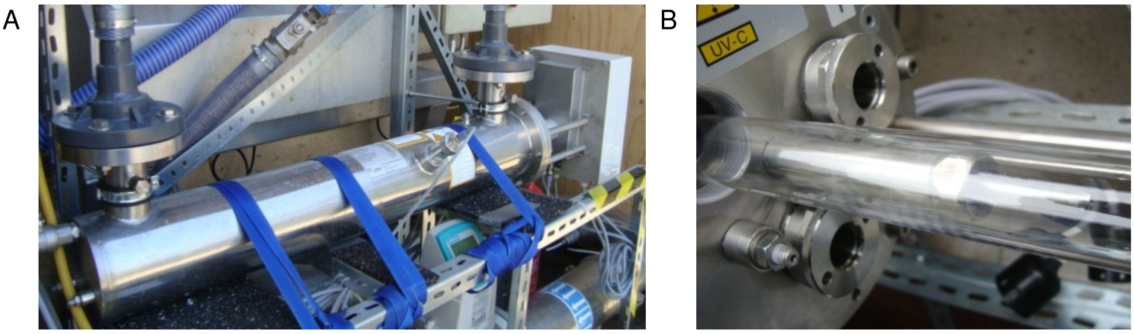
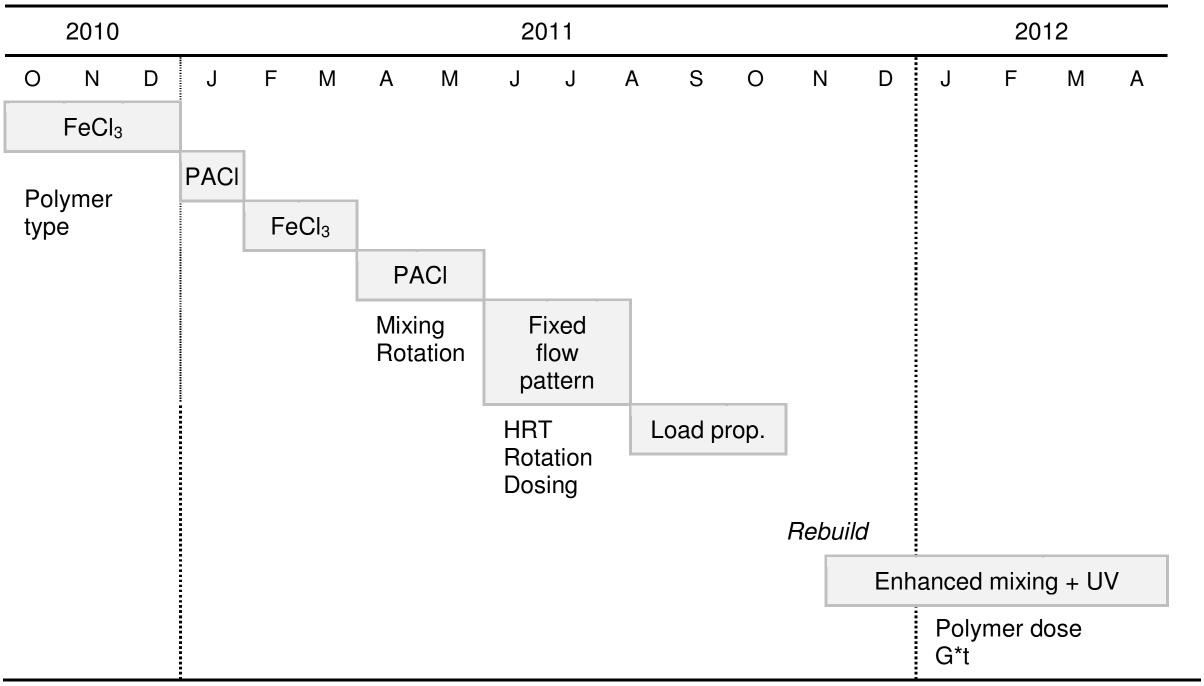


Figure 22: UV reactor (A) and the quartz sleeve (B)

3.4 Trial planning

Table 11 gives an overview of the main microsieve trial phases. After the optimization of the chemical dosing (types and doses) different mixing conditions and hydraulic retention times (HRT) were tested. During the summer of 2011 the microsieve pilot unit could be operated with a dynamic flow rate and load proportional dosing could be introduced. The reconstruction of the pilot unit (as described in Chapter 3.2) and the installation of the UV disinfection were conducted in November 2011. In the trial extension (Oxeram 2b) further optimization of the hydraulic conditions and the chemical dosing were performed.

Table 11: Microsieve trial phases



3.5 Analytical methods

Most parameters were measured in the accredited BWB laboratories (Ruhleben and Jungfernheide). Besides, several parameters were determined on site. For the analysis of phosphorus, phosphate and COD the testing site was provided with a spectrophotometer DR 2800, a heating block Thermostat LT200, both from HACH LANGE, and a small filtration unit. Table 12 presents the methods for the determination of the sum parameters that were applied.

Table 12: Methods for water analysis (sum parameters)

Parameter	BWB laboratories	On site
Chemical oxygen demand (COD)	DIN ISO 15705;	HACH LANGE LCK 414 according to DIN ISO 15705
Dissolved organic carbon (DOC)	DIN EN 1484 (H03)*	
Suspended solids (SS)	DIN EN 872 (H02)*	

* Filtration with glass fiber filter

An overview over the measured physico-chemical parameters, aluminum, iron and phosphorus compounds and their methods of determination are shown in Table 13.

Table 13: Methods for water analysis (other parameters)

Parameter	BWB laboratories	On site
Turbidity		ULTRATURB plus sc online turbidity photometer from HACH LANGE: 0.0001 to 1000 NTU according to DIN EN ISO 7027 (C02); AL250-IR portable turbidity photometer from AQUALYTIC: 0.01 to 1100 NTU
UV absorbance (254 nm)	DIN 38404-C03**	
Transmittance	DIN 38404-C03	
Temperature		DIN 38404-C04
pH	DIN 38404-C05	
Total aluminum / iron	DIN EN ISO 11885 (E22)	
Ortho phosphate	Malachite green method according to Motomizu* for < 30µg/L	HachLange LCK/ LCS 349 according to DIN EN 1189*
Total phosphorus	DIN EN ISO 11885 (E22)	HACH LANGE LCK/ LCS 349 according to DIN EN 1189
Intestinal Enterococcus	DIN ISO 7800-1***	
Escheria Coli	DIN EN ISO 9308-3***	
Coliphage	BWB-08-96	

* Filtration with 0.45 µm cellulose nitrate filter

** Filtration with glass fiber filter

*** The detection limit varies between 15 and 38 MPN/100 mL

Some sludge samples out of the flocculation tank and filtrate pipe were also analyzed according to aluminum or iron content, total phosphorus, dry matter content and volatile suspended solids (see Table 14) to compare the floc composition before and after the microsieve.

Table 14: Methods for sludge analysis

Parameter	BWB laboratories
Aluminum (Al)/ iron (Fe)	DIN EN ISO 11885 (E22)
Total phosphorus (TP)	DIN EN ISO 11885 (E22)
Dry matter content	DIN EN 12880
Volatile suspended solids (VSS)	DIN EN 12879

The considered eco-toxicity tests are displayed in Table 15. The tests were conducted at the Technische Universität Berlin (TU Berlin) at the laboratory of Prof. Dr. Pflugmacher Lima.

Table 15: Eco-toxicity tests

Parameter	Laboratory of Prof. Dr. Pflugmacher Lima (TU Berlin)
Enzyme activity test (glutathione S-transferase) in *C. demersum* (24h)	According to (Habig et al. 1974)
Enzyme activity test (peroxidase) in *C. demersum* (24h)	According to (Bergmeyer 1986)
Ames test	ISO 11350, OECD 471
UMU-Chromo test	DIN 38415 T3, ISO 13829
Acetylcholinesterase (AchE) inhibition test	DIN 38415 T1
Luminescent bacteria test	HACH LANGE LCK 487 according to DIN 38412 T34
Algae growth inhibition test (72h)	OECD 201
Daphnia acute immobilization test	OECD 202
Daphnia magna reproduction test	OECD 211, draft ISO 20665

3.6 Applied chemicals

In this work only ferric chloride and polyaluminum chloride were used (see Table 16).

Table 16: Product data of applied coagulants

Coagulant	Ferric chloride	Polyaluminum chloride
Supplier	Brenntag	Brenntag
Concentration [%]	40	40
Metal content [%]	13 (Fe)	6 (Al)
Density [g/cm^3] at 20°C	1.42	1.2 to 1.3

Due to the multitude of available polymer characteristics and the manifold demands and influences of the application prior tests in lab scale are essential. Preliminary jar tests identified the polymers with high cationicity and high molecular weight as the most effective. The polymers applied in the jar tests and the pilot plant are listed in Table 17.

Table 17: Product data of applied polymers

Polymer	C1	C2	C3	C4	A1
Supplier	PolyChemie	Kemira	Kemira	GE	Kruse
Charge	cationic	cationic	cationic	cationic	anionic
Charge density	70 %	60 %	60 %	60 %	15 %
Molecular weight	HMW	HMW	HMW	HMW	HMW
Form	solid	solid	solid	liquid	solid

3.7 Calculations

3.7.1 Filtration velocity

For the jar test the filtration velocity F is determined out of the filtrate volume produced within the first 10 sec and calculated with equation (6). According to the test set-up the filter area is totally submerged.

$$F = \frac{V}{t \cdot A_{sieve}} \tag{6}$$

V Filtrate [m^3]

t Filtration time [sec]

A_{sieve} Total filtration area of mini microsieve [m^2]

The filtration velocity of the pilot plant is defined as the filtration velocity through the filter media referring to the submerged filter area. It is assumed that the filter area of the microsieve is submerged to 50 %. The filterability is calculated with equation (7).

$$F = \frac{\dot{V}}{0.5 \cdot A_{sieve}} \tag{7}$$

$\dot{V}$ Flow rate [m^3/h]

A_{sieve} Total filtration area of microsieve [m^2]

The filtration velocity in jar test and pilot plant are not directly comparable in the way they are determined here. The effluent flow of the pilot plant is measured every 2 seconds at continuous filtration. In the jar test only the average flow within the first 10 seconds of each batch test is determined. Furthermore, the miniature microsieves used in jar tests are chemically cleaned before each test. Thus, the filtration velocities determined via jar test are much higher than those determined in the pilot plant.

3.7.2 Backwash time

The backwash time percentage is a counter value and determined every 5 minutes (see equation (8)) and is a useful indicator of the quality of the chemical pre-treatment.

$$BW\ time = \frac{t_{BW}}{300\ sec} \tag{8}$$

$BW\ time$ backwash time [%]

t_{BW} operating time of the backwash pump within 300 sec [sec]

The objective is to avoid a continuous backwash (= higher energy demand and backwash water production) by creating shear resistant and easily removable flocs. The pilot plant was designed assuming there is 100 % backwash time at 30 m^3/h and accordingly 33 % at 10 m^3/h for the given influent water quality.

As the backwash time is depending on the flow rate the specific backwash time was defined. It is the backwash time percentage related to the influent flow. The specific backwash time allows the comparison of different operating conditions during dynamic operation and thus, varying flow rates.

3.7.3 G value

The G value before and after the rebuild was measured for the flocculation and coagulation stirrer. Figure 23 shows the experimental set-up. The applied force at a certain lever arm on the central shaft of the stirrers was determined with a force meter (Lutron Force Gauge FG-5005). The G value was then calculated with equation (9). With the measured G values the impeller power number NP was calculated with equation (10).

Figure 23: Experimental set-up for the G value measurement

$$G = \sqrt{\frac{P_{water}}{V \cdot \eta}} = \sqrt{\frac{M \cdot \eta \cdot \pi}{30 \cdot V \cdot \eta}} = \sqrt{\frac{f \cdot a \cdot \pi}{30 \cdot V}} \tag{9}$$

P_{water}	power input into water [W]
V	volume of the water [m³]
η	dynamic viscosity [kg/(m·s)]
M	torque moment [Nm]
f	applied force [N]
a	lever arm [m]

$$N_P = \frac{P_{water}}{\rho \cdot \left(\frac{\eta}{60}\right)^3 \cdot d^5} \tag{10}$$

N_P	impeller power number [-]
ρ	water density [g/cm^3]
d	diameter of the propeller [m]

3.7.4 Specific energy demand

The specific energy demand of the flocculation and coagulation stirrer was calculated with equation (11).

$$ED = G\ value^2 \cdot \eta \cdot t \tag{11}$$

ED	Energy demand [Wh/m³]
t	retention time [s]
η	dynamic viscosity [kg/m·s]

3.7.5 Fluence

The fluence is estimated with equation (12). In this equation the theoretical calculated parameters with the MPSS method are set in correlation with the measured parameters. Furthermore, a lamp aging factor (CA) and a geometry factor (CG) are considered.

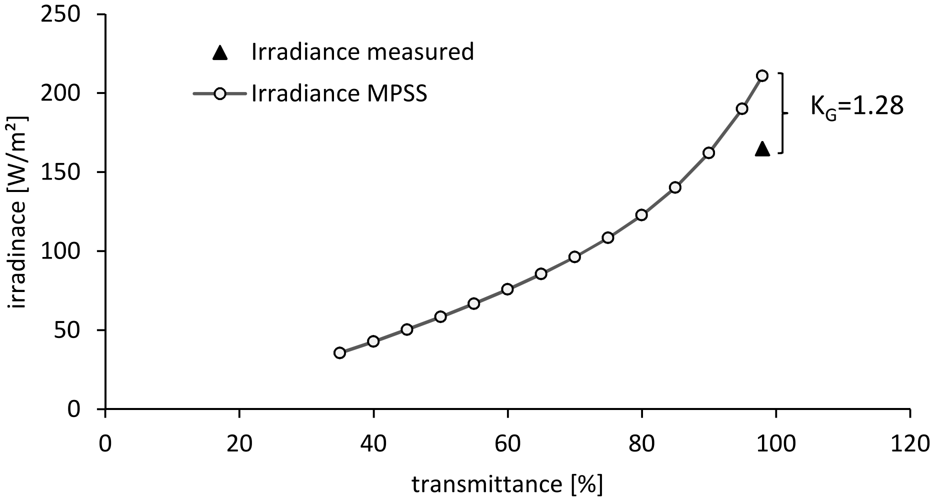


Figure 24: Geometry factor

The lamp aging factor is calculated with equation (13). The decrease of the lamp output is regarded as linear as the lamp degradation of new UV lamps proceeds linearly at the beginning (see Figure 13). The geometry factor is set to 1.28. It takes into account the deviation from the theoretically calculated irradiance and the irradiance measured right at the beginning of the UV reactor operation (see Figure 24).

$$H_0 = \frac{I_{measured}}{I_{MPSS}} \cdot \frac{H_{MPSS} \cdot \dot{V}_{MPSS}}{\dot{V}_{measured}} \cdot C_A \cdot C_G \qquad (12)$$

$I_{measured}$	Measured irradiance in the UV reactor[W/m²]
I_{MPSS}	Calculated irradiance with the MPSS method [W/m²]
H_0	Fluence [J/m²]
H_{MPSS}	Fluence calculated with the MPSS method [J/m²]
$\dot{V}_{MPSS}$	Flow rate calculated with the MPSS method [m³/s]
$\dot{V}_{measured}$	Measured flow rate in the UV reactor [m³/s]

$$C_A = 1 - \left(\frac{h}{8750 \cdot 0.3} \right) \qquad (13)$$

C_A	Aging factor [-]
h	Time of operation of UV lamps [h]

Chapter 4

Results and discussion

4.1 First results and challenges

In the first two months of operation there was a wide variation of the dosed chemical concentrations and polymer types. Mostly the cationic powder polymer C1 was applied. For short periods two other cationic polymers (C2 and C3) and one anionic (A1) polymer were tested. All the time ferric chloride was applied. After the installation of the second filter disc (19.10.2010) the flow was increased from 10 to 20 m^3/h. The average effluent TP concentration amounted to 60 µg/L (see Figure 25). 26 out of 35 samples contained less than 80 µg /L of total phosphorus. Due to high influent concentrations, the TP removal was high (86 %). The first two months also showed good removal of COD (36 %) and SS (34 %). There were 0.49 up to 2.70 mg/L residual iron in the effluent, which means that partially over 60 % of the dosed iron was found in the effluent. It should be noted, that extreme total iron values above 2 mg/L are caused by polymer under-dosing (< 1 mg/L polymer). At 20 m^3/h good backwash times below 66 % were achieved. The backwash time was in the range of 40 to 65 % with a mean value of 50 %.

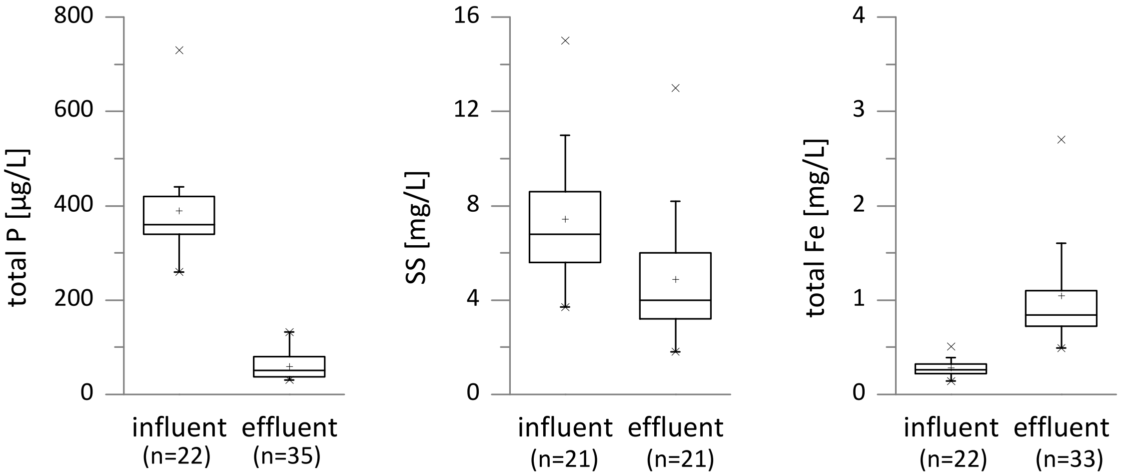


Figure 25: First pilot plant results (October/ November 2010)

Although there was twice as much TP in the influent compared to May 2009, effluent TP values < 80 µg/L could be achieved. The SS content in the effluent (3 to 6 mg/L) was slightly higher than observed in the preliminary tests (2.5 to 4 mg/L). This was probably due to higher influent concentrations. On average there were 3 mg/L more SS compared to May 2009. During the first two months of operation the mean total iron content in the effluent was similar high (around 1.0 mg/L Fe), but the measured values were less widely spread.

The effluent water quality achieved in the first two months of pilot plant operation was very similar to the results of the preliminary test. But operating the pilot plant continuously over several weeks instead of only few hours, revealed a lot more insights into the technology. The high iron concentrations in the effluent were not only analytical determined but also visible as orange iron deposits in the filtrate tank and the filtrate pipe (see Figure 26). After only two weeks of operation the inner surface of the filtrate pipe was completely covered with a thick layer.

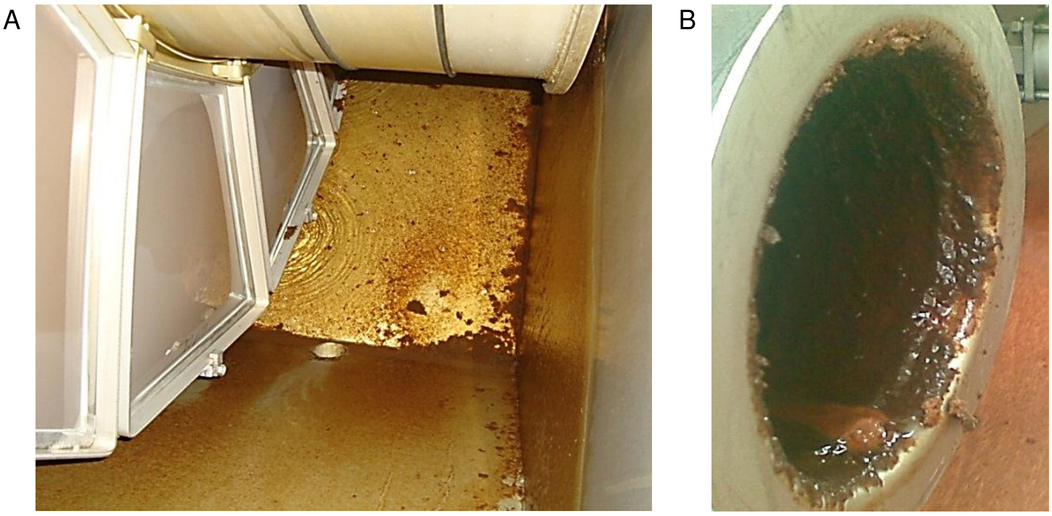


Figure 26: Iron deposits in filtrate tank (A) and filtrate pipe (B) in October 2010

The deposits in the filtrate pipe were removed and analyzed. Additionally sludge from the flocculation tank was collected and likewise analyzed in order to compare the composition of the flocs created during the chemical pre-treatment to the flocs formed after the filtration. The results are given in Table 18. The contents of Fe, TP and volatile suspended solids (VSS) were very similar.

Table 18: Sludge composition in the flocculation tank and filtrate pipe when $FeCl_3$ were applied - Contents of total iron (Fe), total phosphorus (TP) and volatile suspended solids (VSS)

Sludge from	Fe [mg/kg]	TP [mg/kg]	VSS [%]
Flocculation tank (Fe)	280	14	50
Filtrate pipe (Fe)	260	20	51

After this discovery post-flocculation tests were set up and regularly conducted, in order to survey the post-floc formation potential. In 13 of 15 tests a floc formation was observed. As shown in Figure 15, the flocs were mostly found on the water surface. Although there is no monitoring value about discharging iron for communal wastewater treatment in Berlin, the occurrence of such iron deposits needs to be reduced or avoided entirely. Otherwise a downstream UV disinfection will not be applicable.

Summing up, the first two months of operation confirmed the results of the preliminary tests. As the phosphorus removal proved to be easy to perform with $FeCl_3$ attention had to be turned to the reduction of coagulant residues in the effluent. The following chapters deal with the parameters influencing the effluent water quality and how the optimization efforts resulted in improved SS removal (2 mg/L SS average effluent content), improved utilization of the dosed coagulant and polymer (dose reduced by 63 %) and a reduction of the overall footprint (volume for coagulation reduced by 74 %).

- First results already confirmed the general process performance observed during the pre-trials in 2009
- Process optimization necessary with regard to the removal of phosphorus and suspended solids and the reduction of coagulant residues

4.2 Influence of the influent water quality

4.2.1 Phosphorus

During the microsieve trials the average concentration of TP in the influent of the pilot unit amounted to 323 µg/L (see Table 2). The TP values varied between 110 and 1400 µg/L. Extreme TP values (above 1 mg/L) were caused by storm weather or sludge drift from the secondary clarifiers. A variation of the TP in the range between 200 and 500 µg/L is probably due to seasonal changes. Figure 27A gives an overview of the TP variation for the years 2010/2011 and 2011/2012. During the changeovers from winter to summer and summer to winter an increase of the TP was observed. Generally, in the winter period the TP concentration (Oct to Mar: 375 µg/L) in the influent of the pilot unit was higher than in the summer period (Apr to Sep: 283 µg/L). As Figure 27B shows, the seasonal variation of TP is primarily caused by the particulate P fraction. The soluble reactive P (srP) varied without correlation to the season (see Figure 27C) and averaged to 67 µg/L.

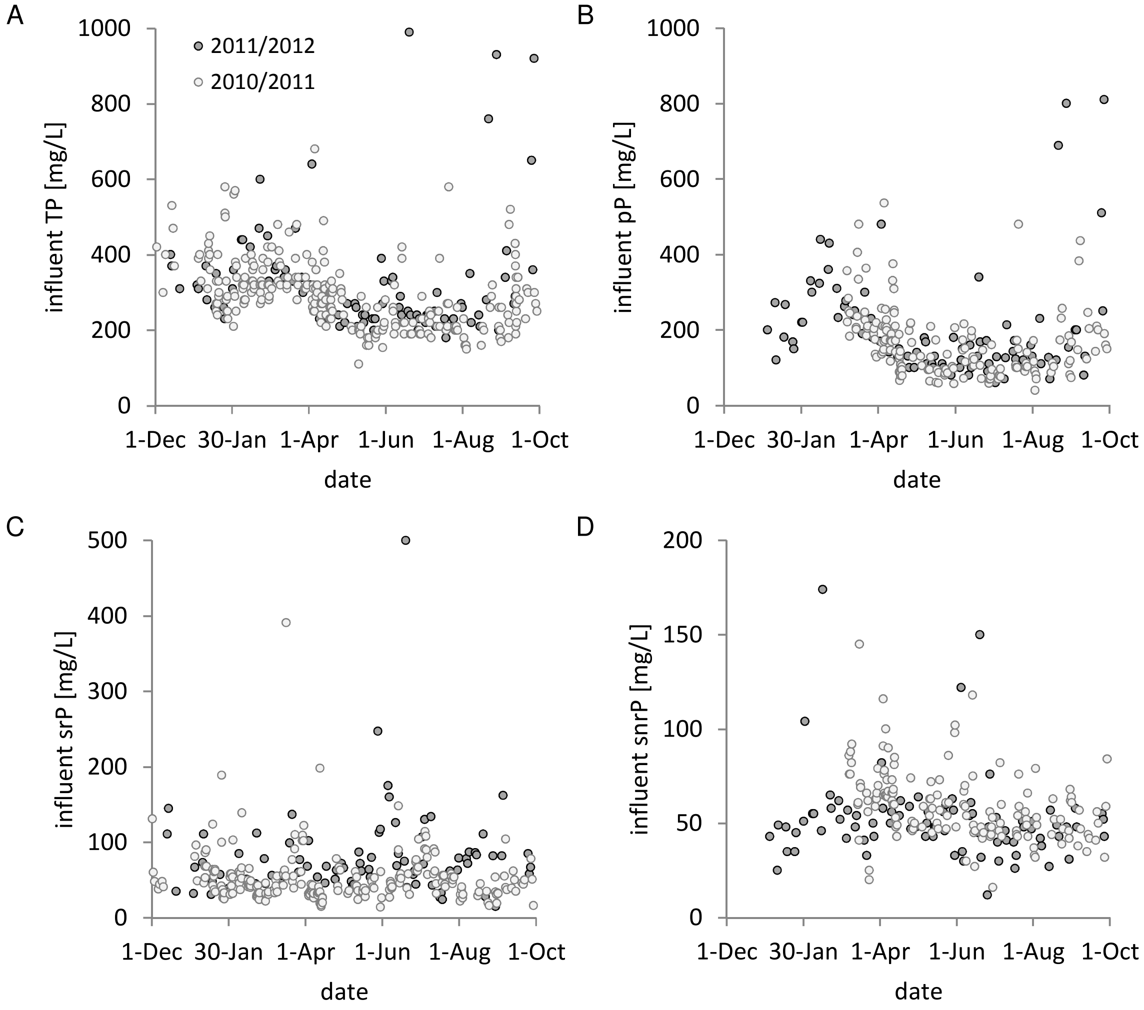


Figure 27: Seasonal variation of the influent phosphorus concentration: total phosphorus (A, n=382), particulate phosphorus (B, n=248), soluble reactive phosphorus/ortho P (C, n=338) and soluble non-reactive phosphorus (D, n=236). Grab samples.

The variation of the soluble non-reactive P (snrP) is not clearly a result of seasonal changes (see Figure 27D). On average, there were 57 µg/L snrP in the influent of the pilot unit. The snrP is the only P fraction which shows a direct correlation to the overall TP removal. The effluent TP concentration of the microsieve pilot unit increases with increasing amounts of snrP in the influent (see Figure 28A). Effluent TP values below 40 µg/L were only achieved when there were less than 50 µg/L of snrP in the influent. With an influent snrP concentration about 40 µg/L the TP removal rate averaged 82 % (see Figure 28B). When the snrP concentration was higher than 60 µg/L the TP removal rate only amounted to 73 %. The average removal of the snrP was 38 % (n=9).

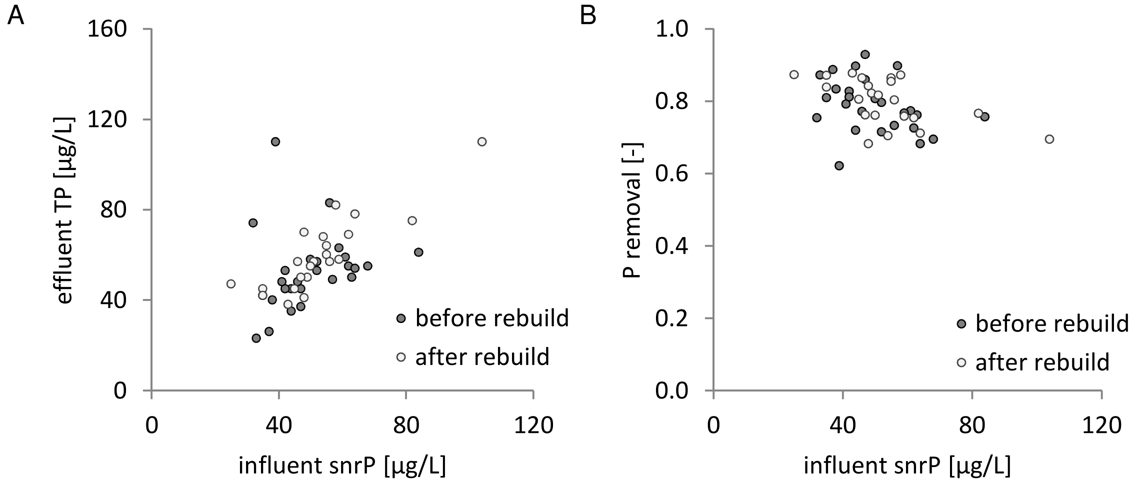


Figure 28: Effluent TP concentration (A) and TP removal (B) as a function of the influent content of the soluble non-reactive phosphorus during load proportional dosing before (n=26) and after (n=24) the rebuild. Grab samples.

During the microsieve pilot trials it was observed that the different P fractions rather than the amount of TP influence the P removal. Especially the amount of snrP in the influent of the pilot unit affects the TP elimination as the snrP can only poorly be removed through the treatment.

4.2.2 Suspended solids

Like the influent concentration of TP also the amount of SS in the influent shows a clear tendency towards seasonal variation (see Figure 29A). On average there were 6.6 mg/L SS in the winter period and 4.8 mg/L in the summer period.

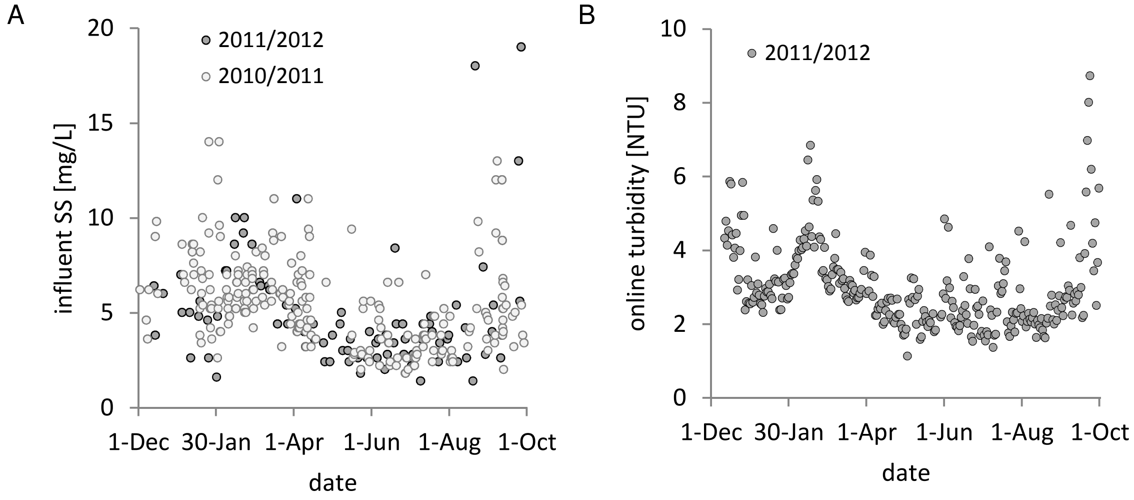


Figure 29: Seasonal variation of the influent SS concentration (A, grab samples) and the influent turbidity (B, online 1d mean values)

Even more clearly visible were the seasonal changes of the particulate water compounds in the online measurement of the influent turbidity. Figure 29B gives the daily mean turbidity values of the year 2011/2012.

During the trials the hypothesis arose that very high (> 10 mg/L) as well as very low (< 3 mg/L) influent concentrations of suspended solids negatively influence the coagulation and flocculation processes. An example is the recording of the diurnal water quality variation in September 2011 (see Figure 37). The highest SS effluent values were observed at the minimum and the maximum SS influent concentrations. Thus, in Figure 30A the effluent water quality of the microsieve is displayed as a function of the influent SS concentration. Neither for the data collected before the rebuild nor for those collected after the rebuild there is a clear tendency visible that would support the stated hypothesis.

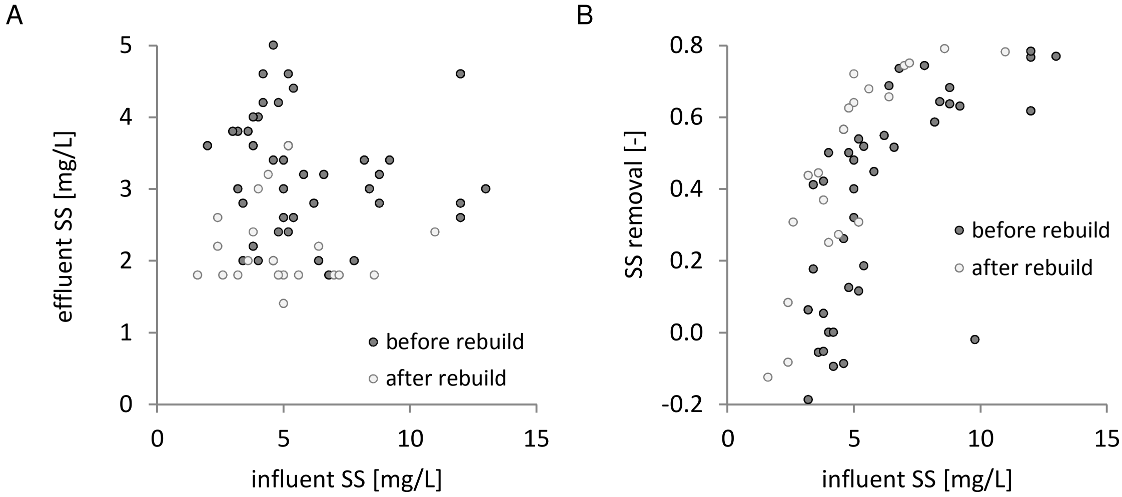


Figure 30: Impact of the influent concentration of suspended solids (SS) on the effluent water quality (A) and the suspended solids removal (B) for the trial phases with load proportional dosing of PACl before and after the rebuild. Grab samples.

The removal rate of SS (related to the influent concentration of the chemical pre-treatment) on the other hand shows a clear correlation to the influent SS amount. Before as well as after the rebuild of the pilot unit the SS removal showed a logarithmic relation to the influent concentration resulting partly in negative removal rates for influent SS concentrations below 5 mg/L. For high influent SS concentrations (8-14 mg/L) the removal rate seemed to converge towards 70 % (before rebuild) or rather 80 % after the rebuild.

4.2.3 Dissolved organic carbon

The influent concentration of dissolved organic carbon (DOC) was very quickly identified as important parameter influencing the coagulation and the coagulant consumption at Ruhleben WWTP. Figure 31 gives the effluent concentration of TP and coagulant residues (Fe, Al) in dependence on the ratio of the coagulant dose and the influent DOC content. For the trials presented in Figure 31A the pilot unit was operated with 0.036 to 0.179 mmol/L of Fe or Al, 1.5 mg/L of the cationic polymer C1 and a constant flow rate of 20 m^3/h (also see Chapter 4.3.1). During the trial phase there were 11.2 to 16.9 mg/L DOC in the influent water with an average value of 13.6 mg/L. For both $FeCl_3$ and PACl the effluent TP decreased with an increased Me to DOC ratio. At a ratio of 0.014

mmol/mg there were only 30 to 50 µg/L TP. No significant impact on the amount of Al and Fe in the effluent could be observed.

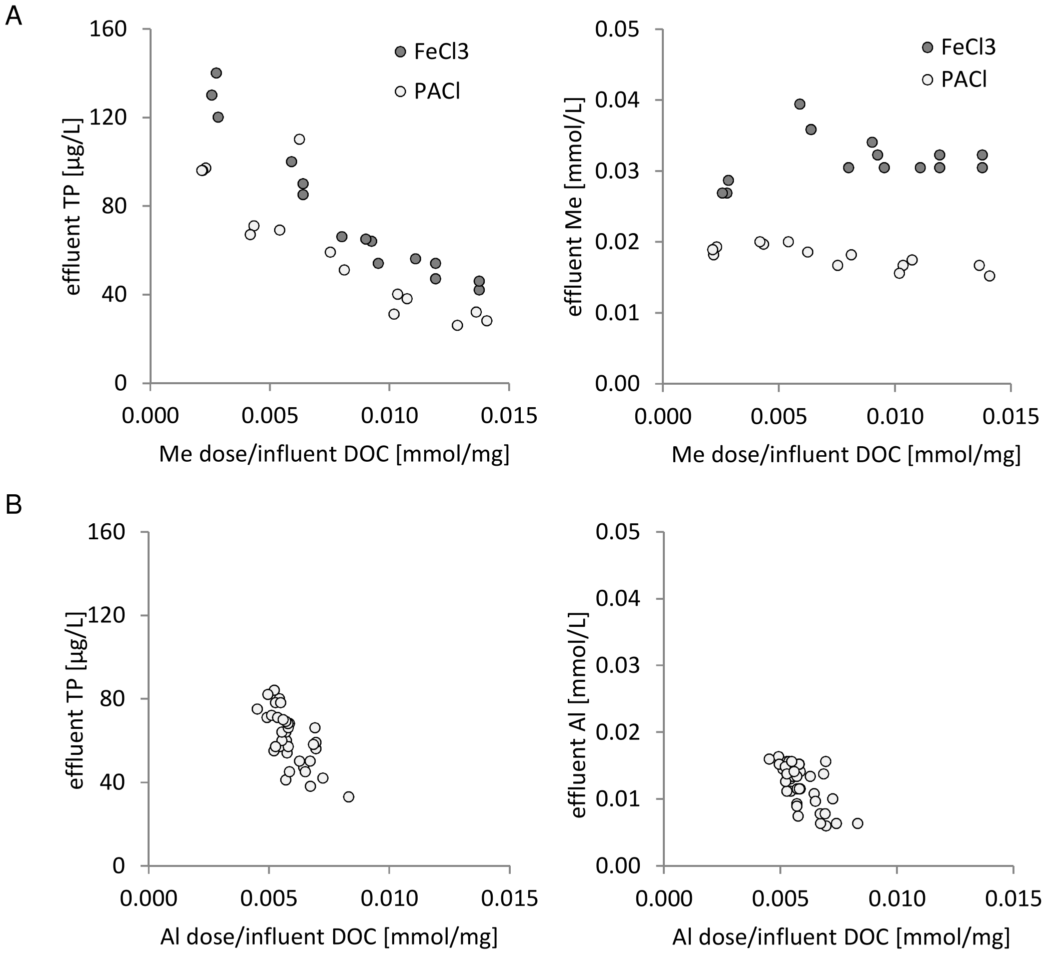


Figure 31: Effluent amounts of total phosphorus and coagulant residues (Fe and Al) related to the ratio of the coagulant dose and the influent DOC concentration during a comparative trial with $FeCl_3$ and PACl (A) and after the reconstruction of the pilot unit (B, load prop. dosing of PACl). April 2011

Figure 31B presents the data collected after the rebuild of the pilot unit. The microsieve was operated with real-time flow variation and load proportional dosing of PACl (0.070-0.107 mmol Al/L) and cationic polymer C2 (0.57-1.13 mg/L). The influent DOC concentration was in a range between 8.9 and 15.5 mg/L and averaged 12.4 mg/L. Although the dosing range was much tighter than in the example A the influence of the DOC concentration on the phosphorus removal is clearly visible. The lowest TP effluent concentration (33 µg/L) was achieved at the highest Al/DOC ratio (0.008 mmol/mg). The data from example B also indicate an influence of the Al/DOC ratio on the amount of Al residues. A lower ratio resulted in higher Al effluent values (> 0.01 mmol/L). Most probably the impact of the DOC on coagulant residues could not be observed in example A due to the non-optimized mixing conditions in the coagulation tank in comparison to example B.

4.2.4 Transmittance

The transmittance of UV radiation is a crucial parameter for the design of UV disinfection plants. Figure 32A gives the seasonal variation of the influent transmittance. In 2011, the transmittance was mainly determined during the non-bathing season (between October and April) and averaged 46 % (n=132). In 2012 it appeared that there are slight seasonal changes of the influent transmittance. Off the bathing season the average transmittance amounted to 51 % (n=28) and to 47 % (n=62) during the bathing season.

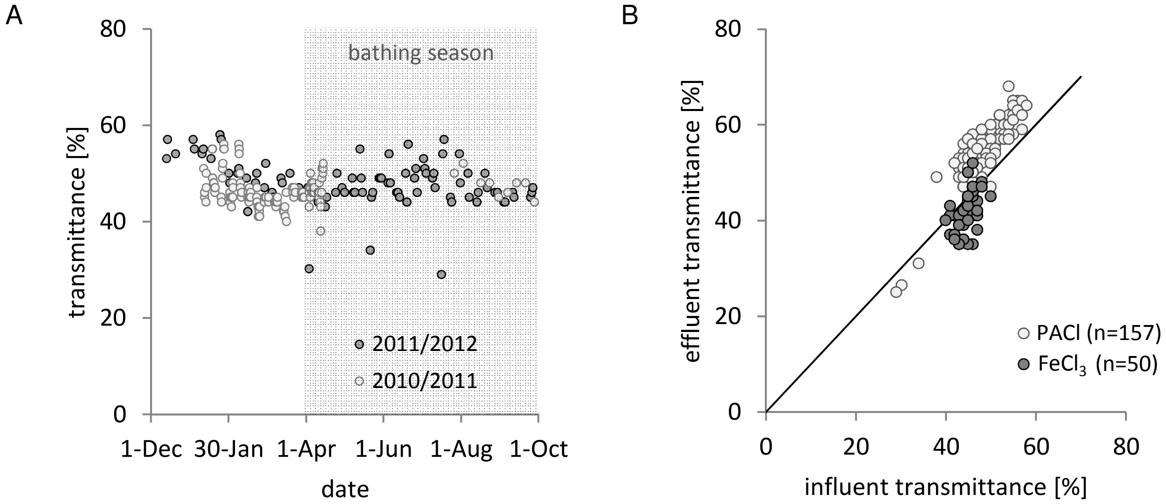


Figure 32: Seasonal variation of the transmittance (A) and the effluent transmittance as a function of the influent transmittance (B, average dosing: 0.07 to 0.09 mmol Me/L)

This is of importance as the value of transmittance after the microsieve filtration is mainly influenced by the initial influent value (when PACl is applied). In Figure 32B the effluent transmittance is displayed as a function of the influent transmittance for the trials with PACl and $FeCl_3$. For the data gained during the dosage of PACl there is a clear correlation between the influent and the effluent transmittance. The relative increase of the transmittance through the treatment averaged 13 %. When $FeCl_3$ was applied the transmittance observed in the microsive effluent was frequently unchanged or even decreased in comparison to the influent value. On average the transmittance was decreased by 6 % (relative) through the treatment when $FeCl_3$ was dosed. The transmittance seemed to be impaired by residues of $FeCl_3$. Thus, no clear correlation of the effluent with the influent transmittance could be observed during the application of $FeCl_3$.

4.2.5 Chemical oxygen demand

The results above indicate that especially the particulate compounds of the influent water seem to vary with seasonal changes. Although only 14 % of the COD is caused by particulate matter the COD also showed seasonal behavior (see Figure 33A) with peak concentrations during the changeover periods between summer and winter. During the summer period the average influent COD value amounted to 38 mg/L and to 42 mg/L in the winter period. The effluent COD concentration seemed to be predominantly dependent on the influent concentration (see Figure 33B). When $FeCl_3$ was dosed the COD removal averaged 23 %. During the application of PACl the COD removal rate was comparatively constant and averaged to 19 %.

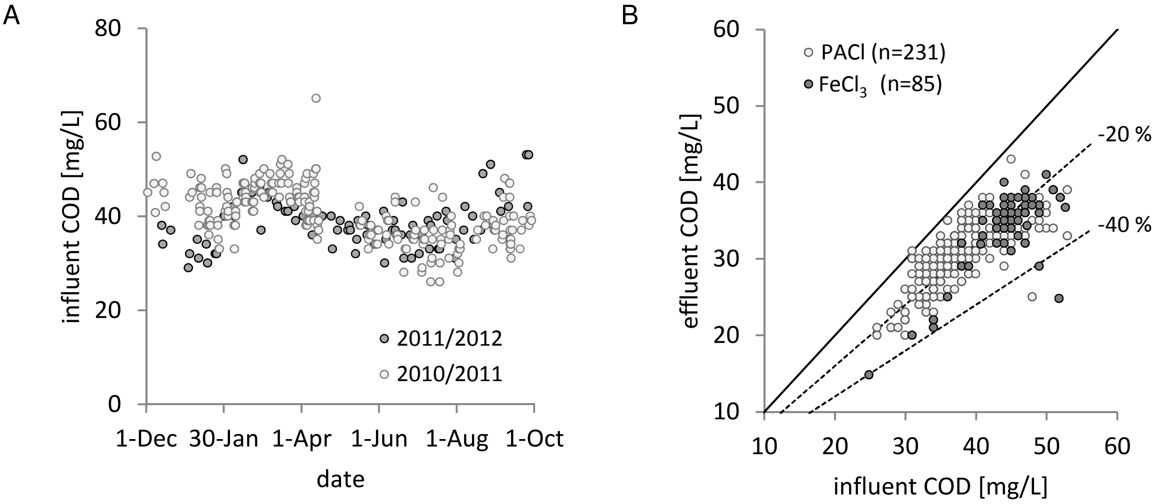


Figure 33: Seasonal variation of the influent COD (A) and the effluent COD as a function of the influent COD (B, average dosing: 0.07 to 0.09 mmol Me/L)

- Especially the particulate compounds of the influent water (secondary effluent from WWTP) show seasonal changes: increase of SS (turbidity) and particulate P during the changeovers from winter to summer and vice versa. Higher amounts of SS and TP during the winter period were observed.
- The results from different trial periods revealed a close relation of the TP removal and the amount of coagulant residues with the concentration of DOC.
- COD removal was comparatively stable (19 to 23 %). Thus, the effluent COD concentration mostly depends on the influent concentration.

4.3 Coagulant type and dose

4.3.1 $FeCl_3$ versus PACl

In the pilot plant an extended test series of two weeks duration with increasing coagulant dosing (0.036 to 0.179 mmol/L metal) was conducted with $FeCl_3$ and PACl. During the whole test series 1.5 mg/L polymer C1 and a flow of 20 m^3/h were applied. With an increasing coagulant dose TP in the effluent clearly decreased (see Figure 34). 0.107 mmol/L Fe were needed to achieve the treatment goal of 80 µg/L. With 0.179 mmol/L Fe the effluent TP amounted to 45 µg/L. With PACl there was generally less TP in the effluent. Already with 0.072 mmol/L Al 80 µg/L TP were under-run. With an Al dose of 0.179 mmol/L TP values between 26 and 32 µg/L could be achieved. On the day of the third sampling with 0.107 mmol/L Al, a peak of SS and TP in the secondary effluent of 22 mg/L and 1.2 mg/L TP respectively caused an effluent TP value of 110 µg/L. This proves the process stability even under high loading conditions.

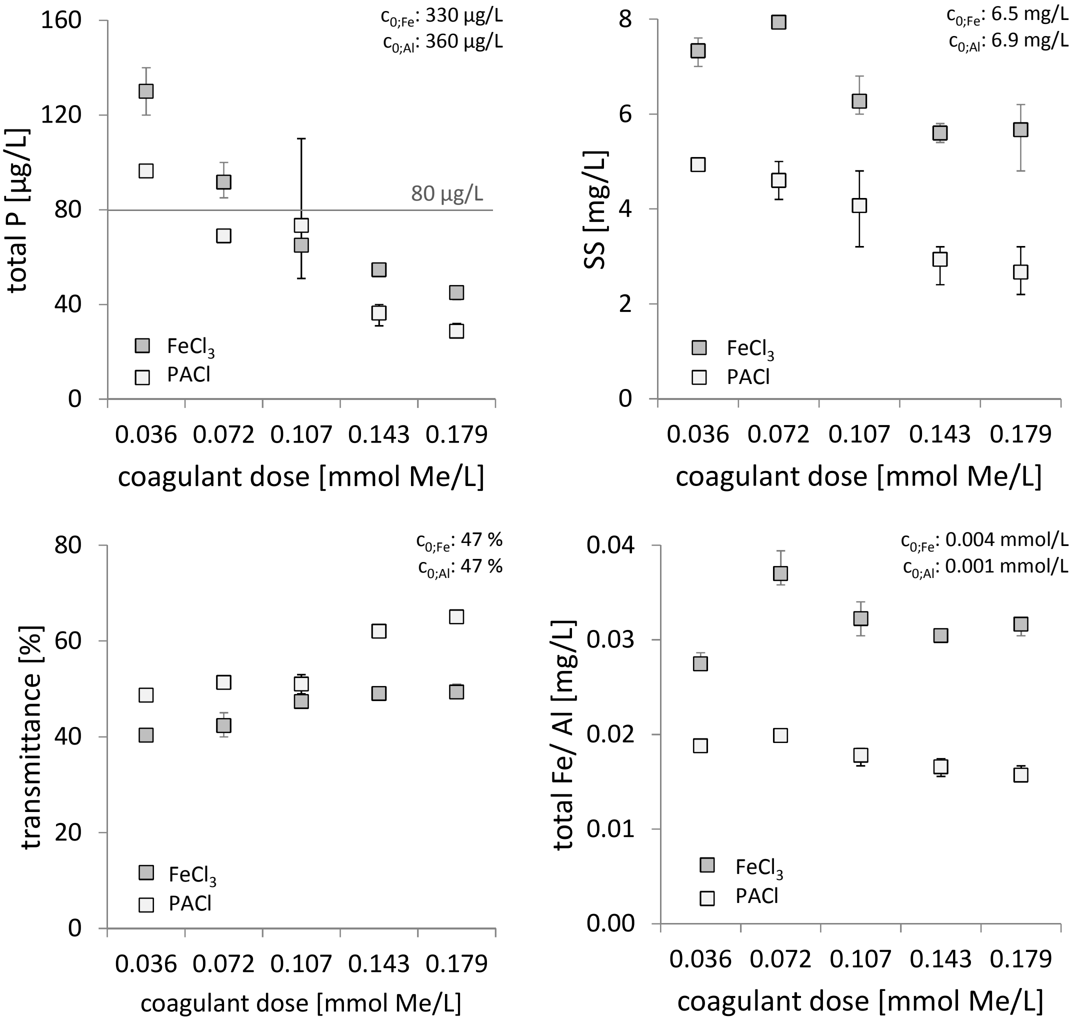


Figure 34: Variation of coagulant type and dose in the pilot plant in April 2011- Results for total P, SS, transmittance and total Fe/Al in the effluent with the average influent concentration c_0. n=3

When $FeCl_3$ was applied the SS content in the effluent was comparatively high. Only with doses of 0.143 to 0.179 mmol/L Fe the effluent SS content under-run the influent content. The highest SS content (8.0 mg/L) appeared with a dosing of 0.072 mmol/L Fe. A reduction of SS in the effluent through increasing the Fe dose could be observed. A decrease of the SS could also be observed while increasing the dose of PACl. Regarding the whole test period there were 2.2 to 5.0 mg/L SS in the effluent.

The average influent transmittance amounted to 47 %, which means less than 50 % of the irradiated energy would not be available for disinfection. With Fe doses of 0.143 and 0.179 mmol/L there was an improvement of the effluent transmittance through the treatment. Below a dose of 0.107 mmol/L Fe there was even a decrease of transmittance. During operation with PACl as coagulant the effluent transmittance was always higher than in the influent.

Due to the high DOC contents of the influent water and their consumption of coagulant, it was expected that higher coagulant doses would result in a lower coagulant breakthrough. At 0.072 to 0.179 mmol/L iron the total iron content in the effluent decreased with increasing iron dosing. But the lowest total iron concentrations in the effluent (0.027 to 0.029 mmol/L iron) were observed at the lowest coagulant dose and all iron values were generally on a very high level. At a dose of 0.072 mmol/L iron up to 0.039 mmol/L iron remained in the effluent water. During dosage of PACl there were less metal coagulant residuals found in the effluent than with ferric chloride. With aluminum doses of 0.036 and 0.072 mmol/L total aluminum in the effluent amounted to 0.018 to 0.020 mmol/L. Through an increase of the coagulant dose the residual aluminum was very slightly reduced to 0.016 to 0.017 mmol/L. Based on mass the difference between $FeCl_3$ and PACl was even more significant. During the test periods there were 1.5 to 2.2 mg/L Fe and 0.4 to 0.5 mg/L Al in the pilot plant effluent. This was reflected in the magnitude of the layer on the filtrate pipe surface formed by residual coagulant (see Figure 35). Whereas with iron there was a thick layer covering the whole surface of the pipe, with aluminum only one third was covered and moreover the layer was less thick.

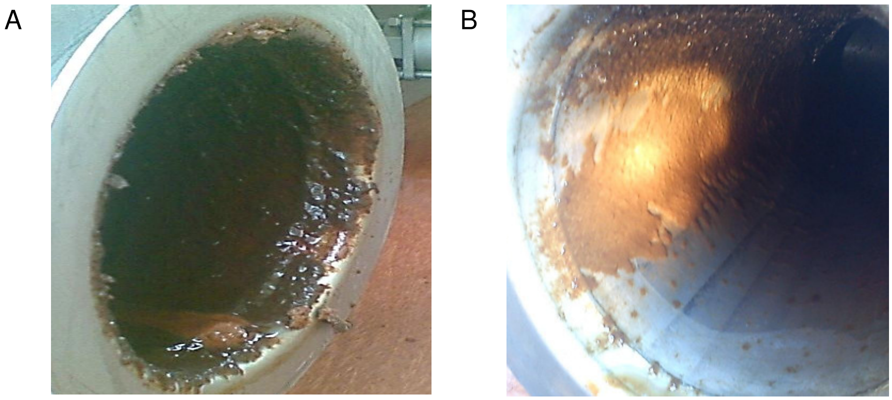


Figure 35: Sludge layer in the filtrate pipe when $FeCl_3$ (A) and PACl (B) were applied

With 0.036 mmol/L Fe the BW time was very low with 20 to 50 % (see Figure 36A). When the Fe dose was increased to 0.072 mmol/L there was an increase in BW time by more than factor two. With doses above 0.107 mmol/L Fe the BW time was mostly in the range of 90 to 100 %. At the lowest Al dose the BW time amounted already to 40 - 50 % (see Figure 36B). With an increase of the Al dose the BW time also increased, but reached the 100 % only with a dose of 0.179 mmol/L Al. Iron (140 mg SS/ mmol Fe) creates more SS than aluminum (108 mg SS/ mmol Al) and therefore creates more load on the system (ATV-DVWK-A 2004). Thus during operation with iron salts the SS load was higher compared to operation with aluminum salts. This resulted in higher BW times, indicating a reduced filtration capacity, and higher effluent values for SS.

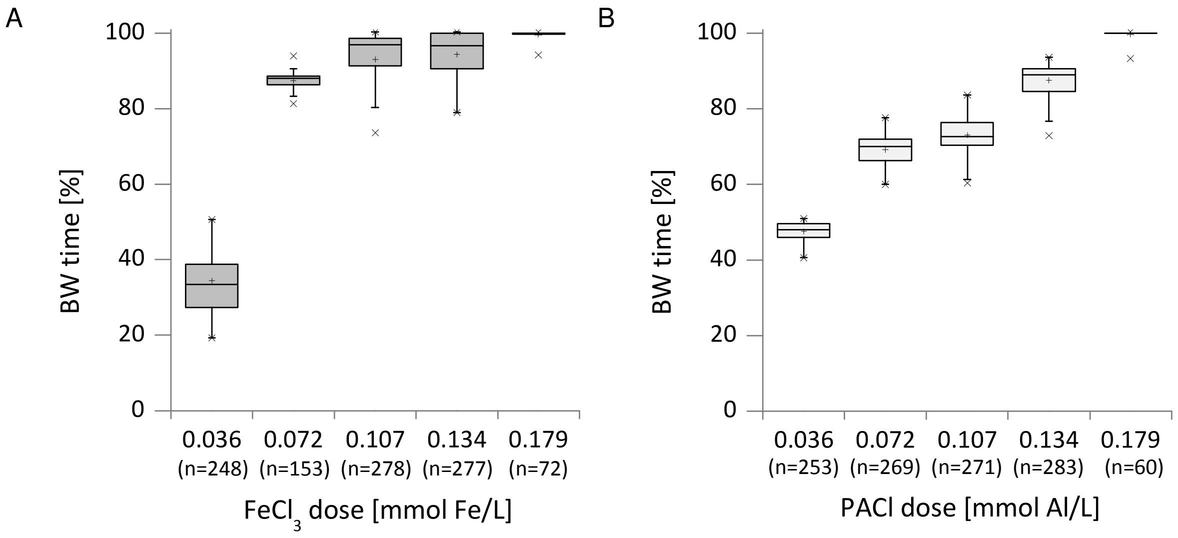


Figure 36: Variation of coagulant type and dose in the pilot plant - Results for backwash time with $FeCl_3$ (A) and PACl (B)

Figure 37 gives the calculated SS load on the microsieve during this test. As iron (140 mg SS/mmol Fe) creates more SS than aluminum (108 mg SS/mmol Al) the microsieve was charged with a higher SS load in comparison to when PACl was dosed (see Chapter 2.3.1). Through the increase of the coagulant dose the ratio precipitates to total SS in the inlet of the microsieve was increased from 50-60 % to 80 %. This resulted in higher backwash times, indicating a reduced filtration capacity.

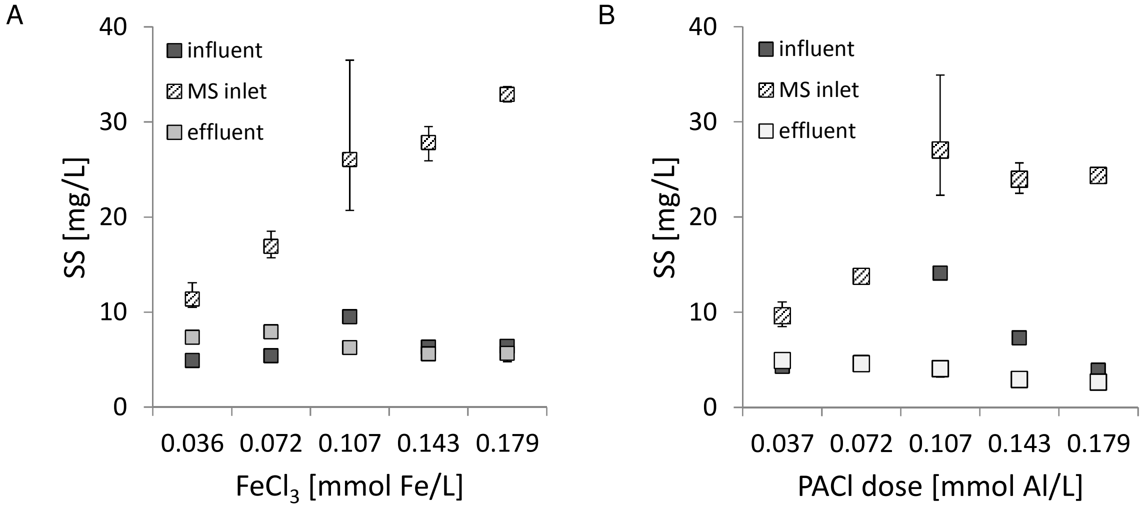


Figure 37: Variation of coagulant type and dose in the pilot plant - Caclulated SS content after coagulation (influent SS + SS created by coagulant) according to (ATV-DVWK-A 2004). n=3

The comparison of the two coagulant types in the pilot plant showed that the performance with PACl was clearly better than with $FeCl_3$. During the test series of both coagulants the influent water qualities were very similar. The pH was in the range of 7.2 to 7.6 and was not decreased through high coagulant dosing. During the dosage of PACl in all concentrations a better removal of total phosphorus and suspended solids was achieved. Furthermore, with PACl an increase of the transmittance, lower amounts of residual coagulant and lower backwash times were attained.

During another trial the ratios of the particulate and dissolved fractions of phosphorus, iron and aluminum in the microsieve filtrate were analyzed. For each FeCl$_3$ and PACl two samples were analyzed. With PACl 40 % less TP was in the filtrate (see Figure 38).

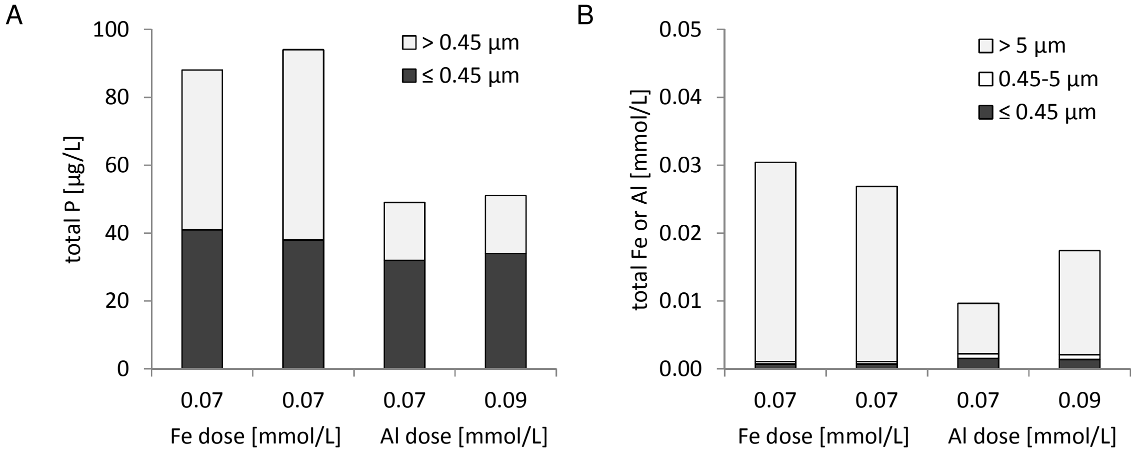


Figure 38: Fractionation of the microsieve filtrate when FeCl$_3$ and PACl were dosed (1.5 mg/L polymer; 20 m^3/h) and analysis of total P (A) and total Fe or Al (B).

The amount of dissolved phosphorus (30-40 µg/L) was similar for all four samples. Thus, the lower TP values achieved with PACl were due to a superior removal of the particulate phosphorus fraction in comparison to FeCl$_3$. The secondary effluent of the Ruhleben WWTP contains 60 to 70 µg/L of soluble non-reactive phosphorus. Approx. 40 % of this phosphorus fraction can be removed by coagulation and filtration for the applied dosing range of coagulant (Miehe 2010). Thus, about 40 µg/L dissolved phosphorus remain in the water presenting a lower limit for conventional filtration technologies. When dosing PACl there was also significantly less coagulant breakthrough in comparison to when FeCl3 was dosed (see Figure 3B). Nearly the whole amounts Fe and Al were found in the size fraction > 5 µm. Only 0.001 mmol/L of the iron and 0.001-0.002 mmol of the aluminum were dissolved.

Flocs formed during the chemical pre-treatment are exposed to high shear forces when approaching the pores of the microsieve. Consequently, flocs can break and the fragments < 10 µm will most probably not be retained by the microsieve. Due to the thinness of the filter media a microsieve offers less chances for a particle to be captured compared to a dual media filter. Thus, particles > 5 µm containing phosphorus and coagulant occur in the microsieve filtrate.

4.3.2 Load proportional dosing

In August 2011 a 24h trial with load proportional dosing of PACl (1.9 to 2.9 mg/L Al) and polymer (1.5 to 2.0 mg/L) was conducted in the pilot plant. The dosage of PACl was linked to the online PO$_4$-P measurement whereas the polymer was applied in a ratio of 0.78 to PACl. Additional polymer was dosed at high influent turbidity values (> 6 NTU).

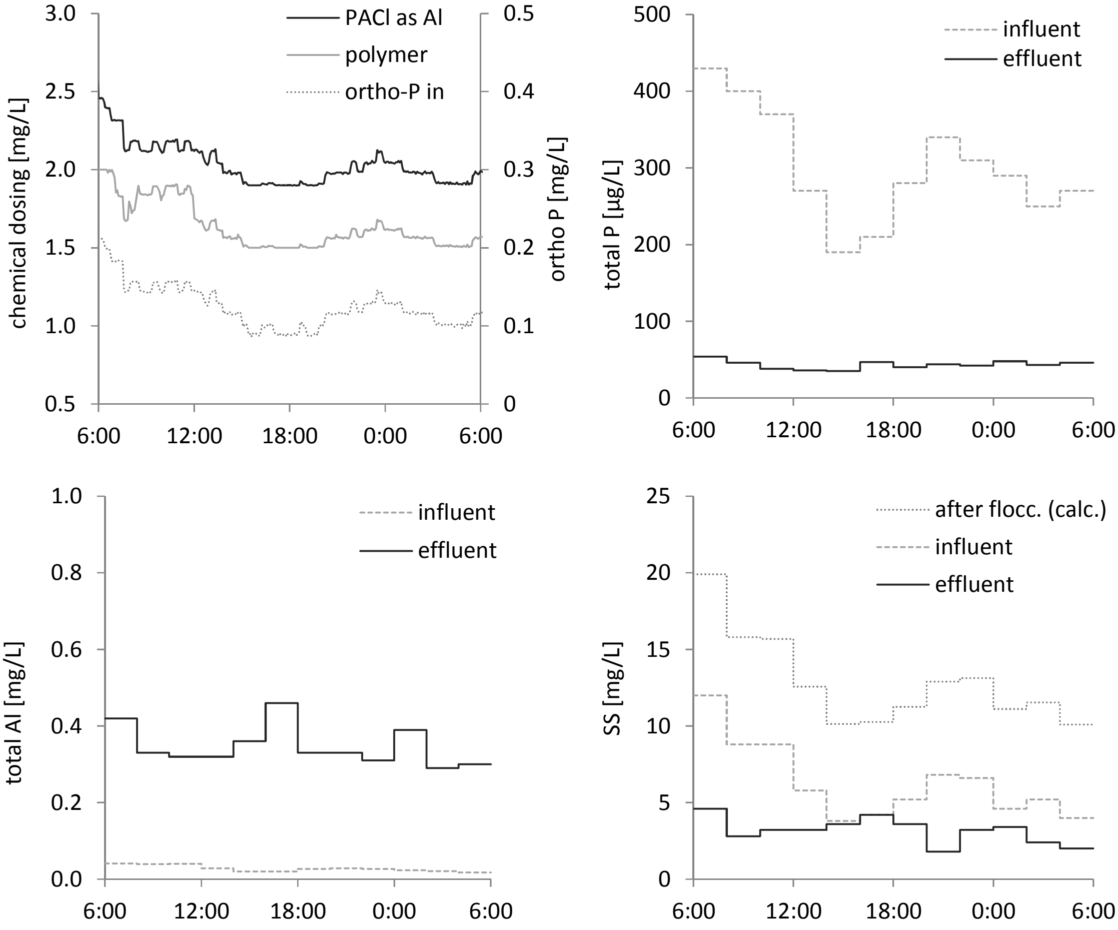


Figure 39: Diurnal variation of the water quality during load proportional dosing (12.09.11). Operating conditions (dosing range: 1.9-2.9 mg/L Al and 1.5-2.0 mg/L polymer).

Figure 39 gives the variation of diurnal water quality for total P, total Al and SS and the corresponding doses of PACl and polymer. 2 h mixed samples were taken during 24 h. It should be noted that during the previous night there was a peak load of turbidity (15 NTU) and PO_4-P (280 µg/L; normally about 100 µg/L). Thus, high concentrations in the first 2 to 3 samples were caused by the peak load. The effluent content of TP was only slightly influenced by the variation of the influent content and on a very low level around 50 µg/L. The variation of the residual Al in the effluent holds three peaks.The first peak can be ascribed to the nightly peak load. The other two Al peaks were accompanied by very low SS influent contents and thus, could be caused by poor floc formation. Initially the high influent contents of SS caused increased effluent contents. From 8:00 h on, the effluent SS content develops contrary to the influent SS content. A decrease of the SS in the influent resulted in a slight increase of SS in the effluent. At SS influent contents below 4 mg/L there is no SS removal when comparing influent and effluent contents. The dotted curve presents the SS load on the microsieve including the estimated SS formed by precipitation (4 g SS per g of dosed Al in accordance to (ATV-DVWK-A 2004)). The actual SS removal through the filtration step amounts to 59-86 %.

Volume proportional dosing was conducted over 8 weeks with 2.4 mg/L Al and 2.0 mg/L polymer. During load proportional dosing on average only 2.1 mg/L Al and 1.65 mg/L polymer were consumed. Thus, 13 % of coagulant and 18 % of polymer could be saved. During both testing periods very good phosphorus removal was achieved. With volume proportional dosing TP in the effluent averaged 41 µg/L (see Figure 40). Applying load proportional dosing there were on average 50 µg/L of TP in the effluent. The increased TP effluent concentration probably was related to the higher influent TP concentration (on average + 80 µg/L) and the reduced coagulant dose. However, sufficient phosphorus removal was achieved.

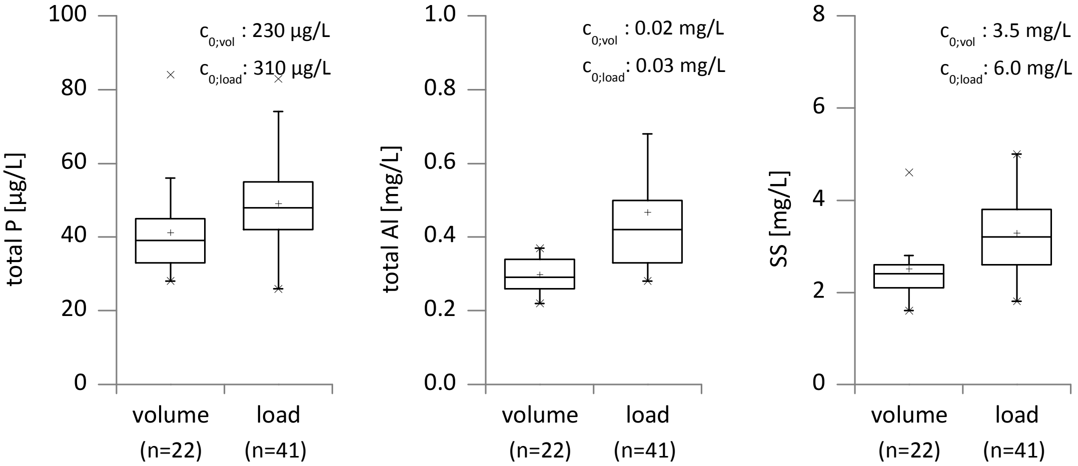


Figure 40: Achievable effluent water quality with volume and load proportional dosing (Fixed daily flow pattern and reduced HRT for coagulation)

The amount of total Al and suspended solids was increased during the period with load proportional dosing. On average there were 0.1 mg/L more total Al and 1 mg/L more suspended solids in the microsieve effluent. But again it has to be considered that the influent water was not of the same quality as during the trials with volume proportional dosing. There were on average 6.0 mg/L suspended solids instead of only 3.5 mg/L when volume proportional dosing was applied.

First tests with load proportional dosing showed that the amount of coagulant and polymer could be reduced while maintaining good effluent water quality.

- Improved performance through change from $FeCl_3$ to PACl: better removal of SS and TP, increase of transmittance and reduction of coagulant residues.
- Load proportional dosing reduces under as well as over-dosing of chemicals. The coagulant and polymer dosing could be reduced by 13 and 18 % while maintaining good phosphorus removal (effluent TP << 80 µg/L).

4.4 Polymer type and dose

4.4.1 Anionic polymer

In preliminary jar tests mostly cationic polymers were tested. As anionic polymers are widely used and known to be less toxic, additional jar tests with anionic polymer were conducted. Figure 41 shows results from jar tests with increasing doses of anionic and cationic polymers at a high iron dose. 0.5 to 1.5 mg/L of the anionic polymer A1 and 1.0 to 2.0 mg/L of the cationic polymer C1 were applied with 0.179 mmol/L Fe.

All test conditions resulted in very low effluent TP contents (45-50 µg/L). Regarding the amount of residual iron the performance of the anionic polymer was decidedly superior. With A1 there were 0.4 to 0.6 mg/L Fe, whereas with the same dose of C1 there was three times more iron (1.1 to 1.4 mg/L Fe). The lower effluent iron contents with A1 were also reflected in significant lower effluent turbidities. The turbidity was between 0.5 and 0.8 NTU. With C1 the turbidity was between 1.4 and 1.8 NTU. The cationic polymer C1 only dominated regarding the filterability. With the cationic polymer C1 the filterability amounted to 24 m/h. By dosing A1 a maximum filterability of only 16 m/h was achieved. By increasing the polymer dose of C1 up to 2.0 mg/L the filterability was improved. By increasing the dose of A1 from 0.5 to 1.0 a slight improvement of the filterability was observed, and none at a further increase to 1.5 mg/L. With the anionic polymer A1 a better effluent water quality was achieved, especially with regard to total iron and turbidity. But crucial for the polymer choice is the filterability as the pilot plant tests will show.

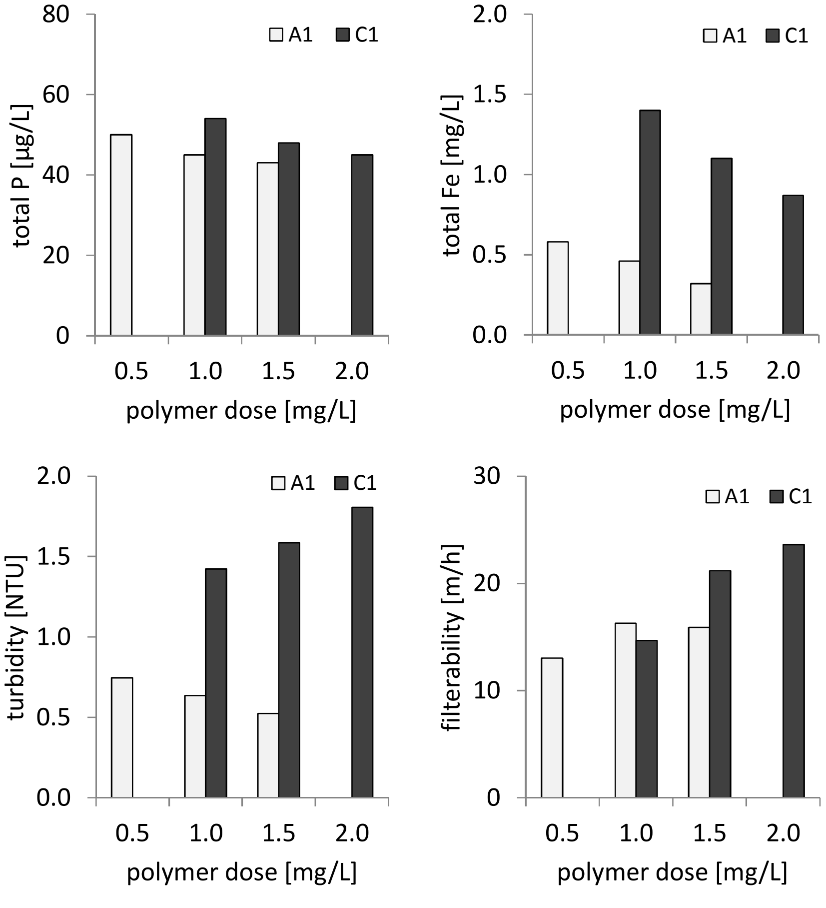


Figure 41: Jar test results - Cationic vs. anionic polymer at a high iron dose (30.03.2011). 0.179 mmol/L (10.0 mg/L) Fe, 0.5 to 2.0 mg/L polymer A1 or C1. n = 1

The anionic polymer A1 was applied in the pilot plant in order to verify the results from the jar tests. The polymer was tested at different dosing conditions with $FeCl_3$ and PACl. In Table 19 the results of the effluent water quality for all test conditions with polymer A1 are summarized. The achieved water quality was similar to the average quality achieved with cationic polymers. No significant superiority to cationic polymers was observed with regard to turbidity and total iron content in the effluent. Both were as high as with cationic polymers. But all this was achieved at significantly lower polymer doses (0.25 to 1.0 mg/L).

Table 19: Effluent water quality in the pilot plant when dosing anionic polymer. 0.090 or 0.185 mmol/L Fe or Al, 0.25 to 1.0 mg/L polymer A1, 10 to 20 m^3/h

Parameter		Mean value	Minimum	Maximum	n
total P	[µg/L]	67	10	100	12
SS	[mg/L]	3.4	1.6	8.2	10
Turbidity	[NTU]	1.9	1.3	4.2	12
total Fe	[mg/L]	1.5	0.8	3.0	5
total Al	[mg/L]	0.5	0.3	1.0	6

Figure 42 gives the results for the backwash time of two different tests with the anionic polymer A1. For test A, 0.90 mmol/L Al were applied with a polymer dose of 0.25 to 1.5 mg/L and a flow of 20 m^3/h. For all four test conditions the backwash time amounted to 100 %. Besides, in all conditions the plant capacity was exceeded, which resulted in a considerable amount of treated water which left the pilot plant through the by-pass and not through the microsieve. At least increasing the polymer dose resulted in a reduction of the by-pass water flow. With 0.25 mg/L polymer about 8 m^3/h of the chemically treated water went into the by-pass, with 0.5 and 1.0 mg/L 5 m^3/h, but with 1.5 mg/L polymer still 4 m^3/h. For test B 0.90 mmol/L Fe and 1.0 mg/L polymer A1 were dosed. During the test the flow was reduced from 20 to 10 m^3/h. At 20 and 15 m^3/h the backwash time amounted to 100 %. When reducing the flow to 12 m^3/h the backwash time was slightly below 100 %. Decreasing the flow by the factor two to 10 m^3/h, the average backwash time amounted to 76 %. At 20 m^3/h approximately 4 m^3/h chemically treated water went through the by-pass, at 15 and 12 m^3/h 0.1 to 0.2 m^3/h and none at 10 m^3/h.

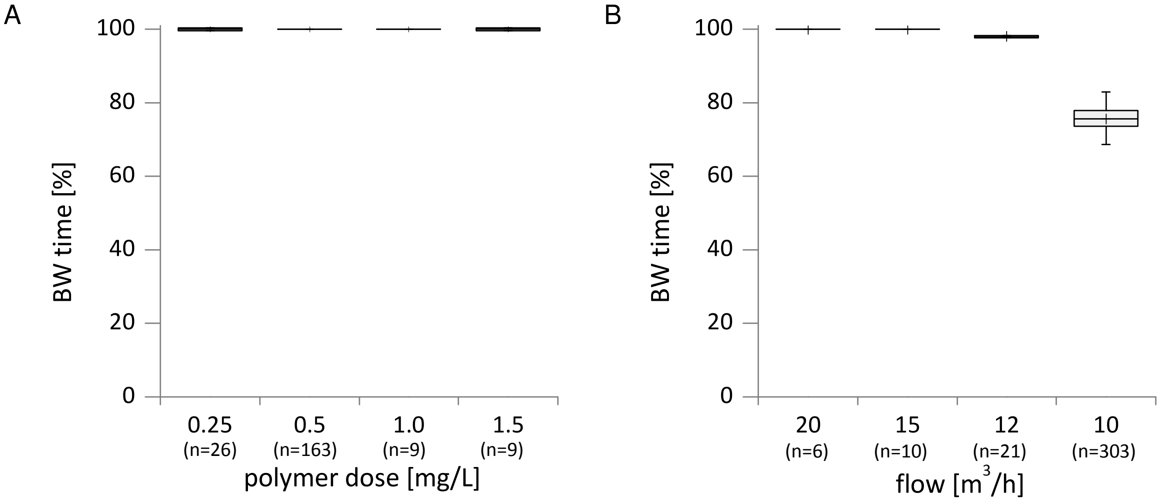


Figure 42: Pilot plant results - Backwash time when dosing anionic polymer. A1: 0.090 mmol/L Al, 0.25 to 1.5 mg/L polymer A1, 20 m^3/h. B: 0.090 mmol/L Fe, 1.0 mg/L polymer A1, 10 to 20 m^3/h

The jar test showed that applying anionic polymers improves the effluent water quality but leads to a reduced filterability. In the pilot plant this resulted in a reduction of microsieve filtration capacity by 50 %. At a dose of 1.0 mg/L of polymer A1 the flow had to be reduced from 20 to 10 m^3/h to achieve a backwash time comparable with results achieved with cationic polymers. Implementing this technology would mean double investment costs and energy consumption for the filtration step. In the given case, the tested cationic polymers are to be favored over the polymer A1.

4.4.2 Comparison of cationic polymers

During the preliminary jar tests a large variety of cationic polymers was tested in combination with $FeCl_3$. With regard to phosphorus removal, filterability and turbidity removal the cationic polymers C1, C2 and C4 were identified as the best flocculant aids. C4 is the only liquid polymer product that was tested. It gave results very similar to C1 (see Figure 64 in Appendix B). As liquid products are generally more expensive powdered polymer products are to be preferred. The polymers C1 and C2 are very similar and only differ in their charge density: C1 (70 %) has a slightly higher charge density than C2 (60 %).

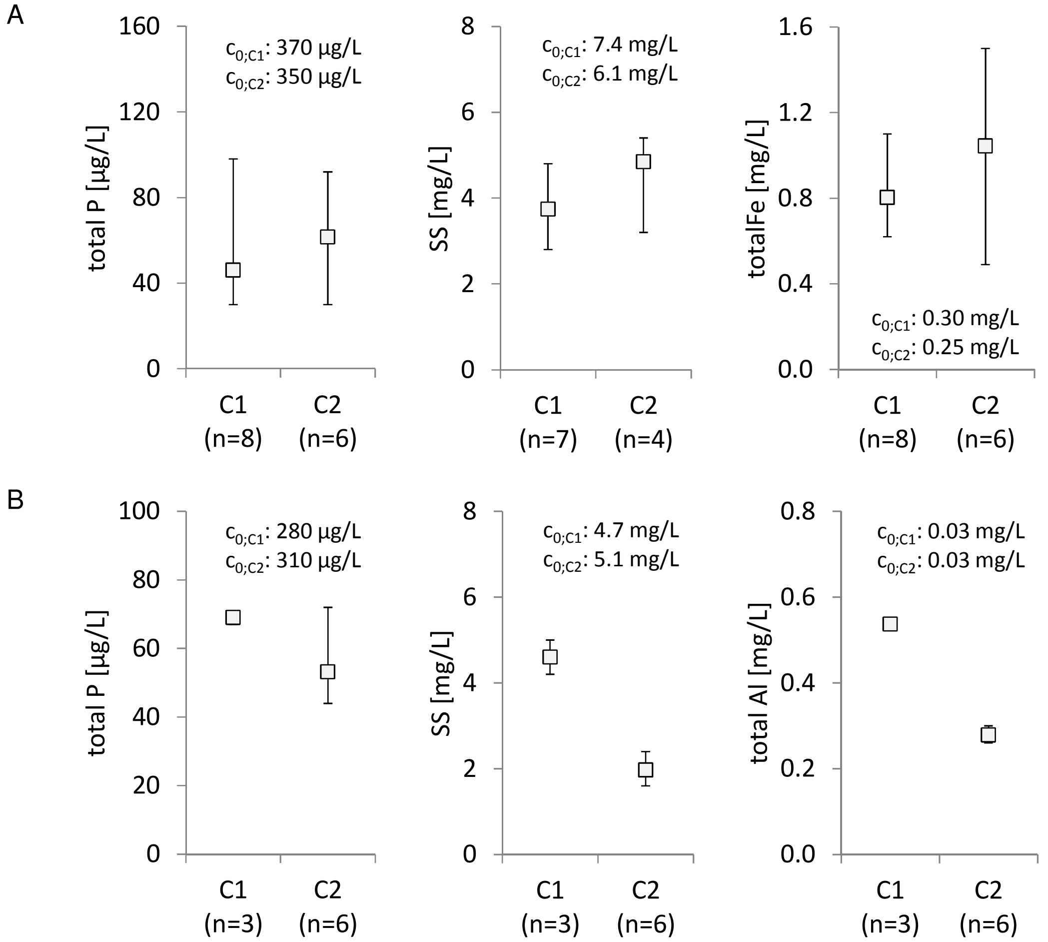


Figure 43: Variation of polymer type in the pilot plant – C1 vs. C2. A: 0.072 mmol/L Fe, 2.0 mg/L polymer C1 or C2, 20 m^3/h. B: 0.072 mmol/L Al, 1.5 mg/L polymer C1 or C2, 20 m^3/h

Figure 43 shows the results from pilot plant tests with $FeCl_3$ and PACl. When applying $FeCl_3$ (A) with polymer C1 a slightly better removal of total phosphorus and SS was achieved. At similar influent contents of TP and SS for both test conditions, with C1 there were on average 40 µg/L TP and 3.7 mg/L SS whereas with C2 there were 60 µg/L TP and 5.0 mg/L SS in the effluent. Moreover, less total iron was found in the effluent when C1 was applied (0.75 in comparison to 1.05 mg/L Fe). In combination with PACl (B) C2 showed a better performance than C1. With C2 an average effluent TP value of 50 µg/L. Under the same conditions with C1 mean effluent TP values of 70 µg/L were achieved. Dosing the polymer C2 resulted in a higher removal of SS. Compared to dosing C1 there was less than half the amount of SS (2.0 mg/L) in the effluent. Regarding coagulant residuals with the polymer C2 a better effluent water quality was also achieved. When polymer C1 was dosed total aluminum in the effluent amounted to 0.54 mg/L and with polymer C2 only to 0.27 mg/L.

The backwash time for the comparison of the polymers C1 and C2 is presented in Figure 44. In combination with $FeCl_3$ (A) the polymer C1 achieved a very good backwash time << 66 %. The results with C2 and $FeCl_3$ were considerably worse with an average backwash time of 75 %. But this was due to prior filter clogging. When PACl was applied (B) with both polymers similar backwash times of about 70 % were achieved. Although the difference in the achieved effluent water quality with the polymers C1 and C2 was only slight, it is recommended to prefer the application of polymer C1 to C2 when $FeCl_3$ is dosed due to the very low backwash times obtained with C1. However, if PACl is dosed, polymer C2 should be preferred.

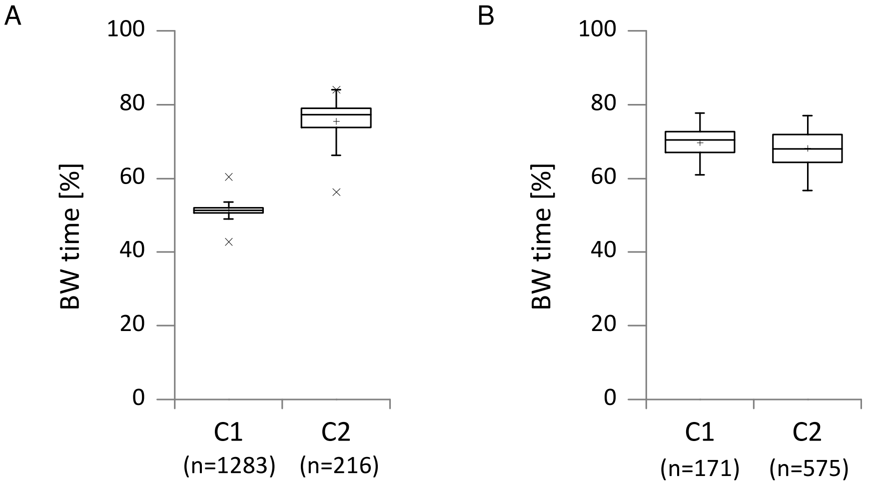


Figure 44: Variation of polymer type in the pilot plant – C1 vs. C2. A: 0.072 mmol/L Fe, 2.0 mg/L polymer C1 or C2, 20 m^3/h. B: 0.072 mmol/L Al, 1.5 mg/L polymer C1 or C2, 20 m^3/h

4.4.3 Polymer dose (before hydraulic optimization)

Figure 45 gives an example of increasing the polymer dose in the pilot plant (January 2011). Applied were 1.0, 1.5 and 2.0 mg/L polymer C4 with 0.090 mmol/L Al and 20 m^3/h. When dosing 1.5 or 2.0 mg/L polymer C4, low TP effluent values (< 50 µg/L) were achieved. With only 1.0 mg/L there was a wide variation of the effluent TP values. 70 to 190 mg/L TP were found in the effluent. Whereas with 1.5 and 2.0 polymer there were approximately 300 mg/L TP in the influent, with 1.0 mg/L polymer there were > 500 µg/L TP in the influent. With 1.5 and 2.0 mg/L polymer SS effluent values around 3 mg/L were achieved. The dosing of only 1.0 mg/L polymer resulted in SS effluent values between 6 and 7 mg/L. In fact the SS influent content was increased during the trials with 1.0 mg/L polymer but SS values of 10 mg/L often promote the floc formation and thus the process performance. The insufficient SS removal with 1.0 mg/L C1 resulted in higher TP effluent values and moreover in higher total aluminum contents in the effluent (of about 0.9 mg/L Al). When dosing 1.5 mg/L or more there were only 0.4 mg/L Al in the effluent. Increasing the polymer dose resulted in decreased backwash times. With 1.0 mg/L C4 there was 100 % BW time. Dosing 1.5 mg/L C4 the backwash time could be reduced to an average of 63 %. The lowest backwash time (average of 56 %) was achieved with 2.0 mg/L. The large difference between 1.0 and 1.5 mg/L C4 was probably caused by the high loads of TP and SS in the influent during the dosage of 1.0 mg/L C1.

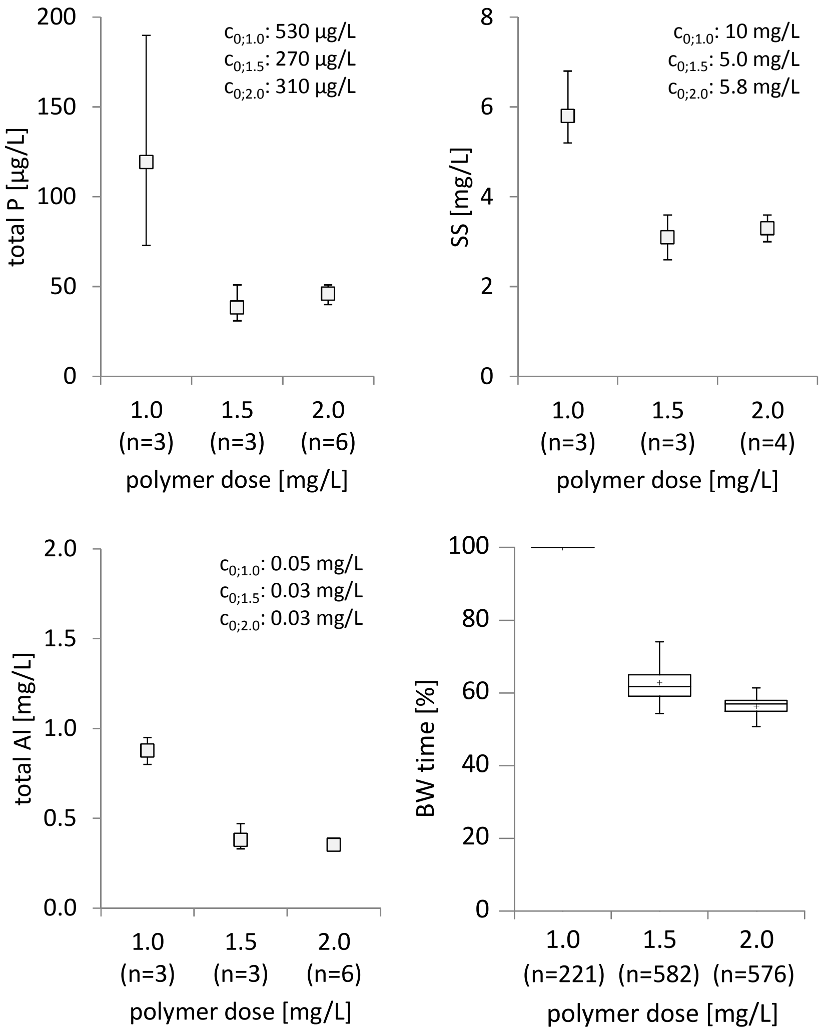


Figure 45: Variation of polymer dose in the pilot plant during winter period – Effluent water quality and backwash time with polymer C4. 0.090 mmol/L (2.4 mg/L) Al, 1.0, 1.5 and 2.0 mg/L polymer C4, 20 m^3/h

Preliminary to the introducing the load proportional dosing another test series with a varying polymer dose was performed. 1.0 to 2.0 mg/L of polymer C1 were applied in combination with 2.4 mg/L Al. During the early summer of 2011 there were very low total P concentrations in the WWTP effluent (average 230 µg/L). A polymer dose of only 1.0 mg/L was sufficient to achieve total P values below 70 µg/L (see Figure 46). The lowest total P values (26 to 41 µg/L) were observed during a polymer dose of 1.75 mg/L. With regard to removal of suspended solids and Al it is reasonable to dose at least 1.5 to 1.75 mg/L polymer. When 1.75 mg/L polymer were dosed on average there were only 1.6 mg/L SS and 0.33 mg/L total Al in the pilot plant effluent.

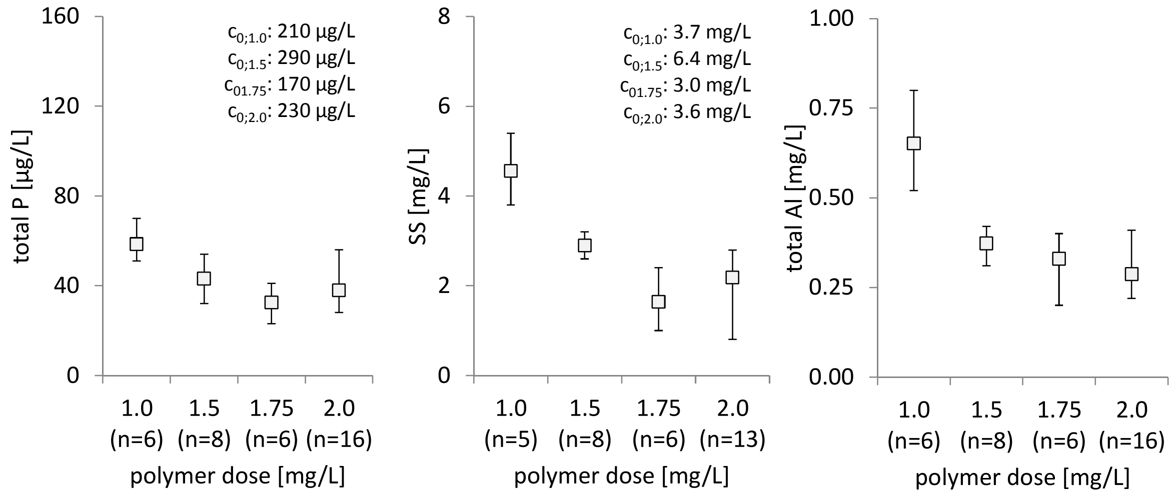


Figure 46: Variation of polymer dose in the pilot plant during summer period. 0.090 mmol/L (2.4 mg/L) Al, 1.0, 1.5. 1.75 and 2.0 mg/L polymer C1, 20 m^3/h, reduced HRT for coagulation.

The results from the pilot plant showed, that 1.5 to 1.75 mg/L cationic polymer were sufficient to achieve good effluent water quality and backwash time. Under the given configurations 1.0 mg/L of polymer were only sufficient at appropriate influent water quality. Thus, the load proportional dosing was conducted with a polymer dosing range of 1.5 to 2.0 mg/L. After extreme changes of the hydraulic conditions during the reconstruction of the pilot plant the polymer dosing could be reduced to 0.6 mg/L on average (see next chapter).

- Anionic polymer probably produces a better effluent water quality than cationic polymer but severely reduces the filtration capacity of the microsieve. Thus, cationic polymer is recommended for the application prior to microsieving.
- During the trial period before the rebuild of the pilot plant a polymer dose of 1.5 to 2.0 mg/L was identified to ensure safe operation and an adequate backwash time.

4.5 Filtration performance and backwash

4.5.1 Filtration and backwash dynamics

The effluent flow of a single microsieve unit fluctuates in accordance to the backwash (see Figure 47A; the grey area). Just before the backwash is activated, the effluent flow is at its minimum (1) due to the formation of a cake layer. As described above, the backwash starts when the maximum water level is reached and thus at the maximum operational differential pressure. The high pressure in combination with clean filter panels during backwash leads to an increase of the effluent flow (2). As long as the panels are backwashed the filtrate flow lingers on a high level. When the backwash is finished (3) the filtrate flow decreases due to progressing filter clogging until the backwash is activated again (4).

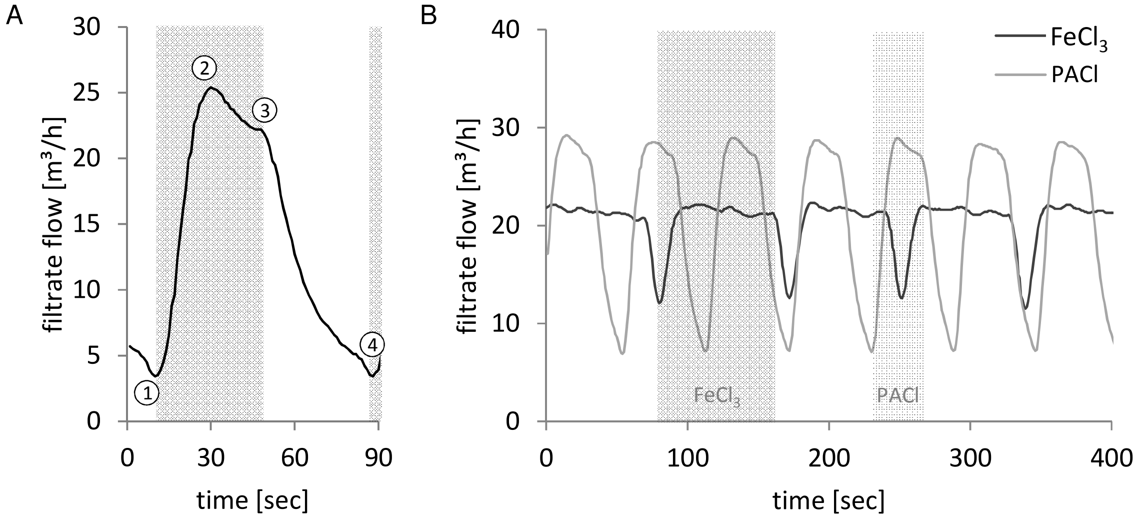


Figure 47: Dynamics of backwash and filtrate flow in detail (A, influent flow 15 m^3/h) and when the coagulant type was varied (B, 0.07 mmol/L Fe or Al; 1.5 mg/L polymer; influent flow 20 m^3/h). 25 d ($FeCl_3$) and 30 d (PACl) after chemical cleaning.

Figure 47B presents the fluctuation of the filtrate flow when $FeCl_3$ (A) and PACl (B) were applied. During the trials different effluent flow dynamics could be observed. With $FeCl_3$ the backwash intervals were longer and the effluent flow fluctuated in a smaller range. The effluent flow was in the range between 12 and 22 m^3/h whereas, with PACl the effluent flow fluctuated between 7 and 29 m^3/h. Because of the larger backwash intervals it can be assumed that $FeCl_3$ formes flocs of different size, structure and porosity compared to PACl. This flocs could not be easily removed from the filter panels through the backwash. Although the maximum flow and thus the shear forces on the flocs and the probability of floc breakage are higher with PACl, a better water quality was achieved (effluent turbidity < 1 NTU instead of 3 NTU). Most probably PACl creates less sticky but stronger flocs than $FeCl_3$.

With advancing time of operation the backwash dynamics change due to increasing irreversible fouling. An example with PACl is given in Figure 48. When the filter panels are brand new or just chemically cleaned the maximum filtrate flow is very high. Usually the maximum filtrate flow on the first days of operation is more than twice the influent flow. In the given example the maximum filtrate flow on the first day amounted to 34 m^3/h while the pilot unit was operated at a flow rate of 15 m^3/h. Due to the high filtrate flow the backwash is of short duration (20 sec).

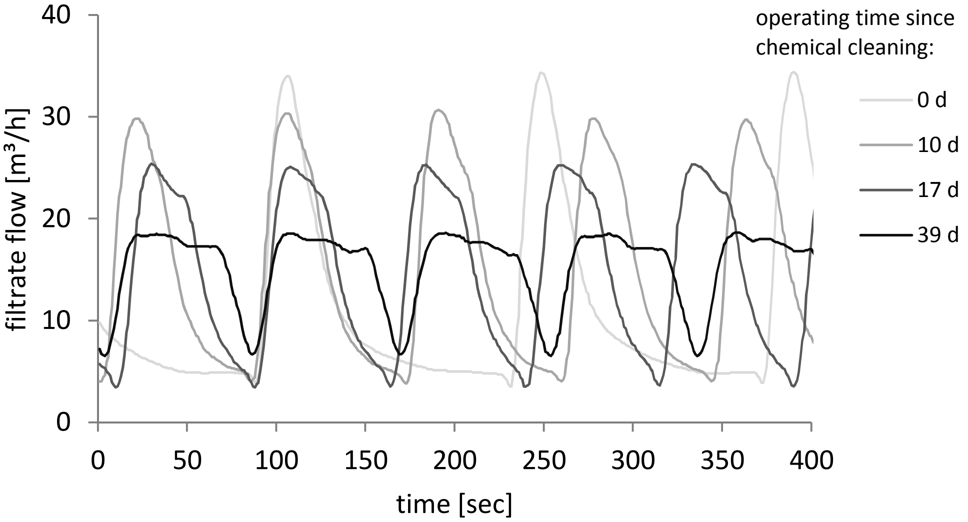


Figure 48: Change of filtration dynamics over time of operation (load proportional dosing of PACl and polymer C2, 15 m^3/h)

Furthermore, the backwash activated only every 140 sec. During the following weeks the irreversible fouling increases and the filter permeability decreases. Thus, the maximum filtrate flow decreases. As a result, the water level in the microsieve declines more slowly and a longer duration of the backwash (40 sec after 17 d of operation) is necessary to regain the initial water level stopping the backwash. Also the time interval between the backwash cycles decreases, e.g. from 120 sec to 35 sec (after 17 d). Thus, after a certain time of operation (depending on water quality and flow rate) there is a turning point and the backwash is activated again less often due to the fact that the backwash duration further increases until continuous backwash is necessary for treating the whole amount of water. In the example given in Figure 48, after 17 d of operation the backwash was activated every 75 sec and after 39 d every 80 sec.

4.5.2 Optimization of the disc rotation

During normal operation the filter discs only rotate during the backwash. In order to create a homogenous particle layer on the whole filter disc and thus even the strong fluctuation of the filtrate flow during the backwash the microsieve was operated with continuous disc rotation. In both cases the discs rotate with a frequency of 50 Hz.

Figure 49 shows the development of the backwash time with continuous disc rotation compared to normal operation. During the tests the pilot unit was operated with a fixed daily flow pattern (see dotted curve). Thus, the backwash time also follows this pattern. With continuous disc rotation it was observed that the backwash time increased to 100 % within only five days. On May 28 at 9 p.m. the filtration capacity of the pilot unit was reached. From that point on more and more water left the pilot unit through the by-pass until finally only 5 m^3/h filtrate was produced and the test was concluded. After a short interruption the pilot unit was restarted again with the usual discontinuous disc rotation. Although no cleaning of the filter panels was conducted, the backwash time was abruptly lower and also did not increase during a whole week of operation, apart from the daily variation. This indicates that the filter clogging observed during the operation with continuous disc rotation was reversible and that using the whole filter area instead of only the submerged area (50 %) is unfavorable for the backwash.

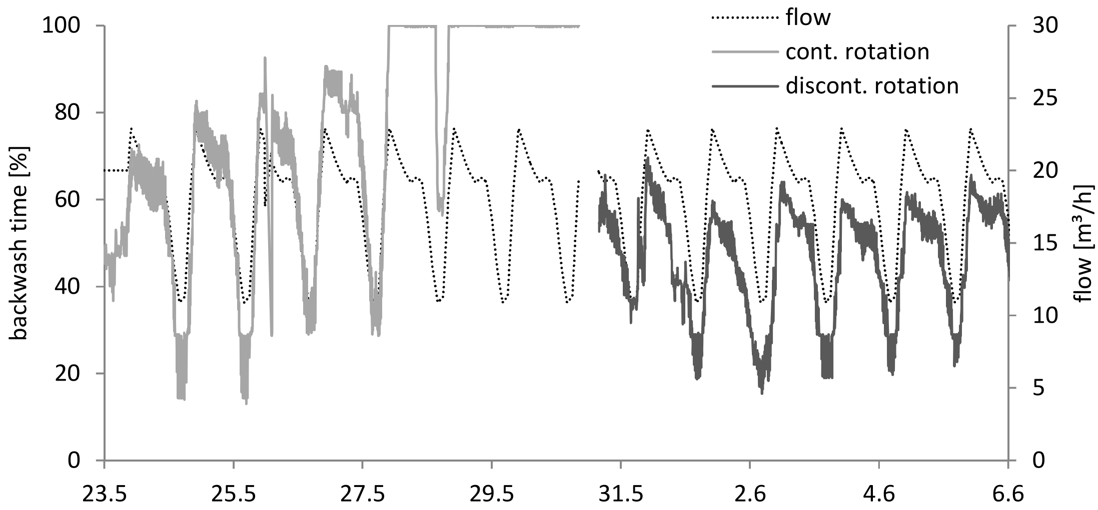


Figure 49: Development of the backwash time with continuous and discontinuous disc rotation (fixed flow pattern, 2.4 mg/L Al, 2.0 mg/L polymer C1)

A detailed look into the filtration dynamics reveals that the fluctuation of the filtrate flow can be evened through continuous disc rotation (see Figure 50) as was anticipated. With continuous disc rotation the peak filtrate flow during the backwash amounted to 26 m^3/h whereas under normal operating conditions the peak flow was 31 m^3/h. As consequence of the reduced peak flow longer backwash duration is necessary to regain the initial water level on the feed side of the microsieve to stop the backwash resulting in an increased overall backwash time. Figure 50 also gives the effluent turbidity for both operating modes. When the disc rotation is discontinuous (Figure 50B) the effluent turbidity slightly fluctuates (between 1.3 and 1.4 NTU) with the effluent flow. During the operation with continuous rotation the effluent turbidity was lower (1.0 NTU) and extremely stable.

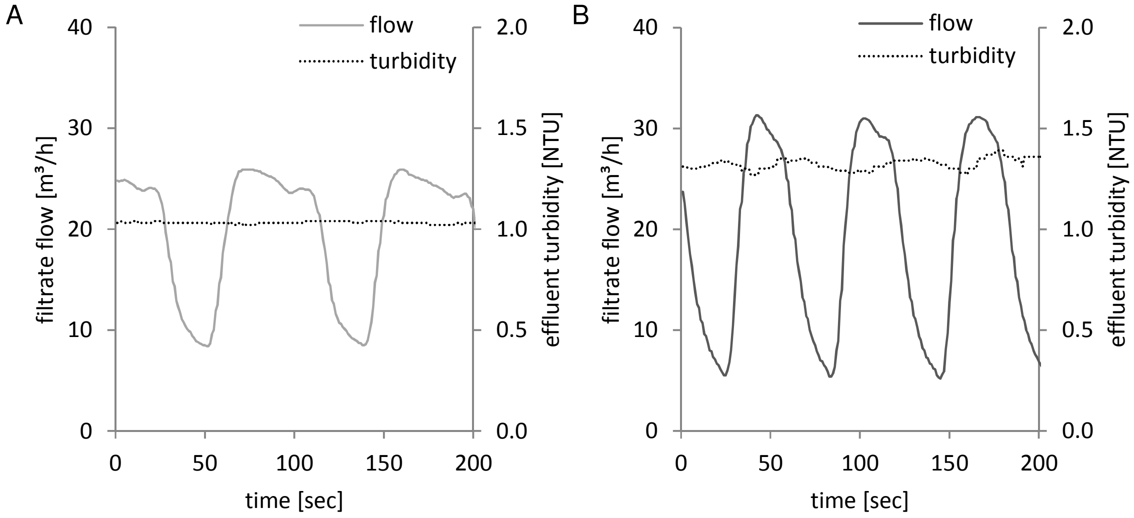


Figure 50: Filtrate flow dynamics with continuous (A) and discontinuous (B) disc rotation (19 m³/h, 2.4 mg/L Al, 2.0 mg/L polymer C1)

In order to estimate the impact of the altered filtrate flow dynamics on the effluent water quality the amounts of total P, SS and total Al were determined (see Figure 51). During both trial phases the total P influent and effluent concentrations were extremely low. With continuous rotation the average total P effluent value amounted to 42 µg/L and to 48 µg/L with discontinuous rotation. Different from the observed effluent turbidity, during the continuous rotation the amount of total Al in the effluent was slightly lower. On average there were 0.33 mg/L total Al compared to 0.41 mg/L. For both conditions the effluent SS content averaged out at around 3 mg/L.

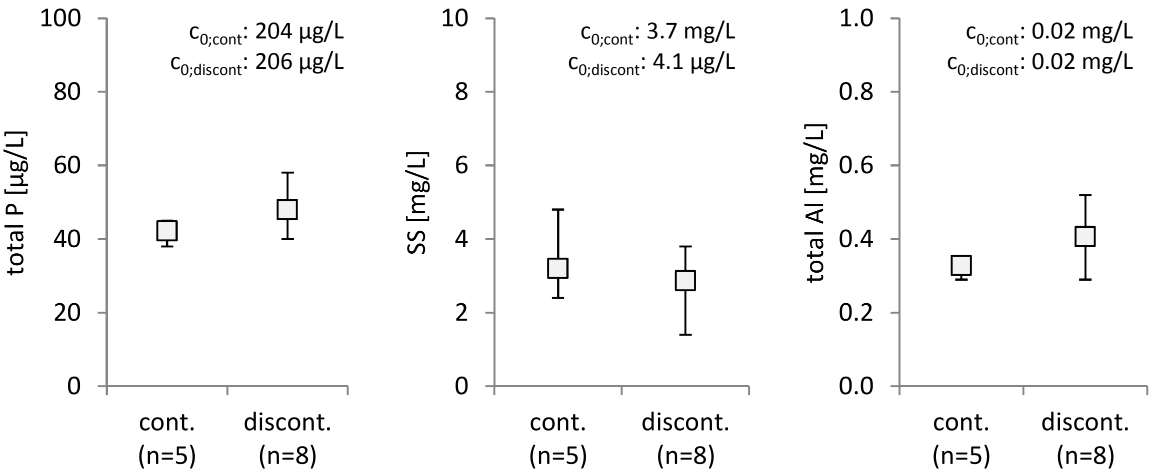


Figure 51: Comparison of the effluent water quality with continuous and discontinuous disc rotation (fixed flow pattern, 2.4 mg/L Al, 2.0 mg/L polymer C1). Mean values and range.

As expected, the fluctuation of the filtrate flow can be softened through the continuous rotation of the filter discs. Possibly due to the changed filtrate flow dynamics the effluent water quality was slightly improved. Unfortunately, a decrease of the filtrate peak flow also results in an increase of the backwash time. The improvement of the effluent water quality was not as significant as would justify the loss in filtration capacity and the increased energy demand for the backwash. Thus, the continuous rotation was not further investigated.

4.5.3 Backwash time and intervals of chemical cleaning

The pressurizing of the filtrate (8 bars) makes the backwash the most energy-intensive element of the process. For a full-scale application a specific energy demand of 18 Wh/m^3 (manufacturer calculation) of treated water is estimated. Thus, the process should be run in a manner that keeps the backwash low. As shown above, the backwash time is influenced by the character of the formed flocs and thus the quality of the chemical pre-treatment, e.g. through the polymer dose (see Figure 45) or the mixing energy. Furthermore, the load of solid matter affects the backwash frequency.

Figure 52 displays the backwash time as a function of the influent flow during dynamic operation. Additionally, the data is classified according to the time of operation since the last chemical cleaning. In the first week, the backwash time was on a low level. At a flow rate of 15 m^3/h the average backwash time amounted to 42 %. Besides, the backwash time showed a clear linear correlation to the influent flow. The coefficient of determination R^2 was 0.94. With progressing time of operation the backwash time increases. After four weeks the backwash time averaged 66 % at a flow rate of 15 m^3/h. But still there was a very good linear correlation (R^2= 0.91). In the seventh week after the chemical cleaning the backwash time increased to 100 %.

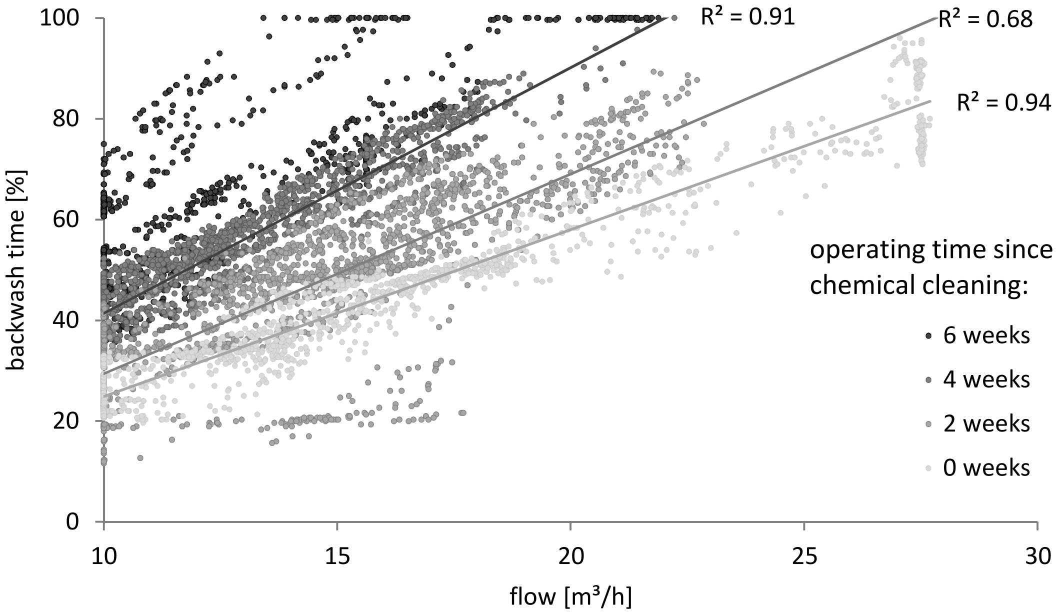


Figure 52: Backwash time as a function of the influent flow and the operating time (Feb/Mar 2012; dynamic operation, load proportional dosing of PACl and polymer C2)

For large scale installation, e.g. Gothenburg, cleaning cycles of 4-5 weeks are reported (manufacturer information). Furthermore, nowadays large-scale applications of the microsieve are equipped with fully automated cleaning systems and additionally offer intermediate high pressure (80 bar) backwashing to lower the backwash time. As the expense for a chemical cleaning are low it is worthwhile to apply a chemical cleaning more often e.g. every 2-3 weeks in order to keep the backwash time and thus the energy demand low (compare D6.2 (Remy 2013)).

- The filtrate flow of a single microsieve fluctuates in accordance to the backwash. The dynamic of the backwash changes with advancing operation time and progressing fouling: the maximum filtrate flow decreases and the backwash duration increases.
- Predominantly, the backwash time correlates with the influent flow. Secondarily, the backwash time depends on the influent water characteristics and the quality of the chemical pre-treatment, especially on the resulting load of suspended solids.
- Chemical cleaning of the filter panels was necessary every 4 to 7 weeks. A short cleaning interval (e.g. 4 weeks) might be beneficial due to a decreased energy demand for backwash.

4.6 Microbiological water quality

Due to the rather broad filter opening (10 µm) no relevant disinfectant effect is expected through the application of the microsieve technology combined with chemical pre-treatment. Only after the reconstruction of the microsieve pilot unit and the start-up of the UV disinfection the microbial water quality was investigated. In Figure 53 the concentrations of the indicator organisms *E. coli*, *Enterococci* and *Coliphages* in the influent and effluent of the microsieve and in the effluent of the UV disinfection are displayed. The average influent concentration of *E. coli* was $8.8 \cdot 10^4$ MPN/100 mL, of *Enterococci* $1.7 \cdot 10^4$ MPN/100 mL and of *Coliphages* $4 \cdot 10^3$ PFU/100 mL. To a large extent the amounts of *E. coli*, *Enterococci* and *Coliphages* in the microsieve effluent correlated with the amounts determined in the influent. Through the microsieve filtration and the chemical pretreatment *E. coli* were reduced on average by 0.82 logs, *Enterococci* by 1.34 logs and *Coliphages* by 0.09 logs.

The highest log reduction through the microsieve was determined for *Enterococci*. For 13 out of 38 grab samples from the microsieve effluent the *Enterococci* concentration was already in the range of the limit of quantification (LOQ). (Koivunen et al. 2003) stated, that the reduction of MOs during coagulation might be explained by the adsorption of microorganisms onto hydroxide flocs. (Matsushita et al. 2011) discovered viruses lost their infectivity when coagulated with Al. The removal of the Qβ bacteriophage was 2 logs higher than of the MS2 coliphage. Their difference in removal was rather due to the sensitivity to the virucidal activity of Al coagulant than to differences in the entrapment of Al flocs (Shirasaki et al. 2009). A disturbance of the analytical measurement by Al cannot be excluded. Thus, it can be assumed that the removal of *Enterococci* is not as strong as the data implies.

After the microsieve filtration, the microbial concentration was further reduced via UV disinfection. The design fluence of the LBX 10 disinfection unit can only be altered by adjusting the flow and thus the contact time in the reactor. The first months after the start-up of the UV disinfection unit a design fluence of 1000 J/m^2 was applied (see Figure 54). The at Ruhleben WWTP installed UV disinfection unit (1 m^3/s) treating a partial flow of the secondary effluent is operated with a design fluence of 1000 J/m^2. As the microsieve combined with chemical pre-treatment improves the water quality, especially the UV transmittance, and sufficient disinfection was achieved with 1000 J/m^2 the design fluence of the pilot unit was reduced to 730 J/m^2. This corresponds to a decrease of the specific energy demand from 51 to 37 Wh/m^3.

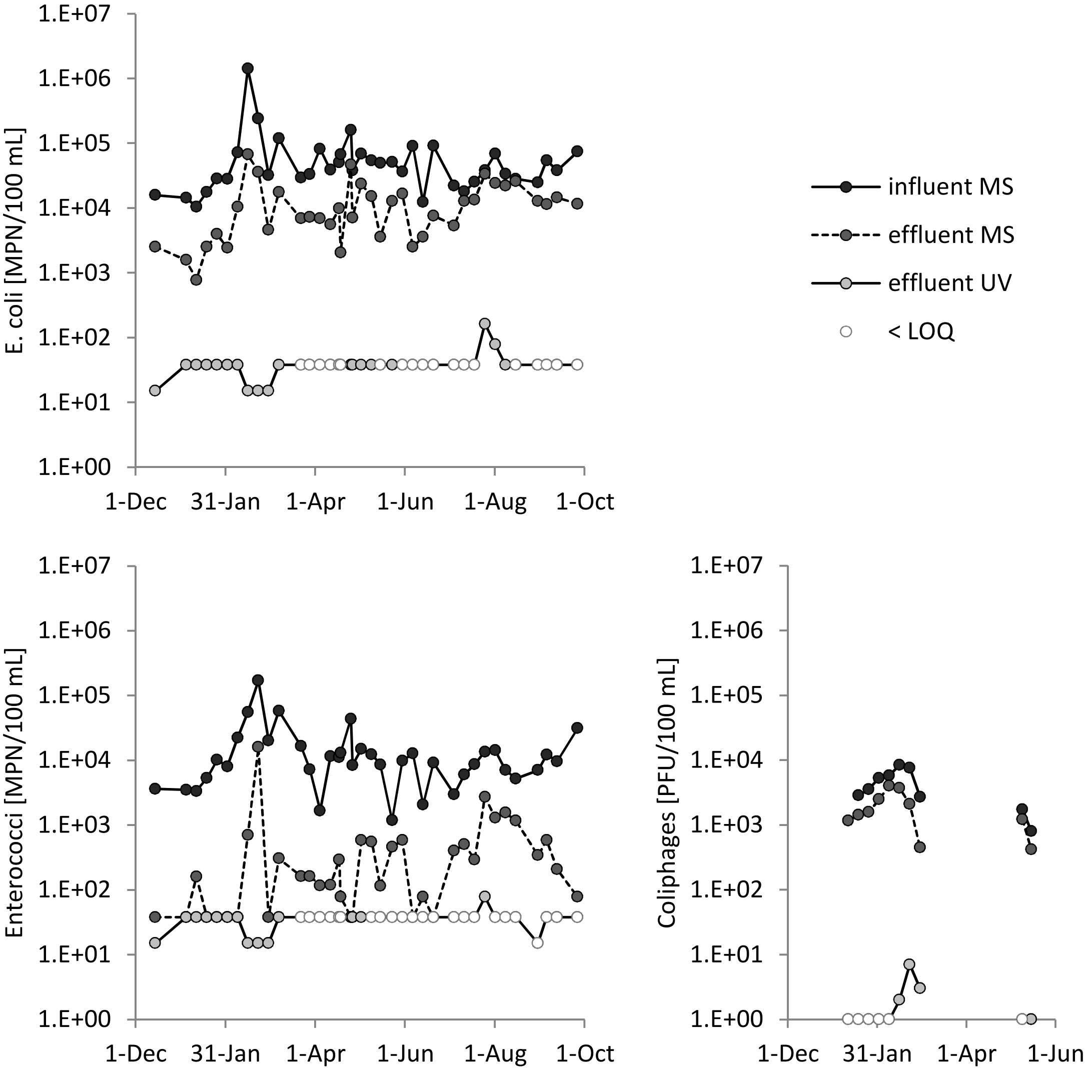


Figure 53: Concentrations of the indicator organisms *E. coli*, *Enterococci* and *Coliphages* in the influent and effluent of the microsieve and after the UV disinfection. The limit of quantification (LOQ) for *E. coli* and *Enterococci* was 15 or 38 MPN/100 mL in dependence of the dilution.

On average *E. coli* were further reduced by 2.52 logs through the application of UV irradiance (see Figure 53). Only two samples contained more than 38 MPN/100 mL *E. coli*. The total log removal through microsieve filtration and UV disinfection was 3.35. In 23 of the 38 samples taken in the effluent of the UV disinfection an *Enterococci* concentration below the LOQ (15 and 38 MPN/100 mL respectively) was determined. For only one singe sample the Enterococci number was higher than the LOQ. Here, the possibility of uncertainty of analytic needs to be kept in mind. *Coliphages* were reduced by 2.97 logs through UV disinfection. Their total log reduction was 3.33.

Apart from the design fluence Figure 54 also gives an estimation of the actual fluence in the UV reactor taking into account the water transmittance, lamp aging and the measured UV irradiance (see equation (12)). Despite a steadily decreasing calculated fluence in the UV reactor, the concentration of *Enterococci* and *E. coli* in the effluent of the UV disinfection did not increase until the end of July 2012. When the calculated fluence was below 550 J/m^2 *Coliphages* (≤ 7 PFU/100 mL) were quantified in the effluent of the UV reactor. However, in May two more samples were analyzed and the number of *Coliphages* was as low as in the beginning (≤1 PFU/100 mL) although the calculated fluence amounted to only 320-330 J/m^2. Thus, the variation of the number of *Coliphages* can possibly be ascribed to the uncertainty caused by sampling and analytic and less possibly to the fluence. The highest counts of *E. coli* (163 MPN/100 mL) and *Enterococci* (78 MPN/100 mL) in the effluent of the UV reactor were detected at the end of July 2012. At this time the calculated fluence reached a minimum of 209 J/m^2. The measured UV irradiance amounted to only 14 W/m^2 (see Figure 59). Thus, on August 7 the glass sleeves of the UV lamps were chemically cleaned, the fluence increased again and the counts of *E. coli* and *Enterococci* were as low as before.

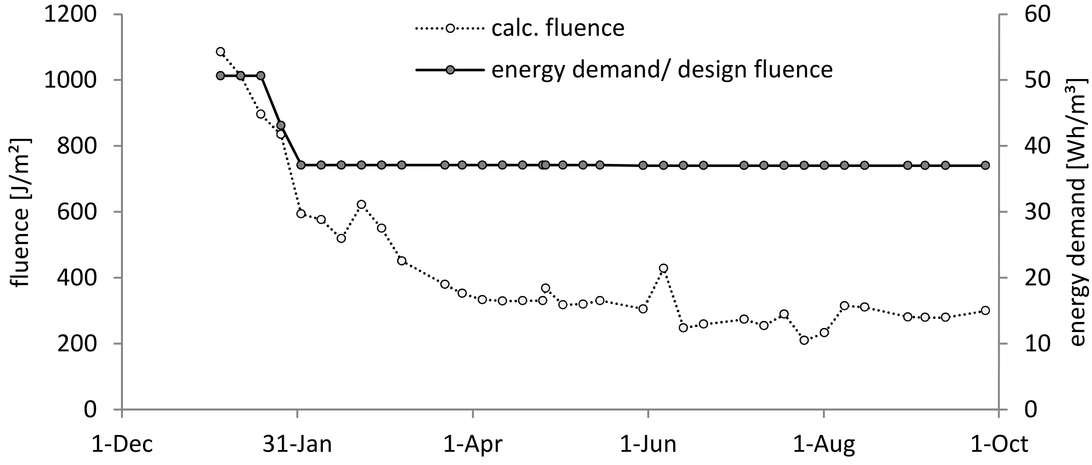


Figure 54: Energy demand and design fluence over the testing period and the estimated fluence at the reactor wall (depending on the lamp aging, water transmittance, measured UV irradiance)

The long-term trials with the UV disinfection showed that through the treatment via microsieve filtration, coagulation and flocculation the applied fluence for UV disinfection can be reduced from 1000 (design for secondary effluent of Ruhleben WWTP) to 730 J/m^2. However, only one out of the 38 samples taken in the effluent of the UV disinfection contained more than 100 MPN/100 mL *E. coli*. For *Enterococci* and *Coliphages* the number was always clearly below 100 MPN/100 mL or rather PFU/100 mL.

- Despite a design fluence of only 730 J/m^2, advancing lamp aging and fouling of the glass sleeves a good disinfection could be provided for 7 months. During this period there were always less than 100 MPN/100 mL of *E. coli* and *Enterococci* (n=30).
- After 7 months a chemical cleaning of the glass sleeves was conducted and the disinfectant effect restored.

4.7 Operational experiences

4.7.1 Performance during peak loads

In Figure 55A the plant performance during peak loading is displayed. During 4 h the influent turbidity exceeded the upper limit of the online measurement (15 NTU) of the pilot plant. The reason for this peak was probably the melting of ice (water temperature: 12.8 °C). Due to the turbidity peak the backwash time sprang to 100 %. On average

2.7 m^3/h of the pre-treated water left the pilot plant through the by-pass. Thus, the peak load exceeded the pilot plant capacity. At 17:00 h the backwash time sprang back to the previous level around 60 % although the turbidity peak lasted until 18:00 h. Only during the peak events there was a clear influence of the influent turbidity on the backwash time. During the whole period displayed in Figure 55A the effluent turbidity was comparatively stable below 2 NTU and not influenced by the turbidity peak.

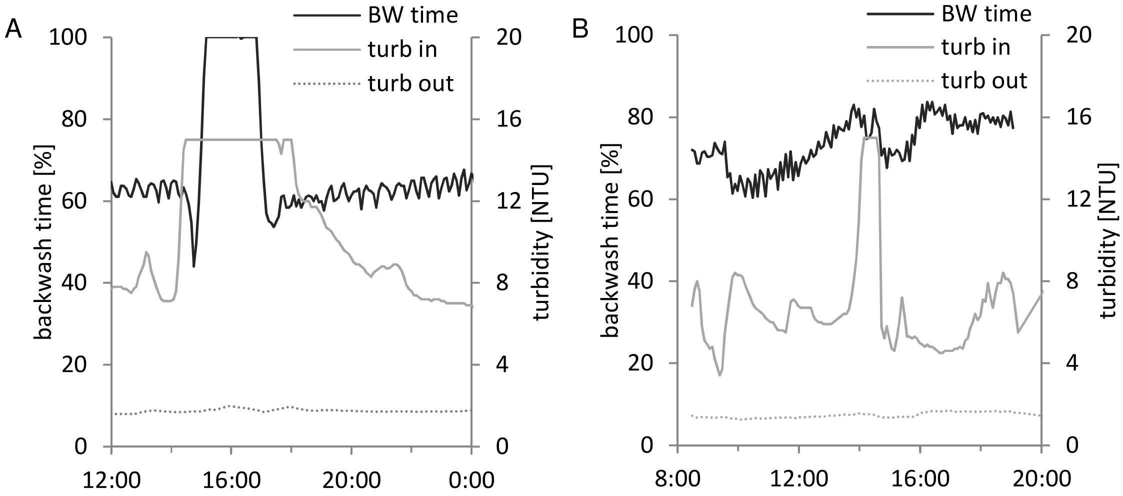


Figure 55: Pilot plant performance at peak loads when dosing 0.07 mmol/L Al (A) and 0.11 mmol/L Al (B). Upper limit for online turbidity measurement: 15 NTU (influent).

Another example where the microsieve had to cope with an influent turbidity peak > 15 NTU is shown in Figure 55B. This event did neither result in 100 % backwash time nor in increased effluent turbidity. Table 20 gives the TP and SS values of influent and effluent of grab samples taken on the same day. During the peak at 14:00 h only a slightly increased content of SS in the effluent (4.8 mg/L) could be observed. The TP removal remained at about 90 % as observed before the peak at 09:00 and 11:00 h. As the influent TP amounted to 1.2 mg/L at 14:00 h this resulted in effluent TP values above 100 µg/L.

Table 20: Pilot plant performance at peak loads when dosing of 0.11 mmol/L Al and 1.5 mg/L polymer (belonging to Figure 55B)

time	TP [µg/L]		P removal [%]	SS [mg/L]	
	in	out		in	out
09:00	410	51	88	11.0	3.2
11:00	490	59	88	9.4	4.2
14:00	1200	110	91	22.0	4.8

The given examples illustrate that the microsieve in combination with chemical pre-treatment can cope with peaks of turbidity, SS and TP. In example B the peak did not lead to an increase in backwash time and the achieved effluent water quality was comparatively good.

4.7.2 Impairment of the backwash system

During the dynamic operation in summer 2011 an unusual increase of the backwash time was observed. An examination of the backwash system revealed that the backwash pressure was strongly reduced due to fouling (see Figure 56) of the strainer in front of the spray nozzles and the backwash pump, a multistage centrifugal pump. With a system pressure of 7 bars or lower there is insufficient cleaning of the filter panels leading to continuous backwash and eventually to by-passing. As the backwash is conducted with the microsieve filtrate the fouling is probably caused by residues of coagulant and polymer.

Figure 56: Cleaning of the backwash pump and the strainer of the backwash system

In the period between June and October 2011, cleaning of the pre-sieve and the pump was necessary in order to maintain a backwash pressure between 7 and 8 bars. Figure 58 gives the recorded pressure of the backwash system during this time period. The backwash system was readjusted as soon as the pressure was 7.2 bars or less. This was the case every 6 to 10 days. Thus, the backwash time cannot be used to evaluate the operation in the period between June and October 2011.

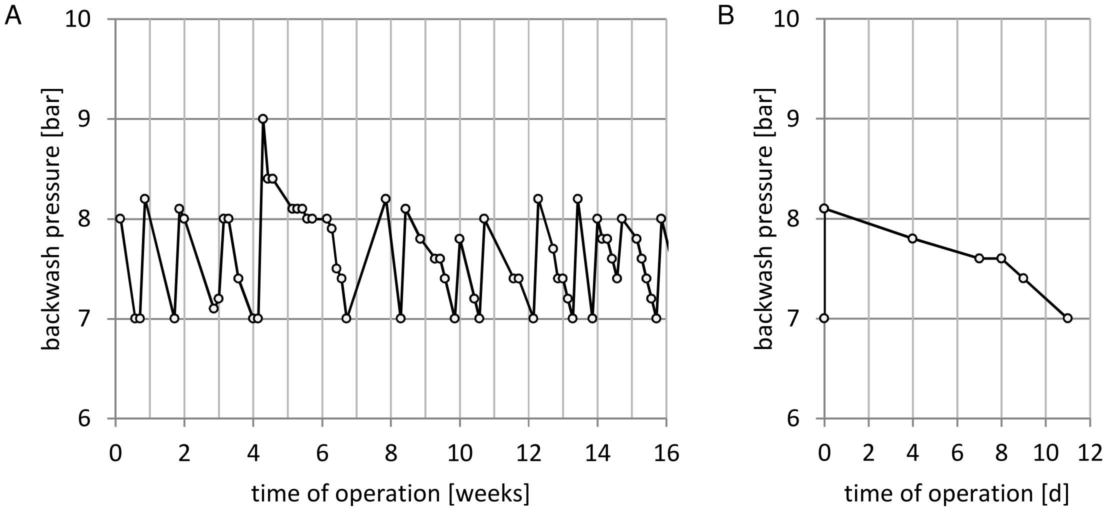


Figure 57: Development of the backwash pressure (June to October 2011)

The problems with the fouling in the backwash system might be an artifact of pilot scale especially of the backwash pump. Thus, during the rebuild of the pilot unit in November 2011 the backwash pump was replaced by a larger pump with larger stage dimenions and the system pressure could be maintained until the end of the piloting without once cleaning the new pump.

4.7.3 Backwash water

As the flow rate of the backwash water in the microsieve pilot unit is too low to be accurately quantifiable the water is collected in a separate tank. A level sensor triggers the sludge pump when the tank is full. Thus, there is a flow rate easy to determine. While chemicals were dosed in 2011 the backwash water was in a range between 0.5 and 3.0 % of the treated water flow (on average 1.7 %). The graph in Figure 58A shows that the amount of backwash water is depended on the backwash time percentage. After the reconstruction of the pilot unit the results were similar (see Figure 58B). On average 1.8 % of backwash water was produced.

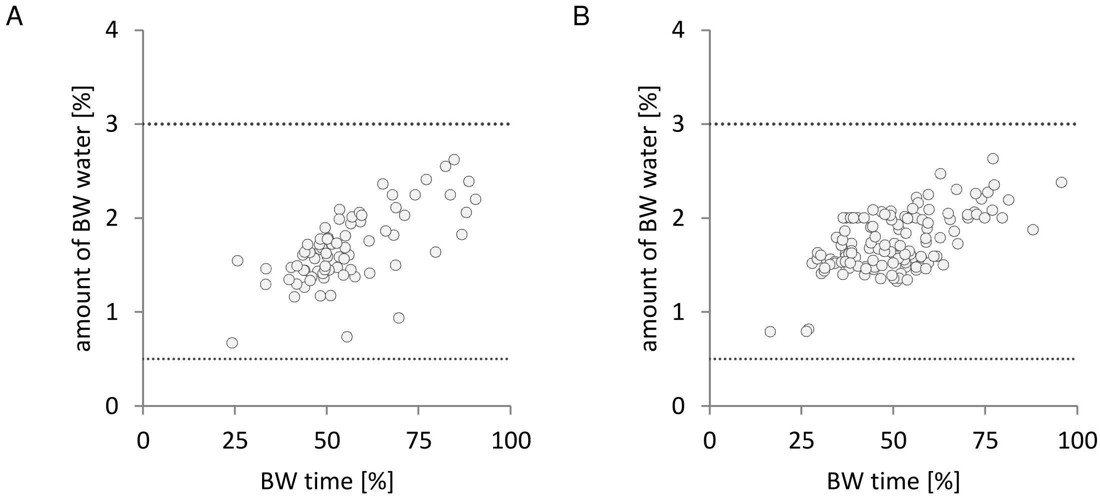


Figure 58: Amount of backwash water as a function of the backwash time before (A) and after the reconstruction of the pilot unit (B). Daily mean values.

During load proportional dosing of PACl in the summer of 2011 the backwash water was further examined (see Table 21). The backwash water contained 580 to 1000 mg/L of suspended solids and 8 to 28 mg/L total phosphorus. The concentration of Al was between 88 and 160 mg/L. The determination of the sludge volume index (SVI) showed that the sludge of the backwash water offers very good settling properties (SVI << 50 mL/g). The supernatant after 30 min of settling contained 24-27 mg/L suspended solids and 2.8-4.3 mg/L total Al. The concentration of total phosphorus was in the range of the content in the influent water (0.3-0.6 mg/L). Thus, the backwash water can most probably be treated via recycling to the primary clarifiers.

Table 21: Composition and characteristics of the backwash water. Mean values (n=3).

parameter		backwash water	supernatant water	sludge
SS	[mg/L]	860	26	6100
total Al	[mg/L]	126	3.5	
total P	[mg/L]	17	0.43	
SV	[mL/L]	140		
SVI	[mL/g]	25		
LOI	[%]			59

4.7.4 Operation of the UV disinfection

The UV disinfection unit has been in operation continuously from mid-December 2011 till the end of September 2012. Due to the intense fouling of the backwash system observed during the summer of 2011 special attention was paid to the fouling behavior of the UV disinfection unit. The UV reactor is equipped with an automated wiping system cleaning the lamp sleeves every hour.

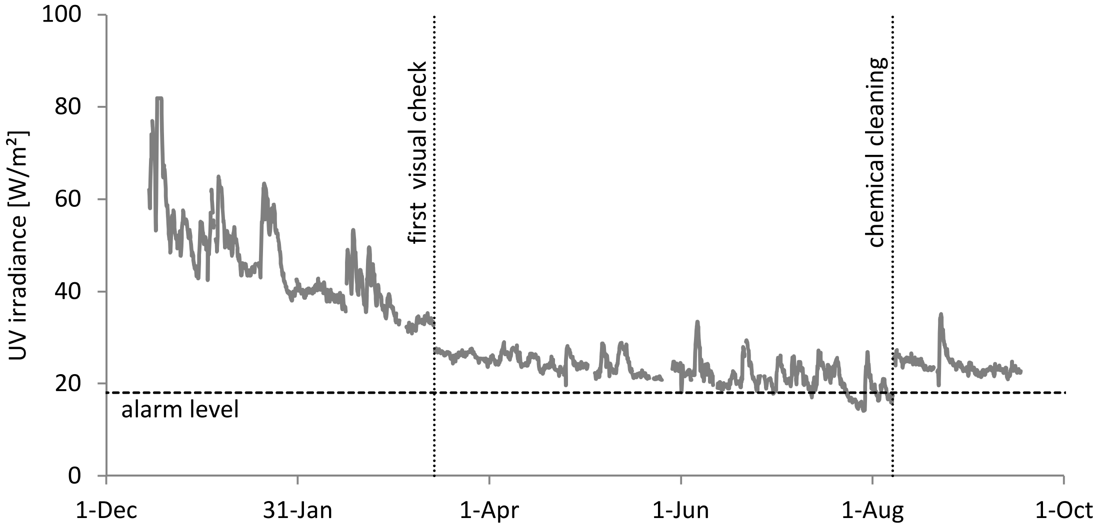


Figure 59: Development of the UV irradiance (measured online)

The development of the irradiance at the reactor wall as a function of the operation time is shown in Figure 60. Over the operating period the irradiance decreased from 82 W/m² to 14 W/m² which is a reduction by 83 %. The sharp drop of the UV irradiance on the March 14 occurred after the UV lamps were temporarily removed for a visual check but could not finally be resolved. Possibly the sensor for the UV irradiance was not reinstalled in exactly the same ankle. In March, after 3 months of continuous operation first evidence of fouling processes was visible (see Figure 60A). A silvery and whitish layer partly covering the sleeves was observed. Calcium and Aluminum were probably the major foulants. In general, the decrease of the irradiance was caused by lamp aging, lamp sleeve fouling and a varying transmittance. The peak values of the UV irradiance occurred during rainy weather. High influent flows led to a dilution of the wastewater and affected the transmittance of the water. The amplitude of the fluctuation was stronger at the beginning of the operation of the UV reactor and declined with the time, due to a probably increasing fouling layer on the lamp sleeves.

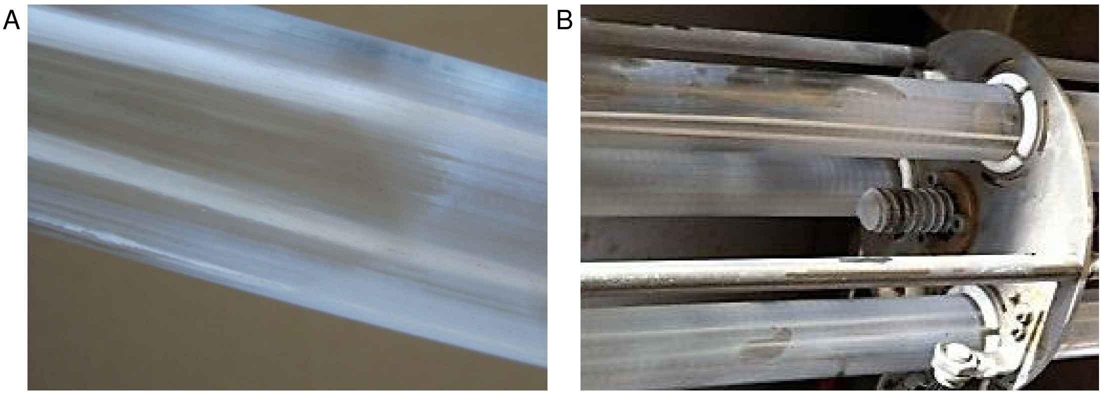


Figure 60: Scaling on lamp sleeves after 3 months of continuous operation (A) and just before the chemical cleaning after 8 months of operation (B)

Due to additional microsieve trials (not part of this report) the UV disinfection was operated 5 more months. After 8 months of operation the fouling of the lamp sleeves was thus advanced that the UV irradiance fell under the alarm level of 18 W/m² and a chemical cleaning was necessary. Opening the UV reactor revealed heavy coating in the glass sleeves (see Figure 60B) that could only be removed with concentrated hydrochloric acid. Although there were some operational problems with the pump the disinfectant effects of the UV irradiance were not impaired throughout the whole testing period.

- The microsieve proved to be a robust technology with little downtime.
- Meanwhile, coagulant and polymer residues affected the backwash system but proved to be an artifact of pilot scale. Coagulant and polymer residues did not impair the disinfection.
- In this application the microsieve produces on average 1.8 % of backwash water. The backwash water contained 580 to 1000 mg/L SS and showed excellent settling properties (SVI << 50 mL/g).

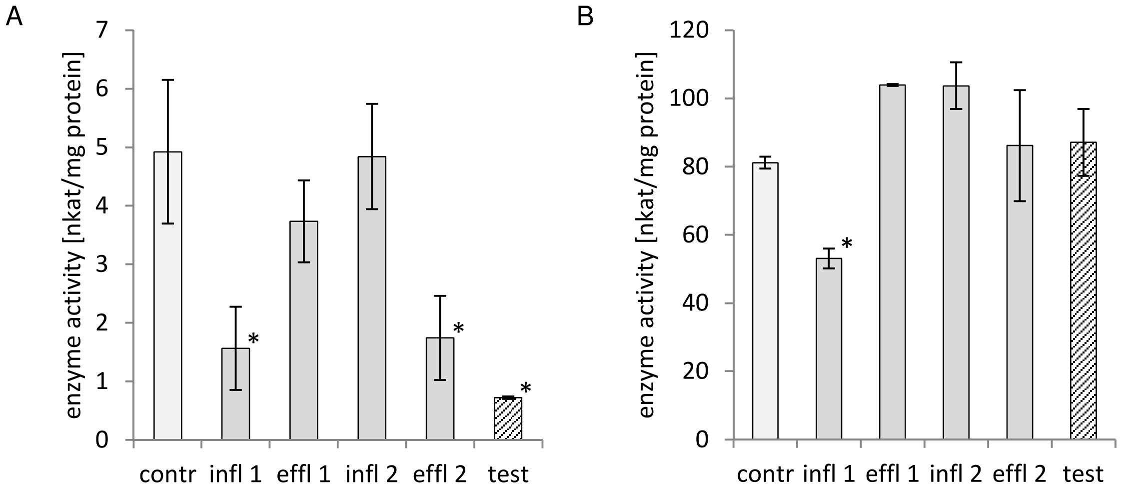
4.8 Eco-toxicity

During the microsieve trials two sampling campaigns for eco-toxicity tests were conducted. In each case two influent and two effluent samples were analyzed as well as one polymer test sample. For the polymer test sample influent water was filtered with a 10 µm filter and stocked with 450 µg/L cationic polymer in order to check the impact of possible residual polymer.

The first sampling was conducted in October 2011. During that period the microsieve was operated with real-time flow variation and load proportional dosing of PACl (1.9 to 2.9 mg/L Al) and cationic polymer (1.5 to 2.0 mg/L). The samples were taken as 24h-mixed samples. The following eco-toxicity tests were conducted:

- Luminescent bacteria test
- Enzyme activity test (glutathione S-transferase)
- Enzyme activity test (peroxidase)
- Ames test
- UMU-Chromo test
- Acetylcholinesterase (AchE) inhibition test.

With regard to mutagenicity, genotoxicity, luminescence and inhibition of acetylcholine-esterase none of the samples showed any abnormality (see Appendix C). Adverse effects were only detected in the two conducted enzyme activity tests. The activity of glutathione S-transferase (enzyme for biotransformation) was significantly decreased through three of the five samples (see Figure 61A). Due to the fact that the influent sample 1, the effluent sample 2 as well as the reference had an influence on the enzyme activity the eco-toxicological effect cannot explicitly be subscribed to coagulant or polymer residuals. In the test with peroxidase (see Figure 61B) the enzyme activity was slightly increased through the effluent sample 1 and the effluent sample 2. The only significant effect was observed in the influent sample 1 where the activity of peroxidase was decreased.



* significant deviation of the enzyme activity in comparison to the control sample

Figure 61: Results from the enzyme activity tests with glutathione S-transferase (A) and peroxidase (B) in *C. demersum* after 24h of exposure. Mean values with standard deviation.

Most of the eco-toxicological tests showed no negative effects of the analyzed water samples. The effects observed in the enzyme activity tests might have had manifold causes and cannot be directly subscribed to the treatment process.

Further research identified the following eco-toxicological tests as extra sensitive towards Aluminum and cationic polymer:

- Algae growth inhibition test (72h)
- Daphnia acute immobilization test
- Daphnia magna reproduction test.

The sampling was conducted in June 2012. During that day the microsieve pilot unit was operated with load proportional dosing of PACl and cationic polymer. Due to a tight time schedule the samples had to be taken between 9 and 11 a.m. on June 5 via grab sampling. This was directly after an ortho phosphate peak resulting in maximum dosing of coagulant (3.9 mg/L Al) and a polymer dose of 0.8 mg/L. For sampling 1 the amount of total Al was determined. In the influent 1 sample there were 0.017 mg/L total Al and 0.140 mg/L in effluent 1. For this microsieve operating mode on average there were 0.02 mg/L Al in the influent and 0.14 to 0.43 mg/L Al in the effluent with an average value of 0.29 mg/L (n=28). Table 21 gives an overview of eco-toxicological tests for aluminum with different organisms and the resulting EC 50, LOEC and NOEC values. It has to be considered that the toxicological effects of aluminium strongly depend on the aluminium species which in turn depend on pH value.

However, all five samples were analyzed undiluted and in 1:2, 1:5 and 1:10 dilutions. Furthermore, a medium control was conducted. Neither one of the original samples nor one of the diluted samples showed significant abnormalities.

With regard to the conducted tests it could not be proven that in this application PACl and cationic polymer have eco-toxicological effects to aquatic organisms.

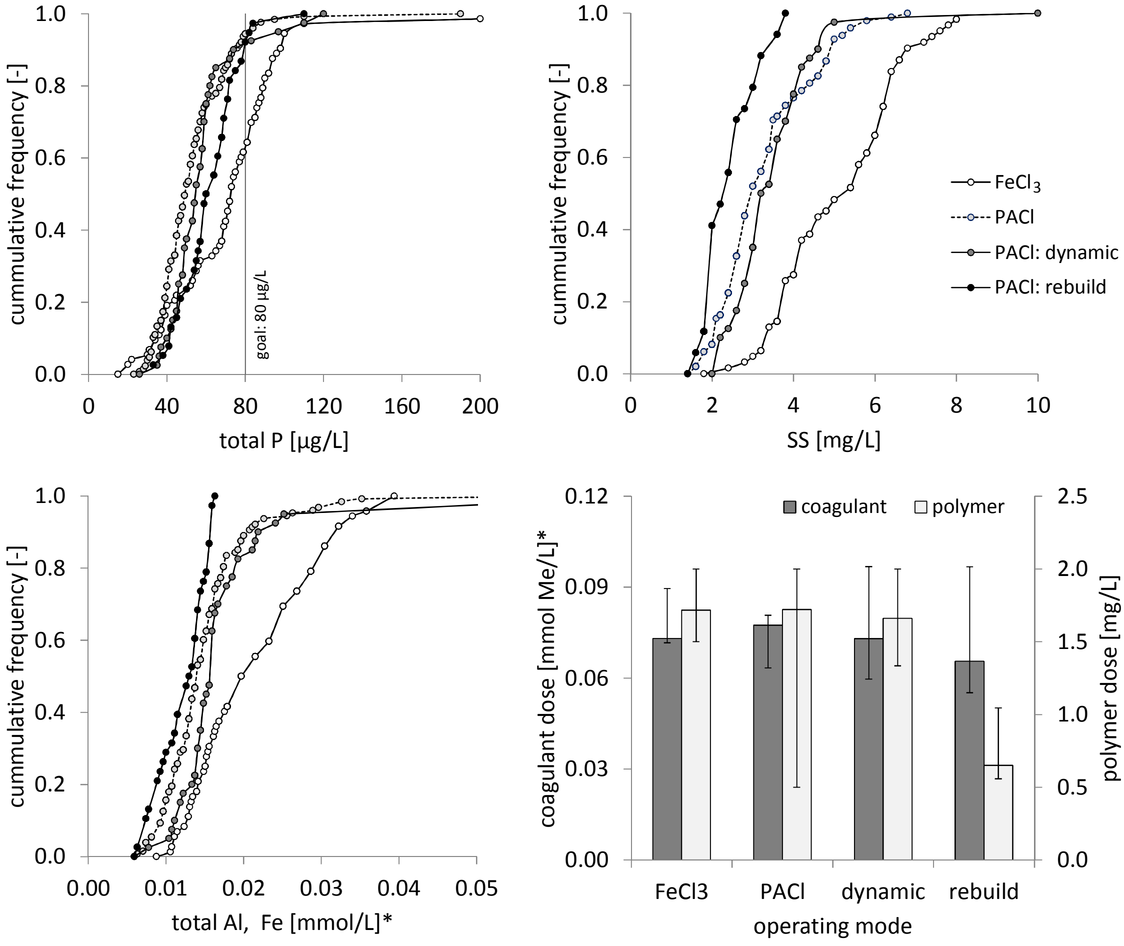
Table 22: Eco-toxicological information of Aluminum (ECHA 2012)

test	effect	duration	pH	c(Al_{total}) [µg/L]
Fish				
LC 50		96 h	6,5	7.400
LC 50		96 h	7,5	14.600
LOEC	growth rate	42 d	7	520
NOEC	growth rate	42 d	7	50
Invertebrates (Daphnia)				
EC 50	mobility	48 h	7,5 – 8,5	5.500
EC 50	mobility	48 h	8,1	2.600
EC 50	mobility	48 h	7,6 – 7,9	1.500
EC 50	mobility	48 h	7,2 – 7,4	1.900
LC 50		21 d	not specified	1.400
LOEC	mobility	21 d	not specified	324
NOEC	mobility	21 d	not specified	162 (=LOEC/2)
Aquatic algae and Cyanobacteria (green algae)				
EC 50	growth rate	96 h	7,6	570
EC 50	growth rate	96 h	8,2	460

- The conducted tests could not prove eco-toxicological effects as a result of the chemical pre-treatment with PACl and cationic polymer. Adverse effects were only observed in enzyme activity tests but they were non-systematic as an influent, an effluent and the polymer test sample were concerned.

Chapter 5
Conclusion

The pilot trials at the Ruhleben WWTP proved that the microsieve technology combined with chemical pre-treatment achieves good and reliable phosphorus removal with effluent values < 80 µg/L TP. The first three months of pilot operation confirmed the general process performance observed during the pre-trials in 2009 but also revealed a need for process optimization with regard to the removal of suspended solids and the reduction of coagulant residues. In particular, up to 60 % of the applied iron was found in the effluent. On average the total iron concentration in the effluent amounted to 1 mg/L. The high iron contents were accompanied by floc re-formation behind the microsieve in the filtrate tank and pipe. An improved performance was achieved through change from $FeCl_3$ to PACl. In the presented case, PACl gave clearly better results for the removal of phosphorus and suspended solids than $FeCl_3$ (see "$FeCl_3$" and "PACl" in Figure 62). Additionally, the occurrence of coagulant residues could be noticeably reduced. In contrast to $FeCl_3$, dosing PACl led to an improvement of the water transmittance simplifying disinfection with UV irradiation.



*0.01 mmol/L: 0.56 mg/L Fe or 0.27 mg/L Al

Figure 62: Comparison of the main optimization advances – Change of coagulant type ($FeCl_3$ vs. PACl), load proportional dosing of chemicals (PACl: dynamic) and constructional change of the hydraulic conditions (PACl: rebuild).

First approaches in the optimization of the hydraulic conditions showed that the hydraulic retention time of the coagulation tank could be reduced by 34 % (3 min at peak flow) while slightly improving the effluent water quality. Reducing the volume of the flocculation tank on the other hand proved to be problematic for the microsieve operation. Intense re-flocculation processes on the filtrate side of the microsieve and spoiling of the backwash system were the consequences. Besides a dynamic operation (real time flow variation) also load proportional dosing of PACl and polymer were introduced in order to avoid under as well as over dosing of the chemicals. Thus, the coagulant and polymer dosing could be slightly reduced while maintaining good effluent water quality (see "PACl: dynamic" in Figure 62).

The dose of cationic polymer had a significant impact on water quality and backwash time: With the initial process configuration 1.5 to 2 mg/L cationic polymer were recommended for a safe and stable operation with adequate backwash time resulting in an average polymer dose of 1.7 mg/L. However, latest results showed that a polymer dose of only 0.6 mg/L is possible without losses in water quality and filtration performance when mixing conditions are optimized. During the rebuild of the microsieve pilot unit the hydraulic retention time of the coagulation could was reduced to 26 % of the initial volume (1 min at peak flow). Due to the installation of a Turbomix^{TM} short-circuiting could be avoided. The increase of the turbulence in the flocculation tank led to an intensification of the floc formation processes that caused a severe sludge accumulation in the microsieve. Thus, the polymer dose had to be decreased in order to reduce the stickiness of the sludge. A polymer dosing range of 0.56 to 1.2 mg/L with an average dose of 0.6 mg/L was identified as optimum. The operation regime of the chemical treatment prior to the microsieve filtration is a trade-off between the energy demand for mixing and the polymer consumption. Despite the noticeably reduced polymer dose the intensified mixing resulted in improved reduction of suspended solids (mean value 2.2 mg/L) and coagulant residues in the microsieve effluent (see "PACl: rebuild" in Figure 62). The effluent phosphorus concentration was slightly increased during this trial period which can probably be subscribed to the seasonal changes of the influent water quality.

Due to the continuous operation over more than 20 months a lot of operational experiences were gained with regard to backwash behavior and cleaning intervals. The backwash of the microsieve is the most energy-intense part of the whole process as the filter panels of the microsieve are automatically cleaned with pressurized filtrate at 8 bars. For the operation of only the microsieve in a full-scale application an energy demand of 18 Wh/m^3 was estimated. The backwash time is an important parameter for the quality of the chemical pre-treatment and an indicator for the energy demand of the process. Predominantly, the backwash time correlates with the influent flow. Secondarily, the backwash time depends on the influent water characteristics and the properties of the formed flocs, especially on the resulting load of suspended solids. Due to progressing fouling of the filter panels chemical cleaning was necessary every 4 to 7 weeks. A short cleaning interval (e.g. every 4 weeks) might be beneficial as the backwash time and thus the energy demand could be kept on a lower level. In this application the microsieve produced on average 1.8 % of backwash water. The backwash water contained 580 to 1000 mg/L SS and showed excellent settling properties (SVI << 50 mL/g) and might be easily treated via returning to the primary clarifiers.

In the course of the pilot plant rebuild an UV disinfection plant was installed behind the microsieve. Like the full-scale UV unit at Ruhleben WWTP (1 m^3/sec) the pilot UV unit was designed with a fluence of 1000 J/m^2. Despite the operation at a fluence of only 730 J/m^2 instead of 1000 J/m^2, advancing lamp aging and fouling of the glass sleeves a good disinfection could be provided for a continuous operation of 7 months. During this period there were always less than 100 MPN/100 mL of *E. coli* and *Enterococci* in the effluent of the UV disinfection. After 7 months of operation a chemical cleaning of the glass sleeves had to be conducted to remove the fouling and restore the UV irradiance. The disinfection was ensured throughout the whole trial period.

As aluminum and cationic polymers are suspected to have toxic effects on aquatic organisms several eco-toxicological tests were conducted. But the tests (e.g. Algae growth, Daphnia acute reproduction, luminescent bacteria test) could not prove eco-toxicological effects as a result of the chemical pre-treatment with PACl and cationic polymer. Adverse effects were only observed in enzyme activity tests. But the observed effects were non-systematic as an influent, an effluent and the polymer test sample were concerned.

Overall, the microsieve in combination with dosing of coagulant and polymer is a robust technology with small amounts of backwash water and a low energy demand of about 21 Wh/m^3 (+ site-specific energy demand for water lifting). Microsieving, together with UV disinfection, can be an alternative to low pressure membrane filtration or dual media filtration for applications targeting phosphorus removal and disinfection, e.g. effluent polishing for sensitive areas or landscape irrigation.

Appendix A

Box plot diagram

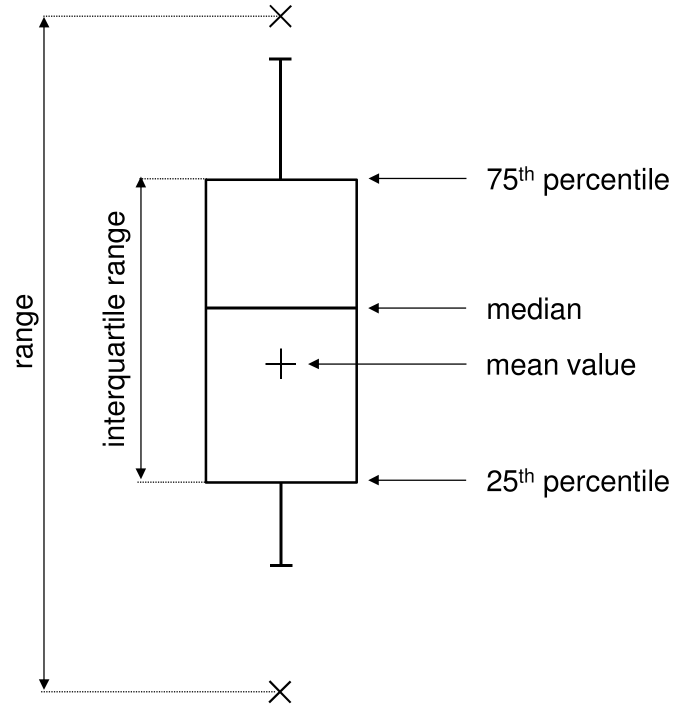


Figure 63: Explanation of the box plot diagram

Appendix B
Polymer type and dose

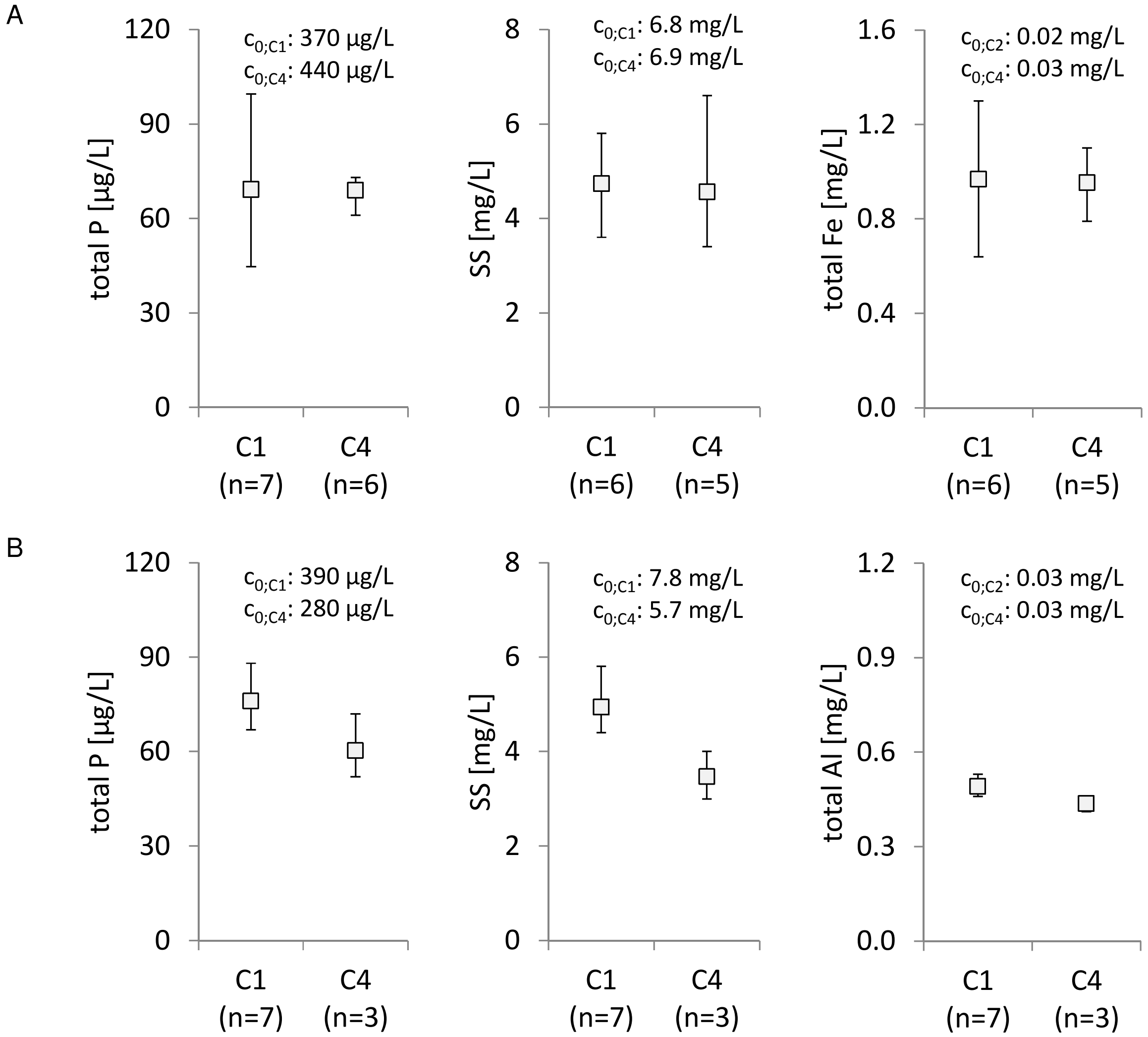


Figure 64: Variation of polymer type in the pilot plant – C1 vs. C4. A: 0.072 mmol/L Fe, 2.0 mg/L polymer C1 or C4, 10 m^3/h. B: 0.072 mmol/L Al, 2.0 mg/L polymer C1 or C4, 20 m^3/h

Appendix C

Eco-toxicity tests

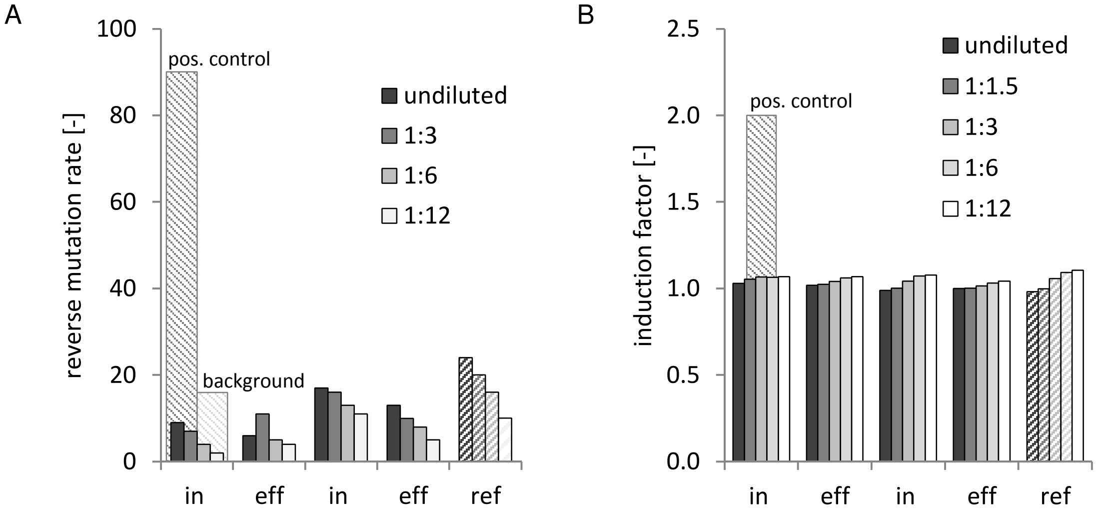


Figure 65: Results from the Ames test (A) and the UMU-Chromo test (B)

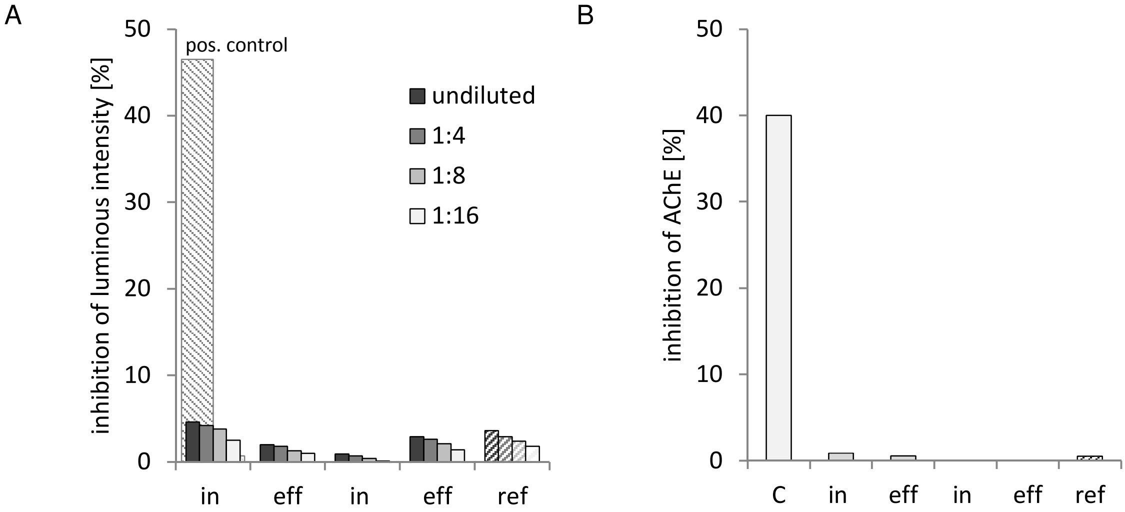


Figure 66: Results from the Luminescent bacteria test (A) and the actelycholinesterase inhibition test (B)

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