MEMBRANE BIOREACTOR TECHNOLOGY FOR DECENTRALIZED WASTEWATER TREATMENT AND REUSE

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Summary

INTRODUCTION
In most industrialized countries, urban water (wastewater, rainwater) is collected in an extended sewer system and treated in a centralized wastewater treatment plant. The original tasks of these plants - prevention of flooding and water-borne diseases - are slowly being replaced by new requirements with respect to water pollution control. Although the current system works very well, it is more and more questioned due to increasing water scarcity, open nutrient cycles, high capital cost demand and the wastewater situation in emerging countries. Alternative approaches with treatment units for single houses to several hundred residents are gaining importance, as no wastewater sewers are necessary, they can be installed stepwise, are independent of each other, and offer the possibility to reuse the effluent on-site. Because of the small spatial footprint and the good effluent quality, MBR-technology seems to be predestined for this task. However, any new system has to comply with the same requirements as the centralized system, i.e. surface and groundwater quality should not be impaired and the high hygienic standards have to be maintained.

GOALS AND METHODOLOGY
MBRs are increasingly used for treatment of domestic and industrial wastewater. But its application in private houses is still relatively new. Within the Aquamin project, a small MBR for wastewater treatment and reuse was installed in the basement of a 4-person household and operated during 3 years. The goals of this work included documentation and optimization of MBR operation, examination of different phosphate elimination processes, identification of chances and challenges for water recycling and investigation of the fate of micropollutants.

RESULTS
The effluent quality with respect to particles, organic material, pathogens and ammonium was permanently on the same or even higher level than what can be observed for large WWTP. Water reuse for toilet-flushing and gardening reduced freshwater consumption by over 35%. However, the recycling of non-degradable substances like salts, phosphate or humics accumulated in the permeate. The latter resulted in a yellow color causing aesthetic problems, which under certain circumstances needs to be reduced. This can be achieved best by adding powdered activated carbon into the MBR or by ozonation of the permeate.

With the conventional flow scheme - a primary clarifier followed by the MBR - around 50% of the nitrogen and 25% of the phosphorus are removed. Especially phosphate elimination is lower than in large treatment plants. Three processes for enhanced phosphate removal were therefore tested throughout the study.

By mixing the first tank and recycling activated sludge from the MBR, the primary clarifier was converted into a first biological compartment. The advantages of this operation were that odor problems were solved, nitrogen removal was increased to 85%, and only stabilized activated sludge was produced which could be dried on-site. In spite of the highly dynamic influent and zero influent periods, enhanced biological phosphorus removal was achieved. Dynamic modeling showed that with EBPR around 60% of the phosphate may be eliminated.

When urine is treated separately by struvite precipitation, elimination rates of over 95% in urine and around 50% overall can be achieved. No problems occurred when further treating the phosphate poor urine in the MBR. However, this technology still bears some practical challenges: especially undesirable precipitation in toilets, pipes and the precipitation reactor reduce overall performance and result in operational problems or even breakdowns. The third
process that was investigated in more detail was the electrochemical dissolution of submerged iron electrodes for phosphate precipitation in the MBR. Advantages of this method are the absence of mechanical parts and the simple control properties by adjusting voltage to the flow. The main challenges are the formation of a passivation layer on the electrodes that negatively impact process performance.

Micropollutants such as hormones or medicine residues in the influent of small WWTP are found in irregular intervals and far higher concentrations than in centralized systems. It was shown that for concentrations similar to municipal wastewater, no adaptation of the sludge to specific substances was necessary and that the removal of pharmaceuticals was in the same range as for centralized treatment. Antibiotic concentrations of 1 to 5 mg l\(^{-1}\) did not inhibit the biological activity of the activated sludge. Due to the high concentrations of pharmaceuticals in the effluent, permeate reuse for irrigation may be undesirable.

Costs for a 4-person MBR are 1.25 to 2.5 times higher than the centralized system. With units of 20 - 50 persons the specific costs will drop to be in a similar range to centralized systems. The main cost drivers are investment costs for equipment, engineering, installation and startup. Automatic failure detection and remote control minimize maintenance requirements and therefore operating costs.

**CONCLUSIONS & OUTLOOK**

Small WWTP with MBR technology for treatment of domestic wastewater are robust and reliable systems. On-site water recycling is encouraged due to the high effluent quality. Only enhanced P-removal cannot yet be guaranteed. Due to this advantage, MBRs are increasingly applied in remote and water-scarce locations like mountains or arid regions. Due to the high energy consumption and relatively higher costs, the technology is not yet an acceptable alternative to the conventional system with sewers and centralized WWTP. However, as the development started only recently and the specific costs per person decrease with larger units, such systems will certainly be applied more frequently, especially in fast growing urban regions of emerging and developing countries, in the future.
Zusammenfassung

EINLEITUNG

ZIELE UND VORGEHEN

RESULTATE
Die Ablaufqualität bezüglich partikulärer und organischer Stoffe, sowie für Ammonium und Keime war permanent auf gleichem bis besserem Niveau als bei grossen Kläranlagen. Durch das Recycling konnte der Wasserverbrauch um über 30% reduziert werden, es wurden aber nicht abbaubare Stoffe wie Salze, Phosphat oder Huminstoffe aufkonzentriert. Letztere führen zu einer gelben Färbung des Permeats, die unter bestimmten Umständen reduziert werden muss. Die einfachste Möglichkeit ist dabei der Einsatz von Pulveraktivkohle im MBR, aber auch eine Behandlung mit Ozon ist möglich, wenn auch aufwändig.
Durch Mischen des ersten Behälters und Rückführen von Belebtschlamm wurde die Vorklärung in ein biologisches Becken umgewandelt. Dadurch wurden rund 80% des Stickstoffs entfernt, die Geruchsprobleme verschwand. Der entstehende gerucharme, biologisch stabilisierte Schlamm konnte vor Ort getrocknet werden. Zudem konnte trotz des dynamischen Zulaufs und Stillstandspérioden eine erhöhte biologische P-Elimination erreicht werden. Eine dynamische Modellierung zeigte aber, dass damit maximal 60% des Phosphors entfernt werden können. Mit der separaten Behandlung des Urins mittels Struvitfällung wird das Phosphat aus dem Urin zu über 95% und die Zulauffracht zur Kläranlage um rund 50%

Spurenstoffe wie Medikamentenrückstände oder Hormone fallen in Kleinkläranlagen in höheren Konzentrationen und in unregelmässigeren Abständen an als auf grösseren ARA. Es konnte gezeigt werden, dass im Konzentrationsbereich von kommunalem Abwasser eine vergleichbare Elimination erzielt werden kann und dass die meisten abbaubaren Substanzen ohne längere Anpassungszeit abgebaut werden. Antibiotikakonzentrationen im Bereich von 1 - 5 mg l\(^{-1}\) beeinflussen die biologische Aktivität des Belebtschlammes nicht. Trotzdem muss zeitweise mit sehr hohen Ablaufkonzentrationen von Medikamentenrückständen oder Hormonen gerechnet werden, was bei der Bewässerung des Gartens oder von Gemüse unerwünscht ist.

Die Kosten für eine Kleinkläranlage für 4 Personen sind im Vergleich mit dem zentralen System um einen Faktor 1.25 bis 2.5 höher, Einheiten für 20-50 Personen werden aber konkurrenzfähig. Der grösste Kostenpunkt sind die Investitionskosten für Material, Engineering und Inbetriebnahme. Mittels Selbstdiagnose und Fernüberwachung können der Wartungsaufwand und damit die Betriebskosten minimiert werden.

SCHLUSSFOLGERUNGEN UND AUSBlick

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### Abbreviations

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<th>Definition</th>
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<tbody>
<tr>
<td>AOB</td>
<td>Ammonia oxidizing bacteria</td>
</tr>
<tr>
<td>AOX</td>
<td>Adsorbable organic halogens</td>
</tr>
<tr>
<td>b</td>
<td>Endogenous respiration rate</td>
</tr>
<tr>
<td>BNR</td>
<td>Biological nutrient removal</td>
</tr>
<tr>
<td>BSE</td>
<td>Bovine Spongiform Encephalopathy (mad-cow disease)</td>
</tr>
<tr>
<td>C</td>
<td>Concentration</td>
</tr>
<tr>
<td>CAS</td>
<td>Conventional activated sludge treatment</td>
</tr>
<tr>
<td>CE</td>
<td>Current efficiency</td>
</tr>
<tr>
<td>COD</td>
<td>Chemical oxygen demand</td>
</tr>
<tr>
<td>d</td>
<td>Interspace distance between two plates</td>
</tr>
<tr>
<td>$E_{eq}$</td>
<td>Equilibrium potential difference</td>
</tr>
<tr>
<td>EBPR</td>
<td>Enhanced biological phosphorus removal</td>
</tr>
<tr>
<td>EE2</td>
<td>$17\alpha$-Ethinylestradiol</td>
</tr>
<tr>
<td>F</td>
<td>Faraday constant (96485 As mol$^{-1}$)</td>
</tr>
<tr>
<td>F/M</td>
<td>Feed to microorganism ratio</td>
</tr>
<tr>
<td>G</td>
<td>Mixing strength</td>
</tr>
<tr>
<td>H</td>
<td>Hydraulic height</td>
</tr>
<tr>
<td>HRT</td>
<td>Hydraulic retention time</td>
</tr>
<tr>
<td>i</td>
<td>Current density</td>
</tr>
<tr>
<td>I</td>
<td>Electric current</td>
</tr>
<tr>
<td>$i_P$</td>
<td>Phosphorus content of the activated sludge</td>
</tr>
<tr>
<td>$K_x$</td>
<td>Half-saturation constant of Monod kinetics for substance $x$</td>
</tr>
<tr>
<td>$K_d$</td>
<td>Solid water distribution coefficient for micropollutants</td>
</tr>
<tr>
<td>$k_{biol}$</td>
<td>Biological transformation coefficient for micropollutants</td>
</tr>
<tr>
<td>$k_{L,a}$</td>
<td>Aeration coefficient</td>
</tr>
<tr>
<td>LOQ</td>
<td>Limit of quantification</td>
</tr>
<tr>
<td>MBR</td>
<td>Membrane bioreactor</td>
</tr>
<tr>
<td>n</td>
<td>Number of persons living in the house</td>
</tr>
<tr>
<td>N</td>
<td>Nitrogen</td>
</tr>
<tr>
<td>$N_{air}$</td>
<td>Energy requirement for the air compressor</td>
</tr>
<tr>
<td>$N_{PP}$</td>
<td>Energy requirement for the permeate pump</td>
</tr>
<tr>
<td>NOB</td>
<td>Nitrite oxidizing bacteria</td>
</tr>
<tr>
<td>ORP</td>
<td>Oxidation reduction potential</td>
</tr>
<tr>
<td>P</td>
<td>Phosphorus</td>
</tr>
<tr>
<td>$P_e$</td>
<td>Phosphorus concentration in the MBR effluent</td>
</tr>
<tr>
<td>$P_{in}$</td>
<td>Phosphorus concentration in the primary clarifier effluent</td>
</tr>
<tr>
<td>$P_{in,raw}$</td>
<td>Phosphorus concentration in the influent</td>
</tr>
<tr>
<td>$P_{PS}$</td>
<td>Phosphorus load in primary sludge</td>
</tr>
<tr>
<td>PAO</td>
<td>Phosphorus accumulating organisms</td>
</tr>
<tr>
<td>PC</td>
<td>Primary clarifier</td>
</tr>
<tr>
<td>PE</td>
<td>Population equivalent</td>
</tr>
<tr>
<td>PPCP</td>
<td>Pharmaceuticals and personal care products</td>
</tr>
<tr>
<td>Pe</td>
<td>Person</td>
</tr>
<tr>
<td>Perm</td>
<td>Permeability</td>
</tr>
<tr>
<td>Q</td>
<td>Water flow</td>
</tr>
<tr>
<td>$Q_{12}$</td>
<td>Water flow from reactor 1 to reactor 2</td>
</tr>
<tr>
<td>$Q_{21}$</td>
<td>Water flow from reactor 2 (MBR) to reactor 1</td>
</tr>
<tr>
<td>Qair</td>
<td>Air flow</td>
</tr>
</tbody>
</table>
Abbreviations

Qe l d\(^{-1}\) Effluent flow (from MBR)
Q_{ES} l d\(^{-1}\) Excess sludge flow
Q_{garden} l d\(^{-1}\) Permeate used for irrigation purposes
Q_{in} l d\(^{-1}\) Water flow to the treatment plant (without recycling)
Q_{Perm} l d\(^{-1}\) Permeate flow (across the membrane)
Q_{WC} l d\(^{-1}\) Water used for toilet-flushing
S g m\(^{-3}\) Concentration of dissolved species
SBR Sequencing batch reactor
SPE Solid phase extraction
t_{Pump} min Runtime of the permeate pump
TMP bar Transmembrane pressure
TSS Total suspended solids
U V Voltage
UR Urine reactor
VSS Volatile suspended solids
WWTP Wastewater treatment plant
X_A g m\(^{-3}\) Concentration of nitrifiers
X_i ng l\(^{-1}\) Concentration of substance i sorbed to the sludge
X_{MBR} g SS l\(^{-1}\) Sludge content of the MBR
Y' Apparent yield

Greek symbols

\(\beta\) \text{ mol}_{Fe} / \text{ mol}_{P} \text{ mol}^{-1} \quad \text{Molar ratio of iron dosed per phosphate}
\(\rho_{\text{anox}}\) g m\(^{-3}\) d\(^{-1}\) \quad \text{Process rate for anoxic processes}
\(\rho_{W}\) kg m\(^{-3}\) \quad \text{Water density}
\(\eta\) - \quad \text{Efficiency}
\(\sigma\) mS cm\(^{-1}\) \quad \text{Conductivity}
INTRODUCTION

GENERAL INTRODUCTION
Introduction

In industrialized countries, the system for wastewater disposal is based on extensive sewer systems and centralized treatment plants. This system has been developed and improved over decades. However, increased demands on elimination rates, emerging problems such as water scarcity, awareness about open water and nutrient cycles and the limited transferability to developing countries put the existing system into question. Alternatives are consequently sought, and several have been proposed. Most of them are based on either source separation of single wastewater streams (e.g. blackwater, graywater, urine), or on decentralized approaches, or a combination of both. Keywords used in this context include DESAR (DEcentralized SA nitation and Reuse, e.g. Wilderer, 2004) and Ecosan (ECOlogical SA nitation, e.g. Langergraber and Muellegger 2005), although the latter may be misleading by suggesting that the current system fails in terms of ecology. Of course, any alternative system has to comply with current standards of hygiene, environmental impact and resource usage.

One of the crucial points when suggesting a decentralized system for urban catchments is the scarce availability of land, therefore requiring a system with a small footprint. From this point of view, membrane bioreactor (MBR) technology would be a good choice. MBRs are increasingly used for centralized and industrial as well as decentralized wastewater treatment. The downscaling of this technology has not yet been scientifically investigated and only little data is available about its performance in such situations.

The central theme of this thesis was to thoroughly test and investigate an alternative approach to decentralized wastewater treatment based on MBR technology. The challenges and differences compared to large treatment plants are manifold, including influent dynamics, matrix variability, short-term peak loads of single substances (e.g. medicines), process reliability and monitoring, or the applicability of well-known processes to different circumstances.

The main contribution of this thesis is to prove the applicability of MBR technology to wastewater treatment on a household scale, and especially to assess the feasibility of three phosphate removing processes (enhanced biological phosphate removal (EBPR), urine source separation with struvite precipitation and electrochemical iron dosing) in this context, to identify the fate of the micropollutants as well as the potential for on-site water recycling.

BACKGROUND

GOALS OF URBAN WATER MANAGEMENT

Historically, urban water management has three goals: to provide a safe supply of drinking water to the urban population, to drain excess water (rainwater and wastewater) from urban catchments and to prevent water-borne diseases (Harremoës, 1999). In industrialized countries, these goals were achieved by the introduction of centralized water supplies and sewerage towards the end of the 19th century. The basic principles of the drinking water system were to draw water from a safe source as well as centralized purification and distribution in pressurized pipes to prevent contamination, whereas excess water disposal was based on an extensive sewer system and discharge into surface water bodies. These systems have been steadily extended and optimized. One of the most important optimization steps was the introduction and stepwise improvement of wastewater treatment due to the decreasing quality of the surface water. Thanks to these developments, the cities of industrialized
countries have been virtually free of water-borne diseases and the surface water quality is generally very high. The history of centralized water and wastewater systems has thus been a success story.

**CENTRALIZED WASTEWATER TREATMENT**

Requirements on wastewater treatment have been increasing over the last 50 years. At the beginning, the main goal was the elimination of particulate and organic material, followed by the implementation of nitrification in the 1970s, phosphorus removal in the 1980s and nitrogen removal (denitrification) in the 1990s. All these requirements could be realized by building and upgrading wastewater treatment plants. With the new millennium, micropollutants (e.g. Ternes, 1998) and disinfection processes moved into the focus of researchers and practitioners around the world. Moreover, many sewers and wastewater treatment plants are reaching the end of their operating life, inducing high investment costs to repair, maintain and upgrade existing systems. The necessity of high water flows for sewer operation, contamination caused by combined sewer overflows, infiltration to and exfiltration from sewer systems are further problems affecting centralized wastewater systems. These challenges, combined with increasing water scarcity, nutrient recycling considerations and the wastewater situation in the large cities of developing countries question the centralized system and call for alternatives. Among the alternatives discussed are source separation (Larsen and Gujer, 1996) and decentralized systems (Otterpohl *et al.* 2003, Wilderer and Schreff 2000).

**DECENTRALIZED WASTEWATER TREATMENT**

In a decentralized approach, the wastewater is treated at or close to the point of origin. Depending on the size of the system, some kind of collection system has to be provided, but the distances are rather short compared to centralized systems. The effluent may be reused on-site for different purposes (e.g. toilet-flushing, irrigation), used for groundwater replenishment, or discharged into nearby surface water (Figure 1). Further advantages of decentralized systems are: they can be built incrementally so that increasing demand can be addressed as it occurs (Fane and Fane, 2005), and the systems can adapt quite rapidly to new technologies, as their lifecycles are shorter than for centralized infrastructures. However, any alternative to the centralized system has to fulfill certain preconditions:

It has to comply with the basic goals of the urban water management system: a safe supply of drinking water, quick disposal of excess water, prevention of water-borne diseases and no deterioration of the quality of the receiving water.

- The system must thus provide advanced treatment, be robust and reliable, as well as easy to operate and control.
- The system needs to be cost-effective. Treatment plants therefore need to be mass-produced and sold in large quantities.
Introduction

The centralized systems of industrialized countries with their highly developed infrastructures will not be replaced by a decentralized approach in the near future. However, in regions with a lack of wastewater infrastructure, decentralized systems may be a real alternative, e.g. Eastern Europe, where only 30-60% of the population are connected to centralized wastewater treatment plants (Figure 2), the United States with approximately 25% of the population being served by small sewage treatment plants (WERF, 2007), or fast-growing cities in the developing countries.

CHALLENGES OF DECENTRALIZED WWTP

A wide spectrum of technologies is applied at isolated locations. Overviews of available technologies are given by Green and Ho (2005) and Boller (1997). Basically, one can differentiate between so called natural (low-tech) and technical (high-tech) systems. Natural systems make use of natural processes with only a minimum of technical installations and are usually characterized by a high land requirement and low energy input (Galvao et al., 2005). They include pond systems, reed beds, soil infiltration and wetlands. Technical systems are based on man-made facilities and technical installations (aeration systems, pumps). They are usually characterized by a lower land requirement and a higher energy input, and include activated sludge systems, rotating biofilm contactors, sequencing batch reactors and biofilters. Natural systems suffer from a limited possibility of engaging with and changing operational parameters, while technical systems usually offer several degrees of freedom (e.g. oxygen supply).
The choice of process depends strongly on specific local circumstances: climate, available area, wastewater load, water and energy availability or effluent requirements. In urban areas of developing countries, for example, systems with a low area and energy input are needed, whereas a high effluent quality is preferred in rural areas close to nature reserves, and the most important feature in arid zones may be water recycling.

With respect to the effluent quality, most natural and technical systems are basically capable of complying with the effluent standards for organic matter and ammonium (e.g. Vilpas and Santala 2007, Luederitz et al. 2001, Kunst et al. 2000), but phosphate removal is often neither required nor achieved. However, they often fail due to inadequate design or insufficient maintenance. None of the above-mentioned technologies can act as a real alternative to the centralized system currently used, because of their high land requirements, poor effluent quality or high energy consumption.

MEMBRANE BIOREACTORS IN WASTEWATER TREATMENT

MBRs have been used for wastewater treatment since the 1960s, mainly on an industrial scale. However, only since the 1990s and the development of immersed membranes has their application become more widespread. Wastewater treatment with MBRs basically relies on the activated sludge system, but the activated sludge is separated from the treated wastewater by filtration instead of sedimentation. The advantages are a higher effluent quality because of complete particle retention (including retention of microorganisms) and a smaller spatial footprint. These advantages are set off against their higher energy consumption and operational costs due to chemical cleaning and membrane lifespan.

Due to their advantages and optimization potential, MBRs are an interesting option for decentralized systems (Fane and Fane, 2005). The short distances between wastewater production and the treatment facility allow easy reuse of the permeate. Several companies around the world have developed technologies for decentralized wastewater treatment with MBRs. However, despite the wide dissemination of this technology, only little data about its performance at household level is available, and performance evaluation is mostly based on data provided from suppliers and public authorities, or extrapolations from lab experiments. Data obtained from synthetic or degritted wastewater from centralized treatment plants may not be comparable to that applicable to raw household wastewater. Closing this knowledge gap is essential if MBRs are to be considered a real alternative.

THE AQUAMIN PROJECT

The Aquamin project was initiated by the environmental agency of Solothurn canton in order to assess the strengths and weaknesses of MBR technology for decentralized wastewater treatment. Within this project, a small MBR was operated over three years in the basement of a detached house with four residents. The basic features of the pilot plant were the MBR treating all household wastewater, a simple sludge collecting system, a urine source separation system, a separate rainwater collection system and a water recycling scheme (Figure 3).
Introduction

Collection and infiltration of rainwater

Rainwater rainwater

Primary clarifier

Excess sludge

Storage tank

Excess sludge

Infiltration

Figure 3: Overview of the single-family house Aquamin. More details can be found in Chapters 1 and 2.

QUESTIONS AND OBJECTIVES

PHOSPHATE REMOVAL

While the elimination of organic matter or nitrification in small wastewater treatment systems is fairly easy, extended denitrification and phosphate removal pose a serious problem. The latter is a precondition for any alternative system competing with the centralized system. In centralized treatment plants, phosphate is either removed by enhanced biological phosphorus removal (EBPR) or by precipitation with iron or aluminum. These methods are not yet widely applied in small treatment plants. Precipitation is problematic due to dosing difficulties and increased sludge production. EBPR has not yet been tested but seems difficult due to the highly dynamic influent. Post-treatment in adsorption beds with high phosphate affinity is also feasible, but requires large volumes. There is a need to devise efficient methods for phosphate removal in small WWTP. Three methods were tested during this project: EBPR, urine source separation with separate treatment and electrochemical iron dosing. The following questions were addressed:

1. What is the P-removal capacity of a conventional MBR process? (Chapter 1 and 3)
2. Is EBPR feasible in a small MBR? What happens during vacation periods? (Chapter 3)
3. Could urine source separation and treatment be a serious alternative in private homes and does the combination of urine and MBR treatment work? (Chapter 4)
4. Is the electrochemical dosing of iron an alternative to conventional chemical (e.g. ferric chloride) dosing? (Chapter 5)

WATER REUSE

On-site systems are well suited for water recycling, as only short pipes are necessary. Depending on the purpose of the reuse, different requirements on water quality must be met. Pathogens, salinity and organic micropollutants are of major concern when treated wastewater is used in agriculture. In the Aquamin project, the permeate was reused for toilet flushing and gardening. The following questions were asked:

5. Which substances accumulate in such a recycling scheme, and which of them cause problems? (Chapters 6 and 7)
6. What is the water-saving potential, and do the benefits outweigh the costs? (Chapters 1 and 2)
MICROPOLLUTANTS

The occurrence of pharmaceutical residues or other micropollutants in decentralized systems differs from that in centralized ones. While the influent load in large catchments depends on the average consumption and a more or less constant load can be expected, it is zero during normal operation and reaches very high levels over short periods. The activated sludge cannot therefore be adapted to biologically transform such substances. By conducting two sampling campaigns with the addition of micropollutants, the fate of these substances was studied:

7. To what extent are micropollutants removed in a non-adapted system? (Chapter 6)

OPERATION OF A SMALL MBR IN A PRIVATE HOUSE

Small wastewater treatment plants are usually applied at isolated locations on relatively large properties, with the plant itself being placed at the edge of the property. The residents therefore have almost no contact with the treatment plant. The situation is completely different when an MBR is placed in the basement of a private house. In practice, however, the residents are not really interested. The following questions are of interest in this context:

8. Which properties influence acceptance? (Chapter 2)

9. Which measures for monitoring and maintenance are appropriate? (Chapter 2)

METHODOLOGY

The main focus of this work was on the continuous operation of the pilot plant and the implementation of different processes on-site. In addition, several technologies were investigated in the laboratory, partly before implementation and partly in parallel. In order to test the various processes, the treatment plant was retrofitted several times and the operating mode of several units was changed over time. At the beginning, the first tank was used as a primary clarifier, followed by a “trial and error” period of almost one year to reduce bad odors and introduce EBPR. After two years, the urine treatment was put into operation and ran for 10 months. Towards the end of the project, electrochemical phosphorus removal was tested in the MBR and two sampling campaigns were conducted to investigate the fate of the micropollutants. The operational conditions of the treatment plant are summarized in Figure 4.

<table>
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<th>Year</th>
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<th>2007</th>
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<tbody>
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<td></td>
<td></td>
<td>1-3</td>
<td>4-6</td>
<td>7-9</td>
<td>10-12</td>
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<tr>
<td></td>
<td>Primary clarifier</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>Sludge recycling (sieve)</td>
<td></td>
<td></td>
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<td></td>
<td>Urine treatment</td>
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<tr>
<td></td>
<td>Electrocoagulation</td>
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<tr>
<td></td>
<td>Micropollutants</td>
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</tbody>
</table>

Figure 4: Operational conditions of the Aquamin pilot plant during the operational period. See the following chapters for more details.

OUTLINE OF THE THESIS

The thesis is structured as follows:

Chapter 1 gives an overview of the treatment plant. The water, sludge, nutrient and heavy metal fluxes are assessed and the consequences for their treatment elucidated.

“Any technical method has to work reliably. That is a basic requirement” (Harremoës, 1998). For decentralized technologies, acceptance and costs are equally important aspects and requirements for dissemination. These subjects are discussed in Chapter 2.
In Chapter 3 the nutrient fluxes are assessed in more detail. While nitrogen can easily be converted biologically into nitrogen gas, phosphorus removal remains a challenge. Dynamic simulations have shown that small systems with a highly dynamic influent and irregular sludge removal can provide conditions suitable for enhanced biological phosphorus removal.

A widely discussed technology for phosphate recovery from wastewater is urine source separation and follow-up treatment. In the Aquamin project, a combination of urine source separation with struvite precipitation and biological wastewater treatment on a household scale was implemented in practice for the first time. The goal was to identify the phosphate removal capacity and its consequences for biological treatment. Chapter 4 summarizes the opportunities and risks as well as the experience gained in this experiment.

Chemical phosphorus removal by precipitation with iron or aluminum is the technology applied most widely in centralized wastewater treatment. Although iron or aluminum solutions can in principle be dosed in small treatment plants, this process is hampered by high maintenance needs and difficulties in ensuring correct ratios. In Chapter 5, a novel method of phosphate precipitation by means of electrochemical iron dosing is described.

In Chapter 6 the fate of the micropollutants is discussed. The influent load of pharmacological residues depends on the health of the residents, and expected concentrations can be far higher than in centralized treatment plants. Moreover, it is not known whether specialized microorganisms are responsible for the degradation of these substances or whether this is inherent to the natural biocenosis. The elimination by sorption and degradation as well as the degradation rates were estimated on the basis of two sampling campaigns spiked with micropollutants.

Chapter 7 addresses water quality aspects in the reuse of treated wastewater. Besides pathogen occurrence, the most important problem observed was the natural color of the permeate. The consequences of colored water and possible methods of color removal are shown.

Finally, the most important findings are summarized in the Conclusions.

This thesis is structured as a paper dissertation, i.e. it consists of a number of scientific articles, except for the first two chapters. Some repetitions are consequently unavoidable, particularly in the introductory sections and the descriptions of treatment plants in individual chapters.

REFERENCES

http://epp.eurostat.ec.europa.eu/portal/page?_pageid=0,1136239,0_45571447&_dad=portal&_schema=PORTAL


CHAPTER 1

WATER, NUTRIENT AND HEAVY-METAL FLUXES IN A DETACHED HOUSE WITH ON-SITE WASTEWATER TREATMENT

Thesis Chapter
Water, nutrient and heavy-metal fluxes in a detached house with on-site wastewater treatment

ABSTRACT
Within the Aquamin project, an MBR for wastewater treatment of a 4 person family was operated during three years in the basement of their house. Recycling of the treated effluent for toilet-flushing and use of rainwater for laundry resulted in 40% freshwater savings. Small treatment plants are characterized by highly dynamic and concentrated wastewater influent. Therefore, they need to provide enough buffer capacity and reliable operation under different loading conditions. The effluent concentrations were well below the threshold values and even similar to large wastewater treatment plants, except for phosphate. Operation of the first tank as primary clarifier resulted in lower excess, but primary sludge production and in nitrogen and phosphorus removals of 50% and 25%, respectively. By mixing the first tank and recycling sludge from the MBR, higher nutrient removal of up to 85% for nitrogen and 50% for phosphorus were achieved. The overall sludge production was smaller, and as only stabilized sludge is produced, it could be stored and dried on-site. If the sludge is used as fertilizer, an area of 500 m² is necessary for a 4-person household in order to prevent over-fertilization with phosphate.

INTRODUCTION
The purification performance of a wastewater treatment plant is best estimated on the basis of mass balances. Its operation cannot be effectively assessed by considering only the effluent concentrations and comparing them with the emission thresholds. This is particularly the case for small wastewater treatment plants, where usually only little data is available and the effluent concentrations depend very strongly on the water flows.
In this chapter, the performance of a small membrane bioreactor treating the domestic wastewater of a 4-person household is assessed on the basis of mass balances. In a first step, the water flows are presented, and these analyses are used as a basis for discussing the sludge production and quality as well as the nutrient balances.

MATERIAL & METHODS
WASTEWATER TREATMENT PLANT
The wastewater treatment plant was in operation for 3 years in the basement of a detached house. It was designed for 4 persons, with two to four persons as permanent residents. Higher (visits) and lower (vacation) occupancies occurred during short periods, but no occupancy data was collected. The tenants usually spent the day at their workplace or at school.
The wastewater treatment plant consisted of two stainless steel tanks with a maximum volume of 1.5 m³. As the house was located within an urban area, a connection to the sewer system already existed. For safety reasons, an emergency spill to the sewer was placed in the first reactor. The first tank was either operated as a conventional primary clarifier or as a biological reactor with mixing by coarse-bubble aeration and sludge recycling. Figure 1 shows the flow scheme of the treatment plant.
The second tank was always operated as a membrane bioreactor. At startup, it was filled with activated sludge from a conventional wastewater treatment plant. The oxygen input was secured by fine-bubble aeration. A flat sheet membrane with an area of 4 m², 40 nm pore size and 6 mm (3 mm during the first year) plate interspace was installed for the separation of the
permeate from the activated sludge. Membrane clogging was prevented by coarse-bubble cross-flow aeration, which was independent of the fine-bubble aeration system. The permeate was pumped into a storage tank of 1.5 m$^3$ placed outside the building. One part of the permeate was reused for toilet flushing or irrigation, the remainder was infiltrated to the groundwater, partly after passing a biotope. Rainwater from the roof was also collected in a storage tank with a volume of 6.3 m$^3$. The rainwater was used for laundry and the excess rainwater was infiltrated with the permeate.

Excess activated sludge was pumped into a filter bag placed in a manhole outside and air-dried. Water dripping from the filter bag was collected and directed back to the MBR. Permeate withdrawal started when the water level in the MBR rose to a certain height and stopped after decreasing to a minimum level. During the night, the plant aeration and pumps stopped for 7 hours due to noise emissions.

**Operation with Primary Clarifier**

At startup and during two periods of 2-3 months, the first reactor was operated as a primary clarifier with a hydraulic residence time of 3-5 days (Figure 1). Within a short time, a sludge blanket formed on the surface leading to strong odor problems both in the basement and outside, although the reactor was airtight. No satisfactory solution was found to handling the primary sludge. As it was not stabilized, transport to and further treatment in a conventional wastewater treatment plant would be necessary.

**Operation with Two Biological Tanks**

The first reactor was intermittently aerated for mixing, and activated sludge from the MBR was recycled by an air-lift pump (Figure 1). By applying this strategy, putrescent conditions and the formation of malodorous substances in the first tank were reduced. Clogging of the membrane by particles was prevented by installing a sieve. A second air-lift pump was installed in the first reactor in order to better control the dosage of the wastewater to the MBR and to buffer hydraulic peak loads. As only two air compressors were available for the fine and coarse-bubble aeration of the MBR, the two air-lift pumps and the coarse-bubble cleaning of the sieve (which also produced the turbulence for mixing of the first reactor), there were some constraints on the control strategies, e.g. mixing of the first reactor was always combined with pumping of activated sludge from the first to the second reactor.

**Measurements**

Grab samples of the activated sludge (MBR), the effluents of the first tank and of the MBR were taken once a week (Figure 1). The samples were transported to the laboratory and analyzed within a short time. The measured parameters and methods are listed in Table B-1 in Appendix B. In addition, several operational parameters were monitored and data were stored electronically (Table B-2, Appendix B). This data could be retrieved through remote control. Samples of the excess sludge were taken periodically and analyzed for dry matter content and heavy metals.
Water balance, daily variation

Measurements of the freshwater, permeate and rainwater consumption and of the water levels in the two reactors were used for the water balance. They had to be checked for plausibility and compared to each other. The permeate consumption measurements did not differentiate between use for toilet flushing and gardening and no daily variation was available. Freshwater consumption did not always correspond to the time of wastewater production. The verification was performed during periods with only freshwater consumption: the daily variations were verified by comparing the fresh and rainwater consumption data (from water meters) with wastewater production data (from water level measurements, pumping capacity and times), whereas the overall water consumption was verified by the phosphorus mass balance. Distribution into the various water consumption categories was based on the criteria listed in Table B-3 (Appendix B).

Mass balances

The main problem in setting up the mass balances was that the influent could not be sampled for chemical analysis. The influent concentrations and loads had to be estimated through literature analysis and correlations with other data. For operation with the primary clarifier, mass balances could be set up for the MBR. By estimating the removal rates for the primary clarifier, this was done for the whole system. As no influent data were available for operation with sludge recycling, the influent concentrations were obtained from the calculated influent loads per person for the primary clarifier operation.
P-balance for primary clarifier operation

The phosphorus balance was the only redundant mass balance. It allowed the water flow and phosphate concentration measurements to be verified. It was based on the water consumption measurements, the P-concentrations in the effluent of the primary clarifier and the MBR and the concentrations and mass fluxes of the excess sludge. The P-balance for the MBR was set up as follows for the period from October 5th to December 6th 2005:

\[ \sum (Q_{in} + Q_{WC}) \cdot P_{in} = \sum Q_{ES} \cdot i_{P} \cdot X_{MBR} + \sum Q_{e} \cdot P_{e} \]

(Eq. 1)

With primary clarifier operation, the daily variations in P-concentration were only marginal, i.e. the weekly measurements of the influent and effluent concentrations were sufficiently precise. For the balance over the whole treatment plant, the P-load over the wastewater recycling and the losses in the primary clarifier had to be considered. For the latter, sedimentation of 10% was estimated on the basis of data from septic tanks (Montangero and Belevi, 2007) and conventional WWTP. The recycling was estimated over the water balance. The final mass balance for the whole system was:

\[ \sum Q_{in} \cdot P_{in,raw} = \sum Q_{ES} \cdot i_{P} \cdot X_{MBR} + P_{PS} + \sum (Q_{e} - Q_{WC}) \cdot P_{e} \]

(Eq. 2)

P-balance for operation with sludge recycling

During the period with primary clarifier operation, 4 persons lived in the house. The influent load per person \( (P_{in,raw,Person,d}) \) was calculated and then used as a basis for operation with biological pretreatment. The P-balance for that operation mode was set up for the period from April 19th to November 15th 2006. The total influent load was calculated as the sum of the P-losses (effluent, excess sludge, storage). The number of persons in the house was then estimated according to:

\[ n = \frac{\sum V_{ES} \cdot P_{ES} + \sum (V_{e} - V_{WC}) \cdot P_{e}}{P_{in,raw,Person,d} \cdot \Delta t} \]

(Eq. 3)

COD balance

The approach for setting up the COD balance was similar to the phosphorus balance. First, the average COD-influent per person was estimated for primary clarifier operation. The sedimentation in the primary clarifier was crucial for estimating the COD influent load. A COD removal efficiency of 50% was assumed on the basis of data from septic tanks (EPA, 2002). The influent load for the second period was then estimated from the P-balance and according to Equation 4 (in [g d+]).

\[ COD_{in} = n \cdot COD_{in,raw,Person,d} \]

(Eq. 4)

Nitrogen balance

The approach for setting up the nitrogen balance was the same as for the COD balance. The elimination rate of the primary clarifier was estimated to be 15% (Montangero and Belevi, 2007).

Heavy metals

The heavy metal content of the excess sludge was originally measured only in order to evaluate additional sludge utilization for agriculture or gardening. The mass balance was set up theoretically, with influent values taken from the literature (Table B-4, Appendix B). Single measurements (between 2-4) in freshwater, permeate and rainwater were used as a reference (Table B-5, Appendix B). The results should be regarded with due caution.
RESULTS

WATER BALANCE

Water consumption

The average water consumption in Swiss households amounts to approximately 162 l Pe\textsuperscript{-1} d\textsuperscript{-1} (SVGW, 2002). Approximately one third is used for toilet flushing, 3% for gardening, and 20% for laundry. In the Aquamin project, the freshwater was replaced by recycled wastewater for toilet flushing and gardening and by rainwater for laundry. The measured average water consumption was approximately 125 l Pe\textsuperscript{-1} d\textsuperscript{-1} with strong variations (min = 0, max = 290 l Pe\textsuperscript{-1} d\textsuperscript{-1}), which was 25% below the reported value. The freshwater savings amounted to 40% over the 3 years, which was slightly lower than expected, but when considering that the permeate was not recycled during several periods and freshwater was used for the last rinse of the laundry, the values match very well.

The distribution of the different usages corresponded quite well to the study of SVGW. Modern installations for laundry and toilets decreased water consumption for these purposes, but water consumption for irrigation was higher (Figure 2).

![Water consumption by source](image)

Figure 2: Water use of various water sources and distribution by consumption classes compared with an average Swiss household.

Seasonal, weekly and daily variation of water consumption

Figure 3a shows the seasonal variation of freshwater and total water consumption (7d average, without vacation periods). While the freshwater consumption shows no clear seasonal trend, the requirements for irrigation purposes are characterized by high peaks of up to 800 l d\textsuperscript{-1} during the summer months (note that from April to June 2005 the permeate pump was out of order and rainwater was used for gardening). In spite of daily permeate production, a few dry periods with high irrigation requirements even resulted in complete emptying of the permeate tank (1.5 m\textsuperscript{3}).

The cumulative frequency curves of daily water consumption separated by the different sources are shown in Figure 3b. Rainwater is only used at 40% of the days in quantities below 200 l d\textsuperscript{-1} (higher values resulted from a period, when rainwater was used for gardening). The permeate curve has a sharp bend at around 250 l d\textsuperscript{-1}. This means that irrigation was necessary at around 5% of the days, in quantities up to 800 l d\textsuperscript{-1}. The median of the freshwater and permeate use are 230 l d\textsuperscript{-1} and 130 l d\textsuperscript{-1}, respectively, while the median for the influent of the MBR was 350 l d\textsuperscript{-1}. 
A comparison of the water consumption between workdays and weekends showed a 15% increase (without laundry and irrigation) on weekends. The difference in diurnal variation between workdays and weekends was also very pronounced, as shown in Figure 4. Both curves showed a minimum during the night, followed by a morning peak. On workdays, this peak was more distinct and occurred earlier in the morning. After the morning peak, the water consumption was more or less equally distributed over the day. Weekends showed a morning and an evening peak.

Figures 3 and 4 show that the wastewater production is highly dynamic and far more pronounced than in centralized treatment, both during single days and longer periods. Due to the large buffer capacity of the treatment plant, the daily variation had only a small influence on the treatment efficiency. For operation with the primary clarifier, the diurnal variation generally resulted in permeate withdrawal in the morning because of the morning peak and the shut-down during the night. For operation with sludge recycling, the hydraulic buffer capacity was larger and the permeate withdrawal start consequently depended less on the wastewater production.

**Influent to the WWTP and storage capacity**

For the design of the hydraulic storage capacity, knowledge about the average water flows is not sufficient. The data of the freshwater consumption was evaluated for peak flows of
different time periods between 15 minutes and 2 days (Figure 5). Within 15 minutes, a maximum water volume of around 250 liters flowed into the MBR; for one hour, this volume increased to 350 liters and for 6 hours to 515 liters (the latter value may be increased by maximum 150 liters because neither toilet-flushing nor laundry effluent were considered in the data). The main reason for the high water flows was emptying of the bathtub.

The maximum effluent capacity varied between 35 and 100 l h\(^{-1}\) depending on the membrane permeability. Taking into account that the effluent pump starts after 45 minutes of pre-aeration, the maximum storage capacity required during normal operation was 380 liters or 530 liters if 150 liters for toilet-flushing and laundry are included. The available storage capacity with primary clarifier operation was between 450 and 700 liters, which was enough during normal operation. With sludge recycling, the storage capacity is even higher.

This means that - if no spillover is foreseen - the reaction time in a worst-case scenario (e.g. pump breakdown) is between 4 and 24 hours. Of course, it should be secured by an alarm system that the wastewater production during failures is minimized.

![Figure 5: Evaluation of peak freshwater flows for different time periods during the 3 years. The maximum water volume entering the MBR within 15 minutes was around 250 l, during a 6 hour period it was between 400 and 515 l.](image)

**SLUDGE PRODUCTION**

*Operation with primary clarifier*

The treatment plant was operated with a primary clarifier only during short periods (a maximum of 3 months). During these periods, no steady state was reached, which means that the sludge production data may not be representative. Biological sludge production with a sludge age of approximately 120 days amounted to 60 g\(\text{COD}\) d\(^{-1}\) corresponding to 15 g\(\text{COD}\) Pe\(^{-1}\) d\(^{-1}\). The apparent yield factor \(Y'\) including biomass decay was 0.33 ± 0.1 g\(\text{COD, X}\) g\(\text{COD, in}\)\(^{-1}\). Conventional wastewater treatment plants have an apparent yield of approx. 0.4-0.5. The primary sludge production was not included in this calculation, but was estimated to be 55 g\(\text{COD}\) Pe\(^{-1}\) d\(^{-1}\) on the basis of the HRT and literature data. The total sludge production added up to 70 g\(\text{COD}\) Pe\(^{-1}\) d\(^{-1}\) (the measured TSS/COD ratio was 0.85), which is slightly lower than conventional treatment (80 g\(\text{TSS}\) Pe\(^{-1}\) d\(^{-1}\)), but was significantly higher than similar small MBRs (Bischof *et al.*, 2005). The difference was most probably due to not reaching a steady state.
Operation with two biological reactors
No primary sludge was produced in operation with two biological reactors, i.e. with mixing, temporal aeration and recycling of activated sludge. This extra COD had to be oxidized. The biological sludge production with this operating mode (sludge age 100 - 350 d) was higher than with a primary clarifier, resulting in 24 gCOD Pe⁻¹ d⁻¹. The apparent yield Y' was slightly lower, with 0.24 (±0.1) gCODX gCOD,in⁻¹ (Figure 6).

Sludge removal and drying
The filter bag was located in a manhole. Due to the limited air circulation, a dry matter content of 10-15 % was reached. After storage in a dry room for half a year, this value was increased to 85%, with a volatile solids content of 50%.
The biological sludge production of a 4-person household amounts to approximately 190 liters per year for operation with a primary clarifier and 300 liters with full biological treatment. For a filter bag volume of 40 liters, this corresponds to 5 and 8 filter bags per year respectively. Sludge handling (replacement of the filter bags) was troublesome because of its great weight (40 kg).

NUTRIENT REMOVAL

Nitrogen
Daily nitrogen production is generally assumed to be 11 gN Pe⁻¹ d⁻¹ (ATV, 2000; 85%-value). A literature review of the nitrogen contents of several wastewater fractions revealed that this value might be too low for pure domestic wastewater (Table 1), although the values found vary greatly because they were assessed in different countries. The mass balance for nitrogen (Figure 6) revealed that the nitrogen load was only 8.4 gN Pe⁻¹ d⁻¹. Urine contains 70-80% of the total nitrogen in wastewater (Larsen and Gujer, 1996). As the tenants spent most of their day at work, they also used the toilet there, which resulted in a lower ratio of urine excreted at home (~60%).

<table>
<thead>
<tr>
<th>Source</th>
<th>Load (av)</th>
<th>Range</th>
<th>Load (av)</th>
<th>Range</th>
<th>Literature</th>
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<tr>
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<td>11</td>
<td>8.7 - 13.3</td>
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<td>0.6 - 1.4</td>
<td>Ciba-Geigy (1977)</td>
</tr>
<tr>
<td>Feces</td>
<td>1.2</td>
<td>1.1 - 5.5</td>
<td>0.6</td>
<td>0.3 - 1.8</td>
<td>Ciba-Geigy, 1977</td>
</tr>
<tr>
<td>Kitchen</td>
<td>0.4</td>
<td>0.08 - 1.5</td>
<td>0.4</td>
<td>0.06 - 1.4</td>
<td>Almeida et al., 1999</td>
</tr>
<tr>
<td>Personal hygiene</td>
<td>0.2</td>
<td>0.08 - 0.4</td>
<td>0.09</td>
<td>0.09 - 0.17</td>
<td>Almeida et al., 1999</td>
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<tr>
<td>Shower/Bath</td>
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<td>0.07 - 0.65</td>
<td>0.01</td>
<td>0.06 - 0.27</td>
<td>Nolde, 1999</td>
</tr>
<tr>
<td>Total</td>
<td>13</td>
<td>8.9 - 15</td>
<td>2.0</td>
<td>1.1 - 2.5</td>
<td></td>
</tr>
</tbody>
</table>

The nitrification functioned very well in both operational modes. The influent water peak in the morning (Figure 4) combined with little aeration during idle periods usually resulted in high ammonium concentrations at the beginning of permeate withdrawal. In order to minimize these ammonium peaks, the MBR was aerated for 45 minutes after reaching the fill level before starting permeate withdrawal.
A greater difference was observed in the denitrification and nitrate effluent concentrations. With primary clarifier operation, the denitrification capacity was limited, because too little organic substrate was available, but around 50% of the nitrogen was denitrified. With mixing in the first reactor and sludge recycling, more substrate for denitrification was available, resulting in 66% denitrification. The overall nitrogen removal amounted to 60% and 85% for primary clarifier operation and biological pretreatment respectively.
Phosphorus
In centralized treatment, the phosphorus load amounts to \( \sim 1.8 \, \text{g P e}^{-1} \text{d}^{-1} \) (ATV, 2000; 85% value). In this study, a value of \( 1.6 \, \text{g P e}^{-1} \text{d}^{-1} \) was measured. At first glance, this appears quite low. When considering the fraction of urine that was excreted at work, a total phosphate load of \( 2 \, \text{g P e}^{-1} \text{d}^{-1} \) was reached. This value corresponds well to that found in a literature review (Table 1). Phosphate is only eliminated by biological incorporation, and, under ideal circumstances, by natural precipitation (Maurer and Boller, 1999). With primary clarifier operation, overall phosphate removal of \( 25 \pm 15\% \) was achieved.

After switching to biological operation with sludge recycling, the phosphate removal was enhanced, due to the higher activated sludge production (which has a higher P-content than primary sludge) and because the environmental conditions in the first reactor were favorable for the growth of phosphate accumulating organisms (PAO, Chapter 3). With this operation, the P-removal was increased to 50% (Figure 6) with maximum removal rates of 80% during a short period.

**Figure 6**: Mass balances for COD, nitrogen and phosphorus for the two operational modes in g P e^{-1} d^{-1}. Values in brackets indicate the estimated range (min/max).

**TEMPERATURE AND ALKALINITY**
As the building was constructed according to the Minergie-P label, a Swiss standard for energy-efficient buildings, and the MBR was located in the basement, the water temperature variations were less pronounced than in large wastewater treatment plants. The temperature curve of the MBR is shown in Figure 7. Quick temperature drops occurred after plant services, when the tanks were filled with relatively cold freshwater. During normal operation, the
temperature never dropped below 15°C. If the treatment plant is outside, temperatures below 10°C have to be expected during longer periods.

Figure 7: Temperature and pH curve from March 2005 to December 2007 (2h interval, moving average). Temperature drops occurred after emptying of a tank and refilling with freshwater or during vacation. pH was in tendency lower during periods with primary clarifier operation (indicated by the arrows) due to lower denitrification.

The water hardness of the freshwater in Zuchwil is almost 30°fH. This results in a good buffer capacity of the wastewater (alkalinity in the influent was around 10 mmol l⁻¹). No problems with pH drops were observed. However Figure 7 shows that the pH was significantly lower during primary clarifier operation because the denitrification capacity was reduced in these periods. With weakly buffered wastewater and high nitrification rates combined with limited denitrification, pH drops resulting in serious inhibition of the activated sludge and operational problems may be expected. It is therefore recommended to operate such a small MBR with intermittent aeration to enhance denitrification.

HEAVY METALS

The copper and zinc found in wastewater from combined sewer systems originate mainly from roof and road runoffs (Boller, 1997; Boulay and Edwards, 2000; Soerme and Lagerkvist, 2002). Only 20-30% of these metals come from domestic wastewater. The elimination rates in mechanical-biological treatment plants reach 75% for copper and 70% for zinc respectively (Boller, 1997). Lower amounts of heavy metals were therefore expected in the excess sludge from a single-family house treating only domestic wastewater (and a small fraction of roof runoff) than in conventional excess sludge.

Copper

The Cu concentrations in the freshwater and MBR effluent varied strongly and were not clearly identifiable. The concentration in rainwater was 75 µg l⁻¹ and in the effluent of the washing machine it was slightly lower at 60 µg l⁻¹ due to dilution with freshwater. The concentrations in the excess sludge were relatively constant at 300 mgCu kgTS⁻¹ (Figure B-1, Appendix B). The higher values at the beginning can be explained by a higher Cu content of the sludge used for the startup and wash-out of Cu installations. A rough estimate of the copper mass flux based on literature data and the measurements is given in Table 2. The estimates show that this treatment plant either had a lower elimination rate or lower influent loads than expected from literature data.
Zinc
The Zn concentrations in the sludge correlated well with the Cu concentrations, i.e. they seemed reasonable. All the other measurements again suffered from great uncertainty. The estimated removal efficiency for zinc was 35%, which is far below the expected value. The zinc load in the sludge is subject to small uncertainty, all the other values are very rough estimates.

Table 2: Estimated average Cu and Zn loads for the Aquamin house (in mg Pe\(^{-1}\) d\(^{-1}\)). Only single measurements were made for freshwater, rainwater and permeate. The uncertainty of these values is very high.

<table>
<thead>
<tr>
<th>Source</th>
<th>Cu</th>
<th>Zn</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freshwater</td>
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<td>12</td>
<td>Measurements</td>
</tr>
<tr>
<td>Rainwater</td>
<td>0.5</td>
<td>0.5</td>
<td>Measurements</td>
</tr>
<tr>
<td>Sludge</td>
<td>7</td>
<td>12</td>
<td>Measurements</td>
</tr>
<tr>
<td>Effluent</td>
<td>2 / 2</td>
<td>20 / 25</td>
<td>Estimate / Measurement</td>
</tr>
</tbody>
</table>

Estimated removal % | 75 | 25 - 35 |
Expected removal     | 75 | 70     | Boller (1997) |

DISCUSSION

EFFLUENT QUALITY
The standards of the effluent quality from small wastewater treatment plants differ for different countries. Generally speaking, the requirements are far lower than for large WWTPs. Table 3 shows an overview of existing effluent standards for the German-speaking countries compared with the effluent quality reached by the Aquamin treatment plant.

Table 3: Effluent standards for wastewater treatment plants in the German speaking countries

<table>
<thead>
<tr>
<th>Requirement</th>
<th>Country</th>
<th>COD (\text{g}_{\text{COD}} \text{m}^{-3})</th>
<th>NH(<em>4)-N (\text{g}</em>{\text{N}} \text{m}^{-3})</th>
<th>N(<em>{\text{tot}}) (\text{g}</em>{\text{N}} \text{m}^{-3})</th>
<th>P(<em>{\text{tot}}) (\text{g}</em>{\text{P}} \text{m}^{-3})</th>
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<tbody>
<tr>
<td>WWTP (&gt;10,000 PE)</td>
<td>CH</td>
<td>15 (^{37})</td>
<td>2</td>
<td>50%</td>
<td>0.8</td>
</tr>
<tr>
<td>WWTP (10-100,000 PE)</td>
<td>D</td>
<td>75</td>
<td>10</td>
<td>13</td>
<td>1</td>
</tr>
<tr>
<td>Small WWTP (&lt; 500 PE)</td>
<td>D</td>
<td>90</td>
<td>10</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>With nitrification (^{29})</td>
<td>D</td>
<td>90</td>
<td>10</td>
<td>25</td>
<td>-</td>
</tr>
<tr>
<td>With denitrification (^{29})</td>
<td>D</td>
<td>90</td>
<td>10</td>
<td>25</td>
<td>-</td>
</tr>
<tr>
<td>With P-elimination (^{29})</td>
<td>D</td>
<td>90</td>
<td>10</td>
<td>25</td>
<td>-</td>
</tr>
<tr>
<td>With nitrification (^{29})</td>
<td>CH</td>
<td>60</td>
<td>3</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>&lt; 50 PE (^{4})</td>
<td>A</td>
<td>90</td>
<td>10</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Average with primary clarifier</td>
<td>25</td>
<td>1.2</td>
<td>45</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>Average with sludge recycling</td>
<td>35</td>
<td>1.2</td>
<td>15</td>
<td>8</td>
<td></td>
</tr>
</tbody>
</table>


The effluent concentrations of the Aquamin MBR conformed to most effluent standards, even for those of large wastewater treatment plants. The only exception was the phosphate concentration, where enhanced removal would be needed because the effluent standards were exceeded by a factor of 10-20. However, two points have to be considered:

- Due to the recycling of the treated wastewater for toilet flushing, the concentrations of nitrogen and phosphate are increased by 25% (the fraction used for toilet flushing). The recycling affects only the effluent concentrations and has no influence on the effluent loads after reaching a steady state.
- The wastewater is not diluted with extraneous water as in most sewer systems.

When comparing the phosphate effluent loads per person with a conventional Swiss wastewater treatment plant \((Q = 0.35 \text{ m}^3 \text{ Pe}^{-1} \text{ d}^{-1}, c_e = 0.5 \text{ g} \text{ Pe}^{-1} \text{ d}^{-1})\), the difference still
remains high, but reaches a factor of only 4-6. So it is questionable whether the current effluent standards specifying concentration threshold values are suitable for wastewater treatment plants with water recycling. Alternatively, elimination rates or effluent loads could be defined on a per-person basis. Of course, it is far easier to just measure the concentrations than to assess the effluent loads or elimination rates.

**EXCESS SLUDGE PRODUCTION**

The excess sludge production was relatively low. For operation with a primary clarifier, the biological excess sludge is best pumped to this clarifier. The primary sludge can then be transported periodically to a larger WWTP for further treatment. The advantage of full biological operation is that only activated sludge is produced; it is aerobically stabilized and does not produce odor problems. The second advantage is enhanced P-removal. A substantial increase of the sludge retention time would additionally reduce sludge production (Pollice et al., 2004), but also P-elimination. So the operator has to decide whether he wants to minimize sludge production or maximize P-removal.

**EXCESS SLUDGE QUALITY AND FERTILIZER VALUE**

*Heavy metals*

Nowak et al. (2003) investigated the sludge quality of wastewater treatment plants in Upper Austria and showed that there was no correlation between plant size and sludge quality. They concluded that the quality of the sludge depended mainly on industrial discharges. With a lack of industrial discharges, roof and road runoff, the sludge quality in terms of heavy metals was expected to be better than for conventional systems. Table 4 shows a comparison of the sludge qualities of several treatment plants and the Swiss Federal Standards (ChemRRV, 2005) for agricultural use. As a complete ban on the agricultural use of sewage sludge in Switzerland comes into effect in October 2008, these figures are only used as benchmark values.

<table>
<thead>
<tr>
<th>Source</th>
<th>Cu mg kgTS⁻¹</th>
<th>Zn mg kgTS⁻¹</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aquamin</td>
<td>320</td>
<td>600</td>
<td>This study</td>
</tr>
<tr>
<td>Kloten-Opfikon</td>
<td>317</td>
<td>1260</td>
<td>Measurements 1992-2006</td>
</tr>
<tr>
<td>Zurich, Werdhölzli</td>
<td>362</td>
<td>780</td>
<td>Measurements 2005-2006</td>
</tr>
<tr>
<td>Denmark, 1997</td>
<td>262</td>
<td>760</td>
<td>Nowak et al., 2003</td>
</tr>
<tr>
<td>Upper Austria, 1997</td>
<td>200</td>
<td>894</td>
<td>Nowak et al., 2003</td>
</tr>
</tbody>
</table>

Surprisingly, the sludge quality of domestic sludge was in the same range as most conventional treatment plants. This is due to the lower amount of sludge production.

*Fertilizer value*

The use of sewage sludge for agriculture has the benefits of nutrient recycling (phosphorus, nitrogen) and improvement of soil quality. This use was banned in many European countries due to the risk of accumulating heavy metals and organic pollutants in the soils and due to health concerns in view of BSE. In Switzerland, however, some special situations still allow the sludge of small WWTP (< 200 PE) to be used in agriculture. The agricultural land that could be fertilized by using only the excess sludge of a 4-person household without exceeding the maximum allowed quantities is calculated in Table 5. The calculations are based on the former sludge quality standards in Switzerland (ChemRRV,
2005) for heavy metals and the nutrient uptake of crops (vegetables, average uptake) for nitrogen and phosphorus.

Table 5: Production, max. application and the resulting area necessary for agricultural use of the excess sludge of a 4-person household for different substances

<table>
<thead>
<tr>
<th></th>
<th>Max. application g m⁻² a⁻¹</th>
<th>Sludge conc. g kgTS⁻¹</th>
<th>Production g a⁻¹</th>
<th>Area necessary m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nitrogen</td>
<td>20</td>
<td>54</td>
<td>1620</td>
<td>80</td>
</tr>
<tr>
<td>Phosphorus</td>
<td>2.1</td>
<td>35</td>
<td>1050</td>
<td><strong>500</strong></td>
</tr>
<tr>
<td>Copper</td>
<td>0.06</td>
<td>0.3</td>
<td>9</td>
<td>150</td>
</tr>
<tr>
<td>Zinc</td>
<td>0.2</td>
<td>0.6</td>
<td>18</td>
<td>80</td>
</tr>
<tr>
<td>Sewage sludge</td>
<td>167</td>
<td>30000</td>
<td>180</td>
<td></td>
</tr>
</tbody>
</table>

To avoid over-fertilization with phosphorus, an area of 500 m² is necessary for a 4-person household, while the required area for heavy metals and nitrogen is below 200 m². The available garden area around the house was approx. 90 m². This means that the sludge either had to be applied to farmland, which requires transport, or had to be disposed of with the normal garbage.

**CONCLUSIONS**

- The effluent quality of an MBR treating the domestic wastewater of a 4-person household almost conformed to the requirements for large WWTP, with the exception of phosphate, where additional measures are needed (Chapters 3-5).
- The treatment plant was operated in two modes: with a conventional primary clarifier and as a fully biological treatment plant. The biological operation mode resulted in nutrient removal rates of 85% for nitrogen and 30-50% for phosphorus, whereas the primary clarifier reached 60% and 25% respectively.
- Due to the accumulation of non-degradable substances in a water-recycling scheme, the effluent concentrations of these substances increased. This should be considered when comparing them with effluent standards.
- By using rainwater for the laundry and recycling the treated wastewater for toilet flushing and gardening, freshwater savings amounted to 40%.
- The daily and seasonal water consumption and wastewater flow showed enormous variations. However, their influence on the treatment efficiency and operation of the WWTP was only marginal as long as there was enough buffer volume to equalize the hydraulic peak loads.
- The limiting substance for the agricultural use of sewage sludge was phosphorus with a land requirement of 120 m² Pe⁻¹.
- The heavy metal content of the activated sludge from a small wastewater treatment plant treating only domestic wastewater is in the same range as for sludge from conventional treatment plants.

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CHAPTER 2

PRACTICAL ASPECTS OF MBR OPERATION IN A DETACHED HOUSE

Thesis Chapter
Practical aspects of MBR operation in a private house

ABSTRACT
Wastewater treatment plants in private houses need to fulfill various requirements regarding emissions, reliability, maintenance and costs. Maximum attention should be given to users and neighbors by minimizing noise and odor emissions. Automatic failure detection and notification of service personnel combined with remote control ensure reliable and cost-efficient monitoring. The membrane usually determines the service interval because of its permeability decrease. For a two-tank treatment plant, operation with a large primary clarifier seems to be superior to operation with sludge recycling with respect to permeability. Costs are dominated by investment costs for the equipment, engineering and installation. Operating costs account for 30 - 50% of total annual cost. Compared to the Swiss average for a centralized system, total annual costs for small MBRs are only 1.5 - 2.5 times higher. From an energy point of view, small MBRs are not yet very efficient, as a lot of energy is used for cross-flow aeration. Under ideal conditions, similar energy use could be achieved to that of large WWTP.

Requirements on operation of household WWTP
Decentralized concepts for wastewater treatment have been praised as alternatives to conventional centralized systems. These proposals are based on several assumptions (Wilderer, 2004):
- The quality of the receiving water bodies is not negatively impacted by the use of decentralized technologies.
- A maximum of convenience is provided to the users.
- The technical solutions are low in cost and reliable in operation.
- Low-cost and efficient measures for operation, monitoring and maintenance are available.
While the first point is examined in the other chapters of this thesis, the main focus of this chapter is on acceptance, operational safety, monitoring, maintenance, energy consumption and costs, based on the three years of operating experience.

Acceptance
Maximum convenience is expected to be provided to the users. This means that they should not be aware of having a small WWTP in their basement (or in the neighborhood), as acceptance would otherwise be seriously impaired. In the Aquamin project, three points with a negative impact on acceptance were identified: noise, odor and frequent maintenance visits. Both noise and odor should already be addressed in the planning phase of such a project because freedom of action during operation is limited. The maintenance requirements depend on the operational strategies applied.

Odor
Although the tanks were air-tight and the off-gas was withdrawn and released over the roof, the tenants complained of odor problems. Bad odors were basically accepted as long as they were restricted to the room where the treatment plant was located. However, depending on the weather conditions, they were also noticed outside and interfered with the use of the garden area. The odors evolved from the primary clarifier. The ventilation system and limited space did not allow installation of an off-gas treatment unit for the primary clarifier (e.g. an activated carbon filter). By partly aerating and mixing the first tank and recycling sludge from the MBR, putrescent conditions and the formation of a malodorous sludge blanket in the first
reactor were minimized and the odor problem was solved. A further concern with respect to odor problems was the installation of the urine treatment unit (Chapter 4). By sucking all the air which had been in contact with urine through a small activated carbon filter, urine odor problems were restricted to periods of maintenance work.

No odor problems originated from sludge handling. Since only secondary sludge was collected in the filter bags, no bad smells were recorded. In contrast, primary sludge is very foul smelling and its removal needs special attention, especially at indoor treatment plants. Avoiding primary sludge was thus a further advantage of operation with sludge recycling.

**NOISE**

The noise emissions of the aggregates (permeate pump, air compressors) were minimized by installing an insulation layer, but no solution was found for the noise caused by the aeration. Ascending air bubbles caused vibrations of the reactor walls (stainless steel) and a humming sound which was highly audible during the quiet night hours. In order not to annoy the residents, the treatment plant was switched off during seven hours at night, which was not ideal from an operational point of view but did not cause any problems either. It would be easier to address this aspect in the planning phase by choosing less sensitive material for the reactor walls (e.g. plastic).

**MAINTENANCE**

The basic idea of a package plant is that no or only marginal maintenance is necessary during normal operation. In the Aquamin project, the state of “normal operation” was only reached after one year. Before that, several breakdowns occurred due to technical or operational failures and retrofitting of the plant, resulting in several maintenance and repair visits from the service company.

Afterwards, the only regular maintenance work was the exchange of the filter bags for excess sludge removal. All other works were related to the experiments (e.g. measurement devices, sampling). A wastewater treatment plant should not cause more work than other household appliances such as heating or ventilation systems. Frequent visits by a service company result in increased costs, as discussed in the cost section.

**OPERATIONAL SAFETY**

In order to provide maximum convenience for users, safe operation must be guaranteed. So the technical installations need to be robust and must cope with diverse loading conditions. In the following sections, the various installations are briefly discussed.

**INFLUENT DYNAMICS AND OPERATION STATES**

The influent water flow fluctuated between 0 and 900 l d\(^{-1}\) and was not predictable. However, the MBR was expected to provide a relatively constant effluent quality. Three operating states were defined: sleep mode during the night, when all aggregates were switched off, standby mode during the filling periods, when mixing and limited aeration were provided, and permeate withdrawal, when the permeate pump and the aeration systems were switched on.

While periods with high influent were critical with respect to the membrane capacity, vacation periods without influent were more critical for the biological system because excessive aeration results in increased decay of the biomass and thus a loss of biological activity (Siegrist *et al.*, 1999). In the Aquamin project, no vacation periods of more than one week were recorded, but standby mode with its limited aeration already proved critical. Therefore it is proposed to provide a special operation mode for vacations with very short aeration periods for mixing and oxidation of decay products.
SIEVE

In the biological operation mode of the first tank with sludge recycling and mixing, particulate material had to be kept away from the membrane compartment. In the course of the project, three different sieves were installed, with diameters of 1 mm (21.4.2005 - 16.9.2005), 6 mm (13.12.2005 - 6.12.2006) and 2 mm (6.12.2006 - 23.10.2007). Together with the 6 mm sieve, an air-lift pump was installed in the first tank in order to pump the water from the first to the second tank, which also increased the water storage capacity. Coarse bubble aeration kept the sieve clean.

The 1 mm sieve clogged several times in spite of regular aeration. Mechanical cleaning with a brush was more successful but had to be done manually. No clogging occurred with the 6 mm sieve. However, the interspace between the membrane plates was densely clogged with particulate material such as hair or kernels. The 2 mm sieve clogged twice due to insufficient crossflow aeration: as the aeration of the sieve was coupled to that of the membrane compartment, the air was mainly blown into the reactor with the lower water level. This was a shortcoming that had to be accepted because the sieve had been installed later. If a sieve is necessary, 2 - 3 mm holes are recommended.

AGGREGATES AND ENERGY

Two compressors and two pumps were installed at the Aquamin plant. The compressors provided air for the oxygen supply in the MBR compartment through fine bubble aeration, for membrane scouring with coarse bubble aeration, for mixing of the two tanks and for the air-lift pumps. The permeate was withdrawn from the MBR and pumped to the storage tank with a rotary pump. The excess sludge was removed with a submersible motor pump, which was used up to twice a week for 20 seconds. All aggregates worked properly and were maintenance-free during the three years of operation.

From an energy point of view, the installed aggregates were not optimized. Approximately 4 kWh d⁻¹ of electricity was used, corresponding to 1.5 kWh Pe⁻¹ d⁻¹ or 10 kWh m⁻³, respectively. Aeration and permeate withdrawal resulted in approximately 65% of the energy demand, the remainder was used for the control unit, in-situ measurements and permeate recycling. The energy consumption is shown in Figure 1.

Conventional large wastewater treatment plants need approximately 0.1 kWh Pe⁻¹ d⁻¹ of electrical energy (VSA, 2006), although, according to Nowak (2003), this could be optimized to 0.05 kWh Pe⁻¹ d⁻¹. Large membrane bioreactors use between 0.3-0.5 kWh Pe⁻¹ d⁻¹ (Wedi et al., 2004) and commercially available small MBRs for 4 persons have an energy requirement of 1-2 kWh d⁻¹, corresponding to 0.25 - 0.5 kWh Pe⁻¹ d⁻¹, according to the suppliers.

![Figure 1: Energy consumption for various purposes in the Aquamin project. 100% corresponds to 4 kWh d⁻¹. Excess sludge removal was below 1%.](image)

The minimum energy requirement for an energy-optimized package plant is estimated in the following. An ideal small MBR requires a control unit, aeration and a permeate suction pump. While the energy requirement for the control unit is hard to predict (perhaps 1 W Pe⁻¹), it can
be calculated theoretically for permeate withdrawal and aeration. The calculations are summarized in Table 1.

The energy requirement of the permeate pump is calculated from Equation 1:

\[ N_{PP} = \frac{Q_{Perm} \cdot g \cdot \rho_w \cdot H}{\eta_{pp}} \]  
(Eq. 1)

where \( N_{PP} \) = energy requirement of the pump [W], \( Q_{Perm} \) = water flow [l d\(^{-1}\)] (assuming 130 l Pe\(^{-1}\) d\(^{-1}\)), \( g = 9.81 \text{ m s}^{-2} \), \( \rho_w = \) water density [kg l\(^{-1}\)], \( H = \) pumping head (including frictional losses in pipes and transmembrane pressure; estimate \( H = 5\) m), \( \eta = \) energy efficiency of the pump [-]. The energy efficiency of the small pumps available today is around 5-15% and may realistically be increased to 60% (Bruderer, 2008). With the indicated values necessary for a 4-person household and a reasonable energy efficiency of 40%, the energy demand of the permeate pump is around 0.005 kWh Pe\(^{-1}\) d\(^{-1}\).

The energy requirement for the aeration is calculated from Equation 2:

\[ N_{air} = \frac{Q_{air} \cdot \rho_w \cdot g \cdot h}{\eta_{air}} \]  
(Eq. 2)

where \( N_{air} \) = power demand for aeration [W], \( Q_{air} \) = air flow [m\(^3\) s\(^{-1}\)], \( \rho_w = \) water density [kg m\(^{-3}\)], \( h = \) water level (including membrane pressure loss) [m], \( g = 9.81 \text{ m s}^{-2} \), \( \eta = \) energy efficiency of the compressor [-]. An energy efficiency of 40% was assumed (the compressors used reached 20-30% and could be increased to approximately 60% (Bruderer, 2008)).

Aeration in MBRs has three functions: mixing, oxygen supply for biological processes and cross-flow aeration to prevent blocking of the membrane. An optimum has to be found between oxygen supply (fine bubble aeration) and membrane scouring (coarse bubble aeration) with low oxygen input. Fine bubble aeration needs more energy than coarse bubble aeration and does not ensure proper scouring of the membrane. According to Judd (2005), the membrane aeration intensity is not yet well understood so that no theoretical relationship is available between aeration and water flux. The cross-flow aeration intensity is based on previous experience or the recommendations of the suppliers. The key contributing factor to energy demand in submerged systems is the specific aeration demand, the ratio of gas flow (\( Q_{air} \)) to membrane area. Judd (2005) reports values between 0.35 and 1.2 m\(^3\) m\(^{-2}\) h\(^{-1}\) (Aquamin: 1.5 m\(^3\) m\(^{-2}\) h\(^{-1}\)). With a membrane flux of 15 l m\(^{-2}\) h\(^{-1}\), a wastewater production of 130 l Pe\(^{-1}\) d\(^{-1}\), an oxygen demand of 70 gO\(_2\) Pe\(^{-1}\) d\(^{-1}\) and a specific cross-flow aeration of 1 m\(^3\) m\(^{-2}\) h\(^{-1}\), the energy demand for membrane cleaning is approximately 0.09 kWh Pe\(^{-1}\) d\(^{-1}\) (\( Q_{air} = 8.7 \text{ m}^3 \text{ d}^{-1} \)). Considering the oxygen supply and mixing, the energy consumption for aeration increases to around 0.12 kWh Pe\(^{-1}\) d\(^{-1}\).

In summary, the total energy requirement for a package plant MBR for household wastewater treatment under perfect conditions is at least 0.13 kWh Pe\(^{-1}\) d\(^{-1}\), with the biggest fraction of energy being used for membrane scouring and aeration.

The theoretical energy demand is – surprisingly – in the same range as for large MBRs. The reason for this small gap is that the calculation was based on ideal conditions, which will hardly be reached, and large MBRs may also be optimized with respect to energy. Not included in the calculation are pumping between different reactors (depending on the flow scheme), water recycling and sludge handling. By far the biggest fraction of energy is used for the cross flow aeration. This air flow is mainly determined by the permeate production. In a small treatment plant, the wastewater is not diluted and a part of the wastewater is produced elsewhere (e.g. at the workplace), which lowers the specific energy demand per person.
Table 1: Summary of the theoretical energy demand for a small MBR

<table>
<thead>
<tr>
<th></th>
<th>Unit</th>
<th>Small MBR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wastewater</td>
<td>gCOD Pe⁻¹ d⁻¹</td>
<td>130</td>
</tr>
<tr>
<td>COD load</td>
<td>gCOD Pe⁻¹ d⁻¹</td>
<td>50</td>
</tr>
<tr>
<td>NH₄-N load</td>
<td>gN Pe⁻¹ d⁻¹</td>
<td>6</td>
</tr>
<tr>
<td>Oxygen consumption</td>
<td>gO₂ Pe⁻¹ d⁻¹</td>
<td>70</td>
</tr>
<tr>
<td>Cross flow aeration</td>
<td>m³ Pe⁻¹ d⁻¹</td>
<td>8.7</td>
</tr>
<tr>
<td>Fine bubble aeration</td>
<td>m³ Pe⁻¹ d⁻¹</td>
<td>3</td>
</tr>
<tr>
<td>Energy demand aeration</td>
<td>kWh m⁻³ Pe⁻¹ d⁻¹</td>
<td>0.01</td>
</tr>
<tr>
<td>Total energy demand for aeration</td>
<td>kWh Pe⁻¹ d⁻¹</td>
<td>0.12</td>
</tr>
<tr>
<td>Pumping (permeate)</td>
<td>kWh Pe⁻¹ d⁻¹</td>
<td>0.005</td>
</tr>
<tr>
<td>Control</td>
<td>kWh Pe⁻¹ d⁻¹</td>
<td>0.005</td>
</tr>
<tr>
<td>Total energy consumption</td>
<td>kWh Pe⁻¹ d⁻¹</td>
<td>0.13</td>
</tr>
</tbody>
</table>

MEMBRANES AND PERMEABILITY

The membranes installed were of the flat sheet type made of polyethylenesulfone (PES) with a pore size of 40 nm. The membrane area was 4 m² and the interspace between two plates was 3 mm. After installing the sieve and switching to the biological operation mode of the first tank, two modules with a plate interspace of 6 mm were installed.

A critical factor affecting operation and maintenance of a small MBR is the permeability, which determines the service intervals. Automatic backflushing or chemical cleaning as in large MBRs is not possible in package plants with flat sheet membranes and can only be done during regular servicing. Figure 2 shows the course of the permeability from the second and third years of operation (data of the first year could not be evaluated). The values were calculated as daily averages according to Equation 3:

\[
Perm_i = \frac{Q_{e,i}}{TMP_{average,i} \cdot t_{pump,i} \cdot 4 \ m^2}
\]

where \(Perm_i\) = Permeability [l m⁻² h⁻¹ bar⁻¹], \(Q_{e,i}\) = Permeate production on day i [l d⁻¹]; \(TMP_{average,i}\) = average transmembrane pressure on day i [bar]; \(t_{pump,i}\) = runtime of the permeate pump on day i [h d⁻¹]; \(A\) = membrane area, 4 m² [m²].

Figure 2: Course of the permeability during the 2nd and 3rd years of operation. Data shown as 7d average. MC = Chemical cleaning of the membrane; Triangles indicate exchange of membrane modules. PC = primary clarifier operation of first tank.

During the three years of operation, the modules were exposed to different operational conditions and cleaned and replaced several times. The effects of different actions are discussed on the basis of the permeability data.
Operation: Primary clarifier vs. sludge recycling
New membrane modules were installed on September 16th, 2005. Until December 14th, the first tank was used as a primary clarifier and subsequently as a biological reactor. According to the data, the permeability remained at a constant level during the three months when the first reactor was operated as a primary clarifier, and showed a strong permeability decrease after changing the operating mode to an anaerobic reactor with sludge recycling and mixing (Figure 2).

On March 22nd, 2007 new modules were installed during anaerobic biological operation of the first tank (with the 2 mm sieve). After 2½ months, however, there is also a sharp bend in the permeability curve (Figure 2). The reason for this is not clear, because nothing changed in the operating mode during that period.

The last period with primary clarifier operation was shorter than three months, but the permeability tended to increase.

It is assumed that operation with primary clarifier results in a significantly slower decrease of permeability. Operation with sludge recycling increases the risk of membrane clogging because particles and more organic material enter the membrane chamber. As the periods with primary clarifier operation lasted no more than three months due to odor problems, this assumption could not be verified.

On-site membrane cleaning
The membrane modules were cleaned mechanically (removal of particulate material between the membrane plates) and chemically (citric acid and caustic soda) on-site on September 7th, 2006. Figure 2 shows that the permeability recovered from 20 l m⁻² h⁻¹ bar⁻¹ to 50 l m⁻² h⁻¹ bar⁻¹ thanks to the cleaning, but was still far from the initial permeability of 200 l m⁻² h⁻¹ bar⁻¹. Moreover, the on-site cleaning of the membrane module required a lot of organization, space and manpower, and is therefore not recommended.

Membrane cleaning and exchange of 1 module
On December 5th, 2006 one of the two membrane modules was chemically cleaned on-site again (citric acid, sulfamic acid, sodium hypochlorite) and the other one was replaced by a new one. For a short period, the permeability reached 100 l m⁻² h⁻¹ bar⁻¹ followed by a sharp decrease within three weeks (Figure 2). Most of the permeate was probably drawn through the new module, thus exceeding the critical flux and resulting in increased fouling and a sharp decrease of permeability.

Membrane exchange
The membranes were exchanged three times: on September 16th, 2005, on March 22nd, 2007 and on October 2nd, 2007. New modules were installed on the first two occasions, and regenerated modules were used the last time. With the regenerated modules, only 75% of the initial permeability was achieved, but it increased during the following weeks and finally reached the initial value with primary clarifier operation (Figure 2).

The regeneration was performed by pumping Javelle water or citric acid through the membrane module.

Monitoring
Decentralized WWTP have to be operated, monitored, maintained and repaired with efficient and low-cost measures. If a breakdown occurs, it should be quickly detected and repaired. Two types of breakdowns may occur: physical and biological. Physical breakdown refers to failures of the mechanic, hydraulic or electronic parts such as pumps, pipes, sieve, membranes or a control unit. It results in reduced or zero permeate flow and, if no action is taken, the risk
of flooding. A biological failure means a reduced treatment capacity of the activated sludge, which may be caused by a toxic shock or insufficient aeration. If the complete biology is affected, permeability quickly decreases due to organic matter accumulating on the membrane surface. But it is hard to detect if only certain bacterial groups (e.g. nitrifiers) are affected.

**APPROVAL FOR INSTALLING A SMALL WWTP**

A high effluent quality under various loading conditions can either be assured by correctly designing the treatment plants or by frequently monitoring the effluent concentrations. Different concepts are applied in Germany and Switzerland. In Germany, suppliers of small wastewater treatment plants need technical approval for their products. This means that they have to test their treatment plant for one year on a certified test site under various loading conditions (Dorgeloh *et al.*, 2005). After successfully completing these test runs, the plant is officially approved and allowed for sale for five years, when an inspection is due again. The plant owner needs a service contract and is obliged to maintain the plant according to the instructions, but no further monitoring by federal agencies is necessary. Despite these rules, many small treatment plants lack thorough service and maintenance and thus cause substantial environmental pollution.

In Switzerland, cantonal agencies approve the design of small WWTP. Service contracts are compulsory, but the agencies are also obliged to monitor the efficiency of the treatment plants. They generally require at least one effluent measurement per year, either by a certified laboratory or by their own staff.

**MEASUREMENTS AND PROBES**

The water quality can be monitored by either online measurement probes or sampling. Within the Aquamin project, a lot of data was collected, either by grab or composite samples (Table B-1, Appendix B), or by online measurements (Table B-2, Appendix B). All online measurement data were saved and could be retrieved electronically.

Within a short time, it became clear that online measurements of chemical parameters are not (yet?) suited for decentralized applications. The sensors were very quickly overgrown by a biofilm (Figure 3) and the cleaning and calibration effort was too high.

![Figure 3: Online signal of the oxygen measurement. After 4 days, the sensor is overgrown by an oxygen consuming biofilm. Grey areas indicate permeate withdrawal with expected high oxygen concentrations.](image)

During the three years experience, useful and informative signals requiring little or no intervention were identified. For the “physical” state of the treatment plant, the most important and useful information were the water level measurements and the transmembrane pressure. The time course of these measurements revealed the development of the permeability as well as information on the wastewater production and operating states (e.g. filling, permeate withdrawal, vacation). Of course, an on-off system releasing alerts would be feasible only if the water level exceeds a certain value for a certain period, or if the permeate flow is too low. However, it would supply no information about an impending breakdown. So
Biological problems are hardly detectable at all unless effluent measurement systems are installed (no reliable ones are yet available at reasonable cost). During a 4-week period in January 2007, the nitrification capacity deteriorated, resulting in very high values of ammonium and nitrite effluent. These were easily detectable by ammonium online measurements and grab samples. Due to the daily variation of the pH and ORP measurements, the slightly higher (pH) and lower (ORP) values did not differ strongly from those during normal operation.

DATA TRANSMISSION AND MONITORING

For many years, monitoring of small wastewater treatment plants has been the responsibility of the users, since frequent plant visits by the service company were costly and information technology for remote control was not well developed. This meant that failures often remained undetected until a plant service was due, as the motivation to check the plant status regularly is not usually high as long as no severe consequences (e.g. flooding, odors) are expected. Advances in information technology have resulted in the development of various remote monitoring concepts, based on either wireless communication (via SMS or WAP) or internet protocols.

A remote control concept was used in the AKWA pilot project in Germany, where the operational parameters were transmitted to the service company on a day-to-day basis (Hiessl et al., 2007). Alex et al. (2003) described two different wireless monitoring concepts for small WWTP, where the plant data was transmitted to mobile phones or pocket PCs. Most small MBRs are now equipped with an SMS alert system that sends an SMS with an error code to the service company and the user in case of breakdown.

The Aquamin project was provided with an automatic failure detection system. It was based on simple measurements or logic errors and detected all physical breakdowns occurring during the experimental period. As the plant data was regularly retrieved via the Internet, no extra failure messages were sent in case of breakdown (e.g. by SMS). The advantage of automatically sent error messages is that only a minimum of manpower is necessary and the errors are immediately identified, whereas regular checking of the data is time consuming and less reliable.

COSTS

In Switzerland, the costs for small MBRs vary strongly depending on the supplier and specific circumstances. As they have to be engineered and designed individually and only a few companies are able to operate them, costs for engineering and maintenance are quite high. Furthermore, they depend on the specific circumstances of the building (e.g. number of users, space available, and location). In order to give a rough estimate, costs are calculated for two examples. Both of them refer to a four-person household and are based on Swiss conditions (i.e. material and labor costs), excluding taxes. Conversion from Swiss Francs (CHF) to Euros (€) was done with a factor 1.6 CHF/€. The accuracy of the cost calculation is 25%.

Example 1 is a package plant like the one installed in Zuchwil with primary clarifier operation of the first tank. The treatment plant is located in the basement of a new building. The effluent is stored in a 3 m³ storage tank and reused for gardening and toilet flushing. It is assumed that no extra costs are associated with the placement of the treatment plant in the basement, i.e. no extra room has to be constructed. The excess water needs to be discharged either to the nearest rainwater sewer or to the nearest surface water. This example is calculated twice, once including the water recycling scheme (a) and once without water recycling (b).
In *Example 2*, an existing three-chamber septic tank is upgraded to an MBR. For this purpose, the last chamber is retrofitted with an aeration system and a membrane module. An effluent pipe already exists and no provision is made for water recycling.

**Investment Costs**

The costs are estimated on the basis of data obtained from several sources. The costs directly associated with the treatment plant were obtained from Picatech Huber AG, a Swiss supplier of small MBRS. Costs connected to construction were estimated by an experienced architect. The costs were compared and validated with the cost estimates performed by Fletcher *et al.* (2007), data from other suppliers of small MBRS and the real costs of the Aquamin project. Package plants with MBR technology for four persons cost between €3000 and €7000. For the membrane modules, control unit and aggregates, a price of €3500 seems reasonable. Tank prices vary between €100 and €500 m$^{-3}$, depending on the material. €300 m$^{-3}$ was chosen for the calculation of Example 1. For the permeate tank, the prize assumed was €200 m$^{-3}$, plus €100 m$^{-3}$ for excavation. The effluent may either be discharged into the nearest rainwater sewer (in urban areas) or surface water body (in rural areas). In Switzerland, the average house connection pipe length per person is around 6m and costs €250 m$^{-1}$ (Maurer and Herlyn, 2006). No data is available about the average distance from houses to the nearest surface water. Construction costs per length for such pipes are less expensive (€80 m$^{-1}$) but the distance covered is expected to be longer. A pipe length of 50m is therefore assumed. Further costs are caused by extra plumbing (10 m recycling pipes, €40 m$^{-1}$), water recycling installations (control and pumps), installation of the MBR, engineering and customer training. Engineering includes all costs related to the planning process (design, coordination etc.). The investment costs are compiled in Table 2.

**Table 2:** Investment cost estimates for two on-site MBRS treating domestic wastewater of a 4-person household. Example 1: Indoor MBR in a new building a) with permeate recycling b) without water recycling. Example 2: Upgrading an existing septic tank with an MBR. The accuracy of the estimates is +/- 25%.

<table>
<thead>
<tr>
<th>Lifetime</th>
<th>Unit</th>
<th>Example 1a)</th>
<th>Example 1b)</th>
<th>Example 2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Wastewater treatment</strong></td>
<td>[a]</td>
<td>€</td>
<td>€</td>
<td>€</td>
</tr>
<tr>
<td>Tanks (3 m$^3$)</td>
<td>20</td>
<td>€</td>
<td>900</td>
<td>900</td>
</tr>
<tr>
<td>Treatment unit 2)</td>
<td>10</td>
<td>€</td>
<td>3500</td>
<td>3500</td>
</tr>
<tr>
<td>Engineering 2)</td>
<td>15</td>
<td>€</td>
<td>1500</td>
<td>1500</td>
</tr>
<tr>
<td>Installation MBR 2)</td>
<td>15</td>
<td>€</td>
<td>1500</td>
<td>1500</td>
</tr>
<tr>
<td>Customer training</td>
<td>15</td>
<td>€</td>
<td>300</td>
<td>300</td>
</tr>
<tr>
<td>Total investment WWTP</td>
<td>€</td>
<td>7700</td>
<td>7700</td>
<td>6200</td>
</tr>
</tbody>
</table>

| **Water Recycling** | | | | |
| Engineering | 20 | € | 500 | |
| Permeate tank (incl. installation) 1) | 20 | € | 5000 | |
| Plumbing (10 m) 1) | 25 | € | 1000 | |
| Water recycling installations 1) | 10 | € | 1000 | |
| Total investment recycling | € | 7500 | |
| Total investment | | € | 15200 | 7700 | 6200 |
| Total investment per person | € Pe$^{-1}$ | 3800 | 1925 | 1550 |

| **House connection (discharge pipe)** | | | | |
| Surface water (50m) / rainwater sewer (25m) | 50 | € | 4000$^{1)}$-6000$^{3)}$ | 4000 -6000 | |
| Total investment (house connection: €4800) | | € | 20000 | 12500 | |
| Total investment per person | € Pe$^{-1}$ | 5000 | 3125 | |

1) Büchel (2008); 2) Picatech Huber AG (2008); 3) Maurer and Herlyn (2006)
OPERATING COSTS AND SAVINGS

The operating costs are more or less identical for both examples. According to the supplier, one service per year should suffice, resulting in costs of about €500 a⁻¹ for travel, time and minor repair work. The energy costs are based on an energy demand of 2 kWh d⁻¹ with costs of €0.15 kWh⁻¹. In addition, chemical analyses are required once a year. COD and NH₄-N measurements in an official laboratory cost around €100. Primary and excess sludge are assumed to be stored in the first tank and removed every second year by a tank vehicle (fee: €100 a⁻¹ for the vehicle plus €20 a⁻¹ for sludge disposal via a large WWTP). By recycling treated wastewater, the freshwater consumption is reduced. The average costs for freshwater in Switzerland are currently around €1.10 m⁻³ (Preisüberwacher, 2008). But it has to be considered that the freshwater price is strongly subsidized and will increase in the future. The operating costs are compiled in Table 3.

Table 3: Operating costs for two on-site MBRs treating domestic wastewater of a 4-person household. Example 1: Indoor MBR a) with and b) without permeate recycling in a new building. Example 2: Upgrading an existing septic tank with an MBR.

<table>
<thead>
<tr>
<th>Service / maintenance \footnote{Picatech Huber AG (2008)}</th>
<th>Unit</th>
<th>Example 1(a)</th>
<th>Example 1(b)</th>
<th>Example 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Service / maintenance</td>
<td>€ a⁻¹</td>
<td>500</td>
<td>500</td>
<td>500</td>
</tr>
<tr>
<td>Energy</td>
<td>€ a⁻¹</td>
<td>120</td>
<td>110</td>
<td>110</td>
</tr>
<tr>
<td>Chemical analysis</td>
<td>€ a⁻¹</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Sludge removal \footnote{Picatech Huber AG (2008)}</td>
<td>€ a⁻¹</td>
<td>120</td>
<td>120</td>
<td>120</td>
</tr>
<tr>
<td>Freshwater savings</td>
<td>€ a⁻¹</td>
<td>-70</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Total operating costs</td>
<td>€ a⁻¹</td>
<td>770</td>
<td>830</td>
<td>830</td>
</tr>
</tbody>
</table>

ANNUAL COSTS

Annual costs depend very strongly on the amortization, which is influenced by the lifetime of the treatment plant and the interest rate. No data about the lifetime of small MBRs is yet available. Various components differ in their lifetime (e.g. pumps and tanks). Including the uncertainties of these data, an average lifetime of 12 - 18 years seems appropriate. The lifetimes used to calculate amortization in the examples are given in Table 2, based on the data given in Fletcher et al. (2007) and lifetimes for other household appliances like refrigerators or heating components and some assumptions. With respect to the interest rate, three different models for financing such a treatment plant are possible. Currently, the costs have to be paid by the house-owner. This means that a private person gets a loan from the bank resulting in an interest rate of 3 - 5%. If a community decides to provide wastewater treatment only by small treatment plants, the plants may be financed and owned by either a public authority or private companies. While the interest rates for public authorities are relatively low (2-3%), the interest rates for companies are higher and they need to assure a minimal profit. Therefore, the interest rate for companies is assumed at 7.5%.

The influence of different amortization times and interest rates is shown in Figure 4 for the investment costs of example 1 without the discharge line and the water recycling components (linear amortization at constant interest rate).
According to the cost calculations, the annual costs for a small MBR are estimated to be between €360 and €560 Pe\(^{-1}\) a\(^{-1}\) for wastewater treatment and discharge only, and if water recycling is included, the costs rise to €555 - 750 Pe\(^{-1}\) a\(^{-1}\). The cost estimates performed by Fletcher et al. (2007) calculated with different interest rates are similar, with costs for a 6 Pe package plant with MBR technology of around €400 Pe\(^{-1}\) a\(^{-1}\) (based on costs in the UK). Furthermore, they showed that the yearly costs for larger units (20 - 50 Pe\(^{-1}\)) are reduced by almost 50%.

For both examples, the annual service costs account for 20 and 50% of the total costs, respectively. If more than one service per year is necessary, the service costs double and this share increases to 40 and 70%, respectively. So it is obvious that the service interval needs to be minimized. It may therefore be wise to install a higher membrane area per person in order to operate at a lower flux and a lower permeability loss in order to obviate two or more service visits per year.

The cost calculation also shows that installation of a water recycling scheme in a four-person household is economically not viable except for remote or arid areas where costs of freshwater production are high. The costs are as high as €170 Pe\(^{-1}\) a\(^{-1}\) compared to savings of €20 Pe\(^{-1}\) a\(^{-1}\). Even with longer depreciation times at lower interest rates, the return on investment remains negative for this scenario. But with larger units, the costs per person for the storage tank decrease, while the cost savings remain on the same level. Therefore it would be beneficial to provide larger recycling schemes. A cost comparison of wastewater reuse to rainwater use shows that the latter is more expensive because of the larger storage capacity required. These considerations exclude extra treatment steps for rainwater or wastewater reuse (Chapter 7).
COST COMPARISON: CENTRALIZED - DECENTRALIZED (WITH MBR)

In Switzerland, the average yearly costs for centralized wastewater treatment are estimated to €315 Pe\(^{-1}\) a\(^{-1}\), including operation costs for the WWTP (€37 Pe\(^{-1}\) a\(^{-1}\)) and sewers (€24 Pe\(^{-1}\) a\(^{-1}\)) as well as capital costs for WWTP (€41 Pe\(^{-1}\) a\(^{-1}\)), sewers (€156 Pe\(^{-1}\) a\(^{-1}\)) and house connections (€56 Pe\(^{-1}\) a\(^{-1}\)) based on current replacement values of the infrastructure with amortization times of 33 years for the WWTP, 80 years for the sewer system and 50 years for the house connections at an interest rate of 3% (Maurer and Herlyn, 2006). In small catchments (< 5000 Pe), costs for the WWTP are higher by a factor 1.5 - 2, while the sewer system is not directly related to the catchment size (VSA, 2006).

Would it be beneficial for a community to base its wastewater treatment on a decentralized instead of a centralized system? This question is not easy to assess, as costs strongly depend on the specific circumstances of a catchment and on the system boundaries considered. Pinkham et al. (2006) give a comprehensive overview on factors influencing the costs of decentralized compared to centralized systems.

In the following, a cost comparison for two extreme scenarios of a decentralized system is presented. In both scenarios, all buildings are equipped with small MBRs. The costs for the decentralized system used for the comparison are taken from example 1b in Table 4 (without water recycling). The scenarios mainly differ in the rainwater and effluent handling.

- **Scenario 1 - rainwater sewers:** In densely populated areas, rainwater needs to be collected in a rainwater sewer, which may also be used for the discharge of the MBR effluents. As no data for the cost differences of combined and separate sewers are available, the costs for a pure rainwater sewer system are assumed at 75% of the average sewer network costs (operation and capital costs). Total costs for a decentralized system amount to €550 - 730 Pe\(^{-1}\) a\(^{-1}\) compared to €310 Pe\(^{-1}\) a\(^{-1}\) for the centralized system.

- **Scenario 2 - infiltration:** In this scenario, which may be applicable in rural areas, rainwater and the wastewater effluent are infiltrated on-site. Therefore, no extra costs are considered neither for the setup of a centralized rainwater sewer system nor for house connections. Costs for the decentralized system in this case are between €390 and 450 Pe\(^{-1}\) a\(^{-1}\).

It is important to note that this comparison only gives a rough overview based on average costs. However, it is interesting to see that the costs only differ by a factor 1.2 - 3. Costs for a real catchment need to be calculated in more detail. If a decentralized system is set up in a catchment, several costs will significantly differ from the ones given in the calculation and in Tables 2 - 4. Costs for small MBR are expected to decline significantly. Standardization, mass production and the setup of service networks will decrease the investment and operating costs for small units. Monitoring of the water quality can not be secured through chemical analysis of single grab samples in a private laboratory. Furthermore, larger units for 20 - 50 persons will decrease the specific costs.

In contrast, several costs that would increase total costs for a decentralized system were not included in this calculation, like taxes, extra land requirements and construction costs for buildings. Further factors that need to be included in a more detailed calculation are real capital costs, effects of population growth, rainwater handling and flood control, sludge handling and the impact on the surface and groundwater quality. Maurer (submitted) states that - from an economic point of view - a decentralized approach is only interesting in small catchments with a high growth rate.

CONCLUSIONS

- The acceptance of a small wastewater treatment plant depends on its emissions and the maintenance work. Noise and odor emissions are problematic and should be addressed during the design phase.
- Reliable and low-maintenance technologies guarantee failure-free operation. The membrane usually determines the service interval because of insufficient permeability. For a two-tank treatment plant, operation with a large primary clarifier seems to be superior to operation with sludge recycling with respect to permeability.
- The energy efficiency of currently available small MBRs can be optimized. However, even under perfect conditions with future technology, the energy demand of a small MBR is at least 0.13 kWh Pe\(^{-1}\) d\(^{-1}\).
- Monitoring of small WWTP has been simplified by recent advances in communication technology. Through Internet-based or wireless monitoring systems, the operating conditions of a treatment plant can be accessed from remote locations. Responsibility for monitoring should be assigned to a service company, which is also automatically notified in case of failure.
- Costs for a 4-person MBR were estimated to be between 360 and 740 € Pe\(^{-1}\) a\(^{-1}\) when considering amortization within 15 years at an interest rate of 2.5 - 7.5%. To achieve this value, in maximum one service per year should be necessary. Compared to average Swiss costs for centralized wastewater treatment, costs for small MBRs are only higher by a factor 1.25 - 2.5. Cost declines in the future and larger units may result in small MBRs being a competitive alternative to centralized wastewater treatment in rural catchments.
- Under Swiss conditions, a wastewater recycling scheme at household level is currently not cost effective.

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CHAPTER 3

BIOLOGICAL NUTRIENT REMOVAL IN A SMALL-SCALE MBR TREATING HOUSEHOLD WASTEWATER

Christian Abegglen, Mario Ospelt, Hansruedi Siegrist

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Biological nutrient removal in a small-scale MBR treating household wastewater

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ABSTRACT
The biological nutrient removal potential of an on-site MBR located in the basement of a 4-person house treating domestic wastewater was investigated. The reactor consists of two tanks in series. This treatment plant differs from other conventional MBRs by a highly fluctuating influent water flow and a lack of pretreatment. During the first period, the first reactor was operated as a primary clarifier, resulting in nitrogen and phosphorus removal of 50% and 25%, respectively. Primary sludge production and bad odors in the basement were further disadvantages. When using the first reactor as anaerobic/anoxic reactor by recycling activated sludge and mixing the first reactor, nitrogen and phosphorus removal of over 90% and 70% were achieved, respectively. By applying a dynamic model of the plant, the return sludge ratio was identified as the most important parameter. With a return sludge ratio of about 1.2, optimal PAO growth and P-removal up to 90% was reached. Since only activated sludge is produced with this operational mode, on-site sludge dewatering is possible. During vacation periods without loading, the Bio-P activity is kept constant if the aeration is reduced to 5-20 min d⁻¹.

Keywords: on-site wastewater treatment; membrane bioreactor (MBR); decentralized; biological nutrient removal; enhanced biological phosphorus removal (EBPR)

INTRODUCTION
The purification capacity of small-scale wastewater treatment plants for the two nutrients nitrogen and phosphorus is often limited. Nitrification and denitrification may occur to a certain degree in biological systems, depending on the plant layout and the operating conditions. In contrast, phosphorus is only removed to a substantial extent with the aid of chemical precipitation. Table 1 compares removal efficiencies of different treatment systems with a central WWTP. These values were estimated by Swiss experts based on measurements in several small-scale treatment plants (VSA, 2005). Flasche (2002) estimates that the 10% of the population connected to small-scale wastewater treatment plants in Lower Saxony (Germany) account for 20% of the organic and nitrogen loads in domestic wastewater, assuming that the plants are maintained and operated properly. In reality, this value is expected to be much higher. Since decentralized wastewater treatment is seen as a possible future technology allowing the sanitation requirements to be met (Green and Ho, 2005; Wilderer, 2005), nutrient removal in small-scale plants should move into the focus of research interest.
Table 1: Removal efficiencies for several small-scale wastewater treatment plants, compared to the
WWTP Werdhoelzli, Zurich.

<table>
<thead>
<tr>
<th></th>
<th>COD removal</th>
<th>N removal</th>
<th>P removal</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Septic tank</td>
<td>20-30%</td>
<td>0-10%</td>
<td>10%</td>
<td>VSA, 2005</td>
</tr>
<tr>
<td>SBR</td>
<td>&gt; 90%</td>
<td>10-70%</td>
<td>10-70%</td>
<td>VSA, 2005</td>
</tr>
<tr>
<td>Trickling filter</td>
<td>&gt; 90%</td>
<td>10-40%</td>
<td>10%</td>
<td>VSA, 2005</td>
</tr>
<tr>
<td>Reed bed</td>
<td>&gt; 80-90%</td>
<td>10-90%</td>
<td>10-60%</td>
<td>VSA, 2005</td>
</tr>
<tr>
<td>Sand filter</td>
<td>&gt; 80-90%</td>
<td>10-20%</td>
<td>10%</td>
<td>VSA, 2005</td>
</tr>
<tr>
<td>WWTP</td>
<td>95%</td>
<td>60%</td>
<td>90%</td>
<td>Plant data</td>
</tr>
</tbody>
</table>

The optimum technology and plant size will depend on the specific circumstances of each site. However, membrane bioreactor (MBR) technology may be of interest for small-scale wastewater treatment (Fane and Fane, 2005). Thanks to their small reactor size and good effluent quality, which makes the effluent available for reuse, MBRs are an attractive option for decentralized sites. Nutrient removal in (decentralized) MBRs has been studied by several researchers who showed that high levels of both nitrogen and phosphorus removal can be achieved by biological means (e.g. Ahn et al., 2003, Lesjean et al., 2005; Monti et al., 2006; Patel et al., 2006). However, all these reactor systems are adaptations of the conventional activated sludge system: they rely on continuous water flows and pretreatment (e.g. screen), and they are designed for 100 to 1000 persons.

Conventional reactor designs cannot be applied on a household scale for several reasons: water flows are subject to high fluctuation, the reactor size needs to be designed for buffering the water flow and it is difficult to implement clearly defined environmental conditions in every part of the plant. Operating stability, maintenance requirements and soft factors such as noise and odors also need to be considered.

In this study, we investigated the operation of a two-chamber MBR used to treat the domestic wastewater of a four-person household. Two reactor configurations were compared with respect to their operational stability and nutrient removal. The configurations studied included a primary clarifier in the first reactor followed by an MBR in the second reactor for the first period, and an anaerobic/anoxic reactor followed by an MBR for the second period.

**MATERIAL AND METHODS**

**WASTEWATER TREATMENT PLANT**

The treatment plant is located in the basement of a four-person household in Switzerland. All the wastewater produced within the building flows through the treatment plant. The effluent is stored in a tank outside: 35% of it is recycled for toilet flushing and irrigation, while the rest is percolated.

The plant consists of two reactors each with a volume of 1.5 m³ (Figure 1). A flat-sheet membrane (4 m², 0.04 μm pore size) with coarse bubble cross-flow aeration is installed in the second reactor. Oxygen is supplied by fine bubble aeration.

The first tank was initially used as a primary clarifier and subsequently as a biological reactor. During the second period, the sludge from the MBR was pumped to the first tank by an air lift pump. Excess sludge is pumped to a filter bag outside the building at intervals of 1-2 weeks.

*Operation with primary clarifier*

The first reactor was initially used as a sedimentation tank. A baffle with an opening at the bottom was installed in the middle of the reactor to prevent short circuits. The hydraulic retention time was approximately three days. This led to an equalization of the pollutant concentration. A stable, malodorous sludge blanket formed within a short time. The primary sludge was removed when operational mode was changed.
Operation with sludge recycling

The first reactor was retrofitted for several reasons: bad odors, limited nutrient removal and primary sludge production. In order to overcome these disadvantages, mixing between the two reactors was enhanced. This was done by pumping activated sludge from the MBR to the first reactor, where a second air-lift pump was installed for aeration, mixing and dosing wastewater to the MBR. A sieve had to be installed in the first reactor to prevent clogging of the membrane. The presence of oxygen (from the air-lift pump), nitrate and activated sludge (from the MBR) in the first reactor was expected to reduce the production of bad odors and to enhance hydrolysis, denitrification and biological P-removal.

![Diagram](image)

**Figure 1**: Modified flow scheme with hydrolysis chamber. During operation with the primary clarifier, no sieve and return sludge pump were in operation and the first reactor was neither mixed nor aerated.

**CONTROL SYSTEM**

All the pumps (aeration, air-lift pumps) are operated intermittently. Aeration and permeate withdrawal are controlled by the water level. Three operational modes can be differentiated:

- **Sleep mode**: Due to noise emissions from the aeration, which the residents perceived as disturbing, the plant had to be switched off during 7.5 hours at night (10 p.m. to 5.30 a.m.). It would switch on only in case of emergency, i.e., if the water level were to rise beyond an alarm level. During sleep mode, all the pumps are shut down.
- **Standby mode**: This mode can be defined as the filling phase. The aeration system is turned on, but is maintained at a low level. During the second period, the air-lift pumps were also switched on.
- **Pump mode**: This mode is switched on as soon as the water level in the MBR reaches a certain height. After a defined pre-aeration time the permeate pump starts, the cross-flow aeration works all the time and the fine bubble aeration system operates at the defined aeration times. The permeate suction pump is switched off when a minimum water level or “sleep time” is reached.

**WASTEWATER COMPOSITION**

The treatment plant is fed by domestic wastewater. In contrast to most other investigations with small-scale WWTPs, the wastewater does not originate from a sewer system. Several difficulties must therefore be overcome: this wastewater is not diluted by rainwater or infiltrated groundwater, it contains hair and particles, the water flow and pollutant load to the plant fluctuates greatly and is not controllable, and neither the wastewater composition nor the
concentrations in the raw influent can be measured. Furthermore, around 30% of the influent is recycled permeate, which leads to an accumulation of non-degradable substances (e.g. salts, organic matter, color).

**MEASUREMENTS**

Grab samples were taken from the influent and effluent of the MBR, transported to the laboratory and analyzed within hours by the cantonal laboratory of Solothurn according to standard Swiss methods (Clesceri et al. 1998). Online measurement devices from WTW (Weilheim, Germany) of the 700 IQ series were used for oxygen (Trioxmatic), ammonium (Ammolyt), nitrate (Varion), temperature, pH and redox potential (Sensolyt).

Several plant parameters are recorded and can be monitored online: water level (both reactors), drinking water consumption, transmembrane pressure and permeate flow. The switch on and off times of the aeration pumps, air-lift pumps, the permeate pump and the excess sludge pump are also recorded.

**MODEL**

A plant model for dynamic simulations based on Figure 1 and Table 2 was set up in Simba (ifak GmbH, Magdeburg, Germany). The biokinetic model used was the ASM3 (Henze et al., 2000) for organic matter and nitrogen with the parameter set from Koch et al. (2000), extended by the Eawag Bio-P module (Rieger et al., 2001) for the phosphorus components. The influent water flow was modeled using plant data from the water-level measurements. Data from September to November 2005 were used for the operation with the primary clarifier, and from March to June 2006 for the simulations of the second period. This period covered three different load situations: vacation (no influent) during the first seven days, followed by normal load for 42 days and a low-loading period of 14 days at the end.

*Model calibration for operation with primary clarifier*

In the first period, the primary clarifier was not included in the model. The primary effluent concentrations were quite stable over the whole period and were assumed to be constant. Measured COD, nitrogen and phosphorus concentrations were used as influent values (Table 2). COD was fractionated according to Table 3.

**Table 2: Model inlet concentrations and plant conditions for the two periods.**

<table>
<thead>
<tr>
<th></th>
<th>1st period</th>
<th>2nd period</th>
</tr>
</thead>
<tbody>
<tr>
<td>COD&lt;sub&gt;tot&lt;/sub&gt; g&lt;sub&gt;COD&lt;/sub&gt; m&lt;sup&gt;-3&lt;/sup&gt;</td>
<td>560</td>
<td>1100</td>
</tr>
<tr>
<td>N&lt;sub&gt;tot&lt;/sub&gt; g&lt;sub&gt;N&lt;/sub&gt; m&lt;sup&gt;-3&lt;/sup&gt;</td>
<td>100</td>
<td>105</td>
</tr>
<tr>
<td>P&lt;sub&gt;tot&lt;/sub&gt; g&lt;sub&gt;P&lt;/sub&gt; m&lt;sup&gt;-3&lt;/sup&gt;</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Alkalinity mol m&lt;sup&gt;-3&lt;/sup&gt;</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>Q m&lt;sup&gt;3&lt;/sup&gt; d&lt;sup&gt;-1&lt;/sup&gt;</td>
<td>0.36; 0.01 - 0.86</td>
<td>0.34; 0.04 - 0.76</td>
</tr>
<tr>
<td>Q&lt;sub&gt;21&lt;/sub&gt; m&lt;sup&gt;3&lt;/sup&gt; d&lt;sup&gt;-1&lt;/sup&gt;</td>
<td>0</td>
<td>0.25 - 0.6</td>
</tr>
<tr>
<td>V&lt;sub&gt;PC&lt;/sub&gt; m&lt;sup&gt;3&lt;/sup&gt;</td>
<td>1.3 (not modeled)</td>
<td>1-1.3 (fully mixed)</td>
</tr>
<tr>
<td>V&lt;sub&gt;MBR&lt;/sub&gt; m&lt;sup&gt;3&lt;/sup&gt;</td>
<td>0.8-1.3 (fully mixed)</td>
<td>1-1.3 (fully mixed)</td>
</tr>
</tbody>
</table>

**Table 3: Resulting COD fractionation for the two modeled periods.**

<table>
<thead>
<tr>
<th>Fraction</th>
<th>S&lt;sub&gt;S&lt;/sub&gt;</th>
<th>S&lt;sub&gt;I&lt;/sub&gt;</th>
<th>X&lt;sub&gt;I&lt;/sub&gt;</th>
<th>X&lt;sub&gt;H&lt;/sub&gt;</th>
<th>X&lt;sub&gt;A&lt;/sub&gt;</th>
<th>X&lt;sub&gt;S&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Koch et al., 2000</td>
<td>0.1</td>
<td>0.06</td>
<td>0.2</td>
<td>0.09</td>
<td>0</td>
<td>0.55</td>
</tr>
<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt; period</td>
<td>0.1</td>
<td>0.06</td>
<td>0.2</td>
<td>0.05</td>
<td>0.004</td>
<td>0.586</td>
</tr>
<tr>
<td>2&lt;sup&gt;nd&lt;/sup&gt; period</td>
<td>0.1</td>
<td>0.03</td>
<td>0.2</td>
<td>0.05</td>
<td>0.004</td>
<td>0.616</td>
</tr>
</tbody>
</table>

For the period with sludge recycling, the raw influent had to be characterized. Its composition depends strongly on water origin (showering, feces, urine, kitchen sink, etc.) and could not be assessed directly within our system. As the hydraulic retention time in the first reactor still
remained around two days, the influent concentrations were assumed to be constant over the day. Nitrogen and phosphorus influent loads could be estimated from the period with primary clarifier; however, for COD this was not possible. The calibration aimed at obtaining a suitable and realistic COD fractionation, because this has a strong impact on the biological activity. The resulting influent composition and plant conditions are shown in Table 2 and the COD characterization with the model in Table 3. The activated sludge concentration was successfully reproduced on the basis of this inlet characterization. Excess sludge removal was implemented according to real plant data. The influent phosphorus concentration was set to 80% of the estimated influent concentrations due to precipitation processes in the MBR (precipitation processes were not included in the model).

Biokinetic model calibration
When the model was applied, the organic and nitrogen components were reproduced adequately, but the phosphate effluent values were too high. One correction to the Eawag Bio-P module (Rieger et al., 2001) had to be implemented: the decay rate of the PAOs was corrected from 0.2 d⁻¹ to 0.15 d⁻¹ for 20°C, which is within the possible range mentioned by Lopez et al. (2006).

RESULTS

OPERATION WITH PRIMARY CLARIFIER

Sludge production
The F/M ratio was in the range of 0.06 - 0.11 gCOD gVSS⁻¹ d⁻¹, with a sludge age of 30 - 50 days. The activated sludge production in the MBR during this period was 60 gCOD d⁻¹ (+/- 20 gCOD d⁻¹). Sludge removal occurred on a weekly basis and was stopped temporarily. With this operation mode, approximately five to six filter bags with a volume of 40 liters would be filled per year. No measurements of the primary sludge production were made.

Nitrogen removal
Due to the high sludge age, nitrification was very stable. In the mornings, an ammonium peak with values of 5-10 gN m⁻³ could be observed (Figure 2), which was minimized by introducing a pre-aeration step before starting the permeate withdrawal. The peak stemmed from the high water consumption in the morning, when people took a shower, and the ammonium-rich water from the primary clarifier flowed to the MBR. The maximum ammonium concentration in the MBR reached 20% of the influent concentration (Figure 2). Total nitrogen elimination was 50%, including incorporation and denitrification. Incorporation was limited by the sludge production and denitrification by a lack of organic substrate. When applying the dynamic model and changing the operating conditions (aeration times), no significant improvement of denitrification could be obtained without impairing nitrification.
Phosphorus removal

The only P-removing process expected was incorporation into the biomass. With an average sludge production of 60 g COD d\(^{-1}\), a P-removal potential of 0.9 g P d\(^{-1}\) or 10% is calculated (\(i_p = 0.015 \frac{g_P}{g_{COD}} \)). However, the observed P-removal was 2 g P d\(^{-1}\) (Table 4), corresponding to 25% and a P-content of the excess sludge of 3.5%, which indicates that other processes were involved. As the sludge is not exposed to anaerobic conditions, EBPR was excluded (no activity could be detected in batch tests either). Recycling of the wastewater means that potassium, magnesium and calcium ions accumulate in the system, leading to higher concentrations than in the drinking water. With this higher ion concentration and the relatively high pH value (7.8-7.9), precipitation of phosphates (apatite, struvite) may account for the higher P-removal efficiency (Maurer and Boller, 1999). Additional P-removal could originate from the presence of additives in the laundry detergents (zeolithes): these release free aluminum ions which precipitate with the phosphate (Wild et al., 1996).

Table 4: Measured influent and effluent values of the MBR for operation with primary clarifier. The samples were taken in the morning, which leads to elevated ammonium values.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Influent (MBR)</th>
<th>Effluent (MBR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>COD (g_{COD} m^3)</td>
<td>562 (± 120)</td>
<td>38.1 (± 6.8)</td>
</tr>
<tr>
<td>NH(_4)N (g_N m^3)</td>
<td>84.3 (± 12.9)</td>
<td>1.04 (± 1.08)</td>
</tr>
<tr>
<td>NO(_3)-N (g_N m^3)</td>
<td></td>
<td>45 (± 6.7)</td>
</tr>
<tr>
<td>P(_{tot}) (in), PO(_4)-P (eff) (g_P m^3)</td>
<td>23.9 (± 2.0)</td>
<td>17 (± 2.8)</td>
</tr>
</tbody>
</table>

OPERATION WITH SLUDGE RECYCLING

Sludge production

As expected, the sludge production was increased when the primary clarifier was used as a biological reactor, reaching 110 g COD d\(^{-1}\)± 50 g COD d\(^{-1}\), including toilet paper and feces, which are normally kept back in the screen in conventional WWTP. This corresponds to a total sludge production of 30 g Pe d\(^{-1}\) d\(^{-1}\) and three to four people living in the house, which is about the same as total sludge production in a conventional WWTP. The sludge age was between 50-200 d, the F/M ratio could not be determined.

Nitrogen removal

The nitrogen removal was strongly improved with recycling, compared to the first period. Nitrification was still stable, but without the distinct morning peaks observed during primary clarifier operation (Figure 2 and 3).
The most remarkable difference was seen in the denitrification. The nitrate effluent concentrations decreased from 45 g m\(^{-3}\) to 5-10 g m\(^{-3}\) (Figure 4), so that a total nitrogen removal of over 90\% was achieved. This happened immediately after the operating mode was changed.

*Phosphorus removal*

Only a slight increase in phosphorus removal was observed at the beginning of the new operating mode with \(Q_{21}/Q = 0.1\), which could be attributed to the higher sludge production. After adjusting the operating conditions on March 15th (return sludge ratio increased to 1.5 ± 0.5, slight decrease of the aeration intensity), the environmental conditions were suitable for Bio-P organisms. After one month, Bio-P activity was clearly detectable with a phosphate reduction of about 5 g m\(^{-3}\) and PAO-activity measured in batch tests. Due to operating problems, the return sludge ratio was reduced to 0.5 - 1 between April 12th and June 14th. \(Q/Q_R\) was increased to 1.5 again afterwards, which resulted in better P-removal of up to 85\% in December 2006 (Figure 4).

*Optimization of nutrient removal*

Several parameters in the existing system were subject to optimization: aeration times, dosing interval from the hydrolysis reactor to the MBR, return sludge ratio and permeate withdrawal times. The influent values of the calibration period according to Tables 2 and 3 were used for
the modeling of the optimization steps (15.4. - 13.6. 2006). The results were always compared with those of the calibration period.

**Return sludge ratio and aeration control**

In the first four weeks of the calibration period with a return sludge ratio of $1.5 \pm 0.5$, the PAOs accumulated, but remained stable after reducing that ratio to 0.5 (Figure 5). This indicates the importance of the return sludge ratio. As the air-lift pumps were operated with fixed time intervals, the return sludge ratio depended strongly on the influent water flow and could not be regulated exactly.

During the calibration period no real steady-state was reached. For the optimization of the nutrient removal, the system was therefore dynamically modeled for about 200 days, to achieve stable conditions. Subsequently, a period of nine weeks was studied in detail. The model was run with return sludge ratios of 0.6-1.7, which resulted in PAO concentrations in the MBR of 300-1500 g m$^{-3}$, respectively (Figure 5). Increasing the return sludge ratio over 1.2 did not affect the PAO concentrations. Figure 6 shows the corresponding phosphate concentrations in the effluent (a) and the degradable substrate concentrations in the first reactor (b) for three return sludge ratios (1.7, 1.06, calibration period (0.5-1.8)). The results show that Bio-P is substrate-limited with a return-sludge ratio greater than 1.2.

By optimizing the return sludge ratio, an average P-removal of 70% was achieved (compared to 25% during the calibration period). As Figures 5 and 6 show, there is a risk of significant P-loss during and after zero or low loading periods. By reducing the aeration intensity during such periods, PAO decay and P-loss could be reduced. However, this requires an adapted aeration control strategy.

![Figure 5: Modeled PAO concentrations for the calibration period and different ratios of Qr/Q (1.7, 1.26, 1.06, 0.64). During the calibration period, the return sludge ratio varied between 0.5 and 1.8. For the runs with different ratios, the activated sludge composition was brought to a steady-state before modeling this period. When minimizing the aeration intensity during low-loading periods, decay of PAO is reduced.](image)

**Permeate withdrawal**

Sleep mode is one of the operational constraints. It should be overcome in the design of a new system, as ammonium, nitrate and phosphate values reach their minimum between midnight and 6 a.m. in their diurnal variation (Figure 3). Implementation of a different process control that withdraws the permeate during the night unless the water level reaches a maximum level enhances the P-removal by another 10% to a total of 80%.
DISCUSSION

OPERATION WITH A PRIMARY CLARIFIER

The disadvantages of the primary clarifier are the primary sludge production and the formation of a malodorous sludge blanket. The activated sludge is preferentially pumped to the primary clarifier and removed together with the primary sludge, which has to be disposed in a central WWTP.

The nitrification capacity is adapted to the average ammonium load, which leads to ammonium peaks during ammonium shock loading. Denitrification is limited because not enough substrate is available during anoxic periods. The optimization of the aeration times in the model showed no relevant improvement of the nutrient removal.

P-removal is relatively low (25%) with this operating mode. If P-removal is required, then simultaneous precipitation with flocculants, sorption in a post treatment step (Westholm, 2006) or urine separation with separate treatment need to be introduced.

The data indicates that a primary clarifier may be advantageous for membrane permeability (Figure 7). This could be explained by the fact that the membrane chamber receives less suspended solids and organic material, which cause blockages and increased membrane fouling respectively.

OPERATION WITH SLUDGE RECYCLING

With the sludge recycling between the MBR and the first tank, only activated sludge is produced that can be treated locally. In our system, the sludge is pumped into a filter bag where it is dewatered to 10% dry matter. This sludge may be mixed with kitchen waste and composted. If the filter bag is placed in a dry room, a dry matter content of 80-90% is reached without causing any odor problems.

Odor problems in the basement disappeared thanks to activated sludge recycling and intermittent aeration. In view of the better nutrient removal and easier sludge handling (no malodorous primary sludge), this operation mode should be favored in the future.
Despite a very favorable COD:P ratio of 75 gCOD/gP, P-removal is limited to 70-80% according to the model results. This is mainly due to high water and load fluctuations which temporarily lead to an overloading of the plant or increased decay of PAOs due to the minimal aeration. Under optimum conditions, daily P-removal of around 90% can be achieved, which was also shown in reality, when measured effluent P-concentrations went down to 1 gP m$^{-3}$ in November 2006 (Fig 4).

**OPERATIONAL CONSTRAINTS**

*Membrane permeability*

The model results are very positive and promising. Not all the proposed changes could be implemented on the plant because further experiments are still going on (e.g. urine separation). Moreover, the goals of a decentralized MBR may differ under various conditions (legal requirements for effluent quality, water reuse). Safe and straightforward operation of the plant is crucial, and one of the cost drivers is its servicing (e.g. membrane replacement or cleaning, sludge removal etc. (Trussell *et al.*, 2006)). The servicing interval depends mainly on the membrane permeability and is necessary every 6-12 months.

![Figure 7: Development of membrane permeability and maximum daily suction pressure (corresponding to the pressure difference over the membrane) during the observed period.](image)

Figure 7 shows the development of the maximum suction pressure and the permeability of a new membrane module during one year of operation. The membrane was replaced at the beginning of September 2006. It had never been cleaned during operation. There is no obvious change in the decreasing gradient of the permeability profile after starting sludge recycling and intermittent aeration of the first reactor. The permeate flow $Q_{\text{membrane}}$ (see equation 1) remains constant until a permeability of about 50 l m$^{-2}$ h$^{-1}$ bar$^{-1}$ is reached.

\[
Q_{\text{membrane}} = A_{\text{membrane}} \cdot \text{TMP} \cdot \text{Perm} \tag{Eq. 1}
\]

with $A_{\text{membrane}} = 4$ m$^2$, TMP = transmembrane pressure [bar] and Perm = permeability [l m$^{-2}$ bar$^{-1}$ h$^{-1}$].

In the last month with a permeability of less than 50 l m$^{-2}$ h$^{-1}$ bar$^{-1}$, the flow declined to 25-30 l/h. This led to a doubling of the pumping time and capacity problems.

The space between the membrane plates, and the hole diameter of the sieve in the first reactor were both 6 mm. Hair, seeds, and other suspended solids partially passed through the sieve and accumulated between the membrane plates, leading to physical blockage of the plates and impaired efficiency of the cross-flow aeration. This effect was temporarily reduced by increasing the cross-flow aeration time during standby mode at the end of May 2006.

The effect of this blocking on the permeability is not quite clear because no significant permeability increase could be observed after the membrane had been cleaned with high-pressure water flows, active chlorine and acids. The removal of the physical blockage without
associated improvement of the permeability indicates that the decrease in the latter must be attributed to membrane fouling.

**Vacation**

As this treatment plant is operated in a one-family house, the loading is very dynamic but the diurnal variations are strongly equalized by the buffer volume of the first tank. The weekly variation with higher water consumption on weekends leads to longer pumping times, but there are also periods without any influent. During such vacation periods, the aeration should be switched off to minimize decay rates, as was shown by Siegrist et al. (1999). When the aeration is switched off, however, anaerobic and putrescent conditions will become established, leading to the production of methane as well as malodorous and toxic substances which impair the activity of the microorganisms. Loss of phosphate must also be prevented: under anaerobic conditions, PAOs release the stored polyphosphate as phosphate, which might be lost after switching to normal operation again.

The behavior of the Bio-P organisms during vacation periods was determined with lab experiments. When kept in an anaerobic state with short daily aeration times (5-20 min d⁻¹), the phosphate is released within a week. Although the activity remains stable for at least three weeks (Figure 8), 2-4 hours of aeration are required for the P-uptake to be completed before starting the permeate pump.

![Figure 8: Development of phosphate concentration during periods without influent (left) and after switching on the aeration (days 0, 9 and 17). When the residents return from their vacation, the aeration should be switched on for a couple of hours to enable the PAOs to take up the released phosphate before starting permeate withdrawal.](image)

**CONCLUSIONS**

- When operating an MBR with primary clarifier, good COD-removal, stable nitrification and N-removal of around 50% are easy to attain. Phosphorus removal was around 30% by incorporation and natural precipitation. When installed indoors, such a reactor needs to be airtight and equipped with off-gas treatment. The activated sludge is best pumped to the primary clarifier, removed with the primary sludge and disposed in a central WWTP.
- The operation of the first reactor with mixing, aeration, sieving of the inlet to the MBR and sludge recycling results in fewer odor problems, a higher denitrification rate (>90%) and partially enhanced biological P-removal of about 70-90%. This operation has the advantage, that a single, stabilized sludge is produced which can be dewatered and air-dried on-site. The influence of the higher organic and suspended solids load on the membrane permeability will be further investigated during longer operation periods.
According to the simulation results, stable EBPR can be achieved in small-scale membrane systems with sludge recycling under highly dynamic influent conditions and irregular excess sludge removal. Due to the high P-storage in the sludge, decay of PAO during low loading periods needs to be minimized by reducing the aeration times to prevent P-loss. The essential parameter for stable operation is the return sludge ratio.

To prevent a substantial loss of the biological activity of the activated sludge during long zero load (vacation) periods, the sludge has to be kept under anaerobic conditions with short daily aeration periods. Bio-P sludge releases the phosphate anaerobically within seven days but takes it up again after 2-4 hours of aeration.

Remote control by internet and regular servicing of the plant by a licensed plant supplier might be the only solution to prevent malfunctioning and low nutrient removal efficiency of such a small plant.

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CHAPTER 4

SEPARATE URINE TREATMENT COMBINED WITH AN MBR FOR DOMESTIC WASTEWATER TREATMENT – 9 MONTHS PILOT EXPERIENCE

Manuscript in preparation
Separate urine treatment combined with an MBR for domestic wastewater treatment – 9 months pilot experience

ABSTRACT
Small wastewater treatment plants generally have a limited phosphorus removal capacity. In order to achieve low effluent concentrations, advanced processes are necessary. One option is separate urine treatment, as around 50% of phosphate in household wastewater originates from urine. Within the Aquamin project, separate urine treatment for phosphorus recovery combined with an MBR for wastewater treatment was investigated on household scale. Over 95% of phosphate from the separated urine was recovered as struvite by dosing magnesium oxide. Due to clogging of the NoMix toilets, the urine separation efficiency was only 30-40% which impaired the estimation of overall system performance. Nitrification and denitrification are not seriously impacted by dosing of the treated urine as long as the urine is dosed in regular (daily) intervals. Shock loads (visits) may result in higher ammonium effluent loads during short periods. Although anaerobic decay of nitrifiers can not be neglected, the activated sludge should be kept under anaerobic conditions during zero-influent periods (vacation) in order to maintain the nitrification capacity on a high level.

Keywords: urine source separation; struvite precipitation; decentralized wastewater treatment; MBR;

BACKGROUND
Urine source separation has been at the focus of considerable research during the last decade. As urine accounts for only 1% of the volumetric flow but contains a large fraction of the nutrients nitrogen and phosphorus, source separation and separate processing offer several advantages over conventional wastewater treatment (Wilsenach and van Loosdrecht, 2004):
- Processing and recovery are easier for more concentrated solutions.
- Nitrification (or nitrogen removal) determines the size of BNR plants and requires around 50% of the aeration energy.
- Phosphate is usually removed by iron or aluminum addition and can (as yet) only be recovered by complex and expensive processes. Even when applying the sludge directly to farmland - which is now prohibited in many countries - only a small fraction of the phosphorus is available for crops (Römer, 2006). Besides, disposal of the sludge incurs high costs.
- Around 50% of the micropollutants originating from human metabolic processes are excreted via the urine. These may be eliminated to the benefit of water bodies and drinking water quality.

Thanks to this intensive research, there is a very broad range of possible solutions for storing, transporting and processing source-separated urine. Maurer et al. (2006) give a broad overview of this topic.
Extensive experience has been gained in laboratory experiments. Several pilot projects dealing with source separation of urine have been conducted, many of them in Sweden, which is the pioneer in the dissemination of this technology. Most projects described in the literature concern urine storage for several months with subsequent transportation and application to agriculture (e.g. Jönsson et al., 1997; Berndtsson, 2006; van Betuw et al., 2006). The same approach was taken in the “Lambertsmuehle” project in Germany (Otterpohl, 2002). Within the Novaquatis project at Eawag, a pilot plant was developed for fertilizer production based on electrodialysis and ozonation (Pronk et al., 2007). Struvite precipitation seems to be the
most popular process for P-removal and recovery. It can be applied to urine (e.g. Ronteltap et al., 2007) as well as to digester supernatant from EBPR plants (e.g. Battistoni et al., 1998; Booker et al., 1999).

Due to a lack of study sites, it has not yet been possible to study the effects of separate urine treatment on wastewater treatment in practice. Several authors stress the positive effect of source separation, but only Wilsenach and van Loosdrecht (2004) modeled the effects of different ratios of separated urine in a catchment. They concluded that the effects of ammonium and phosphate concentrations in the effluent were negligible due to sufficient capacity for nitrification and enhanced biological P-removal. Positive effects of the total nitrogen effluent load and a capacity increase were mentioned.

The Aquamin project investigated on-site wastewater treatment and recycling with a membrane bioreactor for a 4-person household. Conventional MBR systems have limited capacity for P-removal (15-25% depending on the sludge production) in the absence of any special process. Struvite precipitation from source-separated urine was integrated in this research work in order to minimize phosphorus losses in the effluent. The urine reactor was located next to the MBR in the basement of the house and operated during 9 months. The effluent from the urine reactor was conveyed to the MBR. Besides the practical aspects of reactor operation, the main focus of interest was the effect of urine source-separation on biological wastewater treatment.

**PRELIMINARY EXPERIMENTS**

**CHEMICAL REACTION**

Struvite precipitation from various waste streams has been investigated by many researchers. The process is mainly applied in order to recover phosphate from digester supernatants (of EBPR plants: Adnan et al., 2003, Battistoni et al., 1998), cattle or pig manure (Burns et al., 2003) or from human urine (Wilsenach et al., 2007). The chemistry of struvite precipitation is well known:

\[
\text{Mg}^{2+} + \text{NH}_4^+ + \text{PO}_4^{3-} + 6\text{H}_2\text{O} \rightarrow \text{MgNH}_4\text{PO}_4 \cdot 6\text{H}_2\text{O} \quad (\text{Eq. 1})
\]

Struvite precipitates spontaneously in stored urine, as urea is hydrolyzed to ammonia by naturally occurring bacteria (Udert et al., 2003), but the precipitation is limited by the availability of magnesium. The most important parameters affecting P-removal by struvite precipitation from human urine are pH, magnesium concentration, contact time and seeding material (Battistoni et al., 1998; Booker et al., 1999; Wang et al., 2006).

**REACTOR SETUP**

Within the Aquamin project, Eawag cooperated with a partner from industry who had developed a struvite precipitation reactor. This reactor had been in use for some time at their head office with good results. In order to gain more experience with it and in view of a lack of time to develop a customized small-scale solution, the same reactor was installed at the study site after pilot testing in the lab. Although the reactor dimensions were far too large for mass production or use in private homes, this handicap was accepted as the study aimed to investigate pilot operation and compatibility with biological wastewater treatment.

The reactor configuration is described in the following section. Numbers in brackets refer to Figure 1: Source-separated urine is collected in a storage tank (1) with \( V_{\text{max}} = 60 \text{ l} \). After reaching a pre-defined fill level (\( V_{\text{start}} = 37 \text{ liters} \)), a magnetic valve (2) opens and the urine flows into the reaction vessel (3). \( \text{MgO} \) is dosed through a spiral pump (4) before a paddle mixer (5) is started. The reaction (Eq.1) takes place during mixing. Mixing is followed by a
sedimentation period. By opening a second magnetic valve (6), the supernatant is decanted to the second storage tank (10). The ball valve (7) at the bottom of the reaction vessel is opened after short stirring (to prevent clogging), and the precipitates are collected in a textile filter bag (8). Excess liquid drips into a funnel (9) and is conveyed to the storage tank (10). The phosphate-poor urine is then periodically pumped (11) to the MBR.

Figure 1: Flow scheme of the urine reactor. The numbers refer to the text.

STOICHIOMETRY AND REACTION KINETICS

In order to assess the suitability of MgO, preliminary jar experiments were conducted in the laboratory. The stoichiometry and kinetics of the process were investigated. Filtered urine (0.7µm GF/F filter) was used to assess the kinetics of MgO dissolution and phosphate removal. The results are presented in Figure 2a (stoichiometry) and 2b (kinetics).

Figure 2: a) Molar ratio necessary for P removal with MgO dosing. b) Kinetics of the experiments: after 60 minutes of stirring, the reaction is complete. b indicates the molar ratio of magnesium dosed to initial phosphate. C_0 refers to either initial phosphate or dosed magnesium concentration.
All experiments were conducted with male urine from the collection tanks at Eawag (from waterless urinals and NoMix toilets), at room temperature (20-22°C) and without pH control. The experiments showed that the pH rose by 0.1 units with MgO addition.

Our results indicate that a molar ratio of Mg:P of at least 1.2 has to be maintained in order to attain more than 95% P removal. This value is higher than reported by Wilsenach et al. (2007) but in the same range as given by Tettenborn et al. (2007). The reason for the difference might be that Wilsenach worked with synthetic urine. The experimental data and literature values also differ with respect to the kinetics. According to Ronteltap et al. (2007), precipitation occurs within 5-10 minutes of stirring when using MgCl₂ as a magnesium source. In our experiment, the reaction was completed only after 60 minutes (Figure 2b) after a fast initial decrease of the phosphate concentration. Free Mg²⁺ ions are only present at very low phosphate concentrations.

The reason for the longer reaction time may be the slow dissolution of MgO, as it is almost insoluble in water but sufficiently soluble in high-strength solutions (digester supernatant, urine, manure). The slow reaction may also contribute to the production of other crystals, e.g. brucite or montgomeryte, as mentioned by Lind et al. (2000).

**SEEDING MATERIAL**

Four factors affect the struvite precipitation: supersaturation of the chemical components (Mg²⁺, NH₄⁺, PO₄³⁻), pH, mixing energy and seeding material. To reach supersaturation, magnesium has to be added. The pH of stored urine is already in a favorable range (9-9.4), and mixing energy is provided by either aeration or stirring. Stored urine contains a wide range of particles, ranging from hair via crystals to microorganisms. The layout of the treatment unit meant that neither the pH nor the mixing strength could be modified. The effect of seeding crystals was tested by running the urine reactor five times in a row without removing the struvite solution from the reaction vessel. The particle size distribution was analyzed with a Malvern MasterSizer X (Malvern Instruments Ltd, Worcestershire, England). The particle size distribution was hardly affected at all, probably due to the very intense mixing strength (G value estimated to be > 500 s⁻¹). Wang et al. (2006) observed that crystal growth was limited with G values > 76 s⁻¹. A detrimental effect was the formation of a very stable struvite layer above the ball valve, which blocked the outflow of the struvite solution into the filter bag and had to be removed mechanically.

**RESULTS - URINE REACTOR**

**PILOT PLANT INSTALLATIONS**

The MBR started operation as soon as the tenants moved to their new house - in October 2004. Pilot testing of the urine reactor was planned to start in 2006. However, all the installations (NoMix toilets, urine pipes, space) had to be prepared beforehand. Three urine-diversion toilets of the Roediger type (Roediger Vacuum AG, Hanau, Germany) were installed. To prevent clogging, the urine traps were flushed with citric acid every one or two weeks. During the first phase of the project, the urine was conveyed to the MBR. The urine reactor was tested extensively at Eawag to detect any operational problems such as bad odors, malfunctioning, clogging or even flooding. The storage tank and reactor were sealed and all air in direct contact with urine was passed through an activated carbon filter. With this measure, bad odors were successfully avoided except during maintenance work.

Several safety measures were taken to avoid flooding: A spill from the urine storage tank to the MBR was installed in view of possible technical defects. A fill-level sensor in the second storage tank prevented the urine reactor from running as long as the tank still contained urine. Pilot testing was delayed by almost a year and finally started in December 2006.
URINE PRODUCTION AND NoMix TOILETS

Per capita urine production ranges from 1.0 - 2.5 liters person\(^{-1}\) day\(^{-1}\) with an average of 1.5 liters, depending on water uptake and transpiration (Rauch et al., 2003). During the testing period, 2-3 persons lived in the house, all of them going to school/work during the day and thus using the toilets there. The expected urine production was in the range of 2-4 l d\(^{-1}\). The data is shown in Figure 3.

Figure 3: Average daily urine flow and urine reactor runs (batch). The urine flow decreased from over 3 l d\(^{-1}\) in February to 0.5 l d\(^{-1}\) in June, mainly due to clogging of the NoMix toilets. The “batch” series indicates runs of the urine reactor.

During an initial period until mid-April, daily urine production was approximately 3 l d\(^{-1}\), followed by a dramatic decrease and reaching a minimum of only 0.5 l d\(^{-1}\) by the end of June. Unfortunately, this trend was not detected immediately and could not be attributed to changing habits of the tenants. In fact, the decrease was caused by the poor separation efficiency of the NoMix toilets due to clogging of the urine trap. Four experiments (five measurements each) with a hardly used NoMix toilet at the Eawag main building and the toilets in the Aquamin house led to the conclusion that only 30-55% of the urine was separated (see Table 1), the remainder being lost through the normal toilet wastewater.

Figure 4 shows a photograph of the urine trap of a urine diversion toilet. The effluent pipe is connected to this cross section. It can be seen that the cross section is almost completely clogged, except for a small hole (white circle) at the top. Chemical analysis of the sediments in the urine trap revealed that the precipitants consisted of a mixture of calcium and magnesium crystals, with an approximately 1:1 ratio of magnesium to calcium.

URINE REACTOR OPERATION

As demonstrated in the preliminary experiments, the effluent of the urine reactor was virtually P-free, with concentrations of around 2 mg l\(^{-1}\). The stoichiometric Mg:P ratio was chosen to be approximately 1.8. The P loads to be treated were smaller than expected due to a) the poor separation efficiency of the NoMix toilets as mentioned above, and b) precipitation in the storage tank.

<table>
<thead>
<tr>
<th>Toilet</th>
<th>Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eawag</td>
<td>99%</td>
</tr>
<tr>
<td>Aquamin 1a</td>
<td>37%</td>
</tr>
<tr>
<td>Aquamin 1b</td>
<td>32%</td>
</tr>
<tr>
<td>Aquamin 2</td>
<td>52%</td>
</tr>
</tbody>
</table>

Table 1: Separation efficiency for different NoMix toilets

Figure 4: Urine trap of a NoMix-toilet with precipitates. The circle indicates the outlet.
As regards the operation of the urine reactor, the following problems occurred:

- Scaling problems presented a challenge from an operational point of view. Wilsenach et al. (2007) observed excessive scaling: approximately 50% of the total precipitant had to be removed mechanically from reactor walls and impeller blades. Similar problems were observed in our reactor. A thick mineral layer deposited on impeller blades and reactor walls. However, the most crucial and delicate place for deposition was the ball valve. Initially, the struvite remained in the reaction chamber for too long, resulting in a stable struvite layer on top of the ball valve. This layer hindered proper opening and closing of the valve, resulting in either blocking or immediate draining of the reaction vessel. With a sedimentation time of 4 hours, no operational problems occurred.

- Stored urine is a highly malodorous liquid. Although almost no odor problems were noticed during appropriate operation, special operations like changing the struvite filter bag lead to direct contact with malodorous air. It is questionable whether tenants would be prepared to perform this task.

After some startup difficulties during the first three months, the urine reactor was operated without breakdowns for the rest of the testing period (6 months).

**COMBINATION UR/MBR – PHOSPHATE**

The primary goal of the urine reactor was to achieve a treatment scheme with enhanced phosphate removal capacity. However, the effect on the overall removal rate was almost impossible to determine because of the poor separation efficiency of the toilets and the decline in EBPR activity with decreasing P-load. Besides, P-removal was also affected by two events involving operational problems when activated sludge was lost. The time series of the phosphate effluent concentration is shown in Figure 5, with high P-concentrations during the first two months, when several technical problems occurred. The arrows in Figure 5 indicate operational problems in the MBR. The grab samples show higher than average effluent values, because they were taken in the mornings, when the effluent concentrations were increased. The effluent concentrations were similar to the period with only biological phosphorus removal and no urine separation in 2006.

![Figure 5: Phosphate effluent concentrations (grab samples) of the MBR during operation with the urine reactor. Arrows indicate periods with operational problems (sieve clogging). Several failures occurred during the first 50 days 2007.](image)

Due to odor problems, the MBR could not be operated with primary clarifier during the testing period. It was therefore hardly possible to identify the phosphate removing mechanisms. But as the effluent of the UR is virtually P-free, separate urine treatment certainly is a good way to increase phosphate elimination.
COMBINATION UR/MBR – NITROGEN

In the urine reactor, only marginal quantities of nitrogen are incorporated into struvite (1 mole nitrogen per mole phosphate → max. 3%). Nitrogen removal was therefore expected to be unaffected by the urine reactor. However, three weeks after the installation of this reactor, extremely high ammonium and nitrite concentrations were measured in the MBR (Figure 6). As the online sensor was only calibrated between 0 and 10 gN m⁻³ and the output value was limited to 30 gN m⁻³, the absolute values are not representative, but the trend is obvious. Single lab measurements revealed ammonium concentrations of 45 and 55 gN m⁻³ on the 3rd and 9th of January respectively.

Several technical failures occurred at the beginning of the urine experiment. During an MBR service, which was conducted immediately before connecting the urine reactor, approximately 50% of the activated sludge was removed. Afterwards the reactor was filled with tap water and no urine was conveyed to the MBR for 12 days (first urine collection in the storage tank), thus reducing the nitrifier population. After the first run of the urine reactor, influent ammonium loads reached normal levels again. On January 9th, the urine reactor was emptied immediately due to a technical failure.

The whole period was dynamically simulated using ASM3. Temperature, pH, and oxygen concentrations were all favorable for nitirification, and the model predicted that the nitrifiers would recover very quickly even when their growth rate was reduced by 50%. So the reason for the ammonium and nitrite peaks remained unclear.

Following this event, the maximum activities of ammonia (AOB) and nitrite oxidizing bacteria (NOB) were measured weekly. The first measurement at the end of January revealed that the two populations were not in equilibrium. The ammonium oxidizing capacity was substantially higher than the nitrite oxidizing capacity (data not shown). This was also confirmed by relatively high nitrite concentrations in the effluent. Later on, only marginal differences between the populations were detected.

The maximum nitrification capacity followed the average water flow (corresponding to the ammonium load) into the MBR during the preceding 7 days, i.e. it depends on the average ammonium influent load (Figure 7). The deviations derive from the inaccuracy of the measurement and emptying of the treated urine storage tank, which usually took 10 days. For normal operation of the urine reactor, no negative impact on nitrification (and denitrification) was observed. The two distinctive minima in Figure 7 occurred during vacation periods, but the nitrifiers obviously recovered very quickly. As a result of the long hydraulic residence time, irregular peak loads were equalized and the effluent concentrations remained at a low level.
Following the problematic startup period, no further problems of nitrification or denitrification with high effluent concentrations were observed.

**SIMULATION RESULTS**

The nitrification capacity was reduced during low loading periods as shown in Figure 7. As these periods were rather short, the consequences of extreme events (vacation, visits) were assessed with dynamic modeling, using the calibrated model applied for the study in Chapter 3. Surprisingly, the modeled nitrifier concentrations \( X_A \) did not correspond to the measured nitrification rates at all. The measured activity was significantly lower. Siegrist *et al.* (1999) investigated the decay of nitrifiers under aerobic, anoxic and anaerobic conditions and showed that the decay rates are reduced under anoxic and anaerobic conditions. As the anaerobic compartments in activated sludge treatment are generally very small and the biomass decay in them is low, the influence of anaerobic decay is usually negligible. In the Aquamin plant, the sludge is exposed to anaerobic conditions for approximately 50% of the time, so anaerobic decay is expected to play a role. For modeling, therefore, the anoxic decay rate of the nitrifiers (Eq. 3) was replaced by anoxic and anaerobic decay (Eq. 4).

\[
\rho_{anoxic\ decay} = b_{AUT} \cdot \eta_{anox} \cdot \frac{K_{O_2}}{K_{O_2} + S_{O_2}} \cdot \frac{S_{NO}}{K_{NO} + S_{NO}} \cdot X_A \quad \text{Eq. 3}
\]

\[
\rho_{anoxic\ decay,\ new} = b_{AUT} \cdot \eta_{anox} \cdot \frac{K_{O_2}}{K_{O_2} + S_{O_2}} \cdot X_A \quad \text{Eq. 4}
\]

With this model adaptation, the modeled nitrification rates complied well with the measured rates and the influence of vacations and visits could be reasonably modeled. Three scenarios were consequently investigated, both for the WWTP with conventional primary clarifier and for the biological pretreatment (Abegglen *et al.*, 2008):

- **Vacation 1**: Two-week vacation period with emptying of the urine reactor during that period. After the vacation, 12 days elapse until the next run of the urine reactor. This scenario was chosen because of the large size of the studied urine reactor. This scenario is not relevant for smaller urine treatment units.

- **Vacation 2**: Two-week vacation period without any influent. The urine reactor runs the day after the holidays. This scenario corresponds to a treatment plant without a urine reactor during a vacation period.
- Shock-load: Increased urine production due to guests, resulting in emergency emptying of the urine reactor, with 15 liters of urine being discharged to the first reactor within 15 minutes.

**Figure 8:** Modeled nitrifier concentration in the MBR over time for normal operation (reference), two-weeks vacation with emptying of the urine reactor from day 10-18 (vacation 1), two-weeks vacation without urine in the influent (vacation 2). The nitrifier concentration is reduced to one third within 14 days and recovers within one to two weeks.

The model results indicate that neither a visit nor long vacation periods have a large impact on the nitrogen effluent values, although the nitrifier concentration declines by 70% within two weeks (the decay would be minimized if the aeration were completely switched off). The long hydraulic residence time results in slow concentration changes and thus good adaptation of the biomass to the new situation. There is, however, a risk of severe failure, as seen during the start-up phase of the urine reactor, although it seems that a membrane bioreactor recovers by itself as no activated sludge is lost through the effluent.

The dosing of the urine was also assessed with the model. Both in reality and in the simulation, the average daily urine production was pumped into the reactor every morning. The model was used to simulate dosing every two (6 liters), seven (21 liters) and ten (30 liters) days. Dosing every seven or ten days resulted in significantly higher ammonium concentrations up to 10 gN m\(^{-3}\).

The model was also applied to the operating mode with a primary clarifier. The results for nitrification were similar, both for the reference case and the extreme events. As no EBPR can be achieved with this operation mode, there is a greater impact on the overall phosphate removal performance.

**CONCLUSIONS**

- Struvite precipitation is a robust process for P-recovery from urine and can be operated automatically, even in private houses. Precipitation at unfavorable locations in the reactor (e.g. impeller blades, valves) strongly disturbs the operation of the urine reactor and leads to failures.
- Advances in toilet technology are necessary. Despite careful maintenance by the users with dosing of citric acid, the separation efficiency deteriorated, dropping to only 35% after three years of operation.
- Maintenance of the urine reactor and handling of struvite have to be carried out by service personnel. Bad urine odors are a nuisance.
- For biological wastewater treatment with a primary clarifier, separate urine treatment for phosphorus removal or recovery is feasible and reduces phosphate loads in the effluent.
- The discharge of treated urine after a delay has no influence on the nitrification capacity of a small wastewater treatment plant. Even special events like vacations or visits have only a small impact on the treatment efficiency. The main reason for this is the long
hydraulic residence time resulting in slow concentration changes and the adaptation of the activated sludge to changing conditions.

- Anaerobic decay of nitrifiers is lower than aerobic decay but has to be taken into account in small systems with long anaerobic residence times.

REFERENCES


CHAPTER 5

ELECTROCHEMICAL PHOSPHATE REMOVAL FOR SMALL-SCALE WASTEWATER TREATMENT PLANTS

Christian Abegglen, Adrian Sigrist, Hansruedi Siegrist

Manuscript in preparation
**Electrochemical Phosphate Removal for Small-Scale Wastewater Treatment Plants**

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* Corresponding author

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**ABSTRACT**

The phosphorus removal capacity of small wastewater treatment plants is generally limited. In order to achieve low effluent concentrations, advanced processes are necessary. Most commonly, phosphate is precipitated by dosing aluminum or iron solutions. In this study, iron dosing by electrically dissolving submerged iron electrodes (electrocoagulation) was assessed in the laboratory and applied in an MBR treating household wastewater. As neither mechanic equipment nor chemical handling are necessary and due to the simple control by adjusting voltage to the flow, this process minimizes maintenance and is well suited for application in MBRs or SBRs.

The lab results proved the feasibility, efficiency and safety of the process. Phosphate concentrations of 0.3 mg l⁻¹ were reached in batch experiments with an efficiency of around 50% (phosphate removed/current applied). The risk of explosion by hydrogen production is negligible, and no negative impact was detected with respect to biological activity or membrane permeability. The formation of a passivation layer on the electrodes seriously impairs the efficiency of the process but can be reduced by alternating current direction and applying coarse bubble aeration. If this problem can be solved, application of electrocoagulation is a promising alternative for enhanced P-removal in small wastewater treatment plants.

**Keywords:** decentralized wastewater treatment; electrocoagulation; on-site; phosphate removal; precipitation;

**INTRODUCTION**

**NUTRIENT REMOVAL IN SMALL WASTEWATER TREATMENT PLANTS**

The nutrient removal capacity of small-scale wastewater treatment plants is often deficient. While nitrogen removal is usually achieved by nitrification/denitrification, several ways to remove phosphate are available, depending on the plant layout and treatment processes.

The most popular process for phosphate removal in large-scale wastewater treatment is precipitation with iron or aluminum. This process cannot be easily adapted to small-scale biological wastewater treatment plants. Due to the precipitant handling and the large fluctuations of the influent loads, it is not easy to define control strategies for correct dosing. Adsorption processes with different substrates (e.g. granulated iron hydroxide, lightweight aggregate, wollastonite) are also widely used (Adam et al., 2006; Genz et al., 2004; Westholm, 2006). By filtering the pre-treated wastewater through these adsorbing materials, phosphate is bound to the surface of the filter media and thus removed. The sorption capacity of the filter media is usually in the range of a few mg P kg⁻¹, hence this treatment requires a lot of space and the filter material needs to be replaced after reaching saturation.
Enhanced biological P-removal, which is also carried out in municipal wastewater treatment, is also possible in smaller treatment plants, but needs certain preconditions (Abegglen et al., 2008), e.g. anaerobic conditions, aeration control and regular excess sludge removal. A less extensively tested option is urine separation and treatment. The treatment options for phosphate removal are summarized in Table 1.

Table 1: Overview of P-removal processes used in small WWTP

<table>
<thead>
<tr>
<th>Process</th>
<th>+</th>
<th>-</th>
<th>Literature</th>
</tr>
</thead>
<tbody>
<tr>
<td>Precipitation Fe/Al</td>
<td>Well-known process,</td>
<td>Sludge production</td>
<td>Harms and Englert, 2006</td>
</tr>
<tr>
<td></td>
<td>sludge settleability</td>
<td>Chemicals required</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dosing and maintenance</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>difficult</td>
<td></td>
</tr>
<tr>
<td>Adsorption in filter</td>
<td>“No” maintenance,</td>
<td>Land requirement</td>
<td>Adam et al., 2006</td>
</tr>
<tr>
<td>beds</td>
<td>no extra sludge</td>
<td>Clogging</td>
<td>Genz et al., 2004</td>
</tr>
<tr>
<td></td>
<td>production</td>
<td>Replacement after</td>
<td>Westholm, 2006</td>
</tr>
<tr>
<td></td>
<td></td>
<td>reaching saturation</td>
<td></td>
</tr>
<tr>
<td>EBPR</td>
<td>Little extra sludge</td>
<td>Monitoring</td>
<td>Abegglen et al., 2008</td>
</tr>
<tr>
<td>production, no chemicals</td>
<td></td>
<td>Only with MBR / SBR</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>possible</td>
<td></td>
</tr>
<tr>
<td>Urine source separation</td>
<td>Recovery possible</td>
<td>Only pilot testing</td>
<td>Abegglen, 2008</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Toilet and piping</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>maintenance (clogging)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Only 50% of P in urine</td>
<td></td>
</tr>
</tbody>
</table>

In this study, the suitability of electrochemical P-removal (EloP) through precipitation with iron for a small WWTP treating domestic wastewater of a 4-person household was investigated. In contrast to conventional systems, iron was not supplied in the form of iron chloride or sulfate, but by electrically dissolving iron from steel electrodes. This process is called electrocoagulation.

**ELECTROCOAGULATION**

Electrocoagulation involves the in-situ generation of coagulants by dissolving metal ions electrically from an anode, usually iron or aluminum (Chen, 2004). Simultaneously, a reduction process takes place at the cathode, which usually involves the generation of electrolytic gases (i.e. hydrogen). As we worked with steel electrodes, we will only focus on the iron chemistry. The main reactions taking place at the anode, cathode, and in the medium are listed in Table 2.

Table 2: Overview of the most important chemical reactions involved in the electrocoagulation process applied to phosphate removal with steel electrodes (Standard potentials from Mortimer, 1996)

<table>
<thead>
<tr>
<th>Reaction</th>
<th>$E_0$</th>
<th>Eq.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anode</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fe(s) $\rightarrow$ Fe$^{2+}$ + 2e$^-$</td>
<td>0.44</td>
<td>1</td>
</tr>
<tr>
<td>2 H$_2$O $\rightarrow$ O$_2$ (g) + 4 H$^+$ + 4e$^-$</td>
<td>-1.23</td>
<td>2</td>
</tr>
<tr>
<td>2Cl$^-$ $\rightarrow$ Cl$_2$ (g) + 2e$^-$</td>
<td>-1.36</td>
<td>3</td>
</tr>
<tr>
<td>oxidation of organic material</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cathode</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2H$^+$ + 2e$^-$ $\rightarrow$ H$_2$(g)</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>2H$_2$O + 2e$^-$ $\rightarrow$ 2 OH$^-$ + H$_2$(g)</td>
<td>-0.83</td>
<td>6</td>
</tr>
<tr>
<td>Bulk solution</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fe$^{2+}$ + 2OH$^-$ $\rightarrow$ Fe(OH)$_2$</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>Fe$^{2+}$ + 0.25O$_2$ + H$^+$ $\rightarrow$ Fe$^{3+}$ + 0.5H$_2$O</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>(3r-3) H$_2$O $+$ r Fe$^{2+}$ $+$ PO$_4^{3-}$ $\rightarrow$ FePO$<em>4$(OH)$</em>{(3r-3)}$ + (3r-3) H$^+$</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>2 Ca$^{2+}$ + HPO$_4^{2-}$ + 2 OH$^-$ $\rightarrow$ Ca$_2$HPO$_4$(OH)$_2$</td>
<td>10</td>
<td></td>
</tr>
</tbody>
</table>

Electrochemical processes are actually well-known in water and wastewater treatment (Chen, 2004) and were already used in the 1920s (Pearse, 1935). Today, electrochemical methods are
mainly used in industrial wastewater treatment, where the process is often combined with flotation. The electrodes are made from different metals and usually only act as catalysts. Electrocoagulation for P-removal has already been investigated by several researchers (Groterud and Smoczynski, 1985; Bektas et al., 2003; Irdemez et al., 2006; Yu et al., 2006; Sakakibara and Nakajima, 2002). However, they all implemented it as a separate process. The combination of electrocoagulation for P-removal with biological wastewater treatment has only been described by Myung and Yu (2003) and Yu et al. (2006). The reported operational parameters (voltage, current, current efficiency, pH shifts) vary strongly between these studies. One goal of this study was to show the influence of different ambient conditions and operational parameters on process performance. Besides the side effects of electrocoagulation, such as pH shifts, hydrogen production or its influence on bacterial activity were investigated. The experiments were divided into two parts, namely determining the crucial operational parameters in laboratory experiments, and implementing the electrocoagulation in biological treatment.

**MATERIAL AND METHODS**

**ELECTRODES AND CONTROL UNIT**

An electrolytic cell consists of two electrodes in contact with an electrolyte. In this case, the electrolyte was wastewater (or activated sludge), and the electrodes were made of steel, with dimensions 60cm x 30cm x 0.5cm. The setup is shown schematically in Figures 1 and 2.

![Figure 1: Schematic diagram of the electrodes (left) and the redox and precipitation reactions in the bulk (right)](image)

By applying an electric current, electrons are withdrawn from the anode and iron is dissolved as Fe\(^{2+}\). Once in solution, the ferrous ions can react directly with constituents of the wastewater but are very quickly oxidized to ferric ions Fe(III) in the presence of oxygen. Fe(III) then reacts with hydroxide or phosphate ions (Figure 1, right). The iron dosage is determined by the electric current applied to the system. The amount of released metal is theoretically calculated from Faraday’s law of electrolysis:

\[
W_i = \frac{I \cdot t \cdot M_i}{n_i \cdot F} \tag{Eq. 11}
\]

where \(W_i\) is the amount of released substance \(i\) [g], \(I\) is the current [A], \(t\) is time [s], \(M_i\) is the atomic weight of the substance \(i\) [g mol\(^{-1}\)], \(n_i\) is the moles of electrons released per mole of substance \(i\) [-] and \(F\) is the Faraday constant [96485 As mol\(^{-1}\)]. In the experiments, a current of 1 to 6 A was applied.

With the equipment used in this study, the voltage difference \(\Delta U\) could be controlled manually. The resulting current \(I\) is the product of the electrode surface \(A_{\text{electrode}}\) and the
current density $i$ and depends on the potential of the solution between the electrodes, the equilibrium potential difference of the electrodes and the anode and cathode over-potentials (Scott, 1995; Chen, 2004).

$$\Delta U = E_{eq} + \sum \eta_{anode} + \sum \eta_{cathode} + \frac{d}{\sigma} \cdot i$$

(Eq. 12)

The solution potential is a function of the conductivity ($\sigma$), the distance between the electrodes ($d$), and the current density ($i$). The over-potential includes losses due to a passivation layer. The equilibrium potential difference of the electrodes is negligible.

**Batch tests**

Batch tests were conducted in order to identify the most important operational parameters as well as the influence of conductivity and the production of hydrogen. The reactor had a volume of 230 liters and was filled with tap water, MBR effluent or activated sludge from a conventional WWTP. Mixing was performed either by an air-diffuser placed under the electrodes or a submersed pump if no oxygen/air had to be supplied.

**Field test**

The field test was conducted in a small membrane bioreactor located in the basement of a 4-person household. The MBR is described elsewhere (Abegglen et al., 2008) and was operated with a primary clarifier, which means that the wastewater composition was relatively constant throughout the day. The electrocoagulation was only in operation when permeate was withdrawn from the MBR, so the iron dosing was approximately proportional to the volumetric flow.

![Figure 2: Schematic diagram of the EloP application in the MBR](image)

**Measurements**

The batch tests were conducted with tap water, permeate from an MBR treating domestic wastewater or activated sludge from a conventional WWTP. Phosphate was added as K$_2$HPO$_4$ or KH$_2$PO$_4$ (Merck).

Samples for measurement of dissolved constituents (Cl$^-$, PO$_4$-P) were first filtered with 0.7 µm glass-fiber filters (Pall Corp.) and then analyzed by ion chromatography. Fe$^{2+}$ samples were analyzed with phenantroline as described in standard methods (APHA, 1998). Fe$_{tot}$ samples were acidified with nitric acid, solubilized with microwaves and then analyzed by ICP. The voltage and current were measured with a digital multimeter (M-3890D, Metex Instruments, Seoul, Korea). The oxygen, pH and electrical conductivity measurements were conducted with probes of the 330i series from WTW (Weilheim, Germany).

**Hydrogen production**

The hydrogen production was measured with two methods in batch experiments. The reactor was sealed with several layers of plastic film under which a hydrogen probe was placed (Draeger MiniWarn, Draeger Safety AG, Luebeck, Germany). Evolution of the hydrogen concentration in the air compartment was measured during a certain time period. The
diffusion of the hydrogen gas through the plastic film was estimated from the concentration gradient after switching off the current.

The second method consisted of filling a glass beaker with water and inverting it above the cathode. The gas was then collected in the beaker and the resulting gas volume measured. The total hydrogen production was estimated by assuming gas-liquid equilibrium in the volume between the electrodes. As the reactor was not mixed during these experiments, this assumption seems reasonable.

RESULTS

CHEMICAL REACTIONS

Anode
At the anode, iron is dissolved from the steel plates. Measurements of the total iron concentrations in batch tests showed that the efficiency was only about 60% of the theoretically expected value according to Equation 11. The production of oxygen (Eq. 2) or chlorine gas (Eq. 3) is often reported to compete with the dissolution reaction. In view of the higher standard potential of these reactions (Table 2), they are not expected to be of major concern. Measurements of the chloride concentration revealed that chlorine production accounts for 5% of the current consumption at most.

Cathode
The production of hydrogen (Eq. 5 or 6) is the most important reaction at the cathode. Hydrogen is either produced from protons (Eq. 5) at pH < 7 or hydroxide ions (Eq. 6) at pH > 7 (Holt, 2003). Due to the poor solubility of hydrogen, gas bubbles are formed. Measurement of the hydrogen production revealed that between 50-95% of the theoretically expected quantity was formed. In short-term experiments, no difference in hydrogen production was measured with or without activated sludge, which indicates that H₂ is not used as an energy source by microorganisms.

Bulk solution
Phosphate adsorbs to iron(III) hydroxide complexes or precipitates by forming iron(III) phosphates (Stumm and Sigg, 1979). The Fe²⁺ ions dissolving from the anode first need to be oxidized to Fe³⁺. The kinetics of this reaction can be formulated according to Stumm and Lee (1961) and Millero et al. (1987):

\[
\frac{d[Fe(II)]}{dt} = -k_{ox} \cdot [Fe(II)] \cdot [O_2] \cdot [OH^-]^{x(OH)}
\]

(Eq. 13)

where \( k_{ox} \) (8.75*10¹⁵ M⁻¹(1+x(OH)) min⁻¹ at 25°C) is the rate constant including temperature and ionic strength effects and \( x(OH) \) describes the dependency on pH, with values ranging from 1.9 to 2 according to our experiments with tap water, MBR effluent and activated sludge (Millero et al., 1987 found \( x(OH) = 2 \) in seawater). The equation indicates that the reaction depends strongly on the pH value. As a rule of thumb, one can say that with a pH > 7.5 and oxygen concentrations above 1 mg l⁻¹, the half time of Fe²⁺ is at most 25 minutes for wastewater, and the production of Fe²⁺-complexes can be neglected. With activated sludge, the process is significantly faster.

At high Ca²⁺ and PO₄³⁻ concentrations, hydroxydicalcium phosphate (HDP) may be formed (Maurer and Boller, 1999) at pH values above 7.3 (Eq. 10). HDP will then either dissolve again or may be transformed to hydroxyapatite Ca₅(PO₄)₃OH. These reactions are very sensitive towards pH, with higher precipitation rates at high pH values. According to an
Austrian supplier of electrolytic wastewater treatment (Europhat, 2008), local pH shifts near the electrode surface force the precipitation of calcium phosphates as a predominant P-removal process. This mechanism was also investigated in batch tests. With a bulk solution pH of below 8.4, the calcium concentration was not reduced. So precipitation of calcium phosphates only plays a minor role under the environmental and process conditions studied here.

**KINETICS**

Kinetic experiments were conducted in order to find out which of the three steps, namely iron dissolution, oxidation of Fe$^{2+}$ and phosphate precipitation, is rate-limiting. As both oxidation and precipitation are very fast processes under the prevailing ambient conditions (pH $> 7.5$, oxygen concentration $> 1$ mg l$^{-1}$), the iron dissolution was found to be the rate-limiting step.

**REACTION EFFICIENCY**

There are two main contributions to the overall efficiency of phosphate removal. Firstly, the current efficiency $CE$ defines the yield of the electrolytic process (Equation 14, Scott, 1995). And secondly, the efficiency of the precipitation reaction, usually expressed by the molar ratio $\beta$ (Eq. 15).

$$CE = \frac{\text{charge consumed in forming Fe}^{2+}}{\text{total charge consumed}}$$ (Eq. 14)

$$\beta = \frac{\text{Fe$_{dosed}$ [mol$_{Fe}$]}}{\text{P$_{precipitated}$ [mol$_{P}$]}}$$ (Eq. 15)

$$\eta_{tot} = \frac{CE}{\beta} = \frac{P_{precipitated} \text{[mol$_{P}$]}}{\frac{I \cdot t}{n_i \cdot F}}$$ (Eq. 16)

In an ideal system, the current efficiency equals 1, as observed by Vik et al. (1984) and Yu et al. (2006). The current efficiency decreases when many side reactions take place. For the batch experiments with permeate, the measured current efficiency was between 60-80% (Figure 3), whereas current efficiencies of only 20-50% were measured with activated sludge. The decrease in the current efficiency with activated sludge compared to permeate may be partly attributed to measurement errors due to a more heterogeneous distribution of the iron (e.g. settling of iron containing sludge toward the reactor bottom) or to the oxidation of organic material at the anode (Eq. 4) respectively. Another reason may be that the experiments with permeate were conducted with new electrodes, whereas the electrodes used for the activated sludge experiments were already covered with a layer of iron oxides.

The molar ratio $\beta$ is strongly affected by the final phosphate concentration, with measured values ranging from 1.05 - 3.

The total reaction efficiency is calculated by combining the two efficiencies for iron dissolution and precipitation (Eq. 16). In the batch experiments with permeate the total reaction efficiency was around 50%.
ENERGY EFFICIENCY
The energy consumption is determined by the applied voltage, current and runtime. Current and runtime depend on the current efficiency, while the voltage depends on the electrode over-potentials, the electrode interspace and the conductivity of the solution (Eq. 12). A small distance between the electrodes is preferred to save energy but mixing has to be provided in order to transport the iron to the bulk solution. A distance of 2 cm +/- 1 cm was found to be reasonable. The most crucial point regarding EloP operation was the formation of a passivation layer on the electrode surface. This layer mainly consisted of iron oxides (e.g. Fe$_3$O$_4$, γ-Fe$_2$O$_3$) and contained hardly any iron phosphates as was seen by scanning electron microscopy. As the layer decreased the current efficiency, the electric resistance and therefore required voltage and energy consumption increased strongly. Alternating the current direction and cross-flow aeration were means to slow down the formation of this passivation layer (Chen, 2004), but could not avoid it. The conductivity was predetermined by the wastewater matrix. Due to the higher salt content, concentrated wastewater was better suited, and it was found that the solution potential accounts for approximately 50% of the voltage under the given operational conditions. The air bubbles slightly increased this resistance by 2-4%.

SIDE REACTIONS
The changes in chloride concentrations were only marginal in batch experiments, but formation of chlorine cannot be excluded. Due to the presence of large amounts of organic material, chlorine gas could react with this to form AOX. During two batch experiments with permeate, neither significant changes in chloride concentration nor AOX production was detected in the liquid phase. Electrocoagulation is often used for the oxidation of organic material. In experiments with treated wastewater the COD remained stable. When applying electrocoagulation to activated sludge, a COD decrease of 60% was observed during a three-day batch experiment (15 hours of electrocoagulation) compared to a 20% decrease in the control.

APPLICATION IN THE MBR
The electrodes were tested over two months in an MBR treating domestic wastewater of a single-family house (Figure 2). The MBR was operated with a variable volume and is
described elsewhere (Abegglen et al., 2008). After reaching a certain water level, a pre-aeration period of 50 minutes started, followed by a permeate withdrawal period, until the water level had dropped to a minimum value. The EloP was only in operation during the pre-aeration and subsequent withdrawal periods in order to provide iron dosing proportional to the volumetric flow. The oxygen concentration during EloP operation was above 1 mg L$^{-1}$ to prevent high Fe$^{2+}$ concentrations, which is suspected to cause fouling problems on membranes (Bagga et al., 2008). Due to very limited space in the MBR compartment, the electrodes could not be placed directly above the aeration system. This resulted in limited turbulence with reduced transport of the iron away from the electrode plate and accumulation of activated sludge between the electrodes, ultimately leading to poor current and reaction efficiency. Furthermore, towards the end of the experiment, oxygen was limited between the electrodes, leading to a biological reduction of the Fe$^{3+}$ complexes or insufficient oxidation of Fe$^{2+}$, which manifested in an intense blue color of the sludge between the electrodes.

**Efficiency of the Process**

The results of the iron production and phosphate removal were somewhat diffuse. Compared to similar periods, the overall phosphate removal capacity increased from 25% to 50%, which indicates that the EloP basically worked. But the $P_{\text{tot}}$ and $P_{\text{tot}}$ measurements in the MBR were not consistent (Figure 4). There are two reasons for this: on the one hand, there were dead zones on the bottom of the reactor where particulate matter was able to settle and was no longer suspended. On the other hand, the space between the electrodes was not mixed very well and iron accumulated there. These effects resulted in an underestimation of the $P_{\text{tot}}$ and the $P_{\text{tot}}$, measurements. Figure 4 shows that the mixing and mass balances were about as expected during the first three weeks. Subsequently, the gap between the theoretically expected and measured values becomes larger. The theoretical $P_{\text{tot}}$ values were calculated from the reactor mass balance, while the expected iron production could only be estimated from the results obtained in the lab experiments.

![Figure 4: Theoretically expected and measured $F_{\text{tot}}$ and $P_{\text{tot}}$ values in the MBR sludge. For $F_{\text{tot}}$, the current efficiency was assumed to be 60% as calculated from the batch experiments. No excess sludge was withdrawn during this period.](image)

The phosphate effluent values were in the range of 6-10 g P m$^{-3}$. The electrochemical phosphate removal was about 7 - 8 g m$^{-3}$ (average flow 0.35 m$^3$ d$^{-1}$) with an applied current of about 3A during ~6 hours and a potential of 6.4 V, corresponding to ~0.3 kWh m$^{-3}$. This increased the energy demand of the treatment plant by only 5%. When including the control unit, the figures double, as it has not been optimized yet. For continuous operation, slightly higher voltages and currents need to be applied.
SIDE EFFECTS: PH, PERMEABILITY, ACTIVATED SLUDGE

Several adverse effects on the operation of an MBR were anticipated, including pH-shifts, permeability loss or inhibition of the activated sludge. While in the batch experiments a pH increase was observed, this was not the case in the MBR due to the higher buffer capacity. A negative effect on membrane permeability can also be excluded (data not shown). Alshawabkeh et al. (2004) observed that the COD removal rate in electric fields of more than 1.14 V/cm (corresponding to a current density of 0.6 mA cm\(^{-2}\)) decreased and concluded that strong electric fields may be harmful to microorganisms. A similar effect was observed by Yu et al. (2006) above a current density of 6 mA cm\(^{-2}\) for COD and a current density of >1 mA cm\(^{-2}\) for nitrogen removal. In this study, the applied electric field was 3.1 V/cm, corresponding to a current density of 1.75 mA cm\(^{-2}\). An influence on the biological activity was not observed as proven by a steady increase of the nitrification activity (Figure 5). The heterotrophic activity was not measured directly, but as neither the COD nor DOC effluent concentrations increased, a negative effect can be excluded. As the electric field was only applied to a very small fraction of the total volume (0.4%), this is not surprising.

![Figure 5: Development of the nitrification capacity during EloP treatment. No adverse effect is observed. The increase stems from a change in operational mode and a higher influent load between Christmas and New Year. AOB = ammonia oxidizing bacteria, NOB = nitrite oxidizing bacteria.](image)

DISCUSSION

PRECIPITATION AND IDEAL OPERATION

Although the MBR experiment was not really conclusive, electrolytic phosphate removal has the potential to be used in small wastewater treatment plants. It is especially suited to MBR or SBR processes, where it can be linked to the aeration control and the water flow (assuming that the inlet phosphate concentration is more or less constant). Several control strategies were tested in a simple dynamic model with respect to minimizing energy requirements and effluent P-load. It was found that coupling the current with the permeate withdrawal was the best option.

The advantage of electrocoagulation is the absence of mechanical equipment and the limited requirement for chemicals. The electrodes themselves need hardly any space and can be constructed in different geometrical forms. For safe and stable operation, good mixing between the electrodes is required.

According to our results, a current efficiency of approximately 50% is reached under operating conditions. Whether this value can be maintained over a longer period still needs to be tested and proven in long-term experiments.

During the dosing of the iron, the oxygen concentration should be maintained above 1 mg l\(^{-1}\) in order to ensure that the ferrous ions are oxidized to ferric ions, as Fe\(^{2+}\) is suspected to cause fouling problems on membranes (Bagga et al., 2008).
**RISKS AND DISADVANTAGES OF THE PROCESS**

The main reaction at the cathode is the production of hydrogen gas (Eq. 4), which is explosive when its concentration reaches 4 Vol%. When operated with the parameters necessary for 90% P-removal at the MBR studied (4.5A, 7 hours), the hydrogen production rate is approximately 0.02 mmol s⁻¹ (corresponding to 0.25 mmol A⁻¹ min⁻¹). The EloP device was only in operation when the aeration with airflow of 80 l min⁻¹ was turned on, so the hydrogen concentration in the gas stream does not exceed 0.03%. In the worst-case scenario, the device would be in operation without aeration or ventilation. The air volume in the reactor was 375 liters. With these assumptions, it still takes 7.5 hours to reach the explosion threshold value for H₂. So the risk of explosion is negligible.

The production of Cl₂ (Eq. 3) and its reaction with organic matter may result in the formation of adsorbable organic halogen compounds (AOX). During two batch experiments with permeate (DOC = 7 mg l⁻¹), the AOX concentration in the liquid phase remained constant. However, the formation of AOX may be of concern. The Swiss threshold value for AOX in sewage sludge is 500 mg kgTS⁻¹. With a daily biological sludge production of 60 g d⁻¹, only 30 mgAOX d⁻¹ have to be formed to exceed this value, corresponding to a change in chloride concentration of less than 0.1 mg l⁻¹, which was below the measurement accuracy. But even if certain amounts of AOX are formed, it is not clear whether they are of environmental concern.

One of the drawbacks of chemical P-removal is the extra production of sewage sludge it involves. In conventional WWTP, sludge production by phosphate precipitation is increased by about 20%. Small membrane bioreactors usually have far lower sludge production due to the large sludge age with aerobic stabilization of the activated sludge. The sludge production for the 4-person household studied was calculated to be 60 g d⁻¹, whereas the sludge production by precipitation is in the range of 30-40 gTS d⁻¹. This is an increase by 50-70% but may be slightly lower due to the oxidation of organic material as observed in batch tests. Furthermore, the extraction of chemically fixed phosphate is difficult. In view of the current discussion of sustainability, where nutrients are supposed to be recycled, this is another drawback of precipitation.

The most crucial point with respect to electrocoagulation is the passivation of the electrode surface. No easy measures to prevent the formation of such a layer have been found in this study, although cross-flow aeration and alternating current direction seem to be beneficial.

**COSTS AND ENERGY REQUIREMENTS**

Is there an economic benefit when applying electrocoagulation instead of dosing of chemicals (e.g. FeCISO₄, product name Sedifos from Feralco, Uerikon, Switzerland) for the removal of phosphate in a decentralized setting? The costs for these two options were calculated including investment and amortization over 10 years for both treatments, as well as operational costs (Table 3). Costs for maintenance and sludge disposal were not included, as they are assumed to be similar for both options. The calculations were based on the data from the MBR investigated in this study and for four persons, with β = 2 and phosphate removal by precipitation of 70% (1.2 gP Pe⁻¹ d⁻¹).

The investment costs were rough estimates, as only little data was available for complete dosing systems and no data could be obtained for the EloP as it was a prototype. Prices of 300 € for chemical dosing (pump, control unit and installation) and 400 € (control unit and installation) were assumed. Product prices (steel, Sedifos and energy) were based on actual prices at end of June 2008. The energy demand for the EloP was estimated with the energy consumption of the prototype with 50% current efficiency, voltage 4.5 A at 9V, and a basis requirement of 15W for the control unit. The cost difference for the two systems is only 25%, although the operation costs for the EloP were based on the prototype with over 50% of its energy demand being consumed by the control unit. Total energy consumption with EloP
increases by 15%, but can certainly be minimized by optimization. While electrocoagulation has higher energy consumption, dosing of Sedifos requires handling of (corrosive) chemicals and either frequent exchange of containers or space for chemical storage.

Table 3: Estimated costs for phosphate precipitation with iron-chloride-sulfate and electrocoagulation (4 Persons)

<table>
<thead>
<tr>
<th></th>
<th>FeClSO₄</th>
<th>EloP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Investment (€ a⁻¹)</td>
<td>40</td>
<td>50</td>
</tr>
<tr>
<td>Energy requirement (kWh a⁻¹)</td>
<td>50</td>
<td>240</td>
</tr>
<tr>
<td>Energy (€ a⁻¹)</td>
<td>5</td>
<td>25</td>
</tr>
<tr>
<td>FeClSO₄/Iron requirement (kg a⁻¹)</td>
<td>50</td>
<td>8</td>
</tr>
<tr>
<td>FeClSO₄/Iron (€ a⁻¹)</td>
<td>35</td>
<td>25</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>80</strong></td>
<td><strong>100</strong></td>
</tr>
</tbody>
</table>

CONCLUSIONS

- Electrocoagulation using iron electrodes for phosphate removal proved to be an interesting option for application in small wastewater treatment plants. As no mechanical components are necessary, the risk of technical failure seems marginal. No chemicals are needed for the operation of such a device.
- Phosphate effluent concentrations of well below 1 mg P l⁻¹ were achieved in laboratory experiments but could not be confirmed due to operational constraints in an MBR treating household wastewater.
- The dosing of the iron should be coupled to the phosphate load. Assuming a more or less constant wastewater composition, the current must be controlled via the water flow. As SBRs and MBRs offer good control of the treated water quantity (cycle length and volume in the SBR or pumping time and water level in the MBR), electrocoagulation is well suited for these technologies.
- The crucial point is the formation of a passivation layer on the surface of the electrodes. This layer substantially reduces the current efficiency and thus the overall process performance. By changing the polarity, this formation can be minimized but not avoided completely.
- No negative impacts of electrocoagulation are expected in membrane bioreactors. Neither pH, permeability nor biological activity were affected. The hydrogen production at the cathode poses no real safety risk as long as aeration and ventilation are ensured.
- The cost difference for installing an iron dosing station or electrocoagulation is small. With the prototype used in this study, the energy demand for a 4-person household would rise by 250 kWh a⁻¹ or 15%, but this can be optimized, while the energy demand of a dosing pump is marginal. Handling of corrosive chemicals as necessary by dosing FeClSO₄ may be undesirable in private homes.

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CHAPTER 6

THE FATE OF SELECTED MICROPOLLUTANTS IN A SINGLE-HOUSE MBR

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The fate of selected micropollutants in a single-house MBR

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ABSTRACT
Membrane bioreactor (MBR) technology is an interesting option for single-house wastewater treatment or small communities. Because typically a very high effluent quality is achieved with respect to pathogens, suspended solids, organics and nitrogen, the permeate is well suited for reuse. Little is known about the fate of micropollutants in such small systems. The differences between centralized and decentralized biological wastewater treatment with respect to micropolllutants are manifold: besides the operational parameters like hydraulic and sludge retention time, the main difference is in the load variation. While the influent load is expected to be more or less constant in large catchments, it varies strongly in small MBRs due to irregular consumption (e.g. of medication by individuals). Concentrations of micropollutants are higher by a factor 50 - 1000 than in centralized treatment. It is also unknown how reliable degradation of micropollutants is in case of irregular exposure.

In this study, two experiments were conducted in a small MBR treating the wastewater of a three-person household. During normal operation of the treatment plant, 25 pharmaceuticals (antibiotics, antiphlogistics, lipid regulators, iodinated contrast media and hormones) that had not been used by members of the household were added in concentrations typical for municipal wastewater. The removal of most substances was in the same range as for centralized wastewater treatment. It was shown that biological transformation was the main elimination process while adsorption to the activated sludge was negligible for most substances due to the low sludge production at high sludge retention time. No appreciable lag for inducing biological degradation was observed. The high hydraulic and sludge residence time had a positive effect on the elimination of slowly degradable substances, but this was partly compensated by the lower biological activity.

An experiment with antibiotics concentrations typical for decentralized treatment (between 500 and 1000 µg l⁻¹; sulfamethoxazole, sulfapyridine, trimethoprim, clarithromycin, roxithromycin) did not show an inhibitory effect on either nitrification or denitrification.

Keywords: Biological degradation; decentralized wastewater treatment; MBR; micropollutants; water reuse

INTRODUCTION
Decentralized wastewater treatment with MBRs is widely discussed in the literature as an alternative to the centralized system (e.g. Fane and Fane, 2005; Wilderer, 2004). Environmental pollution from a single small wastewater treatment plant (WWTP) is generally low. In sparsely populated areas, however, the sum of such diffuse discharges may significantly contribute to the pollution of surface or groundwater, as in several US regions, since 21% of the US population is served by on-site wastewater systems (WERF, 2007). Flasche (2002) estimated that in the German province of Lower Saxony 10% of the population connected to decentralized wastewater systems account for at least 20% of the

1 Decentralized wastewater treatment is mostly defined by the number of connected person equivalents (PE), although the definition may be country-specific: The European Urban Water Directive applies to centralized plants with a minimal size of 2000 PE. The German and Austrian legislation subdivides decentralized treatment into small plants (from 51 PE up to 5000 or 500 PE respectively) and very small installations (up to 50 PE in
nutrient load into the environment (assuming that all systems work properly). Due to environmental concerns, in central Europe septic tanks are upgraded to aerobic biological wastewater treatment plants with sequencing batch reactor, membrane bioreactor (MBR) or biofilm treatment (sand filtration, constructed wetlands). No data is available on the diffuse pollution caused by pharmaceuticals and personal care products (PPCP) from on-site wastewater systems, although their fate during water and wastewater treatment has been intensively investigated in recent years (e.g. Ternes and Joss 2006; Reemtsma and Jekel 2006). Several studies were conducted on biological wastewater treatment in order to compare the elimination rates of PPCP in conventional activated sludge treatment (CAS) with membrane bioreactors (Bernhard et al., 2006; Clara et al., 2005; De Wever et al., 2007; Joss et al., 2005; Smook et al. 2008). MBR treatment generally showed similar elimination rates; only for few compounds significantly higher degradation is documented for MBR treatment (most probably due to the higher sludge age) resulting in a still open discussion on whether this difference is toxicologically relevant or not.

As the removal of organics and nitrogen by small membrane bioreactors is comparable to larger systems, the elimination of micropollutants may also be expected to be comparable. However, several differences between large and small wastewater treatment plants potentially affect the micropollutant elimination efficiency:

- The hydraulic residence time (HRT) in centralized systems is mostly in the range of 8 to 16 h while in small treatment plants it may reach several days.
- The sludge age in conventional centralized WWTP is in the range of 5 - 20 d (mostly 20 to 50 d in MBRs), whereas almost no sludge removal (or only at long intervals) may occur in small MBRs, resulting in sludge ages of more than 100 d. Such a long sludge age results in a lower biological activity but favors the growth of slowly growing organisms (“specialists”).
- In large catchments, the load of a substance corresponds to the average consumption and the load at the influent of the WWTP is more or less evened out according to the frequency of inputs (Ort et al., 2005). In small WWTPs for 4 to 50 persons, the load variation for single substances is significantly higher, resulting in long periods without any micropollutant influent, followed by high load periods of days or a few weeks, when an individual undergoes medical treatment.
- While in large catchments, for instance one out of some hundred or several thousand persons undergoes medical treatment with a specific compound, it is one out of only 4 to 50 for small treatment plants. This results in increased influent concentrations up to the mg l\(^{-1}\) range.
- It is unclear, whether the microorganisms responsible for biological degradation have to adapt to the micropollutant load (i.e. form special enzymes), or if degradation occurs via constitutively expressed enzymes. De Wever et al. (2007) postulate that sludge adaptation is important for specific substances and lag phases of several weeks may occur before degradation starts. For drugs excreted in decentralized plants, this would imply that degradation may be hampered by a lack of time to induce the necessary enzyme expression (e.g. medical treatment of a tenant during only a few days).

In centralized wastewater treatment, the removal of pharmaceuticals is substance-specific and varies between 0 and nearly 100%. Of the three possible removal mechanisms biological degradation, sorption to sludge and stripping, the latter is negligible, while sorption and biological processes are similarly important depending on the micropollutant studied. Joss et al. (2006) set up a model to predict the removal pathways by assessing a biological degradation parameter \(k_{\text{biol}}\) and a sorption coefficient \(K_d\).

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both countries). In this text, centralized treatment only refers to the size of a catchment (> 1000 PE) independent of the treatment technology (CAS, MBR, ...).
Only little information is available about removal of pharmaceuticals in small wastewater treatment systems. Hospital wastewater treatment got into the focus of researchers, but to the authors knowledge no comprehensive overview on removal of pharmaceuticals in such systems is yet available. The objectives of this study were to assess the elimination capacity for pharmaceuticals of a small MBR treating domestic wastewater of a single-family house and to compare this with known elimination rates in centralized CAS and MBR treatment. Further goals were to identify and quantify the removal mechanism (sorption to sludge or biological degradation), to predict effluent loads and concentrations for selected substances in the case of medical treatment of a resident by applying the model of Joss et al. (2006) and to find out, whether inhibition of the activated sludge is to be expected during exposure to concentrations of micropollutants typical for decentralized WWTP (mg l\(^{-1}\)). The investigated substances included several active groups (anti-inflammatory drugs, iodinated contrast media, sulfonamide and macrolide antibiotics, lipid regulators, antiepileptic drugs). As the persons living in the house were not under medical treatment, the target compounds have been spiked.

**MATERIAL AND METHODS**

**WASTEWATER TREATMENT PLANT**

The WWTP was located in the basement of a single-family house with three to five residents in Switzerland. Operation of the treatment plant started in October 2004 with activated sludge from a conventional WWTP. Two micropollutant sampling campaigns of two weeks were conducted: the first in August and the second in December 2007. During both periods, two to three persons lived in the house. The treatment plant consisted of two tanks, the first being used as a biological compartment (first campaign) or as a primary clarifier (second campaign), while the second one was a conventional membrane bioreactor. When reactor 1 was used as biological compartment, activated sludge was recycled from the MBR, and it was intermittently mixed by coarse bubble aeration pulses. All the same, mainly anaerobic conditions prevailed in the first tank, while the MBR was always aerobic or anoxic. The flow scheme from the first period proved to be very promising with respect to enhanced nutrient removal, while the flow scheme with primary clarifier is usually applied in small MBRs. The flow schemes of the treatment plant are shown in Figure 1. More details of the plant can be found in Table 1 and Abegglen *et al.* (2008). The effluent of the MBR was partly reused for toilet flushing and gardening. For the time of the experiment, this recycling stream was interrupted and drinking water was used instead. Excess sludge was usually pumped once a week from the MBR compartment to a filter bag. During the sampling periods, the excess sludge removal was stopped.

Table 1: Characteristics and operational parameters of the small MBR during the two sampling campaigns (average)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Experiment 1</th>
<th>Experiment 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Influent and effluent flow (Q(_{io}, Q_e))</td>
<td>1 dl(^{-1})</td>
<td>306</td>
</tr>
<tr>
<td>Flow from 1(^{st}) to 2(^{nd}) compartment (Q(_{12}))</td>
<td>1 dl(^{-1})</td>
<td>830</td>
</tr>
<tr>
<td>Sludge recycle to 1(^{st}) compartment (Q(_{21}))</td>
<td>1 dl(^{-1})</td>
<td>524</td>
</tr>
<tr>
<td>Active volume, 1(^{st}) compartment (V(_1))</td>
<td>m(^3)</td>
<td>0.65 - 1.03</td>
</tr>
<tr>
<td>Active volume, 2(^{nd}) compartment (V(_2))</td>
<td>m(^3)</td>
<td>0.98 - 1.22</td>
</tr>
<tr>
<td>Suspended solids concentrations (TSS)</td>
<td>g m(^{-3})</td>
<td>3800</td>
</tr>
<tr>
<td>Hydraulic retention time (HRT)</td>
<td>d</td>
<td>6.3</td>
</tr>
<tr>
<td>Sludge retention time (SRT)</td>
<td>d</td>
<td>&gt;150</td>
</tr>
<tr>
<td>Temperature °C</td>
<td>23.1 ± 0.5</td>
<td>16.0 ± 1.0</td>
</tr>
<tr>
<td># samples from 1(^{st}) compartment (R(_1))</td>
<td>8</td>
<td>-</td>
</tr>
<tr>
<td># samples from 2(^{nd}) compartment (R(_2))</td>
<td>7</td>
<td>10</td>
</tr>
</tbody>
</table>

\(^{a)}\) Both reactors had a maximum volume of 1.5 m\(^3\). Active volume means the volume that was filled with water.
Figure 1: Flow scheme of the treatment plant during the two experimental periods. Micropollutants were spiked into the first tank in the first (above) or directly into the MBR (below) in the second experiment. Circles indicate sampling points. Qin: influent; QWC: in-house water reuse (interrupted during sampling periods); Qgarden: water reuse for gardening; QES: excess sludge withdrawal; Qt: treated effluent; Q12: flow from the first to the second compartment; Q21: sludge recycle to the first compartment.

SAMPLING PERIODS AND SAMPLE PREPARATION

During the first sampling period, the micropollutants were spiked into the first reactor followed by 15 min of mixing (initial concentrations: 0.5 - 2 µg l⁻¹, iodinated contrast media: 10 µg l⁻¹). Grab samples were taken from the first reactor and from the MBR effluent (8 samples at both sampling points taken with increasing time intervals). The flow rates (Qin, Qt, Q12, Q21) were calculated from water level measurements (recorded in 1-minute intervals) and verified by a parallel tracer experiment with bromide.

In the second experiment, the micropollutants were dosed into the MBR (initial concentrations: 0.5 - 1 µg l⁻¹, iodinated contrast media: 4 µg l⁻¹). After spiking, the MBR was stirred for 15 min. 11 samples were taken from the MBR effluent with increasing time intervals during two weeks.

Four liters of sample were collected in amber glass bottles and immediately transported to the lab for further processing within a maximum of 2 hours after sampling. The mixed liquor samples (reactor 1 in experiment 1) were centrifuged at 3000 rpm for 5 min to improve the filterability. All samples were filtered with a glass fiber filter (0.7 µm, GF/F, Pall Corp., East Hills, USA), then the pH was adjusted, internal standards were added (Table 2) and the substances were enriched by solid-phase extraction (SPE) on polymeric cartridges. The cartridges were subsequently dried under a nitrogen gas stream and stored at -20° until
analysis. Only the filtered samples (liquid phase) were further processed and analyzed for the parent compounds, i.e. no metabolites or transformation products were measured. Bromide was used as a hydraulic tracer. Aliquots of the filtered samples were analyzed by ionic chromatography.

**MICROPOLLUTANTS STUDIED**

The substances studied included different therapeutic groups (antibiotics, antiphlogistics, lipid regulators, iodinated contrast media and hormones). The selection was done considering the following criteria: a) select several (chemically heterogeneous) representatives of different pharmaceutical groups, b) choose compounds that the activated sludge had never been in contact with and c) availability of analytical method as well as data of fate during centralized treatment. Similar concentrations as expected in wastewater of centralized WWTP (0.5 - 4 µg l⁻¹) were chosen as initial spike concentrations to allow a direct comparison of the observed degradation.

In a realistic scenario, i.e. if a resident is under medical treatment, the expected influent concentrations are in the range of 100 - 2000 µg l⁻¹. Example: ibuprofen is prescribed in daily doses of 1.2 mg d⁻¹. Excretion via urine and faeces are 7 and 23%, respectively. With a water consumption of 320 l d⁻¹ in a 3-person household, this results in an influent concentration of 1125 µg l⁻¹, compared to 1.5 - 2.1 µg l⁻¹ measured in WWTP influents in Switzerland (Ternes and Joss, 2006). The same calculation for 17α-ethinylestradiol results in a concentration of 50 ng l⁻¹ for small WWTP (if contraceptives are used), while the concentration in municipal wastewater is typically ≤2 ng l⁻¹.

**BATCH EXPERIMENT AT HIGH ANTIBIOTICS CONCENTRATION**

In order to elucidate, whether high concentrations of pharmaceuticals have negative impacts on the treatment performance of activated sludge, batch experiments were conducted at antibiotic concentration typical for decentralized applications. Unfortunately, these experiments could not be carried out with the activated sludge from the single-house MBR studied before.

Activated sludge (3 l) from a conventional, nitrifying treatment plant was poured into two 3 L reactors (further named A and B) operated in parallel. For nitrification NH₄HCO₃ was added as substrate to the activated sludge and the whole slurry was aerated during the day from 9h to 15h (1.5 - 3.5 mgO₂ l⁻¹). Denitrification was carried out over night by switching off the aeration after the nitrification was completed and by adding sodium acetate as substrate. pH and temperature were controlled (7.5 - 7.9 and 20°C, respectively). A mixture of sulfonamide and macrolide antibiotics (sulfamethoxazole, sulfapyridine, trimethoprim, clarithromycin and roxithromycin) was spiked to reactor A once a day during the first four days. The initial target concentration varied between 0.25 and 1 mg l⁻¹, corresponding to the expected influent concentrations of a 4 person household. From reactor A 12 samples were analyzed for the spiked antibiotics (5 samples were taken during the first 24h, afterwards one sample was taken per day).

The nitrification and denitrification activity of the sludge was assessed a) by measuring oxygen uptake rates during nitrification periods and b) by measuring nitrate during denitrification periods. The experiments lasted for 7 days.

**SAMPLE ANALYSIS**

Details of the analytical methods can be found elsewhere (Schluesener et al., 2008, in preparation). A summary is given below.

For the separation of antibiotics and neutral drugs a HPLC system consisting of a degasser (Gastorr 152), a G1312A binary pump and a G1329 autosampler (all Agilent Technologies
AG, Basel) was used, while for the other compounds the system consisted of a G1379A degasser, a G1311A quaternary HPLC pump and a G1313A autosampler (all from Agilent Technologies, Waldbronn, Germany). The detection was performed on a triple quadrupole mass spectrometer API 4000 (Applied Biosystems, Rotkreuz, Switzerland or Foster City, CA, USA) with an electro spray ionization source (ESI). Positive mode, a spray voltage of 5000 V and a temperature of 390°C were used for antibiotics and neutral drugs; positive mode, a spray voltage of 5500 V and a temperature of 650°C for contrast media; negative mode, a spray voltage of -4500V and a temperature of 650°C for acidic pharmaceuticals; for steroid hormones, ionization was performed in Heated Nebulizer (APCI) positive mode at the following conditions: CUR, 15 psi; CAD, 6 mTorr; nebulizer current (NC), 5 µA; TEM, 450 °C; GS1, 60 psi; EP, 10 V.

Antibiotics and neutral drugs carbamazepine and primidone
The pH was set to 7.5 with sulfuric acid. For SPE, Oasis HLB (200 mg / 6 cc / Waters) cartridges were used and the analytes were eluted with 4 x 2 ml of methanol. The LC mobile phase was composed of 10% acetonitrile, 5 mM ammonium acetate in HPLC-grade water (Eluent A) and 20% eluent A in HPLC-grade acetonitrile (Eluent B). The analysis was performed with a gradient starting at a level of 100% eluent A, which was subsequently changed to 74%/62%/0%. The flow rate was kept constant at 200 µl/min and the temperature of the column was set to 25°C. The injection volume was 20 µl and a RP YMC column (YMC Europe GmbH, 150 x 2.1mm YMC Pro C18, 120 Å, 3 µm) was used for the separation.

Acidic pharmaceuticals
The pH was set to 2.8 with sulfuric acid. For SPE, Oasis MCX 6(200 mg / 6 cc / Waters) cartridges were used. The acidic pharmaceuticals were eluted from the cartridge by using four times 1 mL acetone. The final extract was dissolved in 10 mM formic acid. For LC separation, a Zorbax Eclipse XDB-C8 column (4.6 x 150 mm, 5 µm; Agilent Technologies, Waldbronn, Germany) was used at 25 ± 1 °C. The flow rate was 0.4 mL/min. The HPLC gradient was established by mixing two mobile phases: phase A: pure acetonitrile and phase B: water with 10 mM formic acid. Chromatographic separation was achieved with the following gradient: 0 min to 6 min: 60% to 5% B; 6 - 14 min: 5% B; 14 min to 15 min: 5% to 60% B; 15 - 20 min: 60% B. Fifty µL of each sample were injected.

Iodinated x-ray contrast media
The pH was set to 2.8 with sulfuric acid. For SPE, ENV+ (500 mg / 3 cc / Isolute) cartridges were used. The iodinated contrast media analytes were eluted four times with 1 mL methanol and then evaporated to dryness. Finally, the analytes were dissolved in 500 µL of phosphate buffer (pH 5.7). The iodinated contrast media were separated using a Chromolith Performance RP-18e column (4.6 x 100 mm, Merck, Darmstadt, Germany) at 25 ± 1 °C. The flow rate was 0.6 mL/min. The HPLC gradient was established by mixing two mobile phases: phase A: water with 10mM ammonium acetate with 1% acetonitrile and phase B: a mixture of 40% water with 10mM ammonium acetate and 60% acetonitrile. Chromatographic separation was achieved with the following gradient: 0 - 6 min: 0% B; 6 min to 12 min: 0% to 20% B; 12-15 min: 20% B; 15 min to 15.5 min: 20% to 0% B; 15.5 - 21 min, 0% B. Fifty µL of each sample were injected.
Table 2: Quality assurance parameters of all analytes including internal standard used for the quantification (IS), limit of quantification (LOQ) and recoveries for wastewater effluent (n=number of samples, rec=recovery, stdev=standard deviation in %).

<table>
<thead>
<tr>
<th>Analyte</th>
<th>IS</th>
<th>LOQ [ng L⁻¹]</th>
<th>Recovery in wastewater effluent matrix [%]</th>
<th>n</th>
<th>rec</th>
<th>stdev</th>
</tr>
</thead>
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<td><strong>Antibiotics</strong></td>
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<td>±10</td>
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<td>±1</td>
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<tr>
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<td>7</td>
<td>89</td>
<td>±27</td>
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</tr>
<tr>
<td>Sulfadimethoxine</td>
<td>d4-Sulfadimethoxine</td>
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<td>8</td>
<td>113</td>
<td>±16</td>
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<tr>
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<td>3</td>
<td>64</td>
<td>±3</td>
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<tr>
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<td>d9-Trimetoprim</td>
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<td>d3-Azithromycin</td>
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<td>Oleanandomycin</td>
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<td><strong>Neutral Drugs</strong></td>
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<td>8</td>
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<td>±30</td>
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<td>Primidone</td>
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<td>±13</td>
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<td>±9</td>
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<td>16</td>
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</tr>
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<td>95</td>
<td>±12</td>
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<tr>
<td>17α-Ethinylestradiol</td>
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<td>3</td>
<td>8</td>
<td>77</td>
<td>±14</td>
<td></td>
</tr>
</tbody>
</table>

**Steroid hormones**
The pH was set to 3 with sulfuric acid. For SPE, a C18 (200 mg / 3 cc / Baker) cartridge was used. The steroid hormones were eluted from the cartridge by using four times 1 mL acetone. The extracts were evaporated to about 500 µL by a gentle nitrogen stream followed by a silica gel clean-up step. The silica gel was dried for 12 h at 150°C. After deactivating with 1.5% MilliQ water, 1 g silica gel was used for the clean-up. Before adding the sample extract, the silica gel was conditioned with hexane/acetone 65/35 (v/v) in a glass cartridge. The sample extracts were added to the conditioned cartridge and both the vial and the cartridges were rinsed three times with hexane/acetone 65/35 (v/v). The final clean-up extract was evaporated to dryness using a gentle nitrogen stream and was resolved in 500 µl MeOH. Steroid hormones were separated using a Synergi RP-MAX column (2 mm i.d., length 150 mm, particle size 4 µm) and a SecurityGuard (Phenomenex, Torrance, CA, USA) at 25 ± 1 °C. The flow rate was 0.2 mL/min. The HPLC gradient was established by mixing two mobile phases: phase A: pure water and phase B: pure methanol. Chromatographic separation was achieved with the following gradient: 0-1 min: 0% B; 1 min to 3 min: 5% to 70% B; 3 min to
23 min: 70% to 100% B; 23-29 min: 100% B; 29 min to 30 min: 100% to 0% B; 30-35 min: 5% B. Ten µL of each sample were injected.

A full quality assurance and quality control (QA/QC) protocol was followed to ensure the quality of the data. Quantification was performed relative to internal standards with internal calibration. The limit of quantification (LOQ) for a signal-to-noise ratio of 1:10 and the recoveries were determined in the wastewater effluent matrix (see Table 2).

**Model setup**

A simple hydraulic model was set up with Berkeley Madonna software (Macey et al., 2001) using completely stirred tank reactors. The water level and bromide concentration measurements were used as a reference to calibrate the hydraulics of both reactors. Removal of the compounds was modeled by including adsorption to and desorption from the sludge and biological degradation according to Joss et al. (2006).

**Adsorption and desorption in bulk liquid:**

\[
\frac{dC_i}{dt} = -k_{sor,i} \cdot \text{TSS} \cdot C_i + k_{des,i} \cdot X_i \tag{Eq. 1}
\]

**Biological transformation:**

\[
\frac{dC_i}{dt} = -k_{biol,i} \cdot \text{TSS} \cdot C_i \tag{Eq. 2}
\]

**Adsorption and desorption on sludge:**

\[
\frac{dX_i}{dt} = k_{sor,i} \cdot \text{TSS} \cdot C_i - k_{des,i} \cdot X_i \tag{Eq. 3}
\]

where

- \( C_i \): Concentration of substance i in the water phase [ng \ l^{-1}]
- \( k_{sor,i} \): Adsorption rate constant [l g_{TSS}^{-1} d^{-1}]
- \( TSS \): Concentration of total suspended solids [g_{TSS} l^{-1}]
- \( k_{des,i} \): Desorption rate constant [d^{-1}].
- \( K_{d,i} \): Solid-water distribution coefficient [l g_{TSS}^{-1}]; \( K_{d,i} = \frac{k_{sor,i}}{k_{des,i}} = \frac{X_{i,eq}}{C_{i,eq} \cdot \text{TSS}} \tag{Eq. 4} \)
- \( k_{biol,i} \): Biological pseudo-first order degradation constant [l g_{TSS}^{-1} d^{-1}]
- \( X_i \): Concentration of substance i sorbed on sludge [ng \ l^{-1}]

Sorption was assumed to be a fast process compared to the hydraulic retention time in each compartment (60-90 h). Therefore, \( k_{sor,i} \) was set to 25 l g_{TSS}^{-1} d^{-1} as proposed by Ternes et al. (2004), and \( k_{des,i} \) was calculated according to Eq. 4 (Ternes et al., 2004; Joss et al., 2006). \( K_d \) and \( k_{biol} \) were estimated for all measured substances using the least-square method. Best estimates and standard errors are given in Table 3. Standard errors were calculated based on the measured datasets without considering systematic errors (e.g. measured data are systematically too low), as these have only a small influence on the estimation of \( k_{biol} \). For rapidly reacting substances (ibuprofen, bezafibrate, estrone, 17α-ethinylestradiol), it was impossible to clearly identify the parameters because of insufficient temporal data resolution. An upper or lower boundary is given for these parameters.

Although the estimated parameters are not directly applicable for far higher concentration ranges (i.e. other degradation rate constants are expected at mg l^{-1} compared to µg l^{-1}), the obtained \( K_d \) and \( k_{biol} \) were used as best estimates to exemplarily model the expected effluent concentrations during typical medical treatment. The dataset from the second measurement campaign was used as the hydraulic input data. Drug uptake and excretion data were taken from Lienert et al. (2007) and the prescription doses were based on the Swiss Pharmaceutical Compendium (Documed, 2008).
RESULTS AND DISCUSSION

ESTIMATION OF MICROPOLLUTANT ELIMINATION BY ADSORPTION AND BIOLOGICAL DEGRADATION

Figures 2 and 3 show the time course of the normalized concentrations of three selected compounds (ibuprofen, iopromide and roxithromycin) compared to the inert tracer for the two sampling campaigns. Table 3 lists the best estimates for $k_{\text{bio}}$ and $K_d$ for both sampling campaigns in comparison with literature data. It is important to note that $k_{\text{bio}}$, $K_d$ and elimination rates differing significantly from those reported may be expected in case of significantly higher pharmaceutical loading (e.g. sufficient to cause biomass inhibition or to act as primary substrate). Moreover, all data is based on measurements of the parent compounds and no transformation products are considered. In the following, the most important findings for the different therapeutic groups are briefly discussed.

**Figure 2:** Normalized concentrations of ibuprofen, iopromide and roxithromycin during experiment 1. Bromide was used as an inert tracer. These compounds were selected as representative for rapid degradation (ibuprofen), strong adsorption with little degradation (roxithromycin), slow biological degradation and little adsorption (iopromide) and inert micropollutants (bromide). a) Reactor 1; b) MBR; $c_0$ refers to the initial concentration in reactor 1.

**Figure 3:** Normalized concentrations of ibuprofen, roxithromycin and iopromide during sampling campaign 2. Bromide was used as an inert tracer. These compounds were selected as representative for rapid degradation (ibuprofen), strong adsorption with little degradation (roxithromycin), slow biological degradation and little adsorption (iopromide) and inert micropollutants (bromide). $c_0$ refers to the initial concentration in the MBR.

*Sulfonamide antibiotics*

Sulfonamide antibiotics were fairly well biodegraded and hardly adsorbed to the sludge (Table 3). The degradation constants were similar for all of the investigated sulfonamides, resulting in elimination rates between 75% and 90%.
Macrolide antibiotics

The fate of the macrolide antibiotics (azithromycin, clarithromycin, roxithromycin) was characterized by low biodegradability and significant adsorption (Figures 2 and 3). More than 50% of the substance disappeared from the liquid phase by adsorption at the beginning of the experiment. Two weeks after spiking, a significant amount still remained sorbed in the system (e.g. clarithromycin: 40%). Because of this large sorption, the model results were very sensitive to the sorption equilibrium constant $K_d$. The estimated $K_d$ values are higher than those reported in the literature, which may be explained by the experimental setup (sorbed concentration at the beginning was zero; the model does not account for irreversible sorption) and the slightly higher affinity of MBR sludge to these compounds (Ternes and Joss, 2006) than conventional activated sludge.

<p>| Table 3: Best estimates for biological degradation constants ($k_{biol}$) and solid-water distribution coefficients ($K_d$) of the substances studied (± standard error). The activated sludge had a higher nitrification activity during the second campaign (12 gN m$^{-3}$d$^{-1}$ and 23 gN m$^{-3}$d$^{-1}$, respectively). Literature data for conventional wastewater treatment is taken from Ternes and Joss (2006). |
|-------------------------------------------------|-------------------------------------------------|
| <strong>Degradation: $k_{biol}$ (l gSS$^{-1}$ d$^{-1}$)</strong> | <strong>Sorption: $K_d$ (l gSS$^{-1}$)</strong> |</p>
<table>
<thead>
<tr>
<th>Campaign 1</th>
<th>Campaign 2</th>
<th>Literature</th>
<th>Campaign 1</th>
<th>Campaign 2</th>
<th>Literature</th>
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</tr>
<tr>
<td>Sulfamethoxazole</td>
<td>0.19 ±0.03</td>
<td>0.2 ±0.02</td>
<td>&lt; 0.2</td>
<td>0.04 ±0.013</td>
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<td>n.a.</td>
<td>0.003 ±0.006</td>
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<td>0.001 ±10$^{-5}$</td>
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<td>0.014 ±0.03</td>
</tr>
<tr>
<td>Diatrizoate</td>
<td>&lt;0.008</td>
<td>n.a.</td>
<td>&lt; 0.1</td>
<td>&lt;0.007</td>
<td>n.a.</td>
</tr>
<tr>
<td><strong>Steroid hormones</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Estrone</td>
<td>&gt;20 $^1$</td>
<td>&gt;20</td>
<td>200</td>
<td>&lt;0.1</td>
<td>&lt;0.1</td>
</tr>
<tr>
<td>Ethinylestradiol</td>
<td>&gt;0.5</td>
<td>&gt;0.7</td>
<td>8</td>
<td>&gt;0.5</td>
<td>&gt;0.3</td>
</tr>
<tr>
<td><strong>Acidic and neutral pharmaceuticals</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Carbamazepine</td>
<td>&lt;0.008</td>
<td>&lt;0.005</td>
<td>0.008</td>
<td>&lt;0.008</td>
<td>&lt;0.075</td>
</tr>
<tr>
<td>Primidone</td>
<td>n.a.</td>
<td>0.022 ±0.007</td>
<td>n.a.</td>
<td>n.a.</td>
<td>0.05 ±0.01</td>
</tr>
<tr>
<td>Ibuprofen</td>
<td>1.33 ±0.02$^1$</td>
<td>&gt; 3</td>
<td>9 -35</td>
<td>0.05 ±0.018</td>
<td>0.006 ±0.004</td>
</tr>
<tr>
<td>Diclofenac</td>
<td>n.a.</td>
<td>&lt; 0.02</td>
<td>&lt;0.1</td>
<td>n.a.</td>
<td>n.a.</td>
</tr>
<tr>
<td>Clofibric acid</td>
<td>n.a.</td>
<td>0.09 ±0.033</td>
<td>0.1 - 1</td>
<td>n.a.</td>
<td>0.007 ±10$^{-5}$</td>
</tr>
<tr>
<td>Naproxen</td>
<td>n.a.</td>
<td>0.08 ±0.016</td>
<td>0.4 - 1.9</td>
<td>n.a.</td>
<td>0.01 ±0.01</td>
</tr>
<tr>
<td>Bezafibrate</td>
<td>0.77 ±0.34$^1$</td>
<td>&gt; 2.9</td>
<td>2.1 - 4.5</td>
<td>0.09 ±0.011</td>
<td>&lt;0.05</td>
</tr>
</tbody>
</table>

n.a. = not available; $^1 k_{biol,anaerobic} = 0$;

Antiphlogistics, lipid regulators, anti-epileptics (acidic and neutral pharmaceuticals)

Carbamazepine, primidone and diclofenac were not significantly degraded, as expected from centralized treatment. Ibuprofen and bezafibrate proved to be easily degradable (>95%), but only under aerobic and anoxic conditions, while no anaerobic degradation was observed (within the first compartment in the first experiment). This differs from the observation of Smook et al. (2008) who observed significant degradation of ibuprofen within the anaerobic...
compartment of an EBPR plant. Naproxen and clofibric acid were slowly degradable with a total removal rate of about 60%.

Iodinated contrast media
While neither iopamidol nor diatrizoate were biologically degradable, a small elimination was observed for iopromide and iomeprol. The estimated values for $k_{\text{biol}}$ were significantly smaller than those reported in the literature. Like ibuprofen and bezafibrate, degradation of iopromide only occurred in the MBR compartment, which indicates that it is not degradable under anaerobic conditions. Under aerobic conditions recently the formation of 12 transformation products was reported for iopromide (Schulz et al., 2008).

Hormones
Estrone was only detected in reactor 1 during the first sampling campaign. This indicates that it is hardly biodegradable under anaerobic conditions, but degrades very quickly in the presence of oxygen or nitrate. 17α-ethinylestradiol was also significantly biodegradable. An estimation of $k_{\text{biol}}$ and $K_d$ was hardly possible due to considerable data uncertainty. It was impossible to clearly identify whether degradation occurred under aerobic or anaerobic conditions.

The experiments and model results indicate that the elimination capacity and pathways of most micropollutants in activated sludge not being regularly exposed to these substances is comparable to activated sludge from centralized treatment (Table 3). Biological transformation started almost immediately (within a couple of hours, Figures 2 and 3), meaning that lag phases and the time required for expressing specific enzymes were negligible. According to the residents, none of the substances studied had been consumed within one year before the sampling period. So the ability to degrade them does not depend on regular exposure to pharmaceuticals or adaptation processes, at least for the micropollutants studied. However, adaptation processes may be important for certain micropollutants or to accelerate the removal rates, but the extent, to which specialized microorganisms are involved in the removal of hardly degradable substances, could not be studied with this experimental setup.

Factors influencing biological degradation of micropollutants
It is well known, that WWTPs at 8 d sludge age or higher (nitrification) have higher elimination rates for micropollutants in comparison with WWTPs without nitrification. With higher sludge age, the proportion of active biomass to suspended solids becomes smaller. This also holds true for the maximum nitrification rate. Due to the change in operation mode between experiment one and two, several operating parameters were changed, among them were the sludge age and the hydraulic residence time. Both were reduced by a factor 2 in experiment 2, resulting in maximum nitrification rates of 13.9 gN g\text{TSS}^{-1} d^{-1} and 24.6 gN g\text{TSS}^{-1} d^{-1}, for experiments 1 and 2 respectively. For conventional WWTP, typical values are around 33-36 gN g\text{TSS}^{-1} d^{-1} (Manser, 2005). The pseudo first order model for identifying $k_{\text{biol}}$ only accounts for the concentration of suspended solids without considering their specific biological activity. Assuming that biological degradation of micropollutants corresponds to the biological activity of the activated sludge this could be an explanation for $k_{\text{biol}}$ being higher in experiment 2 than in experiment 1 for several pharmaceuticals (sulfamethazine, iomeprol, ibuprofen, bezafibrate). A different explanation could be that the sludge was already better adapted to degrade these compounds in experiment 2.
Sulfamethoxazole showed comparable removal rates and clarithromycin appeared even better biodegradable in the first experiment. No explanation was found for this fact.
**SCENARIOS FOR MEDICAL TREATMENT OF TENANTS**

The goal of the experiment was to estimate degradation and sorption constants. The experimental design does not allow a direct assessment of elimination rates, effluent or sorbed concentrations for a realistic scenario since biological process rates may differ significantly with concentration increasing by orders of magnitude (e.g. due to inhibition, adaptation). Nevertheless, typical scenarios for the fate of selected micropollutants were modeled on the basis of the treatment plant design (primary clarifier and MBR), hydraulics \( (Q = 320 \text{ l d}^{-1} \approx 2-3 \text{ persons}) \), using the model parameters \((k_{\text{biol}} \text{ and } K_d)\) of sampling campaign 2 as best estimates and considering the medical prescriptions as well as the expected excretions. The goal of the simulation is to show expected concentrations and elimination rates. It was assumed that one person in the house underwent medical treatment during a certain period and that all the excretion occurred at home. The durations of the treatment were chosen according to typical prescriptions. The 42 days for ibuprofen and naproxen and 180 days for ethinylestradiol were selected to reach a steady state. Different durations were chosen because of the differences in the \(K_d\) values. With the \(K_d\) values for ibuprofen and naproxen, steady-state is reached within a few weeks.

Table 4 gives an overview of the prescription dose, the excretion rates and the parameters used for modeling as well as the model results.

<table>
<thead>
<tr>
<th>SMX</th>
<th>ROX</th>
<th>IOP</th>
<th>IBU</th>
<th>NPX</th>
<th>EE2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dose [mg d(^{-1})]</td>
<td>1600</td>
<td>300</td>
<td>32'000</td>
<td>1200</td>
<td>1000</td>
</tr>
<tr>
<td>Duration [d]</td>
<td>21</td>
<td>10</td>
<td>1</td>
<td>42</td>
<td>42</td>
</tr>
<tr>
<td>Indication</td>
<td>Antibiotic</td>
<td>Antibiotic</td>
<td>X-Ray contrast medium</td>
<td>Inflammation, pain, rheumatism</td>
<td>Rheumatism</td>
</tr>
<tr>
<td>Excretion (^{1)}) via urine [%]</td>
<td>83(^{2)})</td>
<td>6</td>
<td>100</td>
<td>7 (^{2)})</td>
<td>90</td>
</tr>
<tr>
<td>via feces [%]</td>
<td>45</td>
<td>0</td>
<td>23 (^{2)})</td>
<td>5</td>
<td>50 (^{2)})</td>
</tr>
<tr>
<td>Load to MBR [mg d(^{-1})]</td>
<td>1330</td>
<td>150</td>
<td>32'000</td>
<td>360</td>
<td>950</td>
</tr>
<tr>
<td>(k_{\text{biol}}) model</td>
<td>0.2</td>
<td>0.02</td>
<td>0.1</td>
<td>5</td>
<td>0.1</td>
</tr>
<tr>
<td>(K_d) model</td>
<td>0.05</td>
<td>0.4</td>
<td>0.01</td>
<td>0</td>
<td>0.01</td>
</tr>
<tr>
<td>(Q_{\text{in}} [\text{l d}^{-1}])</td>
<td>320 (+/- 85)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Model results**

| Degradation [%] | 77 | 25 | 63 | 98 | 65 |
| Sorption [%] | <0.5 | 5 | <0.1 | 0 | <0.1 |
| Effluent [%] | 23 | 70 | 37 | 2 | 35 |
| \(c_{\text{average, MBR}} [\mu g l^{-1}]\) | 800 \(^{3)}\) | 2 | - 3) | 22 | 1040 |
| \(c_{\text{max, MBR}} [\mu g l^{-1}]\) | 1200 | 240 | 7000 | 120 | 1300 |
| \(c_{\text{max, MBR}} [\mu g l^{-1}]\) without degradation | 4400 | 250 | 13000 | 1200 | 3100 |

\(^{1)}\) Only parent compound and glucuronides; \(^{2)}\) Lienert et al. (2007b) \(^{3)}\) no steady state is reached; \(^{4)}\) including N\(_4\)-Acetyl sulfamethoxazole

Figure 4a compares the influence of \(k_{\text{biol}}\) on the expected removal in a small MBR with the model proposed by Joss et al. (2006) for centralized municipal sewage treatment plants. Due to the high hydraulic residence time (\(> 3 \text{ d})\) in the small MBR, compounds with \(k_{\text{biol}}\) values of \(\geq 0.01 \text{ l g} TSS^{-1} \text{ d}^{-1}\) are degraded by more than 10\%, while for centralized wastewater treatment \(k_{\text{biol}} = 0.1 \text{ l g} TSS^{-1} \text{ d}^{-1}\) represents a threshold value. More than 90\% degradation is achieved with \(k_{\text{biol}} > 1 \text{ l g} TSS^{-1} \text{ d}^{-1}\). It should be kept in mind that these values depend on the activated sludge concentration and activity and that the values given in Table 3 may not be representative for concentrations higher by orders of magnitude.
Under steady-state conditions, the micropollutant removal via adsorption can be calculated from Equation 5 (Ternes and Joss, 2006):

\[
\frac{M_{ES}}{M_{in}} = \frac{SP \cdot K_d}{1 + SP \cdot K_d} \quad \text{(Eq. 5)}
\]

where

\(M_{ES}\) = load withdrawn with the excess sludge [μg d\(^{-1}\)]

\(M_{in}\) = influent load [μg d\(^{-1}\)] and

\(SP\) = sludge production per unit of wastewater (g TSS l\(^{-1}\))

The daily sludge production in small MBRs is relatively low (15 g TSS person\(^{-1}\) d\(^{-1}\)) compared to centralized treatment (40 g TSS person\(^{-1}\) d\(^{-1}\)). However, because the wastewater in small systems is not diluted by extraneous or rainwater, the sludge production per unit of wastewater is comparable: 115 and 130 mg TSS l\(^{-1}\) was assumed for decentralized respectively centralized treatments. Figure 4b shows the influence of \(K_d\) on removal by adsorption and excess sludge withdrawal. For both decentralized and centralized treatment with a low sludge production, sorption is comparable and not relevant (< 10% elimination) for \(K_d\) values < 0.3 l g\(^{-1}\) TSS. In small MBRs with a high sludge age, steady-state conditions for compounds with relevant sorption are only reached if the duration of the medical treatment exceeds the sludge age. With shorter treatment, the removal is smaller than shown in Figure 4b.

This can be illustrated with the results from the experiments, where the macrolide antibiotics showed a strong sorption to activated sludge, resulting in a quick drop in the dissolved concentration (Figure 3) and high \(K_d\) values (0.4 - 1.5 l g\(^{-1}\) TSS). However, the overall removal in a medical treatment scenario showed only a small effect of the adsorption (Table 4, roxithromycin) because of the very high sludge age with corresponding small excess sludge removal. It is concluded that the most important elimination mechanism for micropollutants in a small MBR is biological degradation or transformation.

![Figure 4: a) Influence of biological degradation on micropollutant removal for a small MBR (HRT = 3.2 d, one completely stirred reactor, sludge concentration = 6 g TSS l\(^{-1}\), SRT > 100d), compared to a centralized WWTP according to the model of Joss et al. (2006; recycle ratio = 2, HRT = 0.5d, number of biological compartments = 1 and 3, sludge concentration = 3.5 g TSS l\(^{-1}\)). b) Influence of the adsorption on micropollutant removal in a small MBR compared to conventional centralized treatment with an average sludge production of 130 mg TSS l\(^{-1}\). Steady-state conditions are assumed.](image-url)

**Concentrations of Micropollutants**

The concentrations of micropollutants in the single-house influent are either negligible or several factors higher than in centralized municipal sewer systems, because one out of only a few persons undergoes medical treatment, whereas in a large catchment, the ratio is one out of
500 to several thousand. Modeled maximum concentrations are indicated in Table 4. The concentrations in the effluent depend on the HRT, SRT, $k_{\text{biol}}$, $K_d$, the duration of the medical treatment and the water flow distribution. The maximum effluent concentration may be much higher than in average (e.g. ibuprofen: $c_{\text{max,e}} = 180 \, \mu g \, l^{-1}$, $c_{\text{av,e}} = 22 \, \mu g \, l^{-1}$) because the water flux varies strongly over the day. This effect is amplified with high $k_{\text{biol}}$ values.

The biodegradation rate constants estimated from the experiments are only valid in a concentration range similar to centralized treatment. As shown in Table 4, the expected effluent concentrations without or even with degradation are very high and partly even in the range of the therapeutic concentration within the human body. It is not well known, whether very high concentrations of antibiotics or other micropollutants negatively affect the performance of the activated sludge with respect to C or N removal. Dokianakis et al. (2004) examined the impact of several pharmaceuticals in the mg l$^{-1}$ range (2 - 10 mg l$^{-1}$) on nitrite oxidizers and found a strong inhibitory effect only for triclosan and a slight inhibition for sulfamethoxazole in batch tests, while carbamazepine, clofibric acid and diclofenac did not influence the nitrite oxidizing capacity. The batch experiments performed in this study with sulfamethoxazole, sulfapyridine, trimethoprim, clarithromycin and roxithromycin using activated sludge from a conventional WWTP at concentrations of 0.5 mg l$^{-1}$ and higher did not show any adverse effect on the nitrification nor denitrification capacity either (see supporting information). The experimental setup and temporal resolution of samples did not allow quantification of model parameters with sufficient accuracy (see supporting information). It is therefore concluded, that the model parameters need to be assessed individually for each compound at higher concentrations in order to predict the performance of small treatment plants.

The temporarily high concentrations further imply that the effluent may be ecotoxicologically relevant and that permeate reuse for sprinkle irrigation of raw consumed crops may be undesirable due to potential health hazards. For 17α-ethinylestradiol, average effluent concentrations of 2-5 ng l$^{-1}$ are expected (if the estimated degradation rate constant is applicable, otherwise concentrations up to 50 ng l$^{-1}$ may occur), which is in the same range as in raw municipal wastewater. As known from literature endocrine effects on fish are very likely with concentrations at 5 ng l$^{-1}$ (Kidd et al., 2007). Dilution in the receiving water body or soil passage preceding discharge will reduce the environmental concentrations.

**CONCLUSIONS**

The elimination of organic micropollutants in a decentralized MBR for domestic wastewater treatment of a 3-person household with a sludge age >100d was assessed.

- The observed removal of pharmaceuticals in a small MBR was comparable to centralized WWTP at influent concentrations typical for municipal wastewater. Only slowly degradable substances ($0.01 < k_{\text{biol}} < 1 \, l \, g_{SS}^{-1} \, d^{-1}$) are expected to be slightly better eliminated due to the longer hydraulic and solids residence time of small MBRs. Very persistent pharmaceuticals such as diatrizoate or carbamazepine are neither removed in large nor small WWTP.

- The major elimination mechanism for pharmaceuticals is biological transformation. Adsorption to the activated sludge only plays a minor role because most substances only show a small affinity to the activated sludge.

- No relevant lag phases for inducing biological degradation of pharmaceuticals were observed. This led to the conclusion that the studied substances are metabolized by enzymes constitutively expressed during activated sludge treatment, and that at concentrations of $\mu g \, l^{-1}$ the microbial population does not need to be adapted by prolonged or regular exposure to the micropollutants.
- Certain substances were only biodegradable under anoxic or aerobic conditions: for instance ibuprofen, bezafibrate, iopromide and estrone.
- During medical treatment of a tenant, pharmaceutical concentrations in the influent and effluent of a small MBR are expected up to the mg l\(^{-1}\) range. No inhibition of the activated sludge treatment with respect to carbon or nitrogen removal was observed, but the removal of pharmaceuticals in a high concentration range may significantly differ from that at lower µg l\(^{-1}\) concentrations. However, the high concentrations of micropollutants negatively affect the quality of the wastewater in case of reuse.

ACKNOWLEDGEMENTS
This study was partly financed by the RECLAIM WATER Project. The European Commission is acknowledged for co-funding this project under contract number 018309 in the Global Change and Eco-system sub-priority of the 6th Framework Programme. Moreover, the Solothurn cantonal agency for the environment is acknowledged for its financial support. Many thanks also go to Josef Asmin and Stephan Koepke from the Department for Environmental Chemistry at Eawag for their chemical analysis.

REFERENCES


Supporting information:

The fate of selected micropollutants in a single-house MBR: Antibiotics at mg·l⁻¹ concentration do not inhibit nitrification of activated sludge

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PURPOSE OF THE BATCH EXPERIMENT

The experiment in the single house MBR was conducted at pharmaceutical concentrations similar to municipal wastewater, in the range of 0.1 to 10 µg l⁻¹. However, in a small MBR, pharmaceutical concentrations of up to a few mg l⁻¹ are expected, which is in the same range as the therapeutic concentration within the human body. It is not known, whether such high concentrations result in an inhibition of the activated sludge.

Therefore activated sludge was exposed to high concentrations of antibiotics (sulfamethoxazole, sulfapyridine, trimethoprim, clarithromycin, roxithromycin; total concentration of nearly 5 mg l⁻¹) in a batch reactor (reactor A). As a control, a second batch (reactor B) with activated sludge was exposed to the same environmental conditions like reactor A without spike. The main purpose of this experiment was to elucidate whether inhibition of activated sludge occurs at such high antibiotics concentrations. Nitrification and denitrification were performed alternately and monitored to check for inhibitions. Several samples were analyzed for the spiked antibiotics in order see whether biological degradation occurs and for confirming exposure to the target antibiotics concentrations. The identification of sorption and degradation constants was not a primary purpose of the experiment and no extra effort was done to confirm the reproducibility of the modeled K_D and k_biol.

MAXIMUM NITRIFICATION RATE IN THE BATCH EXPERIMENT

The nitrification rates were assessed in batch experiments through continuous measurements of the oxygen uptake rates (OUR) during nitrification. The nitrification rates were measured daily as follows: the aeration was switched on until the oxygen uptake rate reached a steady state (endogenous respiration). In a first step, sodium nitrite was added (1.5 mg_N l⁻¹) to activate the nitrite oxidizers, thus increasing the oxygen demand. In a second step, another 1.5 mg_N l⁻¹ sodium nitrite plus 10 mg_N l⁻¹ ammonium bicarbonate (NH₄HCO₃) were added. The maximum nitrification rate was then calculated as the difference between the OUR after ammonium addition minus the OUR during endogenous respiration.
The measured OURs for the reactor with antibiotics and for the control are shown in Figure SI-1. At days 3 and 4, the OURs could not be determined due to technical problems. Batch B (control) was exposed to slightly lower oxygen concentrations during these days.

**Figure SI-1:** Measured nitrification rates during the experiments with high (3 - 4.5 mg l\(^{-1}\)) antibiotics concentration. The rates at days 3 and 4 could not be determined due to technical problems.

**ANTIBIOTIC CONCENTRATIONS IN THE BATCH EXPERIMENTS**

5 antibiotics (sulfamethoxazole, sulfapyridine, trimethoprim, clarithromycin, roxithromycin) were spiked daily into reactor A during the first four days. The initial concentration was between 0.25 and 1 mg l\(^{-1}\), followed by daily additions of 80 to 300 µg l\(^{-1}\) (Figure SI-2). The lines in Figure SI-2 shows the model results with the best estimates for \(k_{\text{biol}}\) and \(K_d\) (Table SI-1). Modeling was not a main purpose of this experiment, therefore the best estimates given for \(k_{\text{biol}}\) and \(K_d\) are afflicted with considerable uncertainty. But it is obvious, that the values differ from the ones reported in Table 3. Whether this difference is due to the higher concentration range or due to the differing properties of the activated sludge cannot be answered. Nevertheless, the lines are shown because they help to understand the experimental procedure.

**Table SI-1:** Estimated parameters for the batch experiment with high antibiotic concentrations. The data set does not allow an accuracy estimation of the values (see Figure SI-2).

<table>
<thead>
<tr>
<th>Antibiotic</th>
<th>(k_{\text{biol}}) l g(_{\text{TSS}}) d(^{-1})</th>
<th>(K_d) l g(_{\text{TSS}}) (^{-1})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sulfamethoxazole</td>
<td>0.02</td>
<td>0.1</td>
</tr>
<tr>
<td>Sulfapyridine</td>
<td>0.02</td>
<td>0.1</td>
</tr>
<tr>
<td>Trimethoprim</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>Clarithromycin</td>
<td>0.2</td>
<td>0.1</td>
</tr>
<tr>
<td>Roxithromycin</td>
<td>0.3</td>
<td>0.2</td>
</tr>
</tbody>
</table>
Figure SI-2: Measured concentrations (dots) of the 5 antibiotics (sulfamethoxazole, sulfapyridine, trimethoprim, clarithromycin, roxithromycin) during the experiment with high concentration. Arrows indicate spiking of antibiotics. The lines are the best fits using the model.
CHAPTER 7

REDUCING THE NATURAL COLOR OF MBR PERMEATE WITH ACTIVATED CARBON OR OZONE

Christian Abegglen, Adriano Joss, Marc Boehler, Simone Buetzer, Hansruedi Siegrist

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submitted to Water Science and Technology
Reducing the natural color of membrane bioreactor permeate with activated carbon or ozone

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ABSTRACT
The suitability of two membrane bioreactors for on-site wastewater treatment and reuse in Switzerland was investigated. The treated wastewater was used for toilet flushing and gardening, with water recycling rates of 30% (single family house) and almost 100% (toilets in a cable car station) respectively. Due to the recycling, an increase in a natural yellowish-brown color was observed, leading to double flushing of the toilets, higher cleaning requirements and increased permeate production.
Color removal with ozone, powdered (PAC) and granulated (GAC) activated carbon was assessed in lab and field experiments. PAC was added directly into the MBR, whereas ozonation and GAC were applied to the permeate. The dosage of ozone or activated carbon depended on the recycling rate and color intensity. The ozone requirement for sufficient color reduction was in the same range as for the removal of trace organics. If color removal is necessary, PAC is the option best suited to small treatment plants, with a requirement of 30-50 g m⁻³ for 30% and 100 g m⁻³ for 100% water recycling.

Keywords: decentralized wastewater treatment; color removal; water recycling; membrane bioreactor (MBR)

INTRODUCTION
In Switzerland, toilet flushing accounts for 30% of domestic water use (SVGW, 2002) or 30-50 l P⁻¹ d⁻¹. Although this amount has decreased during recent years due to advances in toilet technology, it could be replaced by water of lower quality. There are several examples of buildings or neighborhoods where the toilets are flushed with rainwater (e.g. Millennium Dome, England: Birks et al., 2004; Eawag building, Switzerland) or reclaimed wastewater (Shinjuku, Japan: Maeda et al., 1996).
No internationally accepted guidelines for the quality of reclaimed water exist. Some countries and states have defined their own standards, which depend on the usage of treated wastewater. Table 1 gives an overview of the water quality criteria for domestic use (including toilet flushing, gardening, cleaning). Most guidelines include hygienic parameters and a selection of physical and chemical values.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Korea</th>
<th>Japan</th>
<th>USA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Turbidity</td>
<td>NTU</td>
<td>&lt;2</td>
<td>&lt;5</td>
<td>&lt;2</td>
</tr>
<tr>
<td>Color</td>
<td>Pt-Co</td>
<td>&lt;20</td>
<td>&lt;10</td>
<td>Clear</td>
</tr>
<tr>
<td>Odor</td>
<td></td>
<td>Inoffensive</td>
<td>Odorless</td>
<td></td>
</tr>
<tr>
<td>COD</td>
<td>mg/l</td>
<td>&lt;20</td>
<td>&lt;3 (BOD)</td>
<td>&lt;10 (BOD)</td>
</tr>
<tr>
<td>Total coliform</td>
<td>cfu / 100 mL⁻¹</td>
<td>N.D. a)</td>
<td>&lt;50</td>
<td>N.D. a)</td>
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<tr>
<td>pH</td>
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<td>5.8-8.5</td>
<td>5.8-8.6</td>
<td>6-9</td>
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</table>


a) Not detected
Reclaimed wastewater is mainly used for toilet flushing in densely populated areas or in regions with water shortages. Its advantage over rainwater is the smaller storage capacity required, because wastewater is steadily produced while rain events may be distributed irregularly over the year. This effect is amplified when the water is also used for gardening. A disadvantage of current recycling schemes is the double piping system required between the treatment facility and the end-users. On-site treatment plants with water recycling minimize the need for piping systems. Membrane bioreactors (MBRs) probably represent the most promising option for on-site water recycling. In this study, two decentralized MBR systems were investigated for various applications in Switzerland (Figure 2). One plant was installed as a study object for alternative wastewater treatment in densely populated areas. This small-scale MBR treats the domestic wastewater of a 4-person household. The effluent is used for toilet flushing and gardening and accounts for 30% of the total water consumption. The second plant is located in a cable car mountain station of a ski resort. It only treats toilet wastewater, and as no freshwater is available, the recycling rate is 80 - 100%.

Despite a good effluent quality in terms of COD and nutrients in both MBRs, a problematic increase in a yellowish-brown color was observed:

- The yellow colored water looked like urine. This led to double flushing and higher water consumption, thus increasing membrane operation times and treatment costs.
- Coloring substances were deposited on calcium carbonate precipitates, causing unaesthetic marks in the toilet bowl (Figure 1) and more frequent cleaning.
- The toilets did not look very pleasant.

Color removal is a major problem in industrial wastewater treatment (textile industry, pulp and paper, food processing industry). Hao et al. (2000) give an overview of various decolorizing techniques. However, only little data is available on domestic wastewater treatment. Park et al. (2005) investigated color removal with UV and advanced oxidation with H₂O₂. Oota et al. (2005) observed a yellowish-brown color in a domestic wastewater reuse scheme and stipulated further treatment by ozonation or activated carbon adsorption to make the water acceptable for reuse. Ohnishi et al. (1998) set up a combined system with an MBR and powdered activated carbon addition to the sludge for a domestic wastewater system and found sufficient color removal by adding 50 gPAC m⁻³. The goal of this paper is to show various treatment methods for decolorizing domestic wastewater that are applicable to small-scale MBR systems. Moreover, we also want to show the advantages and disadvantages as well as costs of these methods. We carried out lab and field experiments with granulated (GAC) and powdered (PAC) activated carbon. Oxidation with ozone, H₂O₂ and chlorine were investigated on a lab scale.

A hygienically safe effluent was important for water recycling used for gardening. In order to assess the hygienic quality, selected microbial parameters were monitored over two years.

**METHODS**

**WASTEWATER TREATMENT PLANTS**

The treatment plant in Zuchwil (Switzerland) was installed as a study object for alternative wastewater treatment in densely populated areas in the basement of a 4-person household. It was in operation from October 2004 until January 2008. The plant in Zermatt (Switzerland) is located at the ‘Hohtaelli’ cable-car mountain station (3273 m asl) and treats wastewater from
toilets (mainly urine) and hand-washing water. It is in operation throughout the year, although most of the load occurs between November and April when the cable car is open to the public. There are also huge flow variations during the tourist season, depending on the vacation schedule and weather conditions.

Both treatment plants have a similar flow scheme (Figure 2): the wastewater flows into the first reactor where hydrolysis and partial denitrification take place. In order to minimize odor problems, activated sludge from the second reactor is recycled. The water is pumped through a sieve to the second reactor that is aerated intermittently to obtain stable nitrification and denitrification. More detailed information about the treatment plants can be found in Abegglen et al. (2008) and Bohler et al. (2007).

**Figure 2:** Flow schemes of the MBRs installed in the basement of a single-family dwelling (left) and in the mountain station of a ski resort (right). For particulate screening, the effluent of the 1st reactor is pumped through a 6 mm perforated sieve (left) and a 3 mm slit sieve (right) respectively.

### Determination of Color

Several standards for measuring the natural color are available. The most common method is the optical comparison of the sample with a Pt-Co standard solution (ISO 7887, Section 4). The value is reported as a “true color”, Hazen units or Pt-Co units, which are all equivalent. This method was not applicable in our setting, as only small sample volumes were available and the uncertainty of the method is quite high (Bennett and Drikas, 1993). The color was determined according to ISO 7887, Section 3, with minor adaptations. Instead of measuring the absorption coefficient at wavelengths of 436, 525 and 620 nm, we did so at 380 nm and 400 nm. This was because the values at the original wavelengths were very small. The absorption coefficient was used as a measure for “color concentration” based on the Lambert-Beer law:

\[ \text{Abs} = \varepsilon \cdot d \cdot C \]

where \(d\) is the optical path length (0.01 m), \(\varepsilon\) the extinction coefficient and \(C\) the concentration of the substance. Assuming that similar substances cause the coloration (substance C), a constant \(\varepsilon = 100 \text{ m}^3 \text{ g}^{-1} \text{ m}^{-1}\) was used for convenience.

Variations in pH of between 7 and 8 did not affect the measurement. The measurements were conducted with a UV-VIS spectrometer (Cary 300, Varian Inc., Palo Alto, USA). The coloring substances were characterized using liquid chromatography coupled to an organic carbon detector, as described by Meylan et al. (2007).

### Wastewater Characteristics

Most experiments were conducted with activated sludge or permeate from the pilot plant in Zuchwil. For comparison, COD and absorbance \((\lambda = 400\text{nm})\) values of four MBRs treating various wastewaters are shown in Table 2. The plant in Duebendorf is an MBR pilot plant at
the Eawag experimental hall treating sewage pumped from the combined sewer system of Duebendorf (Joss et al., submitted). The plant in Meilen is a small-scale MBR treating the wastewater of a 2-person household, where no water recycling takes place.

Table 2: COD concentrations and absorbance (at λ=400 nm) in the permeate of different MBRs

<table>
<thead>
<tr>
<th>Plant</th>
<th>COD g m⁻³</th>
<th>Abs (400nm)</th>
<th>Reuse %</th>
<th>Membrane pore size μm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Duebendorf</td>
<td>19 (± 6)</td>
<td>0.018</td>
<td>0</td>
<td>0.2</td>
</tr>
<tr>
<td>Meilen</td>
<td>20</td>
<td>0.044</td>
<td>0</td>
<td>0.4</td>
</tr>
<tr>
<td>Zuchwil</td>
<td>34 (± 7)</td>
<td>0.066</td>
<td>30</td>
<td>0.04</td>
</tr>
<tr>
<td>Hohtaelli</td>
<td>100</td>
<td>0.129</td>
<td>100</td>
<td>0.04</td>
</tr>
</tbody>
</table>

PAC EXPERIMENTS

All batch experiments were conducted with wastewater from Zuchwil. A fixed amount of activated carbon (PAC) was added to 500 ml of permeate or mixed liquor (activated sludge). After stirring for 24 hours at room temperature, equilibrium was reached. Samples were filtered with 0.7 μm filters (Pall Corp., East Hill, USA), stored at 4°C and measured within 48 hours.

Experiments were conducted at room temperature (20-22°C) and the unchanged pH level was monitored. PAC of type “Adsorber MB4” was commercially available from Faerber-Schmid (Schlieren, Switzerland). The average diameter of the carbon particles was 2.5 μm.

Data for permeate and activated sludge (AS) experiments were not comparable. Firstly, the permeate already passed through a membrane of 0.04 μm pore size before PAC addition, whereas the activated sludge samples were only filtered with 0.7 μm glass fibre filters for measurement. This means that more substances are available in the AS system (molecules > 0.04 μm), causing a more intensive color and acting as competitors for sorption.

Field experiments were conducted during normal operation of the plants. PAC was dosed directly to the MBR and the absorbance was measured over time. At the Zuchwil plant, direct dosing only was tested for two weeks, whereas at the Hohtaelli plant, color removal was practiced during the whole season.

EXPERIMENTS WITH GAC

Experiments with granulated activated carbon were performed at the Hohtaelli MBR. A box filled with GAC was installed between the membrane and the permeate tank. The water trickled through this filter at an average flowrate of 0.06 m min⁻¹ and with a hydraulic retention time of 5 minutes.

OZONE EXPERIMENTS

Two types of ozone experiments were conducted, only with permeate from the Zuchwil plant. In a first series of experiments, distilled water was saturated with ozone. Different amounts of this solution were added to the permeate. The mixture was stirred and absorbance values were determined.

In the second series, ozone gas was injected into the permeate. Ozone was produced with an air-fed ozone generator used in aquariums (model Certizon 300 from Erwin Sander Elektroapparatebau GmbH, Uetze-Eltze, Germany). The lab experiments were carried out with two reactors of different volumes (3 and 10 l) and gas injection depths (0.25 m and 1.2 m). The ozone loads in the feed gas and off-gas were determined by conducting the gas through two washing flasks in series filled with potassium-iodine solution as described in method 2530 E of the Standard Methods (APHA, 1998).

The ozone concentration in the feed gas was between 0.8 - 1.25 gO₃ m⁻³ air
RESULTS AND DISCUSSION

HYGIENIC PARAMETERS

As the membrane pore size (here: 0.04 μm) is generally smaller than the bacteria (faecal coliforms: 0.6 - 1.2 μm diameter), MBRs are expected to produce a hygienically safe effluent. Although several investigations (e.g. Zhang and Farahbakhsh, 2007) could not detect any faecal coliforms in MBR effluents, hygiene is not guaranteed due to re-infection of the permeate.

The effluent of the MBR in Zuchwil was periodically tested during two years on aerobic mesophilic germs, E. coli and Enterococcus. A total of 36 grab samples were analyzed; the results are summarized in Table 3.

Table 3: Results of microbial counts in the effluent of the Zuchwil MBR plant

<table>
<thead>
<tr>
<th>Unit</th>
<th>Aerobic mesophilic</th>
<th>E. coli</th>
<th>Enterococcus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bathing water standard</td>
<td>cfu / 100ml</td>
<td>-</td>
<td>500 (a)</td>
</tr>
<tr>
<td>No cfu detectable</td>
<td># samples</td>
<td>-</td>
<td>18</td>
</tr>
<tr>
<td>Bathing water standard exceeded</td>
<td># samples</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>Range</td>
<td>cfu / 100ml</td>
<td>0.3 - 74 *10^3</td>
<td>0 - 109</td>
</tr>
</tbody>
</table>

(a) Valid in the EU (Directive 2006/7/EC)

Neither E. coli nor Enterococcus were detected in 33% of the samples, but 90% of them conformed to bathing water quality. The excess values occurred after chemical backwashing of the membrane. The contamination possibly occurred due to the membrane being cleaned outside the reactor.

Aerobic mesophilic germs were detected in all samples. Organic carbon and nutrients, which support microbial growth, were still abundant in the effluent of the MBR and biofilm growth was already visible in the permeate mains after a short time of operation. The detected bacteria probably originated from sloughing biofilm.

COLORING SUBSTANCES

There is agreement in the literature that refractory organic substances such as humics are the main coloring agents. However, it is not clear where they originate. Some researchers assume them to be urobilin and stercobilin (Oota et al., 2005), while others suggest that they stem from metabolic byproducts of microorganisms (Park et al., 2005). The absorption spectra show them to be a mixture of substances. However, their source (urine/faeces or metabolic byproducts) could not be determined.

In order to find out whether molecular size plays an important role, as suggested by Takahashi et al. (1995) or Park et al. (2005), several samples were investigated using different types of molecular cutoff and size-exclusion chromatography (SEC). According to the result of this analysis, all fractions contributed to the natural color of the wastewater, and the relative distribution of the fractions remained constant even after applying ozone or carrying out PAC treatment.

OZONATION

The goal of the experiments was to find out whether ozone can ensure sufficient and stable color removal in this setting, and what ozone dosage would be necessary. Several ozone applications in water and wastewater treatment have been described in the literature, e.g. reduction of yellow substances to enhance water quality in aquariums (Otte et al., 1977),
decolorizing industrial wastewater (Hao et al., 2000), degradation of trace organics (e.g. Huber et al., 2005), or processing human urine for toilet flushing (Gulyás et al., 2004). Batch experiments with saturated ozone solution showed that ozonation is an adequate process for color removal. An amount of 5-8 gO₃ m⁻³ was needed for 50% color removal, which was in the same order of magnitude as for reducing trace organics (Huber et al., 2005). Technically, ozone has to be produced on-site and added via a bubble column. Several factors affect the efficiency of color removal: injection depth, bubble size, ozone concentration in the gas, contact time and water quality (background DOC concentration).

Experiments showed a pseudo first-order relation between ozone dosage and color reduction (Figure 3), similar to Takahashi et al. (1995):

\[
\frac{d\text{Abs}}{dt} = -c \cdot R \cdot \text{Abs} \\
\ln \frac{\text{Abs}(t)}{\text{Abs}_0} = -c \cdot R \cdot t
\]

Abs is the absorbance [-], R the ozone to DOC dosage rate [gO₃ gDOC⁻¹ h⁻¹] and c a constant [gDOC gO₃⁻¹]. The dosage rates are expressed as net ozone to DOC consumption rates in order to eliminate the influence of the injection depth (calculated from the difference between in- and offgas concentration). R varied from 0.66 to 3.8 gO₃ gDOC⁻¹ h⁻¹ (DOC concentration 7.5 - 12 g m⁻³). For these rates, c was 0.75 ± 0.2 gDOC gO₃⁻¹. Dosing of around 1 gO₃/gDOC is therefore sufficient for 50% color reduction.

\[0.66\text{ gO₃/gDOC/h} \quad 0.66\text{ gO₃/gDOC/h} \quad 0.8\text{ gO₃/gDOC/h} \quad 3\text{ gO₃/gDOC/h} \quad 3.8\text{ gO₃/gDOC/h} \]

\[0.66\text{ gO₃/gDOC/h} \quad 0.66\text{ gO₃/gDOC/h} \quad 0.8\text{ gO₃/gDOC/h} \quad 3\text{ gO₃/gDOC/h} \quad 3.8\text{ gO₃/gDOC/h}\]

Figure 3: Relative color reduction by ozonation. Abs/Abs₀ = 0.5 appears optically clear in the toilet bowl. The ozone consumption rate is between 9 and 30 mg l⁻¹ h⁻¹. Color decrease over time (left) and depending on the ozone to DOC consumption (right).

Takahashi et al. found c values between 0.4 - 0.5 gDOC gO₃⁻¹ with a different composition of the background DOC (only humic acid) and R ranging from 3 - 8 gO₃ gDOC⁻¹ h⁻¹. So the chemical composition of the coloring substances plays an important role in the decolorization kinetics.

A field experiment at the Zuchwil treatment plant with ozonation by an aquarium ozonator in the permeate tank for color removal was successful during one week (the available installation was not suitable for ozonation experiments over the longer term).

Although ozone is used in the aquarium-owners community, its application in private homes without professional maintenance seems questionable due to its toxicity. As ozonated water has a higher biodegradability, more biofilm growth is expected in the recycling lines.
Powdered Activated Carbon

The suitability of PAC for removing color from domestic wastewater was assessed by batch adsorption experiments in the lab and by evaluating plant data. The PAC dosage ranged from 10 to 300 g_PAC m^-3 wastewater^-3. The initial color concentration was similar for all lab experiments. These experiments allowed the required PAC quantities for domestic wastewater to be determined (Figure 4). The differences between the two data series (permeate and activated sludge) stem from the different wastewater compositions (permeate: membrane filtered before the experiment, no particles; activated sludge: activated sludge flocs, filtration only for measurement). A 40-60% color removal resulted in optically clear water. The batch experiments indicated that sufficient color removal could be achieved with a PAC dose in the range of 20 - 50 g m^-3 for permeate and 80 - 150 g m^-3 for activated sludge experiments.

Figure 5 shows the results of four field experiments: two were conducted in Zuchwil (zw 1 and zw 2) and two at Hohtaelli (ht 1 and ht 2). For each experiment, a certain amount of PAC was dosed and the changes in absorbance were measured over time. The absorbance curve showed a minimum 24 hours after the PAC dosage (Figure 5a). This implied that the maximum adsorption capacity was reached within this time (as confirmed by batch experiments). No (or only little) adsorption took place afterwards and the color increase was proportional to the incoming water flow, which is shown in Figure 5b, where the treated water volume per PAC is plotted against the absorbance. The slope depends on the influent color concentration and the PAC dose. Because of the specific wastewater compositions, the absorbance target values were different for Hohtaelli (0.05) and Zuchwil (0.03). Figure 5b shows that 30 - 50 g_PAC m^-3 were needed for the domestic wastewater treatment with a lower recirculation rate, while Hohtaelli required 60 - 110 g_PAC m^-3 with almost 100% recirculation. Long-term experience at Hohtaelli revealed a PAC consumption of 100 g m^-3. These values may vary depending on the characteristics of the PAC used in a system. However, they are in the same range as reported by Ohnishi et al. (1998), who used 50 g m^-3 for a system similar to that in Zuchwil.

Figure 4: Relative color removal for different PAC dosages for permeate and activated sludge. The gray area indicates a dosage with optically sufficient color removal.
Figure 5 (a) Development of absorbance after single PAC dosages at Hohtaelli (ht) and Zuchwil (zw) over time and (b) in relation to the dosed PAC and treated water volume. After 24 hours, the sorption capacity is depleted (Sa) and the color increases proportionally to the water flow (Sb). The water was optically clear at absorbance values (at 400 nm) of 0.05 for Hohtaelli and 0.03 for Zuchwil.

The membrane manufacturer was concerned about the negative effects of PAC on the membranes, as carbon particles could block the membrane pores. Measurements of the particle size distribution showed that the smallest carbon particles were in the range of 1 µm, so pore blockage could be excluded. In the literature, PAC is said to have a positive effect on membrane permeability (Ohnishi et al., 1998; Kim and Lee, 2003). According to our measurements, the permeability was completely unaffected by the PAC dosage (Figure 6), at least during the short experimental period.

Figure 6: Average daily permeability (based on the pure filtration time) at the Zuchwil plant over time. The PAC dosage (19.4, 26.4; arrow) did not show any impact.

**Granulated Activated Carbon**

Granulated activated carbon was tested as a color-removing agent in the Hohtaelli treatment plant during a period of one week (7.4.2006-13.4.2006). A filter used 11 kg GAC. The water passing through this filter ran crystal clear only for six days (corresponding to 350 filter bed volumes), after which the adsorption capacity was depleted (data not shown). GAC consumption would be approximately 1100 g m⁻³, which was ten times that for PAC. This indicated that color molecules only adsorbed onto the surface of the carbon particles or into macropores, but did not diffuse into micropores, or that all kinds of organic material adsorbed onto the carbon, resulting in low efficiency.
The advantage of GAC is that it can be applied in the recycling stream, whereas PAC requires all the wastewater to be treated. GAC treatment may be promising given a low recycling rate (<10%) and an appropriate reactor design.

**Chemical Oxidation with H₂O₂ and Chlorine**

The suitability of H₂O₂ and chlorine for color removal was assessed in a few lab experiments. Although this treatment worked well in the lab, it was not implemented in real applications because handling chemicals such as H₂O₂ and chlorine in private houses poses a safety risk for the residents.

**Costs**

If color is not removed from public toilets, e.g. at Hohtaelli, users assume that their predecessor forgot to flush, so the toilets end up being flushed several times. This leads to higher water consumption and thus to increased permeate production (i.e. higher energy costs and decreased membrane life). Moreover, the toilets probably need more frequent cleaning to remove precipitates in the toilet bowl.

In Table 4, the costs for color removal by PAC or ozone are compared with the expected extra costs without color removal. The MBR in Zuchwil with 30% water recycling is used as the model plant. The cost comparison is based on experience with the installed and tested equipment for this study. The energy consumption of both plants was in the order of magnitude of 6-10 kWh m⁻³, which is very high compared to similar plants described in the literature (e.g. Harms and Englert, 2006) and due to inefficient aggregates. An energy demand of 5 kWh m⁻³ was assumed for the cost calculations.

Further design parameters and assumptions were as follows:

- **General:** Electricity in Switzerland costs 0.1 € kWh⁻¹.
- **No color removal:** Toilet flushing accounts for 30% of total water consumption. This value will increase by 10% assuming that 1/3 of toilet users are unfamiliar with the system and concerned about the color, so they flush before and after using the toilet. The extra energy demand per m³ of treated wastewater is therefore 0.5 kWh m⁻³. The frequency of cleaning the membrane and toilet is not increased.
- **Ozone:** The ozone gas-water transfer efficiency is 85% (corresponding to a bubble diameter of 1 mm and an injection depth of 1.5 m). With an energy requirement of 120 kWh kgO₃⁻¹, the energy efficiency of the ozone generator is poor. As regards investment costs, only the ozone equipment (air pump, ozone generator, tubing) was considered, assuming a life span of 5 years (no data available): this corresponds to 1500 m³ of treated water. Installation costs and off-gas treatment were excluded.
- **PAC:** No automatic dosage device is required, as PAC is available as slurry and can be dosed manually by the plant operator during his control visits (or by the residents). The excess sludge treatment includes air-drying (15% dry matter) and disposal with conventional garbage. Costs for disposal are 2 € per 50 liters of garbage (costs for a garbage disposal bag in Switzerland).

The comparison shows that color removal is unnecessary from an economic point of view. Even at higher recycling rates, its costs remain higher as long as no extra cleaning (of the toilet bowl or membrane) is required. Fletcher et al. (2007) estimate the yearly operational costs for a membrane package plant to be 2.9-3.1 € m⁻³. So the costs of color removal would raise these costs by 10-30%.
**Table 4: Cost calculation for color removal compared to the extra costs caused by double flushing if no treatment is installed**

<table>
<thead>
<tr>
<th></th>
<th>Ozone</th>
<th>PAC</th>
<th>No treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dosage [g m⁻³]</td>
<td>5</td>
<td>50</td>
<td>-</td>
</tr>
<tr>
<td>Extra energy [kWh m⁻³]</td>
<td>0.7</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>Investment [€]</td>
<td>500</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unit costs [€ m⁻³], (4 € kgPAC⁻¹)</td>
<td>0.07</td>
<td>0.2</td>
<td>0.05</td>
</tr>
<tr>
<td>Disposal costs [€ m⁻³]</td>
<td></td>
<td>0.01</td>
<td></td>
</tr>
<tr>
<td>Total costs [€ m⁻³]</td>
<td>0.40</td>
<td>0.21</td>
<td>0.05</td>
</tr>
</tbody>
</table>

**CONCLUSIONS**

- The natural yellowish-brown color of wastewater is intensified by recycling. Reuse of colored wastewater is an aesthetic rather than an operational problem. Color removal makes sense in public environments where water is scarce, whereas users in private houses may tolerate the color.
- Natural color can be determined successfully by measuring the spectral absorption at 380 to 440 nm. A direct conversion to Pt-Co or Hazen units is not possible.
- Ozone treatment reduces the natural color. The amount of ozone required is in the same range as for reducing trace organics. Depending on the amount of water to be treated, small ozone generators used for aquariums may be used. Residual ozone in the off-gas is corrosive and toxic and needs to be destroyed. The application of ozone in small treatment plants is probably not the best solution.
- Good color removal is also obtained by powdered activated carbon. Direct addition to the MBR reduces color and sorbs dissolved organic matter. No effect on permeability was observed. Its disadvantages are the extra sludge production and the necessity for continuous (weekly) dosing. This is the most promising treatment for small-scale applications.
- The natural color was substantially reduced by granulated activated carbon. The sorption capacity for color was 10 times smaller than for PAC. If only a small fraction of the wastewater needs color removal, GAC may be better suited than PAC.
- Costs for color removal increase operational costs by 10-30%, which is 5-10 times higher than the extra operational costs due to double flushing.
- From a hygienic point of view, MBR effluent is unsafe due to bacterial growth in effluent pipes or storage tanks, although fecal bacteria are retained by the membrane.

**REFERENCES**


CONCLUSIONS
CONCLUSIONS

Several aspects of MBR technology for domestic wastewater treatment in a private house were investigated by running a pilot plant. Different operating conditions were tested during the 39 months of operation. The research focused on downscaling, testing and optimizing available technology under real life conditions. Due to the holistic approach and the wide technology spectrum it was not possible to integrate and investigate every detail. In the following, the results of the study are briefly summarized and classified in a larger context. The questions raised in the introductory part are specifically addressed.

MBR OPERATION AT HOUSEHOLD LEVEL

- Operation of an MBR at household level (4-8 Pe) is not comparable with large MBR or lab applications. The systems mainly differ in their influent composition and dynamics and with respect to monitoring and maintenance. Results from conventional MBR research are therefore often not directly applicable on a household scale.

- The treatment efficiency for particles, organic matter, germs, ammonium and nitrate was very high. Even threshold values for large WWTP were met. Due to this good quality, the permeate is well suited for reuse, thus reducing freshwater consumption.

- Automatic failure detection systems with electronic notification are well suited for identifying breakdowns and allow fast action. In combination with remote access and control with a minimum of measurement devices, the state of the plant can be reliably monitored.

- The most critical point of MBR operation is the membrane permeability. To prevent clogging, particles have to be retained. This can be achieved either by primary sedimentation or a sieve. Primary clarifiers are most commonly used, but result in primary sludge production and odor problems due to anaerobic and putrescent conditions. Transformation of the primary clarifier to a biological reactor by mixing and sludge recycling from the MBR reduced the odor problems, enhanced nutrient removal and provided a larger storage capacity for hydraulic peak loads. Furthermore, only stabilized sludge is produced. The disadvantage of this process scheme is a faster decrease in permeability.

- Only limited phosphorus removal can be achieved using the conventional MBR scheme with a primary clarifier. Legislation usually does not require enhanced P-removal in small wastewater treatment plants. However, this may be necessary in certain regions, and phosphate effluent loads need to be minimized especially if small treatment plants are seen as an alternative to the conventional system. Three methods, namely EBPR, urine source separation and treatment and electrochemical iron dosing were tested during this project.

- In spite of the strongly fluctuating influent and irregular sludge removal, EBPR proved feasible. Periods with zero influent (vacation) do not negatively impact the PAOs if anaerobic conditions prevail. From an operational point of view, it is hardly possible to achieve very low effluent concentrations reliably with the proposed flow scheme because of the influent dynamics. Operation with a primary clarifier prevents EBPR due to a lack of organic material.

- Urine source separation and precipitation of phosphate with magnesium oxide is effective and results in phosphate removal of over 95% in urine and up to 50% overall. Clogging of the NoMix toilets significantly reduced the urine separation efficiency. If dosed equally,
further treatment of the phosphate-poor effluent from the urine reactor in an MBR can be expected to work without problems.

- Instead of dosing chemicals for phosphate precipitation, iron can be added by electrolysis of submerged iron electrodes. Advantages are the robust operation due to lack of mechanical parts and simple control by adjusting the voltage to the flow. Negative effects like hydrogen gas production, permeability loss or impacts on the activated sludge are negligible. The main problem that could not be solved within this project was the formation of a passivation layer on the iron plates. This layer reduces iron dissolution and the energy efficiency of the process, but can probably be better controlled with coarse bubble aeration below the plates.

- Reuse of the permeate for applications without direct contact is safe. In a water recycling system, non degradable organic substances accumulate to a certain degree. This accumulation results in an intense yellow color of the water. Depending on the requirements, this can be removed with ozone or powdered activated carbon. However, the cost of this treatment is not negligible. In the presence of a small MBR, permeate recycling is economically superior to rainwater use because of the steady production and the smaller storage capacity required.

- The occurrence of micropollutants in small WWTP differs fundamentally from large catchments. The tenants are occasionally subject to medical treatment. The activated sludge is therefore not adapted to these substances and the influent concentrations of a single medication are 2-3 orders of magnitude higher than average in centralized treatment. It was shown that most substances are eliminated to the same extent as in large treatment plants, at least in the concentration range usually found municipal wastewater. The main removal mechanism is biological transformation, while sorption to the sludge only plays a minor role due to the low sludge production. Antibiotic concentrations of 1 to 5 mg l\(^{-1}\) do not result in inhibition of the biological activity (nitrification and denitrification). However, the high influent concentrations mean that the expected effluent concentrations are also very high and may impact the receiving water body.

- Specific costs per person for small MBRs are 1.25 - 2.5 times higher for household wastewater treatment than for the centralized system and are expected to be about equal for plants for 20-50 persons. While the technology itself is relatively cheap, the main cost drivers are engineering, installation and maintenance costs. Mass production and simpler installation systems will significantly reduce costs in future. The energy requirements for small plants are also higher, with the biggest share of energy being used for membrane scouring with coarse bubble aeration.

OUTLOOK
Small MBRs are nowadays mainly marketed and sold to private homes at isolated locations. This development is driven mainly by the increased legal requirements on small WWTP. Nevertheless, the market for small MBRs is small. One of the largest suppliers in Germany sold 210 units within five years, but their popularity is increasing as 70 of these were sold in 2005. There are also locations where water scarcity and the possibility of water recycling is the main driver. In Switzerland, this is mostly the case in Alpine regions, where several small MBRs are already in operation or planned, e.g. Säntis (Urnäsch), SAC Monte Rosa, Hohtälli, Klein Matterhorn (all Zermatt), Piz Corvatsch (St. Moritz). Water scarcity and the necessity of better water quality may also be of importance in Mediterranean tourist regions.

In the introduction, the question was raised whether small MBRs are a serious alternative to the conventional system with extended sewer systems and centralized treatment. With the technology applied in the Aquamin project, this question is answered in the negative because of the higher costs, energy requirement as well as for maintenance reasons. However, there is
still a huge optimization potential. Development of MBRs for private houses only started 15 years ago, and the technology has steadily improved. For example, flat-sheet membranes with backwashing have been developed since the start of the project four years ago. This could significantly minimize the energy demand and increase the membrane lifetime. As a parallel: in Germany, the first washing machines for laundries were marketed in 1951, but only from the 1960s they were available for the broad public, and within 20-30 years virtually every building was equipped with one. A similar development could be imagined for wastewater treatment, although the incentives are different. Laundry was a time and work-intensive process, while wastewater treatment does not involve any effort for the consumers (in industrialized countries). The technology could be used in developing countries with fast-growing cities (huge markets), where small and cheap MBRs could be introduced in stages to alleviate wastewater problems.

It is impossible to say whether MBR technology is the right process for decentralized wastewater treatment, or whether other, more promising technologies will be available. The advantages of this technology are that all wastewater fractions are treated in one process, wastewater sewers are no longer necessary, the effluent can be reused, the spatial footprint is small and remote monitoring is possible. If the treated effluent is not reused or infiltrated, it can be discharged to a nearby creek or the rainwater sewer. In combination with urine source separation, the nutrients could be recovered. The challenges are that some collection system has to be provided for the final products (e.g. excess sludge, urine, struvite), possibly in combination with municipal waste collection. The costs and energy demand are probably higher than for a centralized system.

We showed that the technology can basically be applied on a household scale. From an operational point of view, this is probably the most difficult and expensive situation. Semi-centralized systems for 20 - 500 persons could be operated more economically. Of course, the introduction of such a system has to be looked at from a holistic perspective. Questions of transition states, industrial wastewater, freshwater infrastructure, surface water quality or public health need to be addressed in this context but were beyond the scope of this thesis.
APPENDIX A

DOMESTIC WASTEWATER TREATMENT WITH A MEMBRANE BIOREACTOR

Christian Abegglen and Hansruedi Siegrist

Water Science and Technology 53(3), 69-78
Domestic wastewater treatment with a Membrane Bioreactor

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ABSTRACT
A small-scale membrane plant for treating the domestic wastewater of a four-person household is presented. The membrane bioreactor has been in operation for six months and achieves elimination rates of 90%, 95% and 80% for total organic carbon, chemical oxygen demand and total nitrogen respectively. Only little sludge is produced. The permeate is reused for flushing toilets and has a yellowish color. After investigations of the effluent quality, decolorisation of the permeate, energy efficiency and control strategies in the first year, urine will be treated separately in an automated precipitation reactor where struvite is produced to improve the overall phosphate removal of the plant.

Keywords: Small-scale wastewater treatment, urine separation, membrane bioreactor (MBR), water reuse, decentralized

INTRODUCTION
The conventional way of treating domestic and industrial wastewater with a central wastewater treatment plant requires long and expensive sewers. Although this system works very well, the costs of building up a whole system from zero are very high. Decentralized and semi-decentralized systems may be an option for sparsely populated areas as well as for fast-growing cities. An advantage of such systems could be the easier handling of resources recycling (water, nutrients and sludge). The statistics (Figure A-1) show that many regions lack a wastewater treatment and sewer infrastructure, either completely or partially.

Small-scale wastewater treatment plants with high purification efficiency such as membrane bioreactors (MBR) are not yet widely used. This may be due to their relatively high costs compared with conventional small-scale wastewater treatment plants (such as septic tanks) as

Figure A-1: Population of selected European countries not connected to central wastewater treatment plants (independent or no treatment). Data: Wieland (2003)
Appendix A

well as new technology and limited experience with small-scale systems. As regulations concerning small-scale wastewater treatment plants do not exist in most countries or do not demand a high-quality effluent, there has so far been no need for more sophisticated systems. However, new regulations or water scarcity and the need to exploit alternative water sources (recycling of treated wastewater) may bring about a change.

The “Aquamin” pilot study run in Switzerland over a period of three years investigates the potential of small-scale wastewater treatment plants with a membrane bioreactor combined with urine separation.

MATERIAL AND METHODS

THE AQUAMIN PROJECT

The Aquamin house project, implemented by EAWAG and the canton of Solothurn, investigates ideas designed to boost sustainability in urban water management. The project goal is to find a new way of treating wastewater, recycling the resources of water, sludge and phosphate on the smallest scale while meeting the same effluent quality standards as conventional WWTPs in Switzerland. A four-person house was equipped with a small-scale wastewater treatment plant in order to treat the wastewater as close to the source as possible. The plant was designed and installed during the summer and autumn of 2004, went into operation at the end of October and has been running since. A family of two adults working four to five days a week and two teenagers going to school live in the house (only one youth lived there during the first four months). The operation of the plant should not require any extra effort from the inhabitants of the house. Although the plant is connected to the local sewer system for safety reasons, this connection would only be used in case of a major breakdown.

WASTEWATER TREATMENT PLANT

The flow scheme of the treatment plant can be seen in Figure A-2. Two stages of its operation will be investigated. During the first year, all the wastewater flows directly into the membrane reactor, which is divided into two separate tanks. The first tank is used as a primary clarifier with a constant water level. To prevent shortcut flows, a baffle is installed in the middle of the tank with an opening at its base. Intermittent aeration creates some turbulence and injects oxygen to reduce anaerobic conditions and the production of malodorous substances. The second tank is the core unit of the treatment plant, an activated sludge tank with a submerged plate membrane. The membrane has an area of 4 m², the pores are 0.04 μm in size (Table A-1). Fouling is prevented by coarse-bubble aeration, and the oxygen input for the biological processes is secured by fine-bubble aeration. The two aeration systems are independent of each other. Permeate is withdrawn with intermittent control by the maximum and minimum water levels. Activated sludge can be pumped into the first reactor or removed from the system by pumping it into a filter bag where it is statically dewatered and air-dried. Permeate is stored in an outside tank and reused for flushing toilets and irrigating gardens. By recycling the wastewater, freshwater consumption can be reduced by 33%, which corresponds well to the expected value of 30-40% obtained by SVGW (SVGW, 2002). Freshwater consumption can even be greatly reduced if the treated wastewater is stored for irrigation in regions with dry summers (e.g. Eastern Europe).

A urine treatment reactor will be installed in the second stage. Urine is separated in NoMix toilets (Roediger GmbH) and can be stored. The urine reactor is operated in batch mode, and struvite (MgNH₄PO₄·6 H₂O) is precipitated by addition of MgO (Udert et al., 2003). Ammonium-rich and phosphate-poor effluent is passed from the urine reactor to the
membrane bioreactor for further treatment. During the first phase, ideal operation conditions for the urine reactor are tested at EAWAG.

Table A-1: Dimensions of the membrane bioreactor

<table>
<thead>
<tr>
<th>Volume of primary clarifier (normal, max)</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volume of MBR (minimum, maximum)</td>
<td>m³</td>
<td>1.2 (1.5)</td>
</tr>
<tr>
<td>Membrane area</td>
<td>m²</td>
<td>4</td>
</tr>
<tr>
<td>Pore size</td>
<td>µm</td>
<td>0.04</td>
</tr>
<tr>
<td>Distance between plates</td>
<td>mm</td>
<td>3</td>
</tr>
<tr>
<td>Permeate flux (normal load, high load)</td>
<td>m² h⁻¹</td>
<td>15 (25)</td>
</tr>
</tbody>
</table>

Figure A-2: Wastewater treatment plant in the Aquamin project

Measurements

All samples were analyzed by the cantonal laboratory of Solothurn. COD, total phosphorus and phosphate were analyzed with test tubes (Hach Lange GmbH: COD: LCK 514, LCK 914, TP, PO₄-P: LCK 350). Grab samples (1 liter) were taken from the influent and the effluent of the membrane bioreactor, transported to the laboratory and analyzed within hours. No samples could be taken from the influent to the primary clarifier, as raw wastewater flow cannot be measured and a sampling device would clog due to coarse material in the sewage. WTW devices of the 700 IQ series are used for online measurements in the membrane bioreactor (oxygen: Trioxmatic; ammonium: Ammolyt; pH, redox potential, temperature: Sensolyt; conductivity: TetraCon).

There are several accepted methods for measuring the color of a sample. According to ISO 7887 (ISO, 1994), it is measured by spectroscopy at three different wavelengths (436, 525, 620 nm). The American Public Health Association (APHA, 1995) proposes a similar approach, namely analysis of a broad wavelength band. We chose the ISO approach because it is easy to perform, whereas an APHA analysis is very sensitive because the color of the permeate is not very intense. Whole spectra (200-900 nm) were also used. Absorption spectra were analyzed with a Varian Cary 50 UV-visible spectrometer.
RESULTS AND DISCUSSION

OPERATION OF THE PLANT

Depending on the aim of the plant, two control strategies for the first tank can be distinguished:

a) Conventional treatment: The first reactor is operated like a normal primary clarifier. The hydraulic retention time has to be long enough to ensure that particles, which could clog the membrane, can settle. The disadvantage is the formation of a sludge blanket in the primary clarifier, which has to be removed at regular intervals, depending on the storage capacity. The primary sludge is not stabilized and needs further treatment, most likely on a conventional wastewater treatment plant.

b) Advanced treatment: Two differences compared to strategy a) exist. The first reactor is aerated intermittently and small amounts of activated sludge are recycled. By applying this strategy, anaerobic conditions and the formation of malodorous substances should be reduced. As sedimentation is constrained (due to turbulence caused by the aeration), clogging of the membrane needs to be prevented by installing a sieve. The first reactor would have the function of hydrolyzing coarse particulate material, which would be supported by the recycled biomass. With this strategy, a better nutrient removal performance should be obtained, as both denitrification and biological phosphorus removal get more organic substrate than in conventional treatment.

Three phases of operation can be distinguished in the first six months of operation: During the first two months (phase 1), the primary clarifier was operated as a sedimentation tank and adjustments in the aeration system (aeration times) in the membrane chamber were conducted. The permeate and aeration pumps were the source of offensive noise, so tenants requested that the plant be turned off during the night. This was done and an operational break of 7.5 hours (no aeration, no permeate removal) was introduced. After two months the coarse bubble aeration in the first tank started operation. During the five weeks following the second strategy (phase 2), a good effluent quality with improved P- removal was observed, however, due to clogging of the membrane (sieve was not installed yet), and a defective ball valve, we had to return to the conventional treatment (phase 3).

Despite of air-tight reactors and separate ventilation of the plant and the basement, during the second phase odours deriving from the plant could be noticed (due to resuspension of anaerobic primary sludge from the first phase). During the first and third phase, odors were negligible.

Most of the operational problems are put down to minor problems (defective valve). The 3 mm distance between the membrane plates is too low if the primary clarifier does not work perfectly.

NITROGEN REMOVAL

Ammonia: Activated sludge from a conventional wastewater treatment plant was filled into the membrane reactor at its start-up. Complete nitrification was observed within a couple of days. As expected after introduction of the night break without aeration, high ammonium concentrations were observed in the morning (by the influent and biomass decay). In order to eliminate organic matter and ammonium, 20 minutes pre-aeration was introduced before the permeate withdrawal. Model calculations and measurements (Figure A-3) show that the ammonium load in the effluent would be 15-20% higher without pre-aeration.

Total nitrogen: After adjusting the MBR-aeration by reducing the aeration time to only 25%, denitrification increased to 30-40% during the first phase, reaching 60% during the third phase (Table A-2). Based on the oxygen profile in Figure A-3, nitrogen removal could be optimized by reducing fine bubble aeration during permeate withdrawal. Even higher
denitrification rates could be reached by introducing oxygen control, which is only reasonable in larger plants. During the second phase, a better denitrification performance with elimination rates of over 90% was observed. It is not yet clear whether this performance can be reached on a permanent basis. Steady-state conditions did not prevail because the primary sludge blanket from the first phase was resuspended and partly hydrolyzed.

Figure A-3: Typical oxygen and ammonium concentrations (left), water level in the MBR and permeate flux (right) during 24 hours. Aeration runs at a higher capacity during filtration, is switched off during the night 22-5.30h), and runs intermittently in between.

**PHOSPHORUS REMOVAL**

No specific phosphate removal process has yet been implemented. So the only process of this kind is incorporation into the biomass, which implies a removal proportional to sludge production. As the increase of the sludge concentration was very low during the first phase (due to high aeration ratios), in the order of magnitude of 10 g COD PE\(^{-1}\) d\(^{-1}\), a phosphate incorporation of 0.2 g\(P\) PE\(^{-1}\) d\(^{-1}\) can be expected (0.02 g\(P\) gCOD\(^{-1}\), Siegrist and Boller 1999)). However, the observed phosphate removal was 0.7 g\(P\) PE\(^{-1}\) d\(^{-1}\), which cannot only be due to the physiological P-uptake. Calculations made by Maurer and Boller (1999) imply a chemical precipitation of phosphate as hydroxyapatite (Ca\(_5\)(PO\(_4\))\(_3\)OH, HAP), as the dissolved concentrations of calcium (95 mg l\(^{-1}\)) and pH values (7.4 – 7.9) are high. This precipitation process could also be an explanation for the high P content of the sludge (0.03 – 0.06 g\(P\) gCOD\(^{-1}\), average 0.05 g\(P\) gCOD\(^{-1}\)). These questions are currently being investigated further.

During the second phase of operation, phosphate removal rates of 90% were observed (grab samples), which indicates biological phosphate removal. COD values in the influent were high and denitrification was almost complete (temporarily), leading to anaerobic conditions during the night. As already mentioned, the experiment had to be interrupted due to technical problems. Experiments on biological phosphorus removal will be continued after some modifications to the plant scheme. However, even if a phosphate-removing sludge can be cultured, removal of this phosphorus enriched sludge from the system on a regular basis must be ensured.

**SLUDGE PRODUCTION AND HANDLING**

Sludge production is one of the most important parameters for the operation of the plant. The higher the sludge production, the more often does excess sludge have to be removed, which leads to more work for the owner or operator of the plant (e.g. handling and disposal of the filter bags). Sludge production depends on the sludge age. In our case, the sludge production during the first three months was around 30 gTSS d\(^{-1}\), going up to 100 gTSS d\(^{-1}\) during the second phase, resulting in a sludge yield of 0.16 and 0.19 gCOD gCOD\(_{removed}\)^{-1}. Both values correspond
well to the value given by Pollice et al. (2004), who measured a sludge yield of \(0.17 \text{ gCOD gCOD}\text{removed}^{-1}\). Sludge was removed for the first time after four months of operation and on a weekly basis since then, when it has remained at a value of 6-8 kg m\(^{-3}\) (variation due to variation of the water level).

The filter bag is placed in a manhole outside the building. The dewatering properties are good, even though temperatures were very low (down to -10°C), dry matter of 10% was measured. Excess water flows back to the membrane unit.

**Effluent quality**

Besides nitrogen and phosphorus, the total organic carbon (TOC) and chemical oxygen demand (COD) was analyzed. These compounds are degraded to 90% and 95% respectively. The average effluent concentrations of all parameters can be seen in Table A-2. These values can be compared to the effluent of conventional nutrient-removing WWTP, although our wastewater is not diluted by rainwater or extraneous water. The removal rates are also similar to those of larger membrane units.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Phase 1</th>
<th>Phase 2</th>
<th>Phase 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>COD</td>
<td>gCOD m(^{-3})</td>
<td>500 / 30</td>
<td>1200 / 40</td>
<td>500 / 30</td>
</tr>
<tr>
<td>TOC</td>
<td>gC m(^{-3})</td>
<td>130 / 10</td>
<td>300 / 14</td>
<td>120 / 10</td>
</tr>
<tr>
<td>TN</td>
<td>gN m(^{-3})</td>
<td>85 / 65</td>
<td>110 / 12</td>
<td>75 / 33</td>
</tr>
<tr>
<td>NH(_4)-N</td>
<td>gN m(^{-3})</td>
<td>66 / 0.1</td>
<td>85 / 0.1</td>
<td>66 / 3.2</td>
</tr>
<tr>
<td>NO(_3)-N</td>
<td>gN m(^{-3})</td>
<td>0 / 55</td>
<td>0 / 9</td>
<td>0 / 26</td>
</tr>
<tr>
<td>TP / PO(_4)-P</td>
<td>gP m(^{-3})</td>
<td>18 / 13</td>
<td>32 / 12</td>
<td>22 / 16</td>
</tr>
</tbody>
</table>

**Permeate color**

Although organic matter is degraded to a high degree and no suspended solids can be detected in the permeate, it shows a yellowish color (Figure A-4a) which intensified in the course of operation. An activated carbon filter was installed to remove this color, but its operation is not satisfactory. The color is only visible when the water level reaches a few centimeters, which is the case in a toilet, which appears to be dirty. We have been unable to identify the substances that cause this color, but suppose them to be degradation products of the biomass (flavines, polycarboxylates, humic acids). As can be seen from the absorption spectrum (Figure A-4a), a mixture of several substances seems to be responsible for the coloration, as no single peak can be detected. A similar coloration is observed in two MBR pilot plants at EAWAG fed with conventional wastewater. The intensity in our system is much higher, as there is no dilution effect and the permeate is recycled. The color intensity increased with time, since the plant was started with diluted activated sludge (Figure A-4a).

Several strategies for removing the color are possible and first experiments at EAWAG showed the removal capacity of chlorination, ozonation and adsorption on activated carbon and granulated ferric hydroxide. Ferric hydroxide is also used for phosphate removal (Genz et al., 2004). Although the color represents only an aesthetic problem, if we want to replace fresh water with permeate, colored water would be an obstacle to acceptance. Figure A-4b shows the results of preliminary experiments for color removal. All examined processes have been successful in reducing coloration in batch experiments. A scale-up for continuous operation with activated carbon and ozone will be investigated.
Appendix A  139

Absorption of visible light

<table>
<thead>
<tr>
<th>wavelength [nm]</th>
<th>absorbance [b]</th>
</tr>
</thead>
<tbody>
<tr>
<td>400</td>
<td>0.000</td>
</tr>
<tr>
<td>500</td>
<td>0.005</td>
</tr>
<tr>
<td>600</td>
<td>0.010</td>
</tr>
<tr>
<td>700</td>
<td>0.015</td>
</tr>
<tr>
<td>800</td>
<td>0.020</td>
</tr>
</tbody>
</table>

**Figure A-4 a)** Absorption spectrum of the permeate within the first three weeks. The color is intensifying due to household wastewater that is more concentrated than the activated sludge from start-up. On top, apparent color bandwidths. **b)** Absorbance at 436 nm for permeate and after preliminary experiments with granulated activated carbon (GAC), ozone (Oz) and chlorine (Cl)

**RESEARCH TOPICS**

The most important question is: under what conditions is this system (or similar systems) a real alternative to conventional systems. In that context, interest centers on the removal efficiency, energy consumption, operation efforts, nutrient recycling, automatic operation with remote control and economic aspects. However, a whole cluster of research questions needs to be answered:

**Dynamic modeling:** In order to optimize nutrient removal, several control strategies need to be tested. This can be done most easily by dynamic modeling. However, model input information such as influent quantification, flow-scheme characterization, sludge composition and required online measurements (sensor devices) have to be known to set up a good model. An extended bio-P-model including apatite precipitation will be developed.

**Phosphorus removal:** In order to meet the same quality standards as conventional WWTP (which is one of the goals of the project), 80-90% phosphorus elimination must be reached, which means an additional 60-70% P-removal by methods other than P-incorporation. One option is separate urine treatment. However, only around 50% of the phosphate is contained in the urine (Larsen and Gujer, 1996). Enhanced biological phosphate removal may be another option, but this requires either excess sludge to be removed on a regular basis or natural chemical precipitation to hydroxyapatite and finally apatite due to increased P-release in extended anaerobic phases. The third possibility is chemical removal by ferric oxide cartridges or precipitation with iron or aluminum salts, but precipitation would increase sludge production and pollute the stabilized sludge with heavy metals. However, the extra sludge production as well as application and handling of these chemicals are all undesirable.

**Urine reactor:** Operation of a separate reactor for urine treatment makes the plant more complex. The advantage of this reactor is the removal of the phosphate in a reusable form (struvite). Automatic operation conditions for this reactor will be tested before combining it with the MBR.

**Trace pollutants:** We expect significantly lower loads of heavy metals and organic micropollutants (e.g. PAK, pesticides, flame retardants, corrosion inhibitors) than in the wastewater from combined sewer systems that is polluted by atmospheric precipitation, industrial wastewater, agriculture and traffic runoff. Due to the significantly higher sludge age in the membrane reactor, the degradation of organic micropollutants used in the household (e.g. pharmaceuticals, day care products and detergents) is expected to be improved compared with conventional activated sludge treatment (Siegrist, 2003) and will be investigated for
selected pharmaceuticals. We also hope to detect additional removal in the biotope before infiltration by photochemical reactions.

Energy consumption: Energy consumption plays a crucial role in the whole project. Electricity is needed for the coarse-bubble crossflow aeration of the membranes, the fine-bubble aeration and the permeate pump, as well as for the struvite reactor. Conventional WWTPs need around 5-10 kWh PE\(^{-1}\) of external electricity for nutrient and COD removal, including reuse of digester gas (Nowak, 2003). There are no data available on the energy consumption of small-scale membrane bioreactors, the company BUSSE producing small-scale treatment plants specifies a value of 250–300 kWh PE\(^{-1}\) a\(^{-1}\) for their BUSSE-MF-HKA4-system (BUSSE, 2005).

Permeate: A high-quality permeate has to be continuously supplied. Investigations on permeate quality such as hygienic aspects and decolorisation will be investigated in the next few months. Further laboratory tests with different adsorbing materials, oxidation processes and hydraulic retention times will be conducted. The aim is to find an effective method with low costs and requiring little maintenance.

Economic factors: As already mentioned, the costs represent one of the most important factors for the spread of this technology. Costs per capita for small and large wastewater collection and treatment systems in Switzerland are 100 – 450 € PE\(^{-1}\) a\(^{-1}\) and 60 – 180 € PE\(^{-1}\) a\(^{-1}\) respectively (BUWAL, 2003). These costs include the operating costs for the WWTP and the sewer system as well as planning and investment costs for construction and renewal of the infrastructure. Costs for small-scale treatment plants need to be in the same order of magnitude or even lower, which might be possible in the case of mass production. However, these would allow only an annual service with regeneration of the membranes and require highly reliable automated operation combined optimally with remote maintenance.

CONCLUSIONS

The aim of this project is to investigate the practical, environmental, social and economic relevance of a decentralized wastewater treatment system at household level. The first 6 months of operation confirm that the effluent quality of the treatment plant can be compared to a conventional WWTP except for phosphorus. A few operational problems and parameters such as nutrient removal, decolorisation, energy consumption, service and maintenance still need to be investigated in more detail.

From an economical point of view, highly sophisticated decentralized systems are certainly worth installing in rural areas with a sensitive ecology, where long sewer systems are necessary. But they might also be an alternative for regions with water scarcity, as they can decrease water consumption by 30 – 40%. However, an economic optimum for the size of such plants needs to be found. Whether the treatment plant will be installed at household level or in small neighborhoods, indoors or outdoors, depends on several factors and needs to be investigated.

ACKNOWLEDGEMENTS

The present work was funded by the canton of Solothurn, Switzerland, and would not have been possible without the support of Picatech-Huber AG, who built and maintains the treatment plant.
REFERENCES


APPENDIX B

ADDITIONAL PILOT PLANT INFORMATION
Additional pilot plant information

MEASUREMENTS

Table B-1: Grab samples taken weekly. X = every week; * = single measurements

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Primary Clarifier</th>
<th>MBR</th>
<th>Permeate</th>
</tr>
</thead>
<tbody>
<tr>
<td>COD</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>N_{tot}</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>P_{tot}</td>
<td>X</td>
<td>X</td>
<td>*</td>
</tr>
<tr>
<td>NH_{4-N}</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>NO_{2-N}</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>NO_{3-N}</td>
<td>*</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>PO_{4-P}</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>TSS</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TOC</td>
<td>X</td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>BOD</td>
<td></td>
<td></td>
<td>*</td>
</tr>
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</table>

Table B-2: Online measurements (signals, recording intervals)

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<thead>
<tr>
<th>Signal</th>
<th>Unit</th>
<th>Recording interval</th>
<th>Place of measurement</th>
<th>Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oxygen</td>
<td>gO_{2} m^{-3}</td>
<td>1 min^{-1}</td>
<td>MBR</td>
<td>Permanent</td>
</tr>
<tr>
<td>Ammonium</td>
<td>gN m^{-3}</td>
<td>1 min^{-1}</td>
<td>MBR</td>
<td>Until March 2007</td>
</tr>
<tr>
<td>Conductivity</td>
<td>mS cm^{-1}</td>
<td>1/10min</td>
<td>MBR</td>
<td>Until 24.5.2006</td>
</tr>
<tr>
<td>pH</td>
<td>-</td>
<td>1/10min</td>
<td>MBR</td>
<td>Permanent</td>
</tr>
<tr>
<td>ORP</td>
<td>mV</td>
<td>1/10min</td>
<td>MBR</td>
<td>Permanent</td>
</tr>
<tr>
<td>Nitrate</td>
<td>gN m^{-3}</td>
<td>1/10min</td>
<td>MBR</td>
<td>From 24.5.2006</td>
</tr>
<tr>
<td>Water level R2</td>
<td>m</td>
<td>1 min^{-1}</td>
<td>MBR</td>
<td>Permanent</td>
</tr>
<tr>
<td>Water level R1</td>
<td>m</td>
<td>1 min^{-1}</td>
<td>1. reactor</td>
<td>From Nov 2005</td>
</tr>
<tr>
<td>Suction pressure</td>
<td>bar</td>
<td>1 min^{-1}</td>
<td>MBR effluent line</td>
<td>Permanent</td>
</tr>
<tr>
<td>Freshwater consumption</td>
<td>l</td>
<td>1 min^{-1}</td>
<td>Freshwater line</td>
<td>Permanent</td>
</tr>
<tr>
<td>Permeate consumption</td>
<td>l</td>
<td>1 d^{-1}</td>
<td>Permeate line</td>
<td>Permanent</td>
</tr>
<tr>
<td>Rainwater consumption</td>
<td>l</td>
<td>1 d^{-1}</td>
<td>Rainwater line</td>
<td>Permanent</td>
</tr>
<tr>
<td>Permeate pump</td>
<td>-</td>
<td>On/off</td>
<td>Permeate line</td>
<td>Permanent</td>
</tr>
<tr>
<td>MBR effluent flow</td>
<td>l s^{-1}</td>
<td>1 min^{-1}</td>
<td>MBR effluent line</td>
<td>Permanent</td>
</tr>
<tr>
<td>Blower 1</td>
<td>-</td>
<td>On/off</td>
<td>Permeate line</td>
<td>Permanent</td>
</tr>
<tr>
<td>Blower 2</td>
<td>-</td>
<td>On/off</td>
<td>Permeate line</td>
<td>Permanent</td>
</tr>
<tr>
<td>Air lift pump 1-2</td>
<td>-</td>
<td>On/off</td>
<td>Permeate line</td>
<td>From Dec 2005</td>
</tr>
<tr>
<td>Air lift pump 2-1</td>
<td>-</td>
<td>On/off</td>
<td>Permeate line</td>
<td>Permanent</td>
</tr>
<tr>
<td>Excess sludge pump</td>
<td>-</td>
<td>On/off</td>
<td>Permeate line</td>
<td>Permanent</td>
</tr>
<tr>
<td>High-load mode</td>
<td>-</td>
<td>On/off</td>
<td>Permeate line</td>
<td>Permanent</td>
</tr>
</tbody>
</table>

Table B-3: Criteria for allocating water consumption to various uses

<table>
<thead>
<tr>
<th>Consumption</th>
<th>Criteria</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gardening</td>
<td>Q_{perm,i} &gt; Q_{perm,max,winter}</td>
<td>Q_{perm,i} - Q_{perm,mittel,winter}</td>
</tr>
<tr>
<td></td>
<td>Q_{perm,i} &lt;= Q_{perm,max,winter}</td>
<td>0</td>
</tr>
<tr>
<td>Toilet flushing</td>
<td>Q_{perm,i} &gt; Q_{perm,max,winter}</td>
<td>Q_{perm,mittel,winter}</td>
</tr>
<tr>
<td></td>
<td>Q_{perm,i} &lt;= Q_{perm,max,winter}</td>
<td>Q_{perm,i}</td>
</tr>
<tr>
<td>Laundry</td>
<td>Q_{TW}(\Delta T = 5 min) &gt; 30 l</td>
<td>1.25*Q_{Rainwater}</td>
</tr>
<tr>
<td>Shower/Bath</td>
<td>Q_{TW}(0:00 - 2:00h) ≠ 0</td>
<td>Q_{TW}</td>
</tr>
<tr>
<td>Dishwasher</td>
<td>Q_{TW}(0:00 - 2:00) = 0</td>
<td>Q_{TW} (0:00 - 2:00)</td>
</tr>
</tbody>
</table>
Table B-4: Heavy metal contents of several wastewater streams according to literature data

<table>
<thead>
<tr>
<th>Source</th>
<th>Cu (mg Pd^{-1} d^{-1})</th>
<th>Zn (mg Pd^{-1} d^{-1})</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laundry</td>
<td>0.8</td>
<td>4.2</td>
<td>Aonghusa, 2002; Baccini, 1993; Jenkins, 1998; Lampert, 1997</td>
</tr>
<tr>
<td>Hygiene</td>
<td>0.1</td>
<td>1.4</td>
<td>Lampert, 1997</td>
</tr>
<tr>
<td>Food / Toilet</td>
<td>2.4</td>
<td>12.5</td>
<td>Baccini, 1993; Chino, 1991; Ciba-Geigy, 1977; Lampert, 1997; Koch, 2001</td>
</tr>
<tr>
<td>Kitchen / Dishwashing</td>
<td>0.25</td>
<td>0.5</td>
<td>Baccini, 1993; Lampert, 1997</td>
</tr>
<tr>
<td>Cleaning</td>
<td>1</td>
<td>5.5</td>
<td>Baccini, 1993; Lampert, 1997</td>
</tr>
<tr>
<td>Total</td>
<td>4.5</td>
<td>24.1</td>
<td></td>
</tr>
</tbody>
</table>

Table B-5: Cu and Zn measurements, Average values and standard deviation

<table>
<thead>
<tr>
<th>Source</th>
<th>Cu (µg L^{-1})</th>
<th>Zn (µg L^{-1})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freshwater</td>
<td>53 (+/-12)</td>
<td>160 (+/-40)</td>
</tr>
<tr>
<td>Laundry effluent</td>
<td>63 (+/-3)</td>
<td>63 (+/-3)</td>
</tr>
<tr>
<td>Rainwater</td>
<td>80 (+/-30)</td>
<td>66 (+/-60)</td>
</tr>
<tr>
<td>Permeate</td>
<td>70 (+/-70)</td>
<td>250 (+/-90)</td>
</tr>
</tbody>
</table>

Figure B-1: Heavy metal content of the excess sludge. The high values at the beginning are inherited from the activated sludge used for start-up.

REFERENCES


Figure B-2: Aquamin pilot plant in the basement with primary clarifier (1), MBR (2), aggregates (3), control unit (4) and off-gas pipes and ventilator (5).
Figure B-3: MBR from top before start-up of the reactor, with membrane module (1), fine bubble aeration diffuser (2), excess sludge pump (3) and air-lift pump (4). At the beginning, only one membrane module was installed with 3mm plate interspace, later on, it was replaced by two modules with 6mm interspace.
Figure B-4: Urine reactor in the basement. Height ca 1.6 m. Opening for struvite filter bag (1), stirrer engine (2), off-gas pipes (3). The activated carbon filter is not visible.
Figure B-5: Excess sludge collection system with manhole cover (1), filter bag, excess sludge pipe (3). The return pipe for the dripping water is not visible.
**CURRICULUM VITAE**

2004 - 2008  Dissertation at the Federal Institute of Aquatic Science and Technology (Eawag), Dübendorf, and ETH Zürich

2002 - 2004  Full-time teaching assistant at ETH Zürich (Chair Urban Water Management)

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1983 - 1989  Primary school, Speicher AR

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