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The GeoPlanet: Earth and Planetary Sciences Book Series is in part a continuation of Monographic Volumes of Publications of the Institute of Geophysics, Polish Academy of Sciences, the journal published since 1962 (http://pub.igf.edu.pl/index.php).

ISSN 2190-5193 ISSN 2190-5207 (electronic)
GeoPlanet: Earth and Planetary Sciences
ISBN 978-3-319-13793-3 ISBN 978-3-319-13794-0 (eBook)
DOI 10.1007/978-3-319-13794-0

Library of Congress Control Number: 2014956489

Springer Cham Heidelberg New York Dordrecht London


Printed on acid-free paper

Springer International Publishing AG Switzerland is part of Springer Science+Business Media (www.springer.com)

# Preface

Water and wastewater management in urbanized areas has been resolved, although sewage sludge created in the course of sewage treatment causes problems. Against this background rural areas, particularly in areas characterized by dispersed distribution of households suffer from the lack of wastewater treatment systems. The problem is aggravated by the increasing use of water due to rising civilization standards. The problem has grown to a scale that no doubt must be resolved in the near future. The most serious faults caused by untreated wastewater being discharged into the environment is pollution of surface and groundwater, and eutrophication of water bodies even in the touristically attractive regions.

In Europe, a substantial proportion of households in rural areas have the so-called dispersed infrastructure (in Poland 26 % of households are separated from each other by 100 m or more). Construction of a sewerage system in such areas is economically ineffective. Moreover, when constructed the sewerage systems suffer from high operation costs.

Also, collecting sewage in septic tanks is unpractical due to odors, costs, and danger, as on puncturing the surrounding soil is polluted. These are the reasons why on-site systems are gaining in interest. One such method that has been developing in the last four decades is a method based on adapting the natural conditions and treatment processes taking place in marsh ecosystems. Treatment wetlands are engineering facilities that tend to follow these natural conditions but in a more controlled way. Wastewater is treated when flowing through the matrix that consists of soil-like substrate and roots and rhizomes as well as microorganisms. The main treatment processes including adsorption, filtration, ion exchange, biodegradation, take place in the gravel filtration medium, however, they are supported by plants that supply oxygen and uptake some minor part of nitrogen. Thanks to the activity of hydrophytes and their ability for gas transfer and release of oxygen to the root zone various types of bacteria can exist and conduct the treatment processes. The method is attractive also because it fits well into the natural type of landscape. Both wastewater and sewage sludge can be utilized in treatment wetland systems (hydrophyte facilities). These facilities are inexpensive to be constructed and operated. The principles of operation are understandable, in particular to farmers and other inhabitants of rural areas.

Experience gained so far clearly shows that facilities composed of a septic tank and treatment wetland can treat wastewater effectively in the rural areas. However, the development of hydrophyte systems has led to complex facilities enabling efficient removal of not only organic matter and nutrients, but xenobiotics as well. Treatment wetland systems have been applied with success to purposes as distant from the original application as dewatering and stabilization of sewage sludge, treatment of landfill leachate, treatment of reject waters from sewage sludge processing, treatment of surface run-off, treatment of industrial water and wastewater, and others.

In this book, all these applications are described based on the authors' own experience and the literature review. The one subject that is not directly related to treatment is generation of humic-like substances that are produced in the course of treatment of wastewater in treatment wetland systems and traditional plants.

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# Contents

# Abbreviations

| | |
|---|---|
| $A_{2/3}$ | Value of quotient at wave lengths: 260 and 320 mm |
| $A_{4/6}$ | Value of quotient at wave lengths: 465 and 665 mm |
| BOD | Biological Oxygen Demand |
| COD | Chemical Oxygen Demand |
| coli MPN | coli Most Probable Number |
| DM | Dray matter |
| HA | Humic acids |
| HSSF | Horizontal Subsurface Flow |
| HTWs | Hybrid Treatment Wetlands |
| IR | Infra-Red |
| MRR | Mass Removal Rate |
| MSS | Mineral Suspended Solids |
| $MUCT_{system}$ | Modification of University Caption Town System |
| NK | Kjeldahl Nitrogen |
| pe | Person equivalent |
| SFs | Surface Flow systems |
| SFTW | Single Family Treatment Wetland |
| SSF | Subsurface Flow Solids |
| SSFs | Subsurface Flow systems |
| TKN | Total Kjeldahl Nitrogen |
| TN | Total Nitrogen |
| TP | Total Phosphorus |
| TSS | Total Suspended Solids |
| TW | Treatment Wetlands |
| TWSs | Treatment Wetlands systems |
| uv | Ultraviolet |
| vis | Visible light |

| | |
|---|---|
| VSSF | Vertical Subsurface Flow |
| VSSs | Volatile Suspended Solids |
| WWTPs | Wastewater Treatment Plants systems |

# Abstract

The idea of wastewater treatment in artificial and natural wetland systems (TWSs) has been developed for the last 30 years. These systems simulate aquatic habitat conditions of natural marsh ecosystems. In Europe about 10,000 constructed wetland treatment systems (TWTs) exist. In Germany about 3,500 systems are in operation. In other European countries, there are also numerous TWSs in operation, for example in Denmark 200–400, in Great Britain 400–600, and in Poland about 1,000. Most of the existing systems serve as local or individual household treatment systems. TWTs are simple in operation and do not require specialized maintenance. No biological sewage sludge is formed during treatment processes. The TWSs are robust to fluctuations of hydraulic loads. For this reason TWSs are in use mostly in rural areas as well as in urbanized areas with dispersed habitats, where conventional sewer systems and central conventional wastewater treatment plants (WWTPs) are avoided due to high costs. TWSs are usually applied at the second stage of domestic wastewater treatment, after mechanical treatment and/or at the third stage of treatment in order to secure polishing of effluent from conventional biological reactors and renaturalization. New application of TWSs is used for rainwater treatment as well as industrial wastewater and landfill leachate treatment. It is possible due to specific TWSs characteristics that have the potential to remove not only organic matter and nitrogen compounds, but also trace metals and traces of persistent organic pollutants and pathogens.

Based on the gathered practical information, results of new research processes and mechanisms of pollutants removal, and advances in the systems properties and design, TWSs are under continuous development. The aim of this volume is to present an overview of up-to-date knowledge concerning functioning, application, and design of TWSs in order to improve protection of surface water from contamination.

# Chapter 1
# Introduction

In Poland, there is a considerable interest in natural methods of wastewater treatment. The explanation is simple: constructing sewer systems in rural areas is not justified from the economical point of view.

This leads to insufficient treatment (usually only mechanical) or a lack of sewage treatment in villages and small towns, and a lack of methods of pollutant removal from surface runoff.

Rural areas in Poland, with a population of 14.6 million (38 % of the total population), are exposed to the inflow of pollutants from household sewage. Farms are supplied with water from a central water supply system or individual wells. Only 8.2 % of them, however, are equipped with sewer systems. Due to farms being scattered around, central wastewater treatment plants (WWTPs) cannot be a satisfactory solution. Moreover, water consumption per capita in rural areas is substantially smaller than in cities. It usually ranges from 50–100 l/day as compared to 120–150 l/day in cities. Therefore, contaminants in rural wastewaters are more concentrated and more difficult to treat in conventional systems. It is estimated that approximately 25 % of the sewage produced in rural areas in Poland is drained directly to the ground and surface water. In 2014 about 20 % of sewage generated in rural areas were collected, while only 7.0 % were treated before discharging to the recipient.

The problems mentioned above can be solved by treatment wetland (TW) systems. The systems simulate hydraulic and habitat conditions of natural marsh ecosystems. Organic substances, nutrients as well as heavy metals and organic micropollutants are removed in natural processes, supported by heterotrophic microorganisms and hydrophyte plants grown in specially designed soil filters or ponds. It is estimated that over 10,000 systems are in operation all over Europe including some 1,000 systems in Poland.

Treatment wetland systems in Poland are mainly used to provide the secondary treatment of domestic wastewater, after mechanical pre-treatment, and for the protection of surface waters. There are also attempts to use TWs for the treatment of landfill leachate. Due to climatic conditions, subsurface submerged beds (SSF) are



H. Obarska-Pempkowiak et al., *Treatment Wetlands for Environmental Pollution Control*, GeoPlanet: Earth and Planetary Sciences,
DOI 10.1007/978-3-319-13794-0_1

mostly used for wastewater and leachate treatment. For water protection, systems with a surface flow systems (SFs) or with a mixed flow are more often used.

Differences in the operation of treatment wetlands result from physical, chemical and biological conditions, which directly influence transformations in the whole aquatic matrix-plant environment. Depending on the quantity of inflowing organic matter load and the rate of biological processes, pollutants could be removed or/and retained in the system. Thus, the facility should be designed and operated in such a way that the highest possible removal of discharged pollutants is ensured.

The aim of this volume is to present an overview of knowledge concerning the application and functioning of treatment wetland systems for water and wastewater treatment.

# Chapter 2
# Characteristics of the Hydrophytes Method

Hydrophyte wastewater treatment plants are designed on the basis of the systems known as "treatment wetland", introduced in western and North Europe, North America and Australia. These systems simulate aquatic and habitat conditions of natural marsh ecosystems. The term "wetland" refers to the areas where the water level is higher than the ground level for most of the year, which results in soil saturation with water, and causes the growth of characteristic plants species. The hydrophyte method of wastewater treatment is a biological process which proceeds in the presence of various microorganisms, and aquatic and hydrophytes plants grown in specially designed soil filters or ponds. Due to specific conditions enabling the development of hydrophytes, the intensification of alternative oxidation and reduction processes, accompanied with sorption, sedimentation and assimilation processes removing the majority of pollutants from wastewater, can be observed.

Initially, there was some difficulty with accepting the term "wetland", that is why several terms describing water treatment plants were used e.g. hydro-botanic plant, soil-plant, hydrophytes plant, macrophytes plant, reed plant, marsh plant, root plant and others. Thus, it seemed necessary to choose one of the terms or introduce a new one. Taking into account the basic role of hydrophytes in the treatment process, the selected name for that process is "treatment wetland systems" (or treatment plants).

Plants most often used in such types of systems are reed (*Phragmites australis*) and willow (*Salix viminalis*). Reed is used because of its extended system of rhizomes and roots. The stalks and leaves of reed contain an extended porous and gaseous tissue called aerenchyma. Oxygen from the atmosphere goes through that tissue to the underground parts of the plants, where aerobic micro-zones (with $O_2$) around roots and rhizomes are created (Fig. 2.1). Those micro-zones are surrounded by anaerobic micro zones (without $O_2$, however, in the presence of $NO_3^-$). Outside them there are anoxic micro-zones (without both $O_2$ and $NO_3^-$). This results in forming conditions which allow the development of heterotrophic microorganisms taking part in a biochemical transformation of supplied pollutants. Reed is also resistant to frost and extensive summer heat.

Willow is a hydrophite plant which is often used because of its fast growth related to the intense consumption of biogenic compounds. Hydrophites plants do not



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DOI 10.1007/978-3-319-13794-0_2

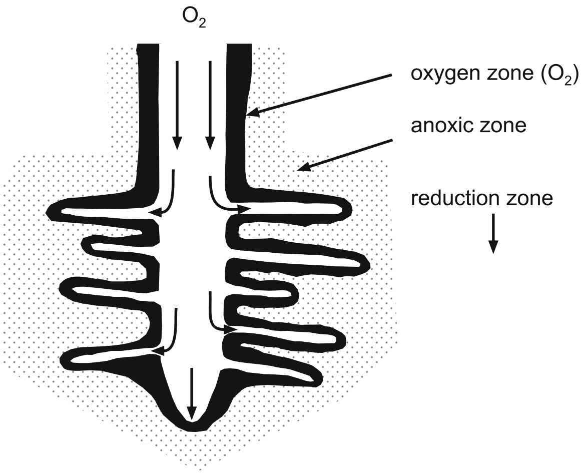


**Fig. 2.1** Redox conditions around rhizomes of hydrophytes (Obarska-Pempkowiak et al. 2010)

transport oxygen to the ground. They grow in the environment of marsh ecosystems. To construct the treatment system for wastewater, the properties of these plants must be taken into account, and conditions for oxygen diffusion must be created.

The main advantages of treatment wetland systems are: simple maintenance, robustness to irregular inflow of wastewater, and lower cost of maintenance in comparison with conventional treatment systems. Moreover, their natural appearance better suits the natural environment. Treatment wetland systems, contrary to traditional biological plants, do not produce secondary sewage sediments and allow the simultaneous removal of biogenic compounds of nitrogen and phosphorus as well as the removal of specific pollutants, for example heavy metals. The main disadvantages of these systems are the following. They need a lot of area, and it takes up to 2–3 years to fully develop the rhizosphere of the plants. Treatment wetland systems are in use mostly in rural areas as well as in urbanized areas with dispersed habitats development, where conventional sewer systems and a central conventional WWTP are avoided because of high costs.

Up till now treatment wetland systems have been used:

1. for removal pollutants from point sources such as domestic wastewater, industrial sewage and landfill leachate,
2. as buffer plant zones for the removal of pollutants from surface runoff,
3. as specially constructed systems for the dewatering and stabilization of sewage sediments.

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# Chapter 3
# Types of Treatment Wetlands

The removal of pollutants in treatment wetland systems is the result of the sorption of biochemical pollutants, redox reactions, and a biological activity of microorganisms as well as hydrophytes plants. Sewage inflowing to a treatment wetland should be pre-treated in order to remove suspension (sand and other mineral and organic solids), and floating (e.g. fats, or substances originated from oil derivatives) substances. Thus, sewage directed to treatment wetlands should be pre-treated in Imhoff tanks, septic tanks or retention ponds. In the case of floating substances, separators of mud and oil, lamellar separators or coalescence separators are used (Fig. 3.1) (Kowalik and Obarska-Pempkowiak 1997).

Small towns with the quantity of sewage below 380 $m^3$/day should be fitted with a simple system of primary treatment, which precedes the treatment wetland system. Most often septic tanks (when the number of users, pe—person equivalent, is below 50), or Imhoff tanks (for settlements above 50 pe) are used.

Primary treatment in Imhoff tanks, septic tanks or separators leads to the production of sewage sludge, which can be discharged to reed beds or to willow plantations for sludge dewatering. If there is no primary treatment, wastewater must be transported to the central treatment system—most often to the municipal or local sewage treatment plants. In Europe, numerous treatment wetlands have screens installed on the wastewater inflow. Solid impurities separated there emit odours, which do not pose a problem when a treatment wetland is located at distance from human habitats. Larger amounts of wastewater require the installation of more complicated primary settling tanks connected to systems ensuring sludge processing (mainly digestion and mechanical dewatering). In the case of industrial wastewater, which contains enormously high loads of pollutants, primary treatment should take place in anaerobic reactors.

Treatment wetlands are usually used for wastewater treatment just after mechanical treatment or after the first stage of treatment, often carried out in conventional WWTPs (Wastewater Treatment Plants).



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DOI 10.1007/978-3-319-13794-0_3

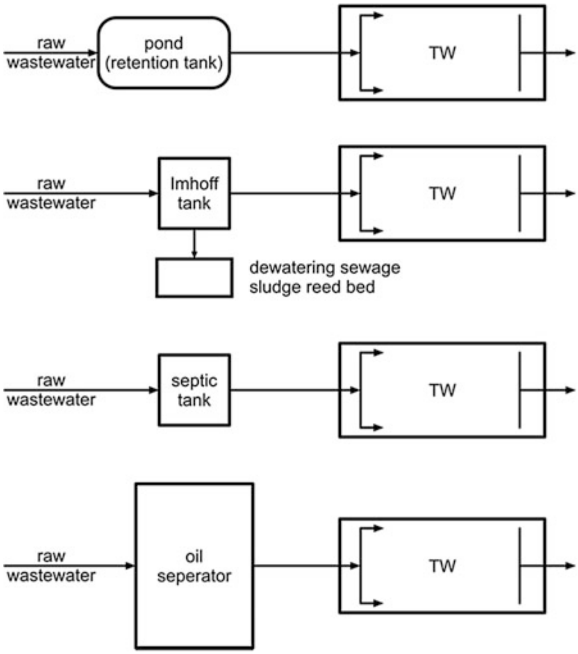


**Fig. 3.1** Types of primary treatment of wastewater inflowing to TWs (Kowalik and Obarska-Pempkowiak 1997)

Treatment wetlands can be built as systems with a surface water flow—free water surface—FWS or a surface flow—SF systems. The second type are systems with a subsurface water flow—vegetated submerged beds—VSB (or a subsurface flow systems—SSF).

In the surface flow system (SFs), the water (wastewater) level is maintained above the ground surface. Plants emerge above the water surface. Water flows above the 30 cm bottom slime. It is recommended that the bottom layer of SFs should have a slope of 0.5 % or less. A gentle slope is required because of maintenance and mosquito elimination. The wastewater flow may be also ensured by the regulation of the outflow level.

Taking into account the way of flow and the predominant treatment processes, SFs are similar to conventional sewage ponds. Because problems with proper flow control arise, SF systems—serpentine ponds or ponds with dykes forcing serpentine flow (Fig. 3.2)—have to be built. In general, such ditches must have sealed slopes and bottoms. The sealing material can be clay or foil. SFs, in temperate climate, are used for treating wastewater inflowing from mechanical, mechanical-chemical or mechanical-biological treatment plants. The system works during the vegetation period.

In SSF systems, the water (wastewater) level is maintained below the ground level, and the flow takes place through the filling material of the bed, which can be composed of gravel, sand or another soil with a high hydraulic conductivity coefficient.

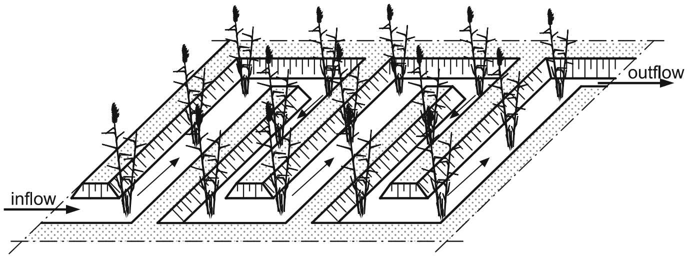


**Fig. 3.2** A typical surface flow system (Obarska-Pempkowiak et al. 2010)

The depth of the bed, depending on both the type of the plants used and on the character of flow (horizontal or vertical), ranges from 0.6 to 1.2 m. The bed is constructed as a layer of soil, which is placed on impermeable subsoil or foil. The bottom slope range from 1 to 3 %. According to Steiner and Watson (1993) the bottom slope of SSF should be 2 %. According to the Darcy's Law, the flow in a nearly horizontal SSF is controlled by the difference between water inflow and outflow levels. In general, the beds with the horizontal subsurface wastewater flow HSSF (horizontal subsurface flow) are recommended. HSSF beds can be periodically submerged to control weed growth, which negatively influences the development of reed (Reed et al. 1998; Cooper and Green 1995). In recent years HSSF beds have been frequently used for wastewater treatment after primary treatment, with unit surface area of at least 5 $m^2$/pe and hydraulic load equal to 40 mm/day (i.e. 40 l/($m^2$·day)). The use of perforated pipes in the bed inflow (the bed surface should be at least 25 $m^2$) ensures the supply of wastewater at a uniform rate. Next the wastewater flows horizontally through the rhizosphere of the treatment wetland, where it is treated in the processes of sorption, filtration and, first of all, by microbiological decomposition. A small amount of nutrient compounds (nitrogen and phosphorus) is periodically assimilated by plants. The plants play an important role in preserving sufficient hydraulic conductivity, enlarging a biological membrane, and stimulating nitrification and denitrification processes.

Wastewater outflows through a gathering ditch, which is filled with broken stone, and through a mechanical device which allows bed submerging highet and outflow regulation. The high level of wastewater in the bed is maintained in summer, whereas a low one—in winter.

The diagram of the submerged SSF system with a horizontal subsurface flow of wastewater is presented in Fig. 3.3.

Further wastewater treatment, after the biological stage (the 3rd stage of the treatment), can be carried out in VSSF beds (vertical subsurface flow treatment wetlands) (Fig. 3.4). In these systems, the treatment process consists of the nitrification of ammonium nitrogen, followed by the denitrification process in the presence of organic matter (transformation of $NO_3^-$ ions into particles of gaseous $N_2$ and $N_2O$, which are released to the atmosphere). In the vertical subsurface flow systems, similarly to horizontal subsurface flow systems, the beds are filled with a filtration

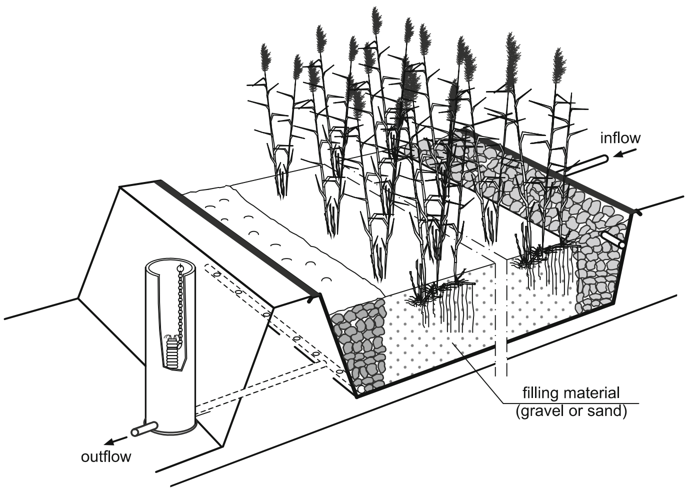


**Fig. 3.3** A typical vegetated submerged system *VSB*, known also as the subsurface flow system *SSF* (Obarska-Pempkowiak et al. 2010)

medium consisting of a few layers with the bottom one laid on impermeable material (clay, foil) (Fig. 3.4). At the bottom of the bed, there is a drainage system, which allows wastewater outflow. Upper ends of the pipes stick out over the bed surface for better ventilation and oxygen access. Additional exhaust pipes are laid out in rows between the wastewater outflow pipes.

The exhaust pipes are perforated only in the bottom part of the bed. Bed compartments should be supplied intermittently with wastewater and left to rest. The surface of a VSSF bed should be at least 5 $m^2$/pe. The vertical subsurface flow systems were introduced for wastewater treatment and they operate during the whole year, also in winter conditions.

Treatment wetlands with beds filled with gravel or coarse, relatively uniform-grained sand, are designed according to EU guidelines, given by Cooper (1990) and Birkedal et al. (1993), whereas the systems with beds filled with fine-grained materials are constructed under Kickuth's licence. In such systems, rhizomes and the roots of macrophyte plants are more important, and biological membrane is less important (Kickuth 1981, 1982).

VSSF beds are characterized by a considerably smaller surface area in comparison to HSSF beds. In several European countries, the suggested surface of vertical subsurface flow beds is determined in different ways. For example in Great Britain, it is assumed that the specific surface of 1 $m^2$/pe allows the removal of organic matter, while the surface of 2 $m^2$/pe ensures effective nitrification (Green and Upton 1995). Under the German ATV regulation, the minimal individual surface of VSSF beds should be 3 $m^2$/pe (ATV A262 1998), while in Austria, according to Haberl et al. (1998), full nitrification demands the surface of VSSF beds equal to 5 $m^2$/pe. For comparison—the most often recommended minimal

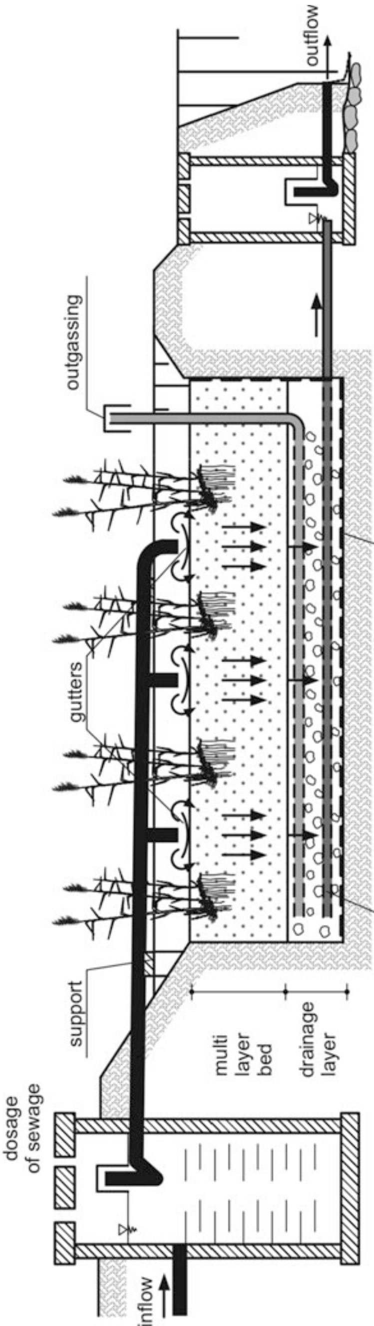


**Fig. 3.4** A typical vertical subsurface flow treatment wetland *VSSF* (Obarska-Pempkowiak et al. 2010)

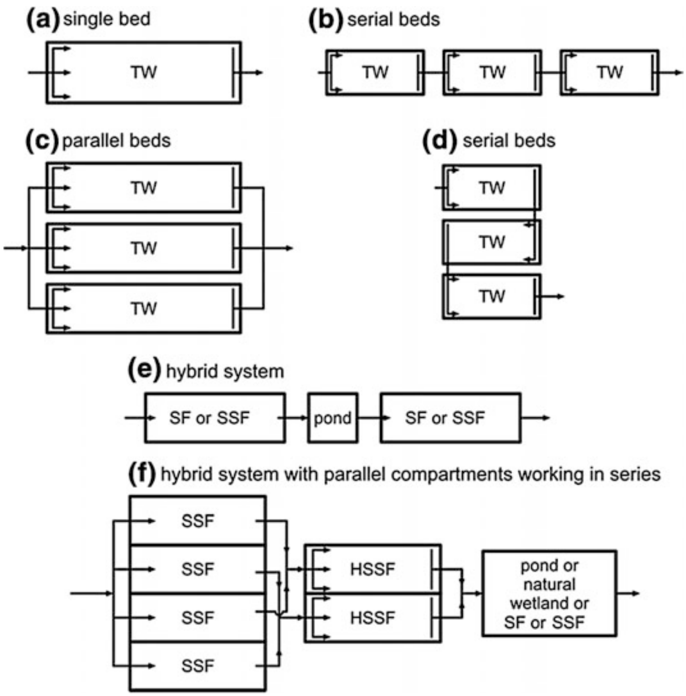


**Fig. 3.5** Examples of bed configuration in TWs, according to Reed et al. (1995)

elementary surface of the HSSF bed is 8 $m^2$/pe (Cooper et al. 1998; Kowalik et al. 1997; Vymazal 1996, 1998).

A treatment wetland may be built as a singular bed, parallel beds, a set of beds (longitudinal or serpentinous). It may also be built as a combination of hydrophyte beds and ponds (Steiner and Watson 1993). The simplest and least expensive solution is a singular rectangular bed (Fig. 3.5a). During the maintenance of singular beds, wastewater is only mechanically treated and discharged through a bypass.

In the case of bigger systems (receiving wastewater of at least 50 pe or 5 $m^3$/day), due to problems with uniform wastewater distribution, a greater number of beds are used. The use of at least two parallel beds allows for maintenance and repairs without shutting down the whole system. While one of the beds is inoperative due to maintenance work, the treatment is continued in the other bed, although the system's effectiveness during maintenance can decrease. The nominal quantity of wastewater inflow is divided equally or proportionally between the beds.

Some beds can be equipped with the surface flow system, and other beds—with a subsurface flow system (SSF). Beds with the subsurface flow system, due to the filling, incur higher investment costs in comparison with surface flow beds, but they can bear higher hydraulic loads.

In practice, sets of beds with a longitudinal or serpentinous flow are used. Joining systems with a surface and subsurface flow in series ensures a stronger effect of treatment due to a greater variety of treatment mechanisms (Fig. 3.5e, f). For example, the removal of suspended solids and $BOD_5$ is more efficient in SF beds, while denitrification runs more intensively in VSB beds. Cascades applied between SF and SSF beds guarantee the aeration of wastewater, which results in the nitrification process being intensified. The last SSF bed can be used to ensure proper conditions for the denitrification process. Aerated wastewater outflow can be recirculated to the first SSF bed in order to extend the duration of nitrification-denitrification processes.

In order to create good conditions for the removal of specific pollutants, phosphate ions and heavy metals, the second stage beds with a submerged flow can have a specific filling material.

In general, treatment wetlands are used for the removal of pollutants from domestic sewage. Depending on pollutant concentration in wastewater inflow to the treatment plant and on terrain configuration, combinations of several different treatment wetlands are used, sometimes coupled with natural ponds or existing marsh ecosystems (Fig. 3.6), and sometimes also with conventional devices (Biernacka and Obarska-Pempkowiak 1996; Osmólska-Mróz 1995). In the systems applied, the pond allows the removal of ammonia with high pH as well as the removal of biogenic compounds. Placing the pond before the hydrophyte ecosystem ensures a uniform wastewater inflow. In comparison with conventional mechanisms, treatment wetlands are used for the removal of nitrogen and phosphorus compounds, e.g. after activated sludge reactors or biofilters.

The use of vertical subsurface flow beds (VSSF) at the beginning of the biological treatment process is considered to be part of primary treatment. It also

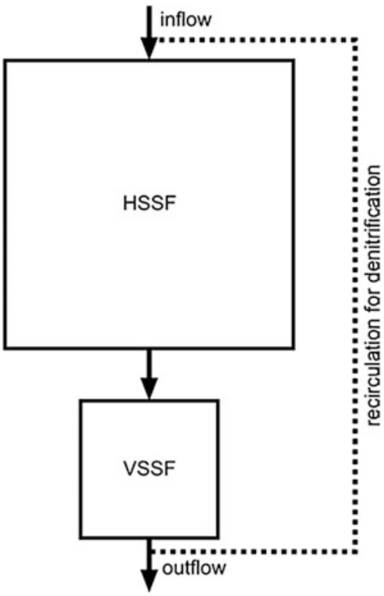


**Fig. 3.6** A hybrid treatment wetland system (HTWs) with a VF bed at the beginning of biological treatment (according to Brix and Johansen 1999)

results in the aeration of inflowing sewage. In HSSF beds, denitrification and organic matter decomposition processes dominate. The result is a significant concentration of ammonia and a negligible ones of $BOD_5$ or COD in the wastewater outflow. Thus, the subsequent stage of biological treatment should take place in a system ensuring proper oxygen conditions for the nitrification process. For that purpose, a natural pond or willow plantations as well as treatment wetlands with a surface flow of wastewater (SF systems) are often used.

At present, there are many examples of HSSF systems with very good operational results if only the removal of $BOD_5$ and suspended solids is required. HSSF beds, working as the third stage of wastewater treatment, are suitable for wastewater nitrification (Cooper et al. 1998). Whereas in HSSF systems working as the second stage of wastewater treatment there are no conditions for the nitrification process, because of limited oxygen supply. This is the reason why in the last ten years an interest in VSSF has increased. The VSSF systems: (a) have a greater ability to transport oxygen, and (b) are considerably smaller than HSSF systems (specific surface is equal to 1–2 $m^2$/pe), when working as the second stage of wastewater treatment. VSSF beds allow the effective nitrification of nitrogen compounds.

Recently it has been proved that the effective removal of pollutants is possible in systems with variable flow beds, so called hybrid systems.

Hybrid treatment wetlands (HTW) can be built as complex systems which consist of two or more HSSF and VSSF beds. HSSF beds ensure the high removal effectiveness of organic matter, suspended solids, and finally they produce favourable conditions for the denitrification process. In VSSF beds there exist good conditions for the nitrification process. Moreover, effective oxygen transport results in the substantial removal effectiveness of $BOD_5$ and COD. According to Cooper et al. (1998), VSSF beds contribute to the effective removal of pathogens including bacteria from wastewater, although due to clogging problems, they are not meant for the removal of suspended solids. The considerable effectiveness of the removal of pollutants in VSSF beds results from additional aeration during the periods following wastewater inflow (pulsatory wastewater inflow). Therefore, it is possible to obtain low $BOD_5$ concentration and full nitrification (Cooper 2004). So far, on the basis of mathematical calculations and the experiences from the operation of the pilot VSSF beds, it has been proved that the pulsatory inflow of wastewater can counteract bed clogging. Batch wastewater inflow causes periods of dryness, which improve treatment efficiency (Kayser et al. 2001).

Currently, the principles regarding types of hybrid systems, which depend on the location of a HSSF or VSSF bed at the beginning of the biological stage of treatment, are being worked out (Cooper and Maeseneer 1996; Birkedal et al. 1993; Brix and Johansen 1999; Brix et al. 2003).

So far there have not been many working hybrid systems and it is difficult to decide which system is better. Some systems such as St Bohaire or Frolois in France (Lienard et al. 1990, 1998) as well as the one in Oklands Park in Great Britain (Cooper and Measeneer 1996) have two-stage VSSF beds, followed by two-stage HSSF beds (Fig. 3.5). Platzer (1996), in Liessen and Merzdorf in Germany used the combination of 10 VSSF beds, followed by 5 or 4 HSSF beds.

The new configurations of hybrid treatment wetlands were introduced by Brix and Johansen (1999). Brix and Johansen (1999) used the following configuration: a HSSF bed followed by the second VSSF bed and recirculation (or alternatively the third HSSF bed—Fig. 3.6).

The unit surface area of the hybrid system is smaller in comparison with single-stage systems. These types of TWs allow the higher effectiveness and more stable removal of nitrogen compounds to be achieved.

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# Chapter 4
# Domestic Wastewater Treatment

## 4.1 Treatment Wetlands Used at the 2nd Stage of Wastewater Treatment

### *4.1.1 SSF Systems*

SSF systems are usually applied at the 2nd stage of domestic wastewater treatment, after mechanical treatment. The number of such installations working at the moment in Europe is estimated to be 100,000. In Germany about 10,000 systems are in operation. In other European countries, there are also a lot of treatment wetlands, for example in Denmark 200–400, in Great Britain 3,000, in Austria about 1,000, in the Czech Republic about 100–400, in Poland above 1,000, in Slovenia about 100 and in Norway—about 100. The majority of European SSF systems were designed for the treatment of domestic wastewater from communities with fewer than 500 inhabitants. However, most of the systems receive sewage from communities with fewer than 50 inhabitants, or even from single households. Only a small part of the systems receive sewage from bigger communities, i.e. with more than 1,000 inhabitants (Paruch et al. 2011; Vymazal 2005; Cooper 1998).

The effectiveness of suspended solids $BOD_5$ and COD removal in HSSF systems with a subsurface water flow was well documented, among others by Cooper and Green (1995), Vymazal (2005), Brix and Johansen (1999), Kowalik and Obarska-Pempkowiak (1997).

Terrain configuration is a common problem that occurs in rural areas and areas with dispersed development, which do not have sewer systems. Those conditions often preclude the installation of local conventional sewage treatment plants, and the achievement of satisfactory effects. Thus, simple, effective, reliable and cost-saving solutions are preferred. Looking for a solution to this problem, in 1986 the Tennessee Valley Authority (TVA) in the United States began advertising treatment wetlands as an alternative to conventional technologies. The systems were used after mechanical treatment, especially in areas with a small number of inhabitants.

 
H. Obarska-Pempkowiak et al., *Treatment Wetlands for Environmental Pollution Control*, GeoPlanet: Earth and Planetary Sciences,
DOI 10.1007/978-3-319-13794-0_4

Treatment wetlands can be used in parishes, schools, campsites, and even in individual households. These systems are effective, simple, not very expensive, with aesthetic qualities. They can look nice in the rural landscape and can also have some educational values. TVA issued clear guidelines about how to design, construct and operate hydrophytes systems (TVA 1991).

The wastewater treatment in SSF systems is possible even in winter. Results of seasonal research for a SSF system supplied with wastewater from septic tanks, which are given by Vymazal (1998), are shown in Table 4.1.

At the beginning of the 1980s, when the first SSF beds were introduced, only one bed was built regardless of the facility size. Nowadays, taking into account problems with the constant inflow of wastewater, this approach has changed. In the case of greater systems (receiving wastewater from 50 or more inhabitants or in quantity above 7.5 $m^3$/day) a greater number of beds is used. The configurations most often applied are presented in Fig. 4.1. Beds are usually rectangular, and the relation between length and width varies from 0.3 to 3 (Treatment Wetlands for Water Pollution Control 2000).

Nowadays beds are usually filled with coarse-grained materials (gravel, small stones), with grain size of 5–32 mm. The choice of bed filling material should be based on hydraulic conductivity and sometimes on the ability of phosphorus compound sorption. In Europe, reed (*Phragmites australis*) is the plant which is most often applied. Other plants used are: reed canary grass (*Phalaris arundinacea*), cattails a (*Typha spp.*) and sweetgrass (*Glyceria maxima*). The enumerated plants are applied individually or together with reeds (Vymazal 1998).

In Great Britain, the analysis of the reed bed operation was performed by Cooper et al. (1998) as well as Cooper and Green (1995). On the basis of the results of organic matter concentration measurements at the inflow and at the outflow, it was found out that the decomposition of organic matter can be described by using first-order reaction constant $K_{BOD5} = 0.1$/day instead of the previous value $K_{BOD5} = 0.19$/day proposed by United Kingdom Reed Bed Treatment Systems Coordinating Groups (Cooper 1990, 1993). It means that elementary surface demand should be 4.6 $m^2$/pe, which allows for obtaining the average value of $BOD_5$ equal to 20 mg $O_2$/l at the outflow (with the average concentration of inflow wastewater $BOD_5$ equal to 200 mg $O_2$/l and with the average flow of 200 l/(pe·day)). According to Cooper (1993), the minimal elementary surface of reed beds must be 5 $m^2$/pe for wastewater which contains $BOD_5$ in the range from 150 to 300 mg $O_2$/l.

In rural areas around Birmingham (Great Britain), serviced by Severn Trend Water Authority, the application of treatment wetlands in the second stage of wastewater treatment was limited to the sites with fewer than 50 inhabitants (Cooper et al. 1998). Till July 1995 three TW systems, with the elementary surface of 5 $m^2$/pe had been built. It was assumed that the average outflow value of $BOD_5$ should be 20 mg $O_2$/l. The treatment wetland in Little Stretton described by Cooper et al. (1998) was constructed of eight beds arranged as terraces. On the basis of the experiences obtained during the pilot system operation, it has been shown that the row arrangement of beds (if there is suitable landscape configuration) is more profitable. The average annual concentrations of characteristic pollutants in the

**Table 4.1** Concentration of pollutants in wastewater after septic tanks and TWs in the Czech Republic according to Vymazal (1998)

| Season | T, °C | $BOD_5$, mg/l | | TSS, mg/l | | TP, mg/l | | TN, mg/l | | Coli MPN, number/100 $cm^3$ | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow |
| Winter | 2 | 251 | 34 | 40 | 9 | 12 | 9 | 86 | 64 | 157,250 | 5,798 |
| Spring | 5 | 242 | 33 | 39 | 8 | 12 | 9 | 80 | 66 | 268,750 | 5,644 |
| Summer | 15 | 176 | 14 | 33 | 9 | 11 | 4 | 71 | 26 | 314,938 | 259 |
| Autumn | 11 | 288 | 17 | 41 | 6 | 12 | 5 | 81 | 35 | 209,438 | 2,632 |
| Yearly | 8 | 242 | 25 | 38 | 8 | 12 | 7 | 80 | 48 | 237,594 | 3,583 |

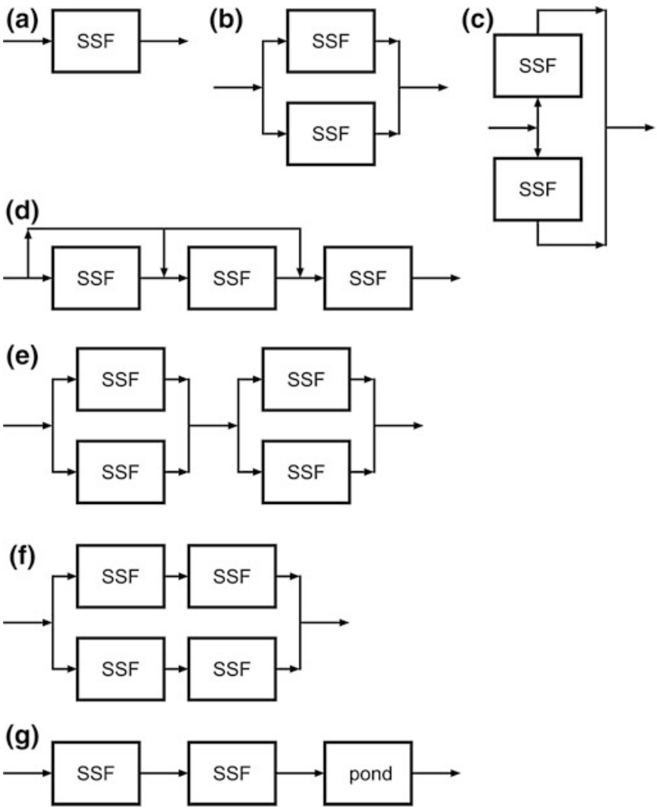


**Fig. 4.1** A typical configuration of a SSF bed: **a** single bed TW; **b**, **c** parallel beds; **d** series of beds with a bypass; **e** two beds in series; **f** two series of parallel beds; **g** beds followed by a polishing pond (Treatment Wetlands for Water Pollution Control 2000)

**Table 4.2** Annual mean concentrations of selected pollutants in the inflow and outflow in Little Stretton, mg/l (Cooper et al. 1998)

| Year | $BOD_5$ | | TSS | | TN | | TP | |
|---|---|---|---|---|---|---|---|---|
| | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow |
| 1987 | 147.0 | 29.0 | 132.0 | 19.0 | 10.0 | 10.0 | 15.0 | 1.0 |
| 1990 | 112.0 | 3.9 | 93.0 | 28.0 | 24.8 | 12.1 | 2.2 | 6.2 |
| 1994 | 58.0 | 2.4 | 62.0 | 16.0 | 15.8 | 0.8 | 4.9 | 8.8 |

wastewater inflow and outflow from reed beds in Little Stretton are presented in Table 4.2.

An operational analysis of the treatment wetland system installed in Wetwood, county Staffordshire, was also carried out (Cooper et al. 1998; Cooper and Green 1995). This system was built on a slope, three beds (5 × 14 m) where installed on terraces. A device for water level control was located at the end of each bed.

Wastewater inflow through a simple pipe that in the inflow zone was replaced by a 0.5 m long distribution layer filled with big stones. The outlet zone was similar—big stones covering a perforated pipe at the end of each bed. The average annual concentrations of characteristic pollutants in the wastewater inflow and outflow of the reed beds in Wetwood are given in Table 4.3.

All the facilities were built by Severn Trend Water Authority. Although pollutant concentrations in purified wastewater were according to the requirements, the deterioration of the purification effects of the reed beds in the facility in Langar (Cooper et al. 1998) was observed. It was a result of soil clogging. Now the majority of filters built in Great Britain are filled with gravel.

According to Börner et al. (1998), treatment wetland systems in Germany have been used for wastewater treatment for over a 40 years. Experiences gained in the last decades have allowed for specifying precise guidelines ATV A262 (1998) for the designing and operation of treatment wetland facilities. Börner et al. (1998), estimated that in rural areas there are a few thousands of treatment wetlands. In Lower Saxony, about 3,000 facilities were located. In Bayern, 150 treatment wetland systems were operating, and another 1,000 were under construction. The majority of the existing facilities were built as subsurface flow beds with a horizontal flow of wastewater, and they are used for the treatment of wastewater from 5 to 1,000 pe. Depending on the role of hydrophytes and on the ability of pollutant removal, the following systems are distinguished: the HSSF beds built according to guidelines provided by Kickuth, the so-called "Kickuth's systems" (Kickuth 1981), and scripus-reed systems, built according to guidelines given by Käthe Seidel, the so-called "Seidel systems" (Seidel 1965). Since 1965, that is, since the building of the first Seidel treatment wetland system in Germany, an analysis of the operating treatment wetlands has been carried out. The analyzed systems have different bed configurations and different variants of primary wastewater treatment.

Initially, coarse-grained materials (ex. coarse-grained gravel) were used as bed filling. Operational experiences proved, however, that there was a possibility of using fine-grained materials, for example sand, which ensured better conditions for the development of microorganisms which decomposed organic mater. The research results proved that in sand, base microorganisms remained active even during winter months (Börner et al. 1998). On the basis of the acquired data, an analysis of 107 treatment wetlands functioning in Germany was made. The results of the analysis proved that 24 VSSF were more effective in pollutant removal in comparison with the remaining 83 HSSF facilities (Börner et al. 1998; Kayser et al. 2001). The average values of characteristic pollutant concentrations in treated wastewater in the VSSF and HSSF are given in Table 4.4.

The concentration of COD and ammonia nitrogen in the VSSF beds equalled 68.2 mg $O_2$/l and 9.5 mg/l respectively, and was lower in comparison with the values received for the HSSF beds—102.5 mg $O_2$/l and 36.0 mg/l. The total nitrogen in wastewater in the VSSF beds was insignificantly higher than in the HSSF beds and equalled 67.1 mg/l and 52.1 m/l respectively. As to nitrate(V), its concentration in wastewater after the VSSF bed was nine times higher in comparison to nitrate(V)

**Table 4.3** Annual mean concentrations of characteristic pollutants in the inflow and outflow in Wetwood, mg/l (Cooper and de Maeseneer 1996)

| Year | COD | | $BOD_5$ | | TSS | | $NH_4^+$-N | | TN | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow |
| 1993 | 146.0 | 52.0 | 53.0 | 6.0 | 44.0 | 6.6 | 12.5 | 8.9 | 3.6 | 1.9 |
| 1994 | 185.0 | 46.0 | 77.0 | 3.7 | 52.0 | 7.2 | 17.2 | 9.5 | 3.7 | 2.1 |
| 1995 | 222.0 | 35.0 | 92.0 | 2.7 | 48.0 | 5.0 | 27.1 | 13.9 | 4.7 | 1.8 |

**Table 4.4** Average concentrations of characteristic pollutants in treated wastewater in the VFTW and HFTW in Germany, mg/l (Börner et al. 1998)

| Parameter | HSSF | VSSF |
|---|---|---|
| COD | 102.5 | 68.2 |
| $NH_4^+$-N | 36.0 | 9.5 |
| $NO_3^-$-N | 7.3 | 65.2 |
| TN | 52.1 | 67.1 |
| TP | 5.0 | 3.2 |

concentration in wastewater after the HSSF bed, and equalled 65.2 and 7.3 mg/l respectively (Börner et al. 1998).

In Lower Saxony, according to the regulations, the following project was developed: "Experiences Concerning Safety and the Effectiveness of Treatment in Small WWTPs with Special Consideration Given to the Use of Treatment Wetlands" (Hagendorf 1996; Von Feld et al. 1996). The average unit surface of the beds used was equal to 6 $m^2$/pe. Research results confirmed that the concentrations of nutrients (nitrogen and phosphorus) at the outlet of the VSSF bed were lower than at the outlet of the HSSF bed. Because of that, all new treatment wetlands were built as beds with a vertical flow of wastewater (Arbeitsblatt ATV A262 1998; Fehr 1998). In Lower Saxony about 90 % of the population are connected to a sewage system, and use the municipal WWTP, whereas only 700 thousand inhabitants use treatment wetland systems for wastewater treatment. In 46 out of 47 private HSSF facilities, wastewater meets German regulations. The monitored facilities removed pollutants from domestic wastewater with high efficiency. The results obtained from seventeen HSSF beds were comparable with the results given by Institut für Siedlungswasserwirtschaft und Abfalltechnik der Universität Hannover (ISAH) (von Felde et al. 1996). Concentrations of nutrients (nitrogen and phosphorus) at the inlet and outlet varied. The concentration of COD at the inlet ranged from 120 to 900 mg $O_2$/l, and ammonia nitrogen ranged from 15 to 300 mg/l. The average values of COD and $NH_4^+$-N at the outlet were 85 mg $O_2$/l and 40 mg/l, and the average concentrations of nitrates(V) and phosphates were 5 and 2.2 mg/l respectively. The average removal efficiency of both COD and phosphates ($PO_4^{3-}$) was about 75 %, whereas that of nitrogen $NH_4^+$-N was about 61 %. Monitoring was carried out in facilities with the unit surface of 5 $m^2$/pe. The reason for the differences in pollutant removal efficiency in the analyzed systems was the daily fluctuation of discharged pollutant load.

In Austria, like in Great Britain and Germany, the majority of rural areas are equipped with a sewage system. In the case of dispersed development areas, building sewage systems connected to the central system is not justified. Treatment wetland systems are the optimal solution for these areas, because they are more effective and require less investment cost in comparison to the central sewage system.

On the basis of the results obtained in Austria and the experiences gained in other countries, Austrian guidelines ÖNORM B 2505 (1995) for designing and building treatment wetland facilities were specified (Haberl et al. 1998).

**Table 4.5** Average concentrations of characteristic pollutants in wastewater treated in TWs in Austria, mg/l, according to Haberl et al. (1998)

| Parameter | VSSF | HSSF |
|---|---|---|
| $BOD_5$ | 7.0 | 15.0 |
| COD | 37.0 | 49.0 |
| $NH_4^+$-N | 7.5 | 15.4 |
| $NO_3^-$-N | 35.0 | 8.0 |

According to Haberl et al. (1998), 23 treatment wetland systems in Austria obtained operating licenses. Out of the systems in operation, 57 % were single-stage, 29 % two-stage and 14 % multistage. The intermittent loading was present in 75 % of the analyzed facilities, whereas in the remaining 25 %, wastewater was inflowing continuously. Wastewater supply in 55 % of the facilities was gravitational. The average unit of vegetative surface was 7 $m^2$ for facilities up to 50 pe and 4.2 $m^2$ for facilities from 50 to 500 pe (all the systems were planted with reed). The majority of TWs were monitored once or more times during a year. The monitoring was commissioned by the government. The obtained measurement results are given in Table 4.5 The VSSF systems showed a higher effectiveness of characteristic pollutant removal in comparison with the HSSF facilities.

The removal of characteristic pollutants in the analyzed treatment wetland systems in Austria was satisfactory. The effectiveness of COD removal from domestic wastewater was about 90 %, whereas the removal of organic matter susceptible to biochemical decomposition ($BOD_5$) and $NH_4^+$-N was about 98 %, $C_{org}$ removal was about 86 % (Scheizer 1998).

In Spain, 39 treatment wetlands are HSSF beds, which constitute 80 % of the total number of systems built in the last 5 years. According to Puigagut et al. (2007), loads of organic matter discharged to HSSF beds were 0.8–23.0 g $BOD_5$/($m^2$·day), and to VSSF beds 12.8–29.8 g $BOD_5$/($m^2$·day). Applied loads were usually higher than loads discharged to other beds in Europe. In spite of high loads, the average effectiveness of pollutant removal, which was observed, was higher and equalled 80 % for HSSF beds and 95 % for VSSF beds.

The systems described above were not effective if the removal of biogenic compounds was taken into account. The average removal efficiency of total nitrogen was 50 %, of ammonia nitrogen 40 %, and of phosphorus 40 %. The obtained effectiveness was comparable to the effectiveness of other facilities working in Europe.

In Norway, treatment wetland systems have been used for wastewater treatment only since the 1990s. Beds are planted with reed and used for the treatment of wastewater in quantities from 2 to 130 $m^3$/day (Mæhlum and Jenssen 1999). Both parallel and multi-stage beds with different hydraulic loads have been analyzed. Granulate called LWA (Light Weight Aggregate, so-called Leca), sand with a high amount gravel were used as filling. The applied materials had high permeability.

Using other countries' experiences and taking into account climatic conditions, a singular deposit with the so-called mixed flow of wastewater: horizontal and vertical, has been designed (Fig. 4.2). Wastewater, after mechanical treatment, flows

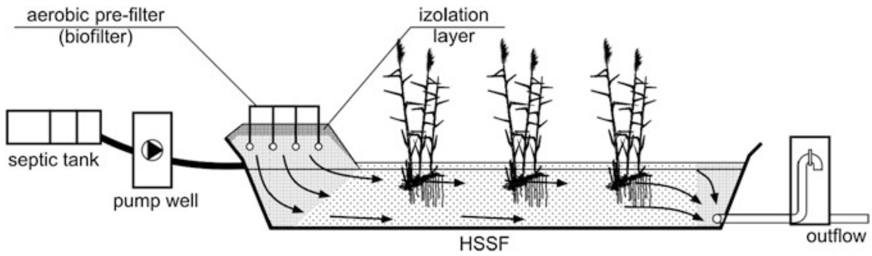


**Fig. 4.2** A schema of single blocked bed, used in Norway (Mæhlum and Jenssen 1999)

periodically, dosed by a pump, through the distribution system into the first part of the bed with a vertical flow. Next, by means of gravitation, wastewater flows into the HSSF system connected with this bed. VSSF beds are usually filled with Leca, and they are located in casing, which provides protection against freezing. HSSF beds' depth is higher, some 0.9 m, than the depth recommended by Cooper et al. (1998), or Johansen and Brix (1996), which was 0.6 m. The higher depth of HSSF beds, according to Mæhlum and Jenssen (1999), prevents the system from freezing, and helps to keep wastewater in the bed for a longer period of time in winter conditions.

The monitoring of treatment wetlands was performed to examine the possibility of the system operation in cold climate conditions, and to verify the freezing susceptibility and removal capacity of biogenic compounds in order to satisfy Norwegian demands. The obtained results of the characteristic pollutant efficiency removal in the treatment wetlands investigated were analysed by Mæhlum and Jenssen (1999) (Table 4.6). The analyzed facilities were characterized by a high effectiveness of pollutant removal: for $BOD_7$ from 67.1 to 90.0 %, for COD from 41.2 to 88.1 %, for TN from 55.2 to 80.3 %, and for TP from 26.0 to 98.3 %. On the basis of the obtained results, it was concluded that HSSF beds with wastewater retention time from 15 to 30 days were characterized by a higher effectiveness of pollutant removal than the beds with shorter retention time—from 12 to 18 days.

VSSF beds, at the first stage of the biological treatment, increased the removal effectiveness of organic matter and total nitrogen. The system working without a VSSF bed was characterized by the 22.9 % removal effectiveness of the total nitrogen, whereas the systems in Haugstein with an additional, separate VSSF bed and a system with compact beds showed a higher ability of the total nitrogen removal—64.0 and 55.0 % (Table 4.6).

The research showed that vertical beds with intermittent loadings, working at the first stage of treatment, had a high removal effectiveness of BOD and COD (over 70 %) as well as of ammonia nitrogen (from 20 to 70 %). The removal efficiency depends on: supplied loads of pollutants, the bed filling material as well as hydraulic regime, (quantities and frequency of loads). On the basis of the carried out research it was proved that the use of VSSF beds was necessary for ensuring the required degree of nitrogen removal. According to Mæhlum and Jenssen (1999), in cold Norwegian climate, beds with vertical subsurface flow should be cased and

**Table 4.6** Removal efficiency of characteristic pollutants in the monitored wetland systems in Norway, (%) according to Mæhlum and Jenssen (1999)

| Object | Type of wastewater | Flow, $m^3$/day | Location and presence of VSSF type bed | Removal efficiency of pollutants (%) | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | $BOD_7$ | COD | TN | TP | TSS | $NH_4^+$-N |
| Haugstein | Domestic | 2.0 | Separately | 90.0 | 82.2 | 64.0 | 98.1 | 79.0 | 71.4 |
| Teter I | Domestic | 2.0 | Blocked | 84.1 | 69.2 | 55.0 | 97.0 | 76.1 | 56.3 |
| Østegård | Domestic | 2.0 | Separately | 90.1 | 41.1 | 79.1 | 93.3 | 30.9 | 80.0 |
| Fagernes | Domestic | 2.3 | Separately | – | 88.1 | 60.2 | 98.0 | – | 62.1 |
| Stange | Domestic | 2.5 | No | 67.2 | 91.0 | – | 74.0 | – | – |
| Teter II | Grey | 1.0 | No | 73.0 | 76.0 | 22.9 | 26.9 | 61.0 | – |
| Jølle | Grey | 0.3 | Separately | – | 97.0 | 8.5 | 91.2 | 96.0 | – |
| Bølstad | Landfill leachate | 2.0 | Aeration lagoon | – | 20.2 | 37.2 | – | 60.3 | 20.7 |
| Esval | Landfill leachate | 130 | Aeration lagoon instead VSSF bed | 35.3 | 4.1 | 11.7 | – | 40.2 | 13.9 |
| Lilleng | Domestic | 15 | Blocked | – | – | 53.0 | 98.0 | – | 80.0 |
| Brømølla | Domestic | 6.0 | Blocked | 98.1 | – | 47.1 | 99.0 | – | – |

covered. Therefore, creating HSSF beds blocked with VSSF beds was suggested. It was also proved that the depth of VSSF beds with gravellier filling should be from 30 to 50 cm, while their hydraulic load should be maintained between 10 and 20 cm/day, with a single dose smaller than 5 mm.

Since 1981 over 75 treatment wetlands have been built in Switzerland, in rural areas with low population density. In general, those systems are small facilities. Billeter et al. (1998), monitored 49 of the systems. Primary treatment in 29 TWs took place in septic tanks, in 9 TWs supplied with domestic wastewater from 3 to 5 pe in Imhoff tanks, while in the remaining facilities it took place in sedimentation ponds. In two thirds of the monitored TWs, biological treatment was carried out in VSSF beds. In the remaining one third, horizontal subsurface flow beds were used while two TW systems had mixed flow beds. Only two out of all the investigated facilities which treated wastewater were bigger than 100 pe, whereas the others were smaller than 20 pe. The entire surface of these systems did not exceed 50 $m^2$ (Billeter et al. 1998). The unit surface of the analyzed beds ranged from 2 to 10 $m^2$/pe (most often from 3.5 to 5.5 $m^2$/pe), and the depth ranged from 0.5 to 1.5 m. The facilities were hydraulically loaded in quantities from 0.156 to 8.0 cm/day. According to legal regulations, individual treatment wetland systems in Switzerland must be under the supervision of local authorities. Therefore, samples of wastewater from TW systems were examined one to four times a year. The concentration of organic matter at the outflows of most of the analyzed systems did not exceed the admissible values of $BOD_5$, equal to 20 mg $O_2$/l in Switzerland (Billeter et al. 1998). In the facilities with a mixed flow of wastewater, the concentration of $BOD_5$ in treated wastewater was below 10 mg $O_2$/l. The removal effectiveness of COD ranged from 78.1 % in the single-stage TWs to 96.8 % in the hybrid treatment wetland systems. The effectiveness of ammonia nitrogen removal ranged from 13.0 to 96.0 % in the hybrid systems.

So far the analyzed HSSF were characterized by a high removal effectiveness of suspended solids, which was over 90 %. Similarly, the removal effectiveness of organic matter was high, it ranged from 71.5 to 94.1 % for $BOD_5$, and from 59.7 to 89.0 % for COD. However, the described treatment wetland systems showed significant differences in the removal effectiveness of nitrogen compounds, from 20 to 70 % (Cooper et al. 1998; Kowalik et al. 1995; Platzer 1996; Kadlec and Knight 1996). Average concentrations of pollutants in the inflow and outflow of HFTW systems in Europe and in the Czech Republic, analysed by Brix (1994, 1998), Schierup et al. (1990) and Vymazal (1998), are presented in Tables 4.7 and 4.8.

It was proved that in the beds with a horizontal subsurface flow of wastewater, there was an insufficient quantity of oxygen to secure the appropriate environment for an effective nitrification process. The beds with a vertical subsurface flow were considerably better aerated (by means of periodic wastewater discharge in particular), and show considerable nitrifying possibilities (Gajewska et al. 2004; Gajewska and Obarska-Pempkowiak 2009; Cooper et al. 1998; Platzer 1995). The conditions prevailing in VSSF beds can be compared to the conditions in unsaturated soil (in the aeration zone), where spaces between the filling material are only periodically filled with treatment wetland wastewater.

**Table 4.7** Average concentrations of pollutants in the inflow and outflow of TWs in Europe, according to Brix (1994, 1998), Coombes (1990) and Schierup et al. (1990)

| Parameter | Inflow | | | Outflow | |
|---|---|---|---|---|---|
| | n[a] | Average | δ[b] | Average | δ[b] |
| *Concentration*, mg/l | | | | | |
| TSS | 77 | 98.6 | 81.6 | 13.6 | 11.1 |
| $BOD_5$ | 80 | 97.0 | 81.0 | 13.1 | 12.6 |
| TN | 73 | 28.5 | 14.7 | 18.0 | 10.7 |
| TP | 67 | 8.6 | 4.5 | 6.3 | 3.5 |
| *Loading*, g/($m^2$·day) | | | | | |
| TSS | 51 | 5.22 | 6.37 | 1.06 | 1.50 |
| $BOD_5$ | 66 | 4.80 | 5.97 | 0.89 | 1.34 |
| TN | 57 | 1.15 | 0.79 | 0.78 | 0.77 |
| TP | 50 | 0.33 | 0.27 | 0.26 | 0.26 |

[a] *n* Number of TWs analysed
[b] *δ* Relative standard deviation

**Table 4.8** Average concentrations of pollutants in the inflow and outflow of HSSF in the Czech Republic (Vymazal 1998)

| Parameter | Inflow | | | Outflow | |
|---|---|---|---|---|---|
| | n[a] | Average | δ[b] | Average | δ[b] |
| *Concentration*, mg/l | | | | | |
| TSS | 37 | 71.9 | 47.2 | 10.8 | 7.1 |
| $BOD_5$ | 39 | 87.4 | 65.7 | 11.9 | 11.4 |
| TN | 26 | 46.1 | 18.5 | 27.6 | 9.7 |
| TP | 27 | 6.4 | 3.8 | 3.1 | 2.1 |
| *Loading*, g/($m^2$·day) | | | | | |
| TSS | 31 | 3.34 | 3.11 | 0.44 | 0.42 |
| $BOD_5$ | 35 | 3.36 | 2.86 | 0.53 | 0.67 |
| TN | 26 | 1.39 | 0.91 | 0.80 | 0.16 |
| TP | 24 | 0.30 | 0.18 | 0.18 | 0.16 |

[a] *n* Number of the TWs analysed
[b] *δ* Relative standard deviation

According to Brix and Johansen (1999), the supplied oxygen quantity in beds with a vertical subsurface flow of wastewater that are periodically irrigated is several times higher than in beds with a horizontal subsurface flow of wastewater. Additionally, during the "rest" period, when sediments are not irrigated by wastewater, oxygen diffusion into soil is 10,000 times faster than in the system in which soil is saturated with wastewater (Brix 1993; Kowalik 2001). Beds with a vertical subsurface flow of wastewater are able to ensure suitable conditions for an effective realization of the nitrification process, which was confirmed by Cooper et al. (1998), Green et al. (1996), Haberl et al. (1998), Laber et al. (1997), Platzer (1995, 1998), Obarska-Pempkowiak et al. (2003) and Gajewska and Obarska-Pempkowiak (2009).

Cooper et al. (1998), indicate that also VSSF beds contribute to the more effective removal of bacteria from wastewater, although they are not designed for the removal of suspended solids because of the possibility of clogging. The significant effectiveness of pollutant removal in VSSF beds is a result of additional aeration during breaks between wastewater inflows. According to Platzer and Mauch (1996), the effective flow of air, the so-called “bed aeration”, is possible only when the upper layers of the bed have good hydraulic properties, and when the bed surface is well drained between doses of wastewater flowing into the bed. Problems related to the clogging of beds with a vertical flow of wastewater were described by Börner et al. (1998) and Platzer and Mauch (1996). According to Börner et al. (1998) and Kunst and Kayser (2000), the removal effectiveness of pollutants in a clogged bed decreases by 35 % for COD and by about 76.2 % for $NH_4^+$-N, when compared to the initial values. The concentration of the inorganic nitrogen compounds (mainly $NO_3^-$-N) decreases over 70 times. The results of the research carried out by Platzer and Mauch (1996) proved that beds with a low hydraulic load and high concentration of organic matter are more sensitive to clogging. According to Platzer and Mauch (1996), the maximum admissible load of organic matter which can be discharged into a VSSF bed in climatic conditions typical of central Europe is 25 g COD/($m^2 \cdot$day).

The VSSF beds which were used have a significantly smaller surface area in comparison to the surface of HSSF beds. The required surface of beds with a vertical subsurface flow of wastewater is determined in a different way by each European country. For example in Great Britain, it is assumed that the specific surface of 1 $m^2$/pe enables the removal of organic matter while, the surface of 2 $m^2$/pe assures effective nitrification (Cooper and Green 1995). According to German guidelines ATV A 262 (1998, the minimal unit surface of a VSSF bed must be 3 $m^2$/pe, whereas in Austria, according to Haberl et al. (1998), the full nitrification process requires the unit surface of a VSSF bed that is equal to 5 $m^2$/pe. For comparison, the most often used minimal unit surface of a HSSF bed is 8 $m^2$/pe (Cooper et al. 1998; Kowalik et al. 1997; Vymazal 1999).

In Poland, the first treatment wetland systems were constructed at the end of the 1980s. In the 1990s several dozens of such facilities were built. The size of those systems varied from 5 pe (individual household systems) to 2,000 pe (wastewater plants for entire villages) (Kalisz and Sałbut 1993a, b, 1995; Kowalik et al. 1997; Kowalik and Obarska-Pempkowiak 1998; Obarska-Pempkowiak and Kowalik 1998). Initially, treatment wetland systems were designed only on the basis of designers’ intuition and did not correspond to the guidelines valid e.g. in Great Britain, Germany and United States (Birkedal et al. 1993; Cooper 1990, 1993; Knight et al. 1993; ATV Arbeitsblatt A 262 1998).

Sadecka (2001), and Sadecka and Kempa (1997), taking into account the results of the 8 years’ research into the removal effectiveness of pollutants in the HSSF system built at the beginning of the 1990s near Gorzów Wielkopolski, proved that the removal of phosphorus compounds was not satisfactory. The obtained results proved a periodic increase of phosphorus concentration in treated wastewater in comparison to its concentration in the TWs inflow.

In Lesznowola near Warsaw, in Bolimowo and Marianowo near Skierniewice, treatment wetland systems designed by ESOS (The Ecological Systems of Cleaning

Sewage), under Prof. R. Kickuth's license for designing soil-root treatment wetlands, were built in 1993. These systems with capacity ranging from a few to 450 $m^3$/day were applied for domestic wastewater treatment. The description of the facilities and first results were given by Błażejewski et al. (1996). In Poland, there are few Kickuth's systems. Due to low effectiveness proved by monitoring results, further implementations of this technology were stopped.

In Poland, 4 out of 11 treatment wetlands (TWs) built for individual farms were located near Lublin, 3 were near Ostrołęka and another 4 were near Ciechanów in the Mazowsze Region. The systems located near Lublin and Ostrołęka were constructed under the UNEP WHO and Polish Ministry of Environmental Protection, Natural Resources and Forestry programme "Sanitation of Rural Areas and Proper Agricultural Practices" (Obarska-Pempkowiak et al. 1997; Obarska-Pempkowiak and Gajewska 2005). Averaged samples of sewage before and after the treatment in TWs were taken once or twice a month for 3 years. The sewage quality was assessed by monitoring physical and chemical parameters, such as the temperature of sewage and air, total suspended solids, $BOD_5$, COD, ammonium nitrogen ($NH_4^+$), nitrate(V) and nitrate(III), organic nitrogen, total phosphorus, and phosphates. Total nitrogen concentration was estimated as a sum of the nitrogen components in all the analysed forms. In all the samples, measurements were performed according to Standard Methods.

The systems near Ciechanów were designed and implemented by the Institute of Building, Mechanisation and Electrification of Agriculture in Warsaw. The major characteristics of the beds in these systems were as follows: (i) the area of the bed was based on a per capita loading rate of 4.5 $m^2$/pe, which means that the specific surface loading of a bed was approximately 29 mm/day, (ii) the length of the bed L = 20 m at all plants, (iii) the width W of the beds was variable, depending on the number of persons, e.g. W = 1.0, 1.1, 1.3, and 1.5 m for 4, 5, 6 and 8 pe, respectively, (iv) the average depth of individual systems was 1 m, and (v) the slope of the bed bottom was 1 %.

The filter systems located near Ostrołęka and Lublin, numbered from 1 to 7, were filled with medium grain sand, whereas systems I, II and IV located near Ciechanów were filled with a mix of gravel (grain size 0.5–8 mm) and artificial aggregate "Pollytag" (grain size 4–8 mm). "Pollytag" is produced from flue ash and containing among other 58 % $SiO_2$, 22 % $Al_2O_3$, 1.4 % Mg and 0.3 % S. The porosity of the granules is about 40 %. The usage of "Pollytag" aggregates as a filter material increased the retention time, sorption capacity of the beds, and the ability to bind toxic substances. One filter bed (III) was filled with coarse sand (grain size 0.1–3 mm). The general characteristics of the pilot farm wastewater treatment plants in the villages in Poland are given in Table 4.9.

Four local community HSSF systems located near Gorzów Wielkopolski were investigated by Sadecka (2001) during 7 years of operation. The characteristics of these systems are presented in Table 4.10. The flow rate of sewage was in the range 26.3–90.0 $m^3$/day. These systems were constructed at the beginning of the 1990s without sufficient knowledge regarding rules governing design. Averaged samples of sewage were collected once a month.

**Table 4.9** Characteristics of individual farm HSSFs near Lublin, Ostrołęka and Ciechanów in Poland (Obarska-Pempkowiak and Gajewska 2005)

| System | Name of the farmer | Number of persons | Characteristics of beds | | |
|---|---|---|---|---|---|
| | | | Area, $m^2$ | Plant | Bed material |
| *Lublin province* | | | | | |
| 1 | Olejnik | 9 | 45 | Willow | Medium sand |
| 2 | Próchniak | 6 | 50 | Willow | Medium sand |
| 3 | Chołaj | 7 | 38 | Willow | Medium sand |
| 4 | Podstawka | 7 + 10 in summer season | 50 | Willow | Medium sand |
| *Mazowsze province* | | | | | |
| 5 | Kesler | 6 | 60 | Willow | Natural soil, mainly (85 % medium sand) |
| 6 | Shiffer | 5 | 35 | Reed | Natural soil, mainly (75 % coarse sand) |
| 7 | Łysakiewicz | 6 + 15 in summer season | 35 | Reed | Natural soil, mainly (75 % coarse sand) |

*Przymorze village near Ciechanów, Mazowsze province*

| System | Name of the fanner | Number of persons | Characteristics of beds | | | | |
|---|---|---|---|---|---|---|---|
| | | | Area, $m^2$ | Plants | Shape, size L × W, m | Substrate material[a] | $k_{10}^{b}$ m/d |
| I | Antczak | 6 | 26.0 | Willow | "U" (2 × 10) × 1.3 | Mix (gravel and Pollytag) $U^a = 3.4$ | 650 |
| II | Kuc | 4 | 18.0 | Willow | Rectangular 18 × 1.0 | Mix (gravel and Pollytag) $U^a = 3.4$ | 650 |
| III | Wikliński Grzegorz | 5 | 22.0 | Willow | "U" (2 × 10) × 1.1 | Coarse sand $U^a = 2.6$ | 100 |
| IV | Wikliński Tadeusz | 8 | 30.0 | Willow | "U" (2 × 10) × 1.5 | Mix (gravel and Pollytag) $U^a = 3.4$ | 650 |

[a] $U = d_{60}/d_{10}$—grain uniformity coefficient, where $d_{60}$ is grain size for which 60 % of grains are finer and $d_{10}$ is the grain size for which 10 % of grains are finer

[b] $k_{10}$ is the hydraulic conductivity estimated using the Hazen formula $k_{10} = C \cdot (d_{10})^2$ , where C is the empirical coefficient dependent on porosity, according to the polish standards

**Table 4.10** Characteristics of local communities' HSSFs near Gorzów Wielkopolski, Poland (Sadecka 2001)

| WWTP | Flow, $m^3$/day | Area, $m^2$ | Depth, m | Unit area, $m^2$/pe |
|---|---|---|---|---|
| Wawrów | 90.0 | 3,500 | 0.8 | 2.7 |
| Gralewo | 46.3 | 3,325 | 0.9 | 3.0 |
| Maryszyn | 26.3 | 4,800 | 0.4 | 4.0 |
| Rokitno | 45.0 | 2,200 | 0.4 | 10.0 |

The removal efficiency in individual and community systems was calculated using formula $\eta = (C_0 - C)/C_0$, where $C_0$ and C are the input and output concentrations of pollutants, respectively.

The $BOD_5$ loading in pilot individual plants near Lublin and Ostrołęka in the first year of operation ranged from 1.21 to 5.76 g $O_2$ $m^2$/day. The loading of COD ranged from 2.79 to 9.06 g $O_2$ $m^2$/day, with the exception of plant No. 7 where it was 18.06 g $O_2$ $m^2$/day. The loading of organic nitrogen was higher in the Ostrołęka region and varied from 0.31 to 0.72 g $m^2$/day, while in the Lublin region it ranged from 0.09 to 0.17 g $m^2$/day. The total phosphorus loading in the investigated plants ranged from 0.09–0.47 g $m^2$/day.

Sewage generated in farms No. 1, 2, 3, and 5 was similar to domestic sewage. The average water consumption in these farms was equal to 55 l/person. The remaining farms produced sewage which is typical of agricultural activities and the average water consumption was 120 l/person (Sikorski 1997). The lower concentration of organic matter in septic tank outflows in plant No. 5 was caused by the improper construction of the outflow of this tank, which led to the decomposition of sewage organic matter before the wetland system.

The average concentration of characteristic pollutants in the outflow from the TWs investigated, and the corresponding maximum permissible concentrations are shown in Fig. 4.3. The results obtained indicated that the concentrations of suspended solids (SS) met the required quality standards in all the TWs. Plant No. 1 slightly exceeded the maximum permissible concentration of $BOD_5$ and exceeded almost twice those of TN and TP for discharge to a lake. Plant No. 6 exceeded the maximum permissible concentrations of organic matter, TN, and TP. Systems No. 2 and 3, planted with willow, fulfilled all the criteria regarding the required outflow quality. Facility No. 5 only exceeded the maximum permissible concentration of TN for discharge to a lake.

An improper operation of septic tanks and the lack of a proper connection between subsequent units were the most frequent reasons for the recontamination of sewage and lower efficiency of sewage treatment in the case of treatment wetlands No. 1, 6, 7 and partially for 4 and 5. Another reason for the poor operation of plant No. 6 was its bed partial clogging. These results indicate that all of the monitored treatment wetlands planted with willow (*Salix viminalis*) achieved higher efficiency of phosphorus removal (over 80 %) compared to those planted with reed (*Phragmites australis*). These suggest that the rhizosphere, i.e. the zone surrounding roots of willow may create conditions for phosphates removal, which does not take place in the case of the rhizosphere of reed. Plants No. 5, 6 and 7, filled with subsoil,

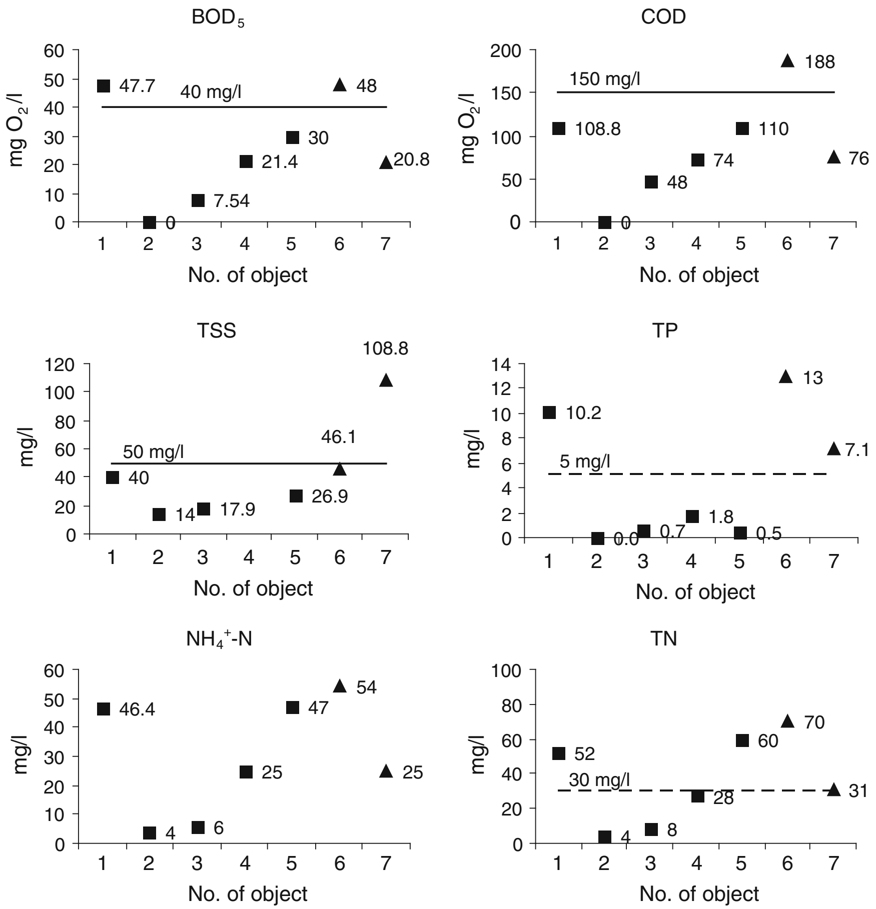


**Fig. 4.3** The average concentrations of characteristic pollutants in outflows from the analyzed treatment wetlands: with willow (■) and reed (▲) near Ostrołęka and near Lublin. *Solid lines* indicate maximum permissible levels of pollutants, whereas *dotted lines* correspond to maximum permissible levels when sewage is discharged to lakes (Obarska-Pempkowiak and Gajewska 2005)

showed a lower efficiency of contamination removal than the plants filled with sorted material, which turned out to have better hydraulic conditions for sewage treatment (e.g. plants No. 2 and 3).

The overall pollutants removal efficiency for the individual TWs near Lublin ranged as follows: 80–99.9, 50–99.9, and 10–99.9 % for $BOD_5$, TN, and TP respectively (see Table 4.11).

In the systems located near Ciechanów, there were no problems with the operation of the septic tanks. In all the plants, the septic tanks were of the same construction. Every tank was a circular concrete structure of the total functional volume 9.6 $m^3$, divided into three equal chambers. The sewage retention time in the septic tanks depended on the number of inhabitants and ranged from 6.1 to

**Table 4.11** Average removal efficiency of selected pollutants in individual TWs in Poland (Obarska-Pempkowiak et al. 2010)

| System | Efficiency (%) | | |
|---|---|---|---|
| | $BOD_5$ | TN | TP |
| 1 | 78.0 | 36.0 | 20.5 |
| 2 | 89.5 | 63.5 | 50.0 |
| 3 | 78.0 | 36.0 | 20.5 |
| 4 | 89.5 | 63.5 | 50.0 |
| 5 | 78.0 | 36.0 | 20.5 |
| 6 | 89.5 | 63.5 | 50.0 |
| 7 | 78.0 | 36.0 | 20.5 |
| I | 19.8 | 18.5 | 50.0 |
| II | 73.8 | 67.0 | 20.5 |
| III | 78.0 | 54.2 | 50.0 |
| IV | 67.5 | 27.0 | 20.5 |
| Wawrów (W) | 66.8 | 37.6 | 40.0 |
| Gralewo (G) | 64.3 | 28.0 | 40.0 |
| Małyszyn (M) | 41.4 | 45.0 | 40.0 |
| Rokitno (R) | 36.6 | 25.0 | 7.7 |

9.8 days. Pre-treated wastewater (i.e. septic tank outflow) was pumped to the willow bed through a submersible pump located in the third chamber of the septic tank. In spite of high concentrations of pollutants in the inflowing sewage, the septic tanks worked properly. The beds were fed with wastewater periodically, usually twice a day. The volume of each dose was equal to 0.5 $m^3$, and the discharge time was only 5 min. Thus, the momentary loading rate was rather high. The average concentrations of the pollutants in the sewage outflowing from the willow beds located near Ciechanów are shown in Fig. 4.4.

The analysis showed that in system I (the Antczak farm) the treatment of wastewater was very poor. This was probably caused by the leakage around the vertical barrier in the bed, which resulted in shortening the retention time (see Table 4.9). In system II (the Kuc farm), treatment efficiency was also rather low, the reason is that the shape of the bed is a long rectangle (18 m × 1 m) and, at lower air temperatures, the wastewater cools quicker than in the "U"—shaped beds.

The overall pollutants removal efficiency for the individual TWs near Cichanów ranged as follows: 20–79, 19–65, 0–78 % for $BOD_5$, TN and TP respectively (see Table 4.11).

Systems II, III and IV effectively removed suspended $BOD_5$ and TN solids and TP. Despite high efficiencies of $BOD_5$ and TN removal the concentrations of TN in the outflows were high due to high concentrations of $NH_4^+$-N (42–113 mg/l) (see Fig. 4.4).

The result indicate that in the investigated willow beds, the sorption of $NH_4^+$ did not take place. This was due to using coarse-grained filling material and the lack of conditions for nitrification in the beds with a saturated subsurface horizontal flow. In the wastewater inflowing to and outflowing from the willow beds, no oxidized

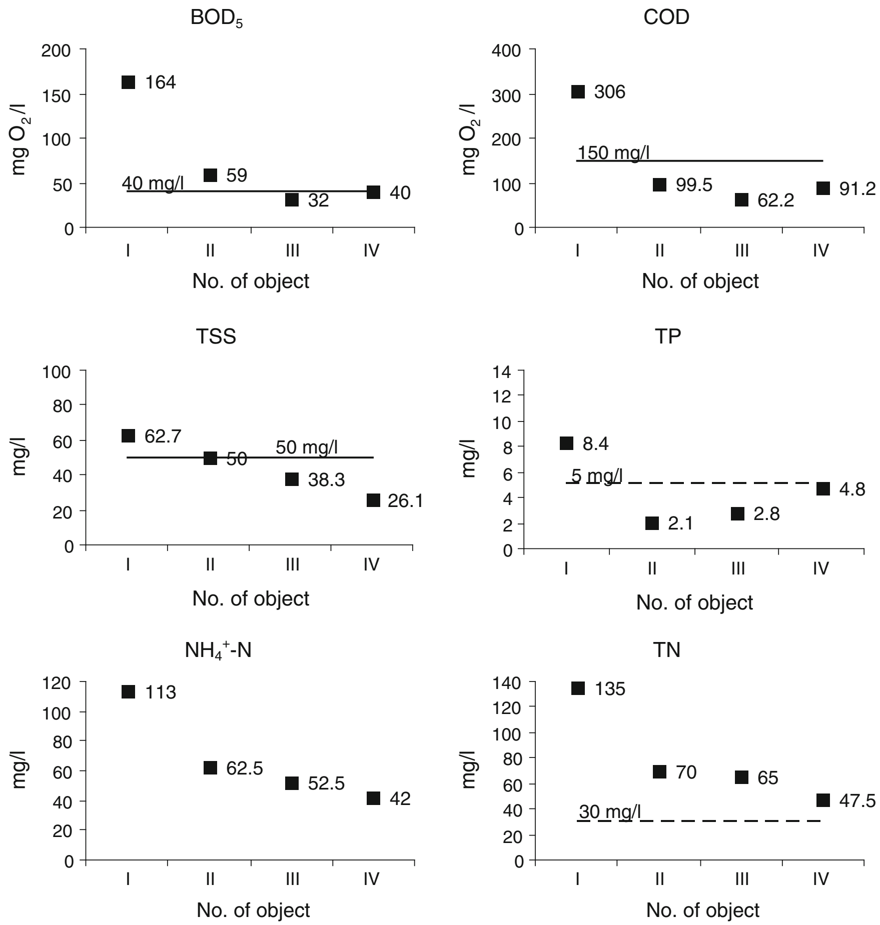


**Fig. 4.4** Average concentrations of characteristic pollutants in the outflow from the treatment wetlands near Ciechanów, Poland. *Solid lines* indicate maximum permissible levels of pollutants, whereas *dotted lines* correspond to maximum permissible levels when sewage is discharged to lakes (Obarska-Pempkowiak and Gajewska 2005)

forms of nitrogen were found, which indicated that ammonification was the dominant process in the beds.

Only systems II and III secured effective removal TP. Similar comparisons were carried out for four local HSSFs located in Wawrów, Gralewo, Małyszyn, and Rokitno (near Gorzów). Comparisons of organic matter ($BOD_5$), total nitrogen, ammonium nitrogen and total phosphorus concentrations in the inflow and outflow of those systems are presented in Fig. 4.5. Only two systems with the lowest concentration of organic matter ($BOD_5$) met the Polish outflow criteria; in one of the systems, this parameter slightly exceeded the permissible value.

The average removal efficiency of selected pollutants in individual and local TWs in Poland is presented in Table 4.11.

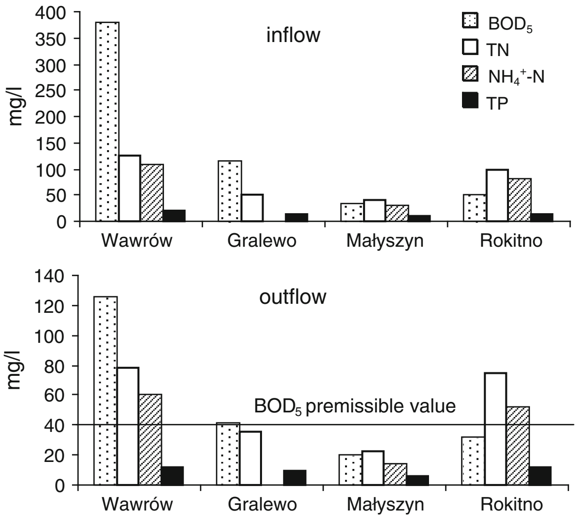


**Fig. 4.5** Average concentrations of selected pollutants in the inflow and outflow in community HSSFs located near Gorzów in Poland (Sadecka 2003)

The mass removal rates of organic matter [g $BOD_5/(m^2 \cdot day)$] in the beds of the analyzed individual farms and local facilities are presented in Table 4.12. The organic matter removal varied from 1 to 108 kg/(ha·day). The mass removal rate nitrogen [g $TN/(m^2 \cdot day)$] in one stage filters are presented in Table 4.13. The total nitrogen removal efficiency in these facilities was low and varied from 22.4 to 84.2 %, with the average value of 44.5 % in load whose discharge ranged from 2.0 to 220 kg/(ha·day).

**Table 4.12** Mass removal rates of $BOD_5$ in individual HSSF systems, $g/(m^2 \cdot day)$

| Local facilities | Farm facilities | | |
|---|---|---|---|
| Near Gorzów Wielkopolski | Near Ciechanów | Near Lublin | Near Ostrołęka |
| $BOD_5$, $g/(m^2 \cdot day)$ | | | |
| Gralewo: 6.5 | I: 0.7 | 1: 4.2 | 5: 1.9 |
| Wawrów: 1.0 | II: 3.4 | 2: 8.2 | 6: 2.7 |
| Małyszyn: 0.1 | III: 2.3 | 3: 10.8 | 7: 0.2 |
| Rokitno: 4.5 | IV: 0.4 | 4: 2.1 | |

**Table 4.13** Mass removal rates of total nitrogen in HSSF household systems, $g/(m^2 \cdot day)$

| Local facilities | Farm facilities | | |
|---|---|---|---|
| Near Gorzów Wielkopolski | Near Ciechanów | Near Lublin | Near Ostrołęka |
| *Mass removal TN*, $g/(m^2 \cdot day)$ | | | |
| Gralewo: 0.2 | I: 2.2 | 1: 0.6 | 5: 0.8 |
| Wawrów: 1.2 | II: 1.1 | 2: 0.2 | 6: 0.4 |
| Małyszyn: 0.1 | III: 0.7 | 3: 0.1 | 7: 0.9 |
| Rokitno: 0.9 | IV: 0.6 | 4: 1.1 | |

The mass removal rates of total nitrogen in the HSSF household systems analyzed are presented in Table 4.13.

The loadings of total nitrogen varied widely from 2.3 to 36.9 g/($m^2$·day).

Several of these facilities did not work properly. The main cause was the improper operation of septic tanks, the lack of proper T-connections that would allow a flow of sewage without fats and suspensions into hydrophyte filters. The substances inflowing with wastewater decreased the hydraulic conductivity of the filters. This sometimes led to the change of the character of the wastewater flow from subsurface to surface. The facilities with subsurface and surface flow should be designed according to different designing and operating principles.

The relationships between the mass loading rates and the mass removal rates of $BOD_5$ and TN are presented in Figs. 4.6 and 4.7. The removal rate of $BOD_5$ and nitrogen in these systems changes substantially—between 1.0–65.0 and 1.0–22.7 kg/(ha·day), respectively.

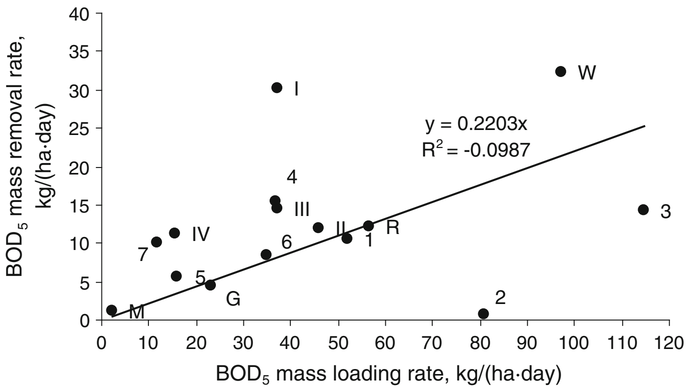


**Fig. 4.6** The dependence between $BOD_5$ mass removal rates and $BOD_5$ mass loading rates

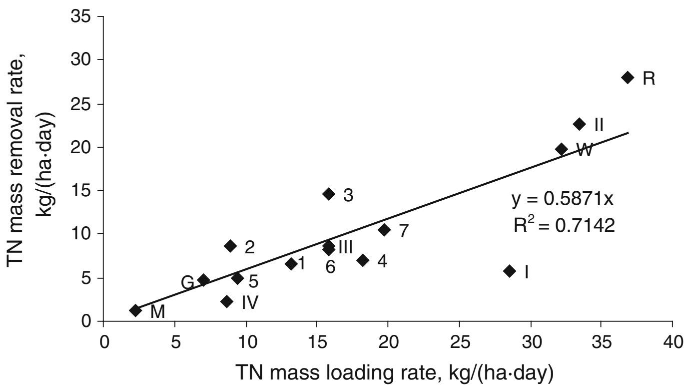


**Fig. 4.7** The dependence between TN mass removal rates and TN mass loading rates

During the 7 years' monitoring of local facilities it was proven that the removal of phosphorus compounds was higher during the vegetation season. It means that TWs had already been clogged and were working like overloaded treatment wetland ponds (Table 4.13). The same could be said about the removal efficiencies of nitrogen compounds. They varied during a year and was usually insufficient (Table 4.14).

The investigations conducted by Soroko (2001) in three pilot HSSF with unit surface equal to 10.0; 7.5 and 6.0 $m^2$/pe during a 6 year period, proved that with higher hydraulic loads and wastewater inflow, the removal efficiency of TN decreased from 57.7 to 46.0 % (Table 4.15).

The monitoring results of the individual household treatment wetlands indicated that the HSSF facilities working at the second stage of sewage treatment provided effective removal of $BOD_5$ and COD as well as TSS. The effectiveness of BOD removal varied from 25.6 to 99.1 % (average 62.4 %) for the loadings from 11.2 to 115.0 kg/(ha·day). However, the removal effectiveness of the total nitrogen was lower and varied from 22.4 to 84.2 % (average 44.5 %), for the loadings from 2.4 to 34.0 kg/(ha·day).

The one-stage vertical subsurface flow systems had not been used until 2004. Only pilot-scale research had been conducted (Soroko 2001; Kowalik et al. 2004). The average $BOD_5$ and total nitrogen removals reported by Soroko (2001) was equal to 97.4 and 41.6 % respectively. Kowalik et al. (2004) reported the removal effectiveness of $BOD_5$ and TN equal to 89.1 and 76.1 % at the second stage of treatment, between 93.8 and 79.1 % at the third stage of treatment.

Vertical subsurface flow treatment wetlands (VSSF) were implemented in the Podlasie region in 2004. The Municipality of Sokoly near Białystok, which is in that region, decided to launch a proper sewage management programme. Since the building of a sanitary sewerage system and its operation were too expensive, the Municipality authorities decided to build treatment wetlands for individual households. At present there are 600 treatment wetlands, consisting of a one-stage bed with a vertical subsurface flow of sewage, in operation in the Podlasie region (Wasiak 2008). The treatment wetlands were built by individual farmers on their own ground, according to the conception and guidelines of the Institute of Applied Ecology in Skorzyn (Halicki 2009). According to this conception, the treatment facility for a single family consists of a septic tank (sewage retention time 5 days), followed by a VSSF bed, periodically supplied with sewage by a pump. If the denitrification of nitrogen compounds is required, a denitrification pond is built after a VSSF bed. The edges of the pond are laid with foil to a certain level (not to the top), which allows sewage to leak into the ground. The sludge from the septic tank is dredged out once every 6 months. The treatment wetlands in Podlasie are being monitored by several research institutes, however, the monitoring results are not obvious. According to the analyses reported by Wierzbicki and Gutry (2009), the average concentrations of characteristic pollutants were as follows: $BOD_5 < 13$ mg $O_2$/l, COD < 93 mg $O_2$/l, TSS < 42 mg/l.

Two treatment wetlands in the Municipality of Sokoły were monitored (Magrel 2009). Effective removed TSS and organics (COD and $BOD_5$) was reported. The

**Table 4.14** Seasonal changes of pollutants efficiences removal in local TWs (%) (Sadecka 2003)

| Parameter | Treatment facility | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Wawrów | | Gralewo | | Małyszyn | | Rokitno | |
| | Autumn–winter | Spring–summer | Autumn–winter | Spring–summer | Autumn–winter | Spring–summer | Autumn–winter | Spring–summer |
| COD | 41.9 | 52.2 | 62.5 | 7.6 | 60.4 | 42.9 | 42.9 | 42.5 |
| $BOD_5$ | 52.2 | 68.1 | 67.6 | 30.5 | 71.4 | 45.2 | 52.2 | 56.1 |
| $NH_4^+$-N | 19.4 | 42.0 | 38.3 | 3.0 | 35.8 | 21.5 | 19.4 | 33.2 |
| TN | 18.6 | 37.4 | 31.5 | 37.8 | 35.2 | 41.6 | 18.6 | 27.6 |
| TP | 0.2 | 13.6 | 1.5 | 40.6 | 20.6 | 26.3 | 0.2 | 11.4 |

**Table 4.15** Average removal of $BOD_5$ and total nitrogen from domestic wastewater as function of hydraulic loads in HSSF (Soroko 2001)

| Hydraulic load, mm/day | Load of $BOD_5$, g $O_2/(m^2$ day) | Load of TN, g/($m^2$·day) | Average removal efficiency (%) | |
|---|---|---|---|---|
| | | | $BOD_5$ | TN |
| 15.0 | 1.2 | 0.78 | 95.9 | 57.7 |
| 20.0 | 1.6 | 1.04 | 95.1 | 49.0 |
| 25.0 | 2.0 | 1.30 | 96.0 | 46.0 |

**Table 4.16** The average concentrations of pollutants in raw and treated sewage and the removal efficiencies in the Municipality of Sokoly, Podlasie region, according (Obarska–Pempkowiak et al. 2010)

| Owner of the farm | Parameter | TSS | COD | $BOD_5$ | TP | TN | $NH_4^+$-N |
|---|---|---|---|---|---|---|---|
| TW 1 | Inflow, mg/l | 239.3 | 707.3 | 443.3 | 30.8 | 94.6 | 75.2 |
| | Outflow, mg/l | 28.4 | 78.3 | 8.3 | 17.9 | 36.4 | 4.9 |
| | Removal effectiveness, % | 88.1 | 88.9 | 98.1 | 41.9 | 61.5 | 93.5 |
| TW 2 | Inflow, mg/l | 574.3 | 467.7 | 260.0 | 13.4 | 84.3 | 79.3 |
| | Outflow, mg/l | 90.4 | 61.7 | 13.3 | 11.6 | 38.7 | 3.5 |
| | Removal effectiveness, % | 84.3 | 86.8 | 94.9 | 13.4 | 54.1 | 95.6 |

outflow concentrations of these pollutants fulfilled the requirements imposed by the Regulation of Environmental Minister from 24th July, 2006 (Table 4.16). However, the removal effectiveness of nutrients, especially phosphorus, was unstable.

The analysis performed by the authors of the article indicated that the investigated facilities were very effective in pollutants removal. The removal effectiveness of $BOD_5$ varied from 86.0 to 98.0 %, and that of COD—from 79 to 94 % (Table 4.17, Fig. 4.8).

The results of the analyses confirmed the low effectiveness of total phosphorus removal. Additionally, it was found that the outflow concentrations of TSS exceeded the admissible value of 50 mg/l (The Regulation of Environmental Minister from 24th July 2006). The share of organic suspended solids in the total suspended solids at the outflow varied from 49.0 to 95.0 % (Obarska-Pempkowiak et al. 2010). Very good conditions for the nitrification process existed in the treatment facilities. This is confirmed by the very low concentrations of ammonia nitrogen at the outflows of the three analyzed facilities. However, the removal of total nitrogen was substantially lower in comparison to that of ammonia nitrogen. As a result, nitrates V were the dominant form of nitrogen at the outflows, which indicates that the denitrification pond failed to play its role.

**Table 4.17** The average concentrations of characteristic pollutants in raw and treated sewage from individual household treatment wetlands in the Municipality of Sokoły, the Podlasie region (Obarska-Pempkowiak et al. 2010)

| Parameter | SFTW 1[a] | | SFTW 2 | | SFTW 3 | |
|---|---|---|---|---|---|---|
| | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow |
| pH | 7.8 | 7.0 | 8.4 | 7.0 | 8.2 | 7.3 |
| TSS, mg/l | 248.1 | 145.6 | 148.3 | 101.6 | 115.6 | 7.6 |
| Organic SS, mg/l | 180.0 | 71.3 | 91.9 | 72.8 | 82.4 | 7.2 |
| COD, mg/l | 931.4 | 191.5 | 517.0 | 62.5 | 722.6 | 37.1 |
| $BOD_5$, mg/l | 496.1 | 25.9 | 180.5 | 26.7 | 274.6 | 4.6 |
| TN, mg/l | 255.2 | 37.9 | 346.1 | 51.0 | 201.7 | 55.1 |
| $NH_4^+$-N, mg/l | 100.0 | 4.3 | 145.5 | 7.5 | 87.5 | 1.3 |
| Org-N, mg/l | 55.0 | 3.6 | 55.0 | 32.8 | 24.8 | 6.9 |
| $NO_3^-$-N, mg/l | 0.2 | 25.7 | 0.1 | 3.2 | 1.9 | 45.6 |
| TP, mg/l | 19.3 | 4.8 | 13.1 | 17.8 | 6.1 | 7.1 |

[a] *SFTW* Single Family Treatment Wetland

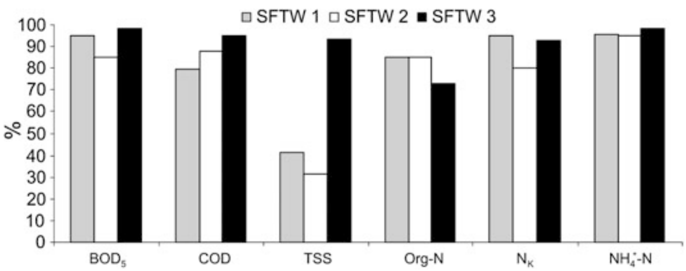


**Fig. 4.8** The pollutant removal effectiveness in the Single Family Treatment Wetlands in the Municipality of Sokoły (the Podlasie region) according (Obarska-Pempkowiak et al. 2010)

Within the research project *Innovative Solutions for Wastewater Management in Rural Areas* supported from the EEA Financial Mechanism and Norwegian Financial Mechanism (PL 0271), and the Polish Ministry of Science and Higher Education (E033/P01/2008/02) the conception of sewage treatment and sewage sludge utilization at the TWs for individual households in a rural area was created. After the review of existing TWs in Poland and in Europe, various configurations of hydrophytes beds are proposed (Fig. 4.9). Three configurations were proposed: two with vertical subsurface flow (VSSF) beds and the third one with a horizontal flow (HSSF) bed preceded by a prefilter (Fig. 4.9):

- Configuration I: primary sedimentation tank with elongated detention time (5–6 days), followed by a single VSSF bed (the unit area of 4 $m^2$/pe) and a pond,
- Configuration II: existing primary sedimentation tank (with short retention time up to 2 days), then two sequential VSSF beds followed by a pond,
- Configuration III: primary sedimentation tank, prefilter (pre-treatment), HSSF bed.

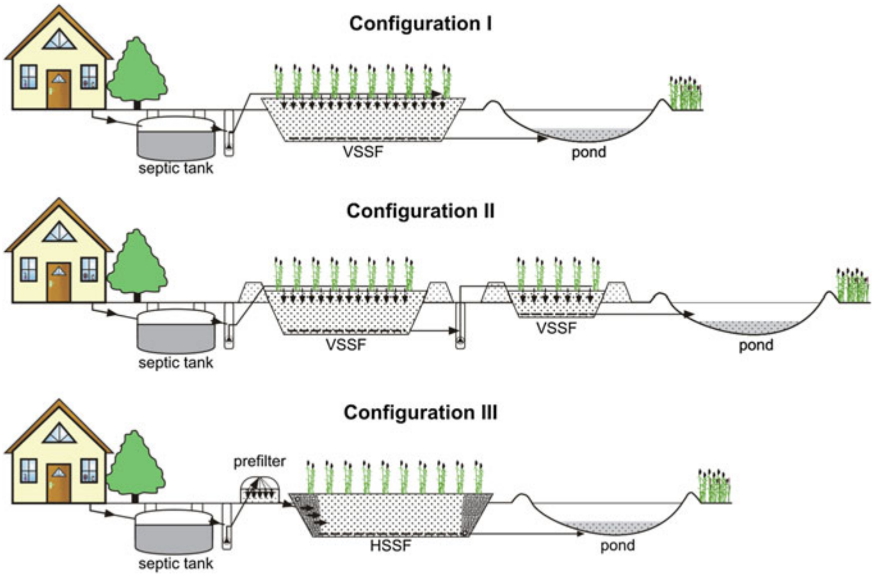


**Fig. 4.9** Layout of three single-farm TW configurations (Obarska-Pempkowiak et al. 2012)

In all TWs additional pumps were used. A timer, overrun by a float switch, controlled the dosing pump. The depth of vertical subsurface flow beds was 0.7 m. The bottom was laid with HDPE foil (1 mm). Common reed was planted on the beds surface with the density of 4 plants/m$^2$. These facilities were constructed in summer 2009 in Kaszuby Lake District (Fig. 4.10).

The single-family TWs were monitored in the years 2009–2013. Although the concentrations of pollutants in discharged sewage varied significantly among the analyzed TWs, they were much higher than reported by Vymazal (2005), Heistad et al. (2006), Steer et al. (2002) and Jenssen et al. (2005). The average concentration of COD varied from 537.3 to 1140.9 mg $O_2$/l and for TN from 107.9 to 134.1 mg TN/l (Table 4.18). These concentrations are two–three times higher in comparison to the values reported by Vymazal (2005) and Heistad et al. (2006) (36.3–77.5 mg TN/l). Similarly high concentration of nitrogen were present in the septic tank outflow in Podlasie region of Poland (Obarska-Pempkowiak et al. 2009). Sewage characterized enormous concentrations of organics COD and $BOD_5$ were discharged to TWs—at four out of the nine analysed farms. COD was exceeding 1,000 mg/l and at another one was 970 mg/l. The inflow $BOD_5$ concentrations were also high—the highest amounting to 1,200 mg/l. So high inflow concentrations could be caused either by improper maintenance and operation of septic tanks or the inflow of high strength wastewater (manure, run-off from the fields or leakages from farmyard).

Although the concentration of pollutants decreased, however the sewage samples collected from the last stage of treatment (the pond) in many cases not fulfil the requirements of the Regulation of Environmental Minister from 24th July 2006.

**Fig. 4.10** The elements applied in individual SFTWs in Stężyca

Relatively low quality treated outflow is likely be a result of short period of operation (low development of roots and rhizomes as well as biofilms) and short period of sewage retention in the pond. Further monitoring of the TWs is necessary in order to explain the sewage treatment mechanisms as well the role of the purification pond in the treatment process.

In spite of so high inflow pollutants concentrations, quite effective removals of pollutants were observed at most TWs. The removal efficiency for organic matter is given in Table 4.19. The biological part (namely treatment wetlands) provided for high and stable efficiency in removing deliverent pollutants, similar to high-effective methods (like trickling filters) applied for treatment of small amount of wastewater.

With reference to $BOD_5$ it varied from 69.7 up to 91.8 %, while—from 53.1 up to 84.1 % with reference to COD. Among organic substances $BOD_5$ was removed with slightly higher efficiency than COD a feature characteristic of both conventional and natural wastewater treatment technologies. The highest effectiveness of $BOD_5$ removal amounting to 88.1 % was observed for Configuration II. Long retention time in HSSF bed applied in Configuration III favour the decomposition of COD.

The analysed TWs showed high variation in total suspended solid removal from 43.6 to 89.1 %. Lower efficiency in removing total suspended solid was caused by high concentration at the inflow. Treatment wetlands facilities, which basing on

**Table 4.18** Average concentrations of pollutants in SFTWs, mg/l

| Parameter | Configuration | | | | | |
|---|---|---|---|---|---|---|
| | I (VSSF + pond) | | II (VSSF I + VSSF + pond) | | III (pre-filter + HSSF + pond) | |
| | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow |
| TSS | 289.7 | 69.9 | 178.3 | 37.2 | 269.4 | 100.3 |
| | 186.7–441.4 | 47.9–100.2 | 161.2–202.7 | 29.3–44.6 | 179.7–413.9 | 83.3–116.2 |
| VSS | 236.9 | 52.7 | 137.5 | 30.5 | 221.5 | 91.4 |
| | 157.0–348.1 | 32.8–77.4 | 113.1–173.2 | 25.7–39.7 | 143.0–346 | 73.3–123 |
| $BOD_5$ | 236.0 | 46.8 | 328.3 | 36.0 | 558.1 | 69.5 |
| | 166.1–330.5 | 37.5–64.2 | 201.9–415.6 | 33.4–40.6 | 466.6–723.9 | 50.6–80.8 |
| COD | 537.3 | 170.3 | 745.3 | 165.9 | 1140.9 | 204.5 |
| | 292.5–679.1 | 101.7–271.8 | 568.1–836.6 | 140.4–179.1 | 940.0–1306.4 | 189.0–206.4 |
| TN | 107.9 | 39.9 | 134.1 | 42.4 | 115.7 | 48.0 |
| | 59.4–159.5 | 26.4–46.9 | 111.2–158.5 | 26.9–72.4 | 80.5–138.6 | 35.6–69.7 |
| $NH_4^+$-N | 76.6 | 17.7 | 78.5 | 15.4 | 61.4 | 16.8 |
| | 38.2–118.2 | 8.15–23.6 | 44.8–97.7 | 7.4–30.1 | 53.8–69.4 | 8.4–29.0 |
| TP | 11.2 | 4.3 | 15.5 | 5.5 | 14.3 | 6.6 |
| | 4.8–17.9 | 3.4–5.7 | 14.6–17.2 | 4.7–6.2 | 13.2–15.9 | 6.0–7.2 |

**Table 4.19** The organic matter efficiency removal in TWs working in analyzed facilities (Obarska-Pempkowiak et al. 2013)

| Facility | $BOD_5$ | | | COD | | |
|---|---|---|---|---|---|---|
| | Configuration | | | | | |
| | I | II | III | I | II | III |
| 1 | 88.3 | 83.5 | 83.3 | 84.1 | 68.6 | 79.9 |
| 2 | 69.7 | 88.9 | 93.0 | 60.0 | 83.1 | 82.5 |
| 3 | 77.4 | 91.8 | 83.5 | 53.1 | 78.6 | 83.3 |
| Average | 78.5 | 88.1 | 86.6 | 65.7 | 76.8 | 81.9 |

**Table 4.20** The total suspended solids removal efficiency in TWs working in analyzed configurations (Obarska-Pempkowiak et al. 2013)

| Facility | TSS | | |
|---|---|---|---|
| | Configuration | | |
| | I | II | III |
| 1 | 89.1 | 85.5 | 43.6 |
| 2 | 58.3 | 72.3 | 79.9 |
| 3 | 67.1 | 78.0 | 45.8 |
| Average | 71.5 | 78.6 | 56.4 |

biological treatment, are predisposed mostly to remove pollutants in dissolved and colloidal form, as well as in residual suspended solid after preliminary tanks. High concentration of total suspended solid at the outflow from the tanks reduced efficiency of the system. Facilities in Configuration II were supplied with wastewater of lower concentration of total suspended solids, which resulted in its removal with average efficiency amounting to 78.6 % (Table 4.20).

The highest efficiency in removing TN was observed in case of facilities operating in Configuration II (69.7 % on average), while in case of Configuration I and III it was 60.7 and 58.5 % respectively. The facilities supplied with wastewater of high concentration of total nitrogen characteristic for agriculture and service activities should have greater area due to kinetics of microbiological changes of nitrogen. The highest efficiency in removing TP was achieved also in facilities operating in Configuration II—64.4 % on average, which provided concentration from 4.5 to 6.2 mg TP/l at the outflow (Table 4.21).

Lower concentration at the outflow was noted in case of facilities operating in Configuration I—from 3.8 up to 5.8 mg TP/l due to receiving the lowest concentrations in inflow (Table 4.18).

**Table 4.21** The nutrients removal efficiency (%) in TWs working in the analyzed facilities (Obarska-Pempkowiak et al. 2013)

| Facility | TN (%) | | | TP (%) | | |
|---|---|---|---|---|---|---|
| | Configuration | | | | | |
| | I | II | III | I | II | III |
| 1 | 70.6 | 75.9 | 49.7 | 68.2 | 67.8 | 57.6 |
| 2 | 55.9 | 79.0 | 69.9 | 65.4 | 61.4 | 54.7 |
| 3 | 55.6 | 54.3 | 55.8 | 29.1 | 64.0 | 50.0 |
| Average | 60.7 | 69.7 | 58.5 | 54.3 | 64.4 | 54.1 |

The treatment wetlands for single-family outflow is a stable and effective method for wastewater treatment in the rural areas.

The TWs operated in Poland receive much higher concentrations of pollutants in comparison to the TWs operated in Europe and USA.

Good treatment effectiveness of BOD (64.0–92.0 %), TN (44.0–77.0 %), TP (24.0–66.0 %) were observed. Comparing the achieved efficiency removal in three configurations of facilities shows:

- importance of TSS removal in prefilter before application of TWs
- double contact time in VSSF beds working sequentially improve the efficiency removal up to 20 % in comparison to the efficiency of single VSSF with bigger unit area.

The lowest concentration of pollutants, and at the same time the highest efficiency of treatment was provided by the facilities operating according to Configuration II. However operation of facilities according to Configuration I and III was also efficient. Nevertheless, in the final comparison it is confirmed that facilities operating in Configuration II were the most efficient (it means using two sequential subsurface beds with vertical wastewater flow). After 3 years the removal efficiency of organic and total nitrogen increased by 12–20 % except for total phosphorous. In the fourth year of explanation the amount of nitrate increased significantly in the outflow from SFTWs with VSSF beds.

Individual construction of the treatment facilities by farmers under the supervision of technical personnel make the framers aware of the significance of each element of the treatment process and guarantees proper future operation of the system.

## 4.2 Hybrid Treatment Wetlands (HTWs)

When quality demands for outflow wastewater, including concentrations of nutrient compounds, are strict, systems which consist of VSSF vertical subsurface flow beds with intermittent loading, and HSSF horizontal subsurface flow beds (so called Hybrid Treatment Wetlands) are the proper solution. HTWs are more expensive than SSFs or SFs as far as investment costs are concerned. However, they can be much cheaper than other systems which use conventional technologies. Presently there are two main hybrid systems, they differ with the bed type—HSSF or VSSF—however at the beginning of the biological treatment process (Figs. 4.11 and 4.12).

A growing interest in hybrid systems has been shown for many years. HSSF beds in hybrid systems ensure the high removal effectiveness of organic mater and total suspended solids. They can also create good conditions for the denitrification process. In VSSF beds, there are good conditions for the nitrification process, and more effective oxidation makes the removal of $BOD_5$ and COD very effective. It is possible to combine the advantages of both the HSSF and VSSF systems, which results in achieving a lower concentration of BOD, full nitrification and partial denitrification, as well as a much lower concentration of total nitrogen (Cooper et al. 1998).

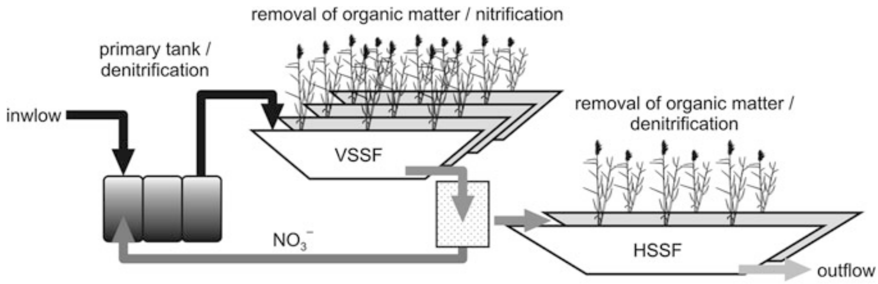


**Fig. 4.11** A schematic of treatment wetlands system with VSSF bed on the beginning of biological stage of treatment

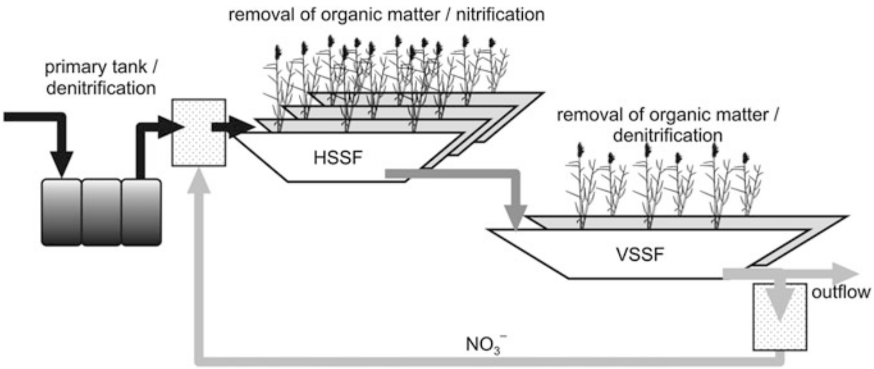


**Fig. 4.12** A schematic of treatment wetlands system with HSSF bed on the beginning of biological stage of treatment

Because of the small number of existing facilities, it is too early to decide "which version, with HSSF" or "VSSF bed at the beginning of the system", has more advantages (Cooper and Maesneer 1996). Some systems like St. Bohaire or Frolois in France (Lienard et al. 1990, 1998) and Oaklands Park, United Kingdom (Cooper and de Maesneer 1996) have two stages of VSSF beds, followed by another two stages of HSSF beds. The results obtained at Oaklands Park, which indicate that nitrification takes place in VSSF beds and denitrification in HSSF beds, are presented in Table 4.22. However, nitrification was not full in the second VSSF because its bed surface was too small.

Urbanc-Berčič and Bulc (1994) describe an excellent pilot system located in Adjovščina, Slovenia. The obtained results are given in Table 4.23.

VSSF beds, A and B, differed in the size of filling material. Bed A was filled with coarse sand (4–8 mm), whereas bed B—with a mixture of fine sand (1–4 mm) and coarse sand (4–8 mm). It was obvious that the bed with the finer sand was more effective. The results presented prove that in the VSSF beds besides nitrification there was also denitrification (Urban-Berćić and Bulc 1994).

**Table 4.22** Concentrations of nutrients after subsequent stages of treatment in HTWs at Oklands Park, United Kingdom, mg/l (Cooper and Green 1995)

| Parameter | Inflow | VSSF I | VSSF II | HSSF II | HSSF II | Pond |
|---|---|---|---|---|---|---|
| $NH_4^+$-N | 50.5 | 29.2 | 14.0 | 15.4 | 11.1 | 8.1 |
| $NO_x$-N[a] | 1.7 | 10.2 | 22.5 | 10.0 | 7.2 | 2.3 |
| $PO_4^{3-}$-P | 22.7 | 18.3 | 16.9 | 14.5 | 11.9 | 11.2 |

[a] $NO_x$-N = N-$NO_2^-$ + N-$NO_3^-$

Lienard et al. (1998) describe the implementation of a hybrid system which consists of a VSSF bed followed by a HSSF bed at Frolois, France. The system was used for dairy sewage treatment, where nitrogen was present mainly in the form of ammonium and organic nitrogen. The total removal effectiveness of nitrogen was 57 %, with the initial concentration 55 mg/l. The load in the inflow initially was equal to 705 g N/($m^2$ year) and later decreased to 346 g N/($m^2$ year). The majority of nitrogen was removed at the first stage, in a VSSF bed. In the inflow of this part of the system, the concentration of nitrogen was 34 mg/l. In the second stage (HSSF bed) removal effectiveness was lower and the concentration of nitrogen in the outflow was 27 mg/l. TKN concentration in the outflow was 21 mg/l, which indicates a small effectiveness of denitrification. The removal of phosphates took place both in the VSSF bed and the HSSF bed. The concentration of phosphates, which was 11 mg/l in the sewage inflow, decreased to 8 mg/l after the VSSF stage, and to 6 mg/l in wastewater after the HSSF bed.

House et al. (1996) describe the use of a hybrid system which consists of two beds with a vertical subsurface flow (surface 125 $m^2$ each) and two beds with a horizontal subsurface flow (134 $m^2$ each) for treatment of wastewater originating from a primary school. The removal effectiveness of TN was 75.3 % (initial concentration was 56.6 mg/l, final concentration was 14 mg/l). Partially treated wastewater with a lower concentration of $NH_4^+$-N (27.8 mg/l) was recirculated into the beds with a vertical flow. Although the VSSF beds decreased the concentration of $NH_4^+$-N to 0.89 mg/l, the concentration of $NO_3^-$-N rose to 21.2 mg/l, which means that full denitrification was not achieved. The average concentration of TP in the inflow was 8.1 mg/l, and after the treatment wetland system it decreased to 4.6 mg/l (average value from 4 years of operation), i.e. by 43 %, the removal efficiency of phosphate compounds decreased every year—in 1993 it was 63 %, whereas in 1996—24 %).

Laber et al. (1999) describe a system which consists of a HSSF bed with 140 $m^2$ surface and a VSSF bed with 120 $m^2$ surface, used in Nepal. The high removal effectiveness of ammonia nitrogen was achieved there: the average concentration of $NH_4^+$-N in the inflow was 37.9 mg/l, whereas after the HSSF bed it was 15 mg/l, and after the VSSF bed it equaled 3.03 mg/l. On the other hand, the concentration of $NO_3^-$-N rose from 0.18 mg/l in the inflow wastewater to 35.2 mg/l in the outflow wastewater, which proved that there were no conditions for the denitrification process. The removal effectiveness of TP was 30.8 %, whereas the concentration of TP in the outflow was 2.7 mg/l.

**Table 4.23** Removal efficiencies of selected pollutants in HTWs at Adjovščina, Slovenia (Urbanc-Berčič and Bulc 1994)

| Parameter | Bed A[a] | | | Bed B[a] | | | Bed C[a] | | |
|---|---|---|---|---|---|---|---|---|---|
| | Inflow (mg/l) | Outflow (mg/l) | Effic. (%) | Inflow (mg/l) | Outflow (mg/l) | Effic. (%) | Inflow[b] (mg/l) | Outflow (mg/l) | Effic. (%) |
| $NH_4^+$-N | 352.3 | 259.7 | 26.3 | 344.0 | 19.0 | 94.5 | 139.4 | 8.7 | 93.8 (97.5) |
| $NO_3^-$-N | 0.081 | 0.057 | 29.7 | 0.104 | 0.524 | 74.5 | 0.29 | 0.023 | 92.1 (74.5) |
| Org-N | 21.2 | 17.0 | 19.8 | 19.5 | 7.7 | 60.5 | 12.4 | 3.1 | 75.0 (84.8) |
| TN | 373.6 | 276.8 | 25.9 | 363.6 | 27.2 | 92.5 | 152.0 | 11.8 | 92.2 (96.8) |
| TP | 25.54 | 18.43 | 28.0 | 25.94 | 1.29 | 95.1 | 9.86 | 0.74 | 92.5 (97.1) |

[a] Beds A and B are VSSF systems (each surface 20 $m^2$), bed C is a HSSF system (surface 40 $m^2$)

[b] Average outflow from bed A and B

In recent years the increase of interest in hybrid treatment wetland systems has been observed (Brix et al. 2003; Cooper 2004; Gajewska and Obarska-Pempkowiak 2011). These systems are composed of two or more filters with mixed flow direction of sewage. Apparently in the HTWs the benefits of both types of bed are merged, resulting in better outflow quality (lower organic matter concentration, complete nitrification and partial denitrification) (Kinsley et al. 2006; Cooper 2004; Alvarez et al. 2006). HTWs require smaller unit area and secure higher efficiency of pollutants removal in comparison to one stage systems. The hybrid designs incorporating multiple stages of TWs is now becoming more popular (Gómez Cerezo et al. 2001; Peng et al. 2005; Brix et al. 2011; Vymazal and Kropfelova 2011) due to the higher tolerance and efficiency of this kind of systems from one side and the often lower footprint in comparison to single stage ones when facing flow and loads variations, or even for giving chances of different outflows quality to be chosen on seasonal basis and depending on the required final concentration of pollutants. Furthermore, single typology TW systems have intrinsically specific limits in terms of processes which occur inside the reactors, for example the commonly scarce nitrification for the HSSF systems or the low denitrification rate for the VSSF unsaturated systems. The main concept of the multistage systems is the assignment of a specific role and process to each stage, in order to reach a final outflow with the best quality. Nowadays probably the biggest HTW system for secondary treatment is located in Italy (one of the biggest in the world). Wastewaters produced by the whole Dicomano municipality (3,500 pe—province of Florence—Italy) is treated in this system, by total surface area of 6,080 $m^2$ (Masi et al. 2013).

Most of hybrid treatment wetlands (HTWs) were used for treatment of primary settled domestic wastewater in local WWTPs.

In the Gdańsk region five of such type TWs were build serving from 150 to 650 pe. The characteristics of the studied HTWs are shown in Table 4.24. The HTW in Darżlubie is probably the biggest facility build in this technology in Poland. It is designed terms for over 650 pe.

All the systems had HSSF as an initial stage of a biological treatment. They differed from one another in the order and number of subsequent stages. The main differences were the number of beds and operating conditions (Table 4.25).

In the case of Sarbsk plant two VSSF beds worked parallely and sewage was discharged continuously. In Wiklino plant, two VSSF worked alternately during 2 weeks, the sewage was periodically pumped into one of two compartments of the VSSF filter. In Wieszyno plant two compartments of VSSF beds worked in series and were loaded continuously.

The average concentrations of several parameters in the inflow and outflow, as well as the corresponding standard deviations, for the five HTWs are presented in the Tables 4.26 and 4.27 respectively (Obarska-Pempkowiak and Gajewska 2005; Gajewska and Obarska-Pempkowiak 2011).

In the Schodno plant approximately 65 % of sewage was pumped intermittently to one of four compartments of VSSF beds. The remaining sewage was pumped directly to the HSSF II bed. Wastewater treated in the HSSF II bed was pumped into two compartments of the VSSF II, which worked alternately with intermittent loading

**Table 4.24** The characteristics of hybrid treatment wetland systems in northern Poland (Gajewska and Obarska-Pempkowiak 2011)

| Plant | Q ($m^3$/day) | Configuration | Area ($m^2$) | Depth (m) | Hydraulic load (mm/day) | Unit area ($m^2$/pe) |
|---|---|---|---|---|---|---|
| Sarbsk | 29.7 | HSSF | 1,610 | 0.6 | 18.5 | 8.5 |
| | | VSSF[a] | 520 | 0.5 | 38.6 | 2.6 |
| | | | Σ 2,130 | | 13.9 | Σ 9.1 |
| Wiklino | 18.6 | HSSF I | 1,050 | 0.6 | 17.7 | 7.0 |
| | | VSSF | 624 | 0.4 | 46.9 | 2.0 |
| | | HSSF II | 540 | 0.6 | 34.4 | 3.4 |
| | | | Σ 2,214 | | 8.4 | Σ 12.4 |
| Wieszyno | 24.5 | HSSF I | 600 | 0.6 | 40.8 | 3.0 |
| | | VSSF | 300 | 0.6 | 81.7 | 1.5 |
| | | HSSF II | 600 | 0.6 | 40.8 | 3.0 |
| | | | Σ 1,500 | | 16.3 | Σ 7.5 |
| Schodno | 2.2 in winter 8.9 in summer | HSSF I | 416 | 0.6 | 5.3–21.4 | 27.8–6.4 |
| | | VSSF I | 307 | 0.45–0.6 | 7.2–28.9 | 20.5–4.7 |
| | | HSSF II | 432 | 0.6 | 5.1–20.6 | 28.8–6.6 |
| | | VSSF II | 180 | 0.45–0.6 | 12.2–9.4 | 12.0–2.8 |
| | | Willow plantat. | Σ 1,300 | | 1.7–6.8 | Σ 20–86.7[b] |
| Darżlubie | 56.7 | HSSF I | 1,200 | 0.6 | 47.3 | 2.0 |
| | | Cascade bed | 400 | 0.6 | 141.2 | 0.67 |
| | | HSSF II | 500 | 1.0 | 113.4 | 0.8 |
| | | VSSF | 250 | 0.6 | 226.8 | 0.4 |
| | | HSSF III | 1,000 | | 56.7 | 1.7 |
| | | | Σ 3,350 | | 16.9 | ∑ 5.6 |

[a] The configuration and water regime of VSSF beds are presented in Table 4.25

[b] In the summertime treated wastewater is directed for irrigation the willow plantation of the area 400 $m^2$

**Table 4.25** The ways of wastewater is discharged to VSSF (Gajewska and Obarska-Pempkowiak 2011)

| Plant | Configuration | Operation condition of VS beds |
|---|---|---|
| Wiklino | VSSF   VSSF | Alternately, intermittent |
| Wieszyno | VSSF ⇒ VSSF | In series, continuously |
| Sarbsk | VSSF   VSSF | Parallel, continuously + recirculation into HF |
| Darżlubie | | Parallel, continuously |
| Schodno | VSSF I · VSSF I · VSSF I · VSSF I | SF I—4 compartments alternately, intermittent |
| | VSSF II   VSSF II | SF II—2 compartments alternately, intermittent |

**Table 4.26** Average concentrations of characteristic pollutants in the inflowing

| Parameters | Unit | Schodno, $n^x = 18$ | Darżlubie, $n^x = 21$ | Wiklino, $n^x = 88$ | Wieszyno, $n^x = 18$ | Sarbsk, $n^x = 38$ |
|---|---|---|---|---|---|---|
| TSS | mg/l | 156.6 ± 51.2 | 359.5 ± 87.9 | 539.3 ± 127.2 | 1269.5 ± 167.6 | 819.9 ± 208 |
| COD | mg $O_2$/l | 880.0 ± 189.2 | 837.5 ± 156.3 | 466.3 ± 92.7 | 1021.9 ± 251.2 | 687.6 ± 162.9 |
| $BOD_5$ | mg $O_2$/l | 448.5 ± 123.2 | 401.5 ± 51.3 | 265.2 ± 51.7 | 657.3 ± 118.5 | 420.0 ± 87.2 |
| TN | mg/l | 96.1 ± 36.7 | 176.3 ± 35.6 | 104.1 ± 10.2 | 114.0 ± 22.1 | 73.8 ± 21.9 |
| $NH_4^+$-N | mg/l | 78.0 ± 28.5 | 82.6 ± 23.4 | 87.3 ± 9.0 | 84.8 ± 15.3 | 7.1 ± 13.7 |
| $NO_3^-$-N | mg/l | 0.1 | 1.3 ± 0.3 | 0.8 ± 0.2 | 1.0 ± 0.4 | 0.9 ± 0.1 |
| Org-N | mg/l | 16.6 ± 3.9 | 90.8 ± 26.8 | 16.2 ± 5.3 | 27.9 ± 8.9 | 25.9 ± 7.3 |
| TP | mg/l | 14.6 ± 3.9 | 15.3 ± 0.8 | 15.2 ± 0.7 | 20.1 ± 1.2 | 11.9 ± 0.9 |

*n* Number of samples

**Table 4.27** Average concentrations of characteristic pollutants in the outflowing

| Parameters | Unit | Schodno, $n^x = 18$ | Darżlubie, $n^x = 21$ | Wiklino, $n^x = 88$ | Wieszyno, $n^x = 18$ | Sarbsk, $n^x = 38$ |
|---|---|---|---|---|---|---|
| TSS | mg/l | 48.6 ± 20.1 | 92.0 ± 27.3 | 36.3 ± 17.2 | 106.4 ± 31.7 | 45.6 ± 49.9 |
| COD | mg $O_2$ l | 178.1 ± 38.1 | 210.5 ± 67.8 | 31.5 ± 8.9 | 175.9 ± 99.3 | 44.2 ± 15.9 |
| $BOD_5$ | mg $O_2$/l | 96.6 ± 20.1 | 72.0 ± 21.4 | 10.9 ± 4.1 | 85.9 ± 53.6 | 19.0 ± 1.7 |
| TN | mg/l | 37.2 ± 9.9 | 56.5 ± 16.9 | 21.7 ± 5.5 | 87.3 ± 14.8 | 27.6 ± 8.5 |
| $NH_4^+$-N | mg/l | 30.6 ± 8.7 | 30.3 ± 11.5 | 6.0 ± 4.3 | 67.1 ± 14.2 | 16.8 ± 11.2 |
| $NO_3^-$-N | mg/l | 0.3 | 5.9 ± 2.8 | 9.6 ± 6.7 | 0.6 ± 0.3 | 5.03 ± 9.38 |
| Org-N | mg/l | 7.2 ± 1.3 | 22.5 ± 5.6 | 4.3 ± 1.7 | 19.53 ± 12.91 | 5.8 ± 2.3 |
| TP | mg/l | 3.5 ± 0.9 | 6.9 ± 2.1 | 7.2 ± 1.6 | 14.6 ± 3.9 | 8.9 ± 3.1 |

*n* Number of samples

regime. In the summer time the treated wastewater was used for the irrigation of the willow plantation (*Salix viminalis*) while in winter sewage was discharged into the soil through a drainage system. Monitoring was performed between 1998 and 2006 and involved the collection of monthly taken wastewater samples in the following points: raw inflow (inflow to the HTW system, after primary treatment), after each stage of treatment and in the final outflow before the discharge point.

The quality of the inflow wastewater for the five HTWs systems differed significantly. The facility in Wieszyno was supplied with wastewater of the highest pollutants concentrations. Very high concentrations of TSS, $BOD_5$ and COD were measured in the inflow to the biological treatment facilities, which may indicate an improper performance of the septic tank. On the other hand, it seems that considerable change in the quality of the inflow to HTW occurred in Schodno plant, since high values of standard deviation were observed (reaching 30 %). The observation of very high organic matter concentrations in the inflow to the Darżlubie and Schodno HTWs suggests that specific kind of sewage was added to domestic wastewater, probably manure liquid from single farm and/or sewage from food industry.

According to present Regulation of Environment Ministry from 24th July 2006 sewage discharged from treatment facilities serving 50–2,000 pe should meet the following standards: $BOD_5 \le 40$ mg $O_2$/l, COD $\le 150$ mg $O_2$/l, and TSS $\le 50$ mg/l. If the discharge point is located in a water body classified as "sensitive" to eutrophication additional nutrient removal should be included in order to meet the following standards: TN $\le 30$ mg N/l, TP $\le 5$ mg P/l (Regulation of Environment Minister from 24th July, 2006, Dz. U. Nr 137, item 984). The final outflow from treatment systems in Wiklino and Sarbsk met Requirements in Regulation of Environment Minister from 24th July, 2006. All the other facilities, in spite of considerable efficiency of pollutants removal, did not provide proper quality of outflow.

The highest organic matter (expressed as COD) removal efficiency was obtained in Wiklino and the lowest in Darżlubie. The ability of the HTWs to organic matter removal decreased as follows:

| | Wiklino | Sarbsk | Wieszyno | Schodno | Darżlubie |
|---|---|---|---|---|---|
| COD removal | 95.5 % > | 93.6 % > | 84.7 % > | 79.8 % > | 74.9 % |

Good removal efficiency of organic matter in all analysed HTWs was observed. For total nitrogen, four of the units reached removal efficiency between 80 and 60 %. In Wieszyno plant the efficiency of total nitrogen removal was very low (near 20 %). The comparison of the average TN removal decreased as follows:

| | Wiklino | Darżlubie | Sarbsk | Schodno | Wieszyno |
|---|---|---|---|---|---|
| TN removal | 79.2 > | 67.9 % > | 62.6 % > | 61.3 > | 23.4 % |

The highest efficiency of nitrogen removal was observed in Wiklino facility in which compartments of the VSSF bed were working intermittently and wastewater was supplied periodically. Likewise to Schodno facility, (where compartments of

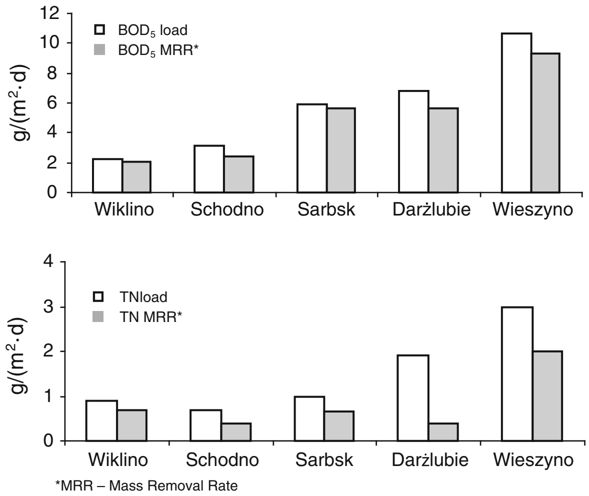


**Fig. 4.13** The comparison of discharged and removed (MRR) loads of pollutants in the analysed HTWs

the VSSF bed were supplied intermittently by means of pumps) high efficiency of TN removal was observed despite very high concentrations of that pollutant in the inflow. It confirms the earlier conclusion that the HTWs are favorable for nitrogen removal. Moreover, the configuration with intermittent loading to VSSF beds which worked alternately with resting periods (e.g. Wiklino) was especially efficient for both nitrogen and organic matter removal (Obarska-Pempkowiak and Gajewska 2005; Gajewska and Obarska-Pempkowiak 2011).

The mean values of organic matter ($BOD_5$) and nitrogen loads discharged to the analysed HTWs in comparison to removed loads (expressed as mass removal rate (MRR)) from 1 $m^2$ are given in Fig. 4.13.

The range of loadings applied in HTWs was wide. The lowest loaded facility was Schodno, whereas Wieszyno had almost ten times higher pollutants loadings. However, in all analysed HTWs maximum allowable loadings given in the literature (for COD = 40 g/($m^2$·day) and TN = 20 g/($m^2$·day), were not exceeded (Langergraber et al. 2006; Sardon et al. 2006). In TWs in Spain HSSF beds load changed from 0.8 to 23.0 g $BOD_5$/($m^2$·day), and for VSSF beds from 12.8 to 29.8 g $BOD_5$/($m^2$·day). According to the Puigagut et al. (2007), description those beds removed adequately: 80.0 and 95.0 % $BOD_5$ loading. In analysed HTWs organic matter bed loading changed from 0.8 (Schodno) to 10.7 g $BOD_5$/($m^2$·day) (Wieszyno) while removal efficiency ranged from 78.5 % (Schodno) to 95.9 % (Wiklino). Basing on analysis of the obtained results it could be concluded that HTW in Schodno, in spite of the lowest loading values, did not provide the highest pollutants removal efficiency. On the contrary, considerably higher organic matter loading in Wieszyno was resulting in poor nitrogen compounds removal efficiency despite satisfactory organic matter removal. The highest MRR of TN was obtained in Darżlubie HTW in spite of quite high values of organic matter loading (Fig. 4.10). The MRR almost three times exceeded the value of 0.7 g N/($m^2$·day) accepted for systems in Danmark (Brix et al. 2003). Obtained results suggest that the MRR of pollutants from 1 $m^2$ changed in proportion to the loading value. At the same time, organic matter in hybrid treatment wetlands was removed effectively in the wide loading range, respectively to the HTW configuration applied. However, the MRR of total nitrogen was related to the applied configuration in a higher degree than to the values of nitrogen loading applied (Obarska-Pempkowiak and Gajewska 2005; Gajewska and Obarska-Pempkowiak 2011).

Seasonal changes in quality of wastewater inflow and outflow were estimated based on monitoring results separated into vegetative (from May to October) and post-vegetative periods. The average results are presented in subsequent figures (Fig. 4.14a, b).

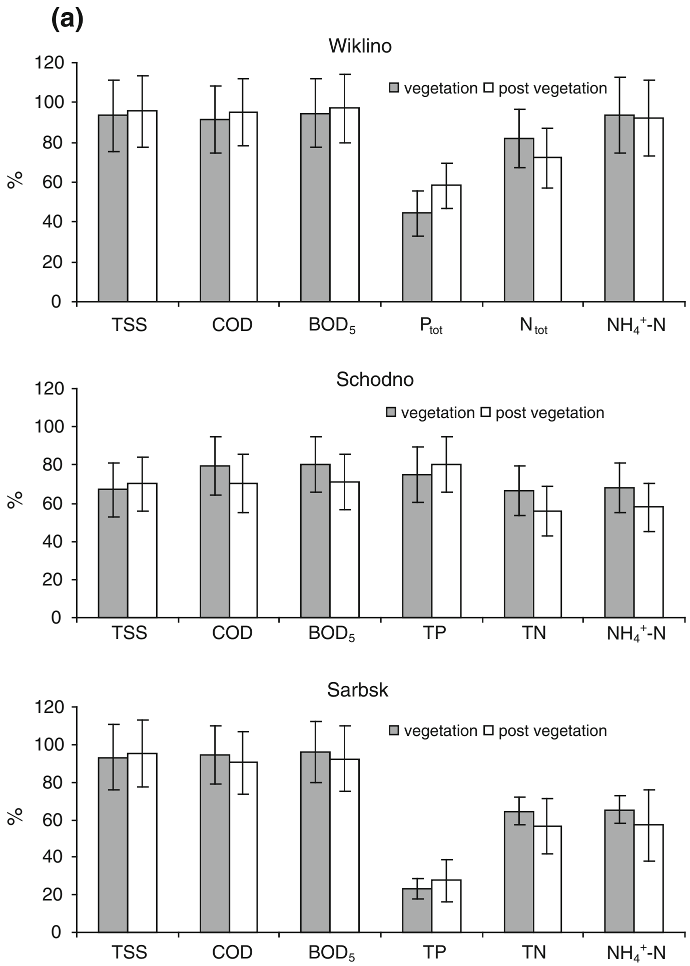


**Fig. 4.14** **a** The comparison of pollutants efficiency removal in vegetation and post-vegetation seasons (Wiklino, Schodno, Sarbsk). **b** The comparison of pollutants efficiency removal in vegetation and post-vegetation seasons (Darżlubie, Wieszyno)

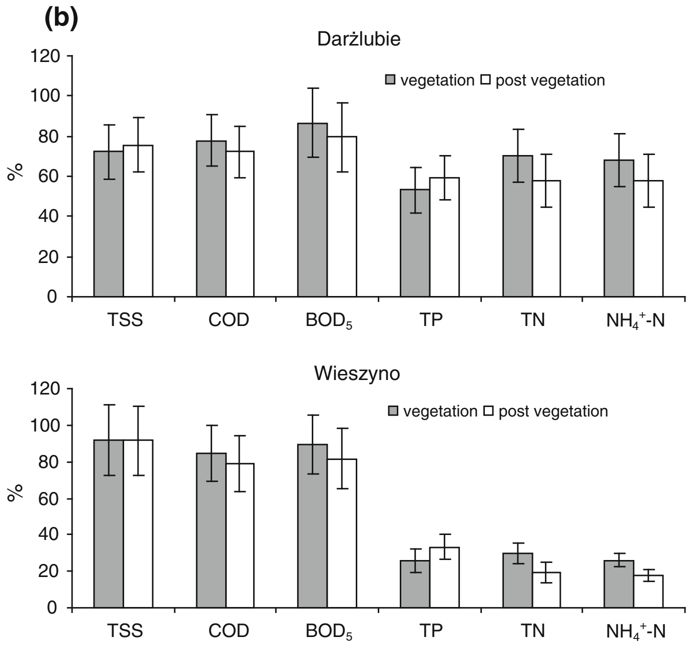


**Fig. 4.14** (continued)

There were not observed significant differences for organic matter removal. In case of total nitrogen about 10 % higher removal efficiency were observed in vegetation season in facilities treating wastewater from local plants.

Other differences were relatively small and could be caused either by flow irregularity or concentrations fluctuations of the supplied pollutants. Based on carried out investigation the following conclusion can be drown:

- hybrid treatment wetlands allowed a stable and effective removal of organic matter in the load range from 1.5 to 17.0 g COD/($m^2$·day), independently of the bed configuration.
- mass removal rates of the total nitrogen ranged from 0.4 to 2.0 g TN/($m^2$·day) and were dependent on both bed configuration of HTWs and sewage supply regime.
- the organic matter and total phosphorus removal efficiencies were not depend on season (Obarska-Pempkowiak and Gajewska 2005; Gajewska and Obarska-Pempkowiak 2011).

## 4.3 SF Facilities

In general, SF facilities do not serve as the second stage domestic wastewater treatment facilities. In Europe and the USA, however, TWs constructed at the early stage of development are still working. They can be treated as reference systems for the estimation of the treatment effectiveness of hydrophyte plants. In some cases (when sewage load is too heavy) it can be beneficial to extend the second stage of the existing treatment plants with a treatment wetland system. As a result, it can be possible to fulfil the requirements for outflow water quality. Such facilities in Columbia, Missouri were described by Broome et al. (1993).

In some regions, facultative lagoons as the second stage of the treatment process were used. During their operation, however, some problems occurred. They were solved by the implementation of treatment wetland systems. One of the problems connected with lagoon exploitation was the development of algae and the increase of the concentration of suspended solids in the wastewater outflow during hot days in the summer. Because of that, additional SF systems were installed as the third stage of treatment in Ouray, Colorado USA (Andrews and Cockle 1996). The operational results from the Ouray system are presented in Fig. 4.15. Using a lagoon in summer conditions was insufficient to meet regulations for suspended solids (max. 30 mg/l). The implementation of a treatment wetland was the solution to that problem.

In Poland the SF system used for domestic wastewater treatment was located in Frombork.

Since 1985 sewage has been directed from an Imhoff tank to a field of reeds. Sewage was discharged through a ∅ 200 pipe (Fig. 4.16). Then it flowed through

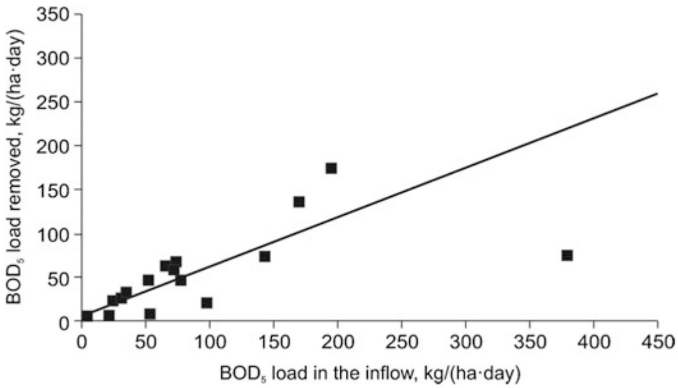


**Fig. 4.15** Unit loads of $BOD_5$ in the inflow and load removal in the SF system in Ouray, Colorado USA, composed of wetland situated after a lagoon (Andrews and Cockle 1996)

the field irregularly. The flow to the sewage treatment plant was 850 $m^3$/day. The biological treatment process took place in an area separated from the Vistula Lagoon (in Northest Poland).

The monitoring results of the investigation conducted in an experimental bed proved that the removal of organic matter inflowing with sewage both in the summer and in the winter was 50 % (Rajkiewicz 1987). In the winter the bed was covered with snow and partially with ice. The outflow water was transparent but with symptoms of full deoxidation.

Research into pollutant removal efficiency was carried out in 1990 (Obarska-Pempkowiak 1991). Samples of wastewater after the Imhoff tank and from the outflow of the reed bed were taken once a week (4–5 times per month). In the summer season (mid June–mid July) samples were taken more frequently (once a day on average). In this period the sewage inflow and outflow were characterized by a relatively stable composition of all the measured pollutants.

The average monthly concentration of suspended solids and $BOD_5$ and COD in the SF system inflow and outflow is presented in Fig. 4.17, concentrations of nitrogen and phosphorus compounds are presented in Fig. 4.18.

In Table 4.28 average monthly percentage removal efficiencies of the pollutants are given supplemented with a year average. The average monthly and annual percentage values of reed biomass, and both nitrogen and phosphorus contents in biomass (Fig. 4.19). The production of land biomass from reed irrigated with wastewater and the average contents of nutrients (nitrogen and phosphorus) in biomass are substantially layer in the reed field irrigated with wastewater (Fig. 4.20). Schematics of nitrogen and phosphorus loads allocation in the Frombork WWTP are presented in Figs. 4.19 and 4.20.

In 1990 the depth of the sewage layer was up to 0.4 m, whereas in the pipe outlet area the depth was below 0.2 m. Assuming that the average value of the depth of the sewage layer was 0.25 m, and the surface flooded by sewage was 22,100 $m^2$ (Fig. 4.16), the volume of sewage retained in the reed rizosphere was 5,525 $m^3$. With 850 $m^3$/day of inflowing sewage, the retention time in the system was 6.5 days.

The nutrients (nitrogen and phosphorus) balance of reed irrigated during the measurement period is presented in Table 4.30 (can be easily deducted from data presented in Table 4.28). The load of nitrogen and phosphorus compounds inflowing into a quasi-natural reed plantation was the sum of loads supplied in certain months of 1990, calculated on the basis of the quantity of inflowing sewage, and the content of total nitrogen and phosphorus in form of phosphates (Fig. 4.20). The calculations were made according to the following equation:

$$L_x = Q_{out} \cdot C_x$$

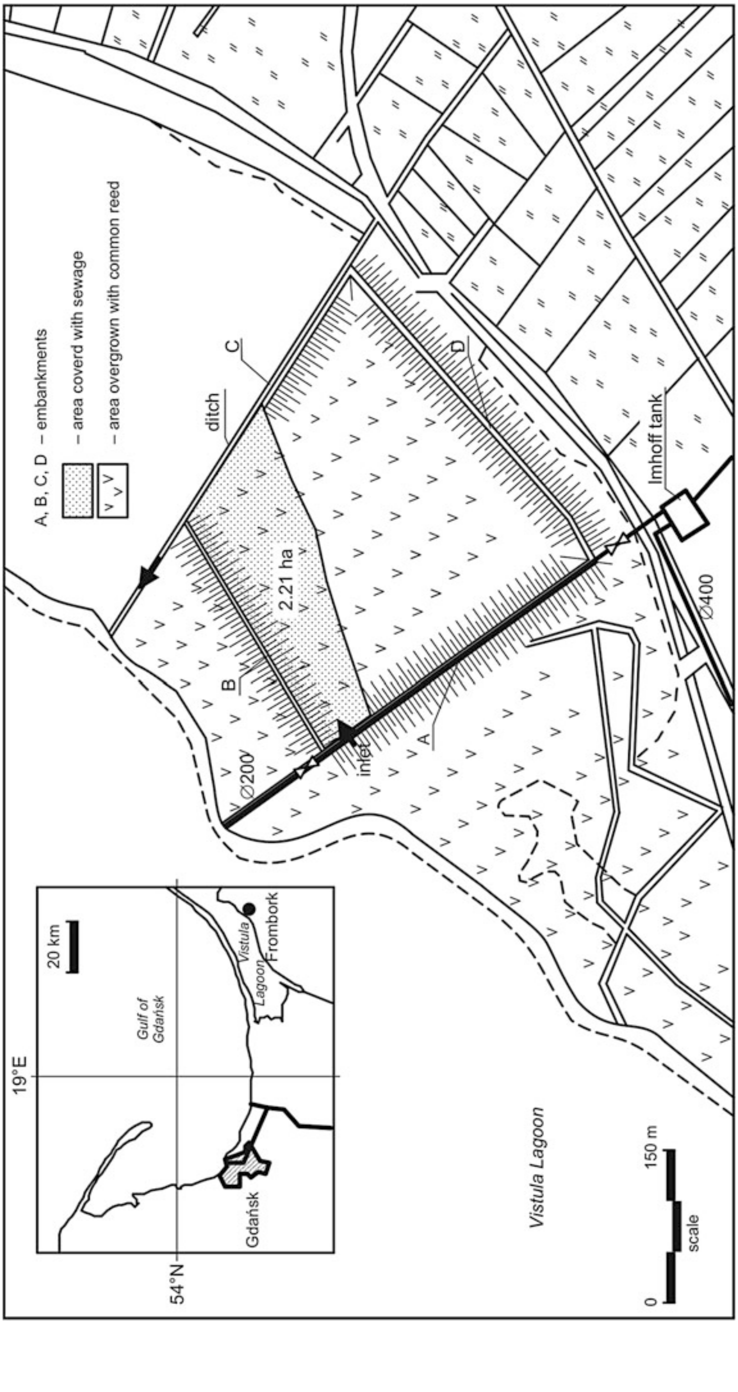


**Fig. 4.16** A scheme of the SF in Frombork, Poland

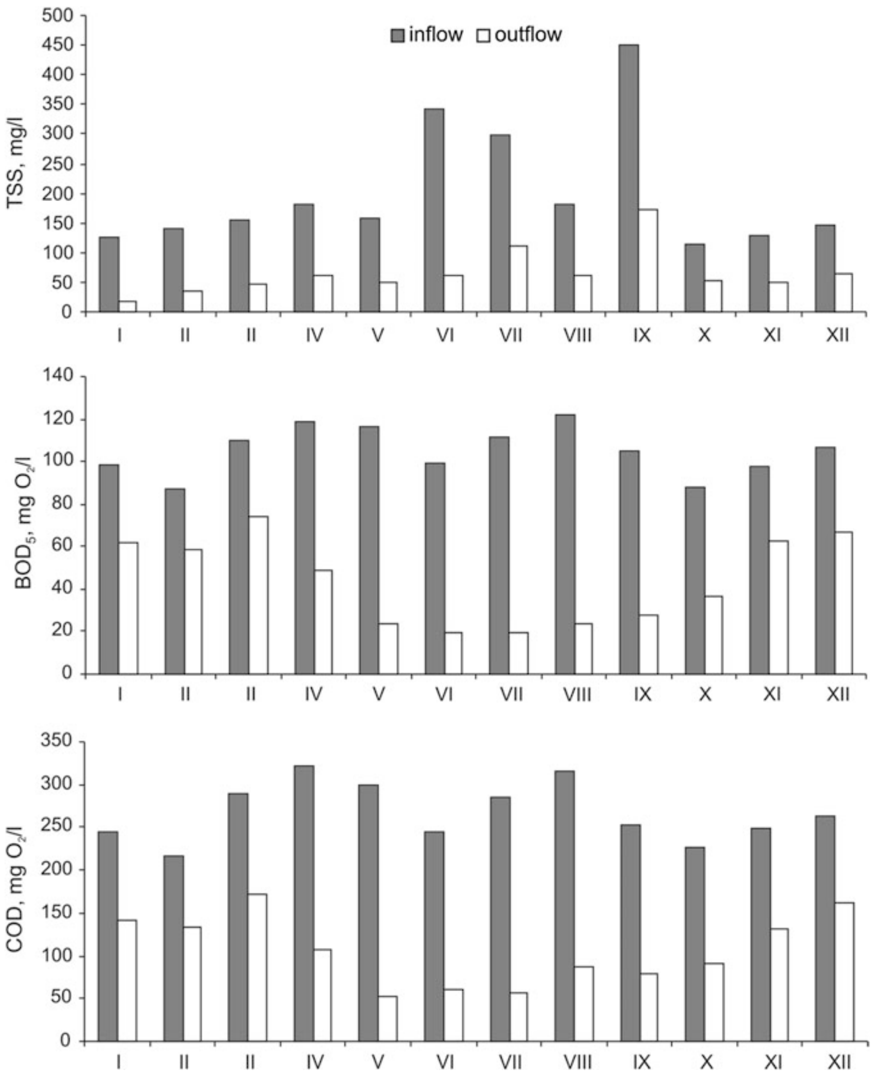


**Fig. 4.17** Average monthly concentrations of TSS and organic matter in the inflow and outflow in the SF system in Frombork, Poland (Obarska-Pempkowiak 1991)

where

$Q_{out}$ hydraulic load of sewage at the outflow to the treatment plant, $m^3$/month,
$C_x$ concentration of total nitrogen or concentration of phosphates, kg/$m^3$,
$L_x$ load of total nitrogen or phosphates, kg/month,

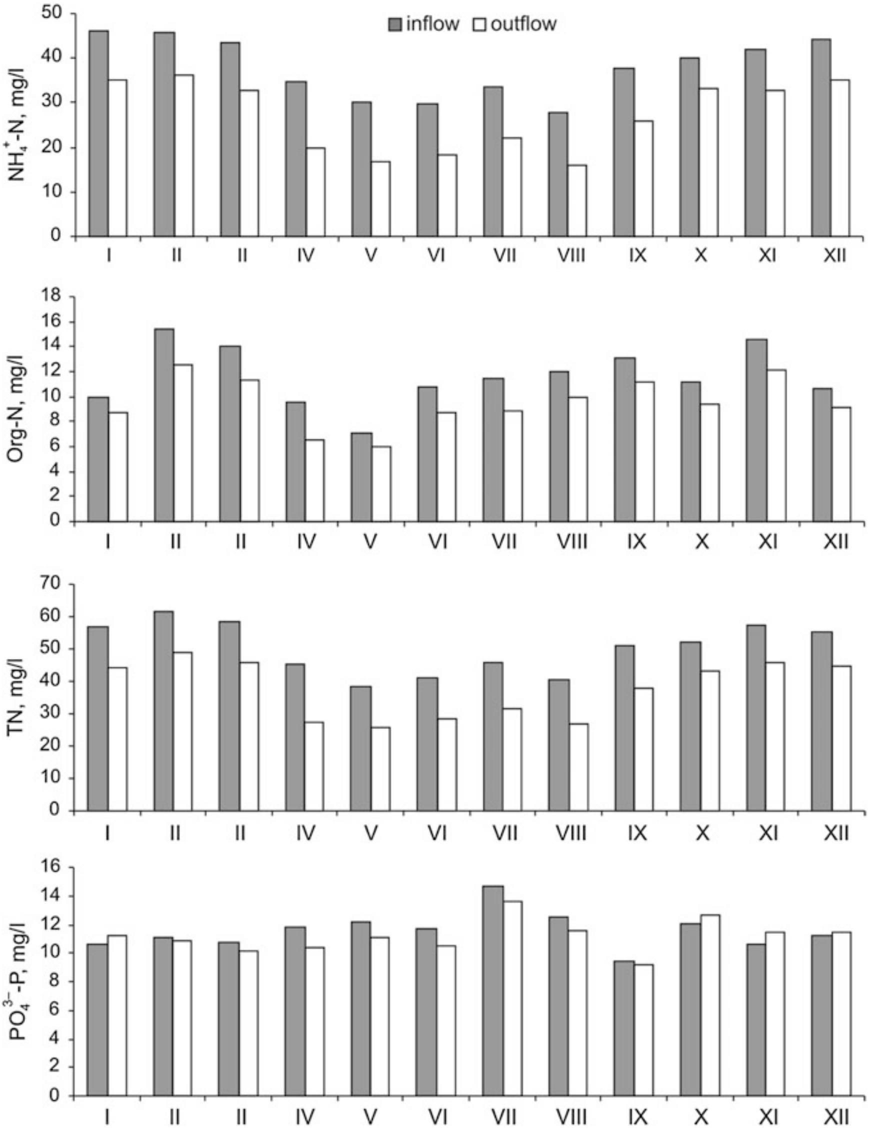


**Fig. 4.18** Average monthly concentrations of nutrients (nitrogen and phosphorus) in the inflow and outflow in the SF system in Frombork, Poland (Obarska-Pempkowiak 1991)

**Table 4.28** Average monthly removal efficiencies of pollutants in the SF system in Frombork, Poland (Obarska-Pempkowiak 1991)

| Parameter | Removal efficiencies (%) | | | | | | | | | | | | Annual average ±SD |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | |
| TSS mg/l | 87.0 | 74.1 | 70.4 | 66.3 | 68.2 | 81.8 | 63.1 | 66.8 | 61.7 | 54.9 | 61.0 | 57.0 | 67.7 ± 9.3 |
| $BOD_5$ mg $O_2$/l | 37.3 | 33.2 | 32.8 | 58.7 | 79.6 | 80.2 | 82.1 | 80.6 | 73.5 | 58.6 | 36.2 | 37.3 | 57.7 ± 21.0 |
| COD mg $O_2$/l | 42.6 | 38.1 | 40.7 | 66.8 | 82.3 | 75.3 | 80.5 | 72.6 | 68.6 | 60.0 | 47.2 | 38.1 | 59.5 ± 17.1 |
| Ammonia nitrogen mg $NH_4^+$-N/l | 24.4 | 21.2 | 24.6 | 43.0 | 44.9 | 37.9 | 34.5 | 42.1 | 31.2 | 17.7 | 22.1 | 20.5 | 30.3 ± 5.2 |
| Organic nitrogen mg Org-N/l | 12.1 | 18.8 | 19.3 | 32.3 | 15.5 | 19.4 | 23.5 | 16.7 | 14.5 | 16.1 | 17.1 | 15.0 | 18.4 ± 5.2 |
| Total nitrogen mg TN/l | 22.1 | 20.4 | 21.9 | 39.6 | 33.2 | 31.0 | 30.4 | 34.6 | 26.3 | 17.1 | 20.1 | 19.3 | 26.3 ± 7.3 |
| Phosphate phosphorus mg $PO_4^{3-}$-P/l | −5.7 | 1.8 | 5.6 | 11.9 | 9.0 | 10.3 | 7.5 | 7.2 | 2.1 | −4.1 | −8.5 | −2.7 | 2.9 ± 6.9 |

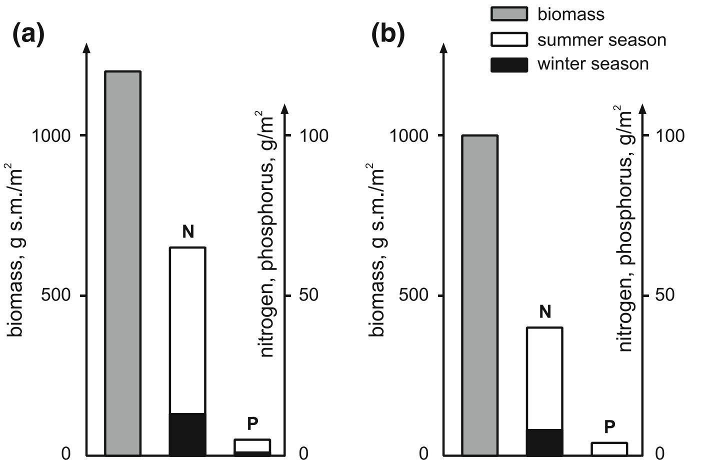


**Fig. 4.19** Contents of nutrients (nitrogen and phosphorus) in **a** a field irrigated with wastewater, **b** a natural field (Obarska-Pempkowiak 1991)

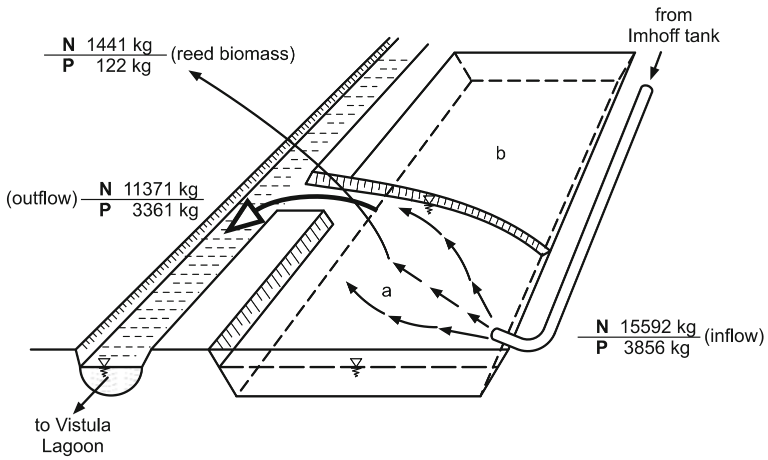


**Fig. 4.20** A schematic of a nitrogen and phosphorus loads in SF system in Frombork, Poland: *a* area irrigated with wastewater, *b* area not irrigated with wastewater

or

$$L_x = 10\,V \cdot A \cdot C_x = \frac{Q}{A} A \cdot C_x$$

where

V hydraulic load, mm/month,

A reed plantation area, ha.

The supplied load of biogenic compounds was also calculated with the use of the equations given above, whereas the quantity of wastewater in the outflow was calculated using the following equation:

$$Q_{out} = \frac{(S + P - E_T)}{1{,}000} F$$

where

$Q_{out}$ quantity of wastewater in the reed outflow, $m^3$/month,
P monthly sum of precipitation, mm,
S sewage hydraulic load of reed, mm,
$E_T$ monthly evapotranspiration of reed field, mm.

Water balance in the water surface of the TW is presented for every month separately in Table 4.29. The presented balance of nitrogen and phosphorus compounds (Table 4.30) shows that reed removes 4,221 kg of nitrogen and 225 kg of phosphorus during a year. Most of the removed nitrogen was absorbed by the system during summer. Phosphorus accumulation took place only in the summer.

The data in Fig. 4.19 show that the accumulation of nutrient compounds in plant biomass was seasonal and equaled 65.2 g N/$m^2$ and app. 5.6 g P/$m^2$ with the obtained biomass harvest equal 1,200 g/$m^2$ in the summer season. The research done in the winter season showed, however, that these compounds were uptaken again by reed in the summer because nitrogen content was only 13.3 g N/$m^2$ and phosphorus app 0.5 g P/$m^2$ then. In Fig. 4.20, the inflow of nitrogen and phosphorus is additionally presented.

Because the sorption process in scarcely dependent on temperature, it could be assumed that an increase of reed activity in the summer indicated the presence of ammonium nitrogen assimilation and denitrification processes. Those results were confirmed by Janota-Besalik and Kermen (1978), and Gambrell and Patrick (1989), who reported that the ammonia ion was absorbable by plants. Wathugala et al. (1987), and Gambrell and Patrick (1989) proved the sorption of the ammonia in soil.

A similar dependence could be observed in the case of total nitrogen, although there were irregular changes of organic nitrogen removal efficiency (value of relative standard deviation is app 25 %, and there is no regular seasonal variation). This together with the ammonia nitrogen removal efficiency makes the seasonal dependence less clear.

The elimination of phosphate phosphorus showed a clear seasonal variation. In winter, negative effectiveness could be observed. Reed field was an exporter of phosphorus compounds. The phosphorus concentration decrease by 10 % was observed only in summer. The average annual phosphorus removal was only 2.7 %, which should be interpreted as the lack of the elimination of phosphorus. The comparison of the nitrogen and phosphorus amount removed from the natural reed

**Table 4.29** Balance of water in the SF system irrigated with wastewater in Frombork, Poland (Obarska-Pempkowiak 1991)

| Element of water balance | Months | | | | | | | | | | | | Σ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | |
| Load of sewages ($m^3$) | 26,350 | 23,800 | 26,350 | 25,500 | 26,350 | 25,500 | 26,350 | 26,350 | 25,500 | 26,350 | 25,500 | 26,350 | 310,250 |
| Outflow from the reed field S (mm) | 1,192 | 1,077 | 1,192 | 1,154 | 1,192 | 1,154 | 1,192 | 1,192 | 1,154 | 1,192 | 1,154 | 1,192 | 14,037 |
| Sum of precipitation P (mm) | 38 | 31 | 31 | 40 | 47 | 62 | 81 | 82 | 69 | 57 | 52 | 48 | 638 |
| Sum of evaporation E (mm) | 15 | 14 | 18 | 30 | 60 | 95 | 98 | 90 | 69 | 34 | 21 | 16 | 560 |
| Evaporation factor for reed $\eta$ | 1.0 | 1.0 | 1.1 | 1.7 | 2.1 | 2.3 | 2.3 | 2.3 | 2.0 | 1.1 | 1.0 | 1.0 | |
| Evapotranspiration of reed, $E_r = E \cdot \eta$ (mm) | 15 | 14 | 20 | 51 | 126 | 219 | 225 | 207 | 138 | 37 | 21 | 16 | 1,089 |
| Water outflow $S + P - E_r$ (mm) | 1,215 | 1,094 | 1,203 | 1,143 | 1,113 | 997 | 1,048 | 1,067 | 1,085 | 1,212 | 1,185 | 1,224 | 13,586 |

**Table 4.30** Balance of nitrogen and phosphorus in the SF system irrigated with wastewater in Frombork, Poland (Obarska-Pempkowiak 1991)

| Element of water balance | Months | | | | | | | | | | | | Σ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | |
| Sewage inflow ($m^3$) | 26,350 | 23,800 | 26,350 | 25,500 | 26,350 | 25,500 | 26,350 | 26,350 | 25,500 | 26,350 | 25,500 | 26,350 | 310,250 |
| Sewage inflow (mm) | 1,192 | 1,077 | 1,192 | 1,154 | 1,192 | 1,154 | 1,192 | 1,192 | 1,154 | 1,192 | 1,154 | 1,192 | 14,037 |
| Load of nitrogen inflow (kg) | 1,491 | 1,471 | 1,539 | 1,158 | 1,009 | 1,046 | 1,204 | 1,073 | 1,308 | 1,370 | 1,461 | 1,462 | 15,592 |
| Load of phosphorus inflow (kg) | 279 | 264 | 282 | 301 | 322 | 298 | 387 | 329 | 240 | 319 | 270 | 295 | 3,586 |
| Sewage outflow (mm) | 1,215 | 1,094 | 1,203 | 1,143 | 1,113 | 997 | 1,048 | 1,067 | 1,085 | 1,212 | 1,185 | 1,224 | 13,586 |
| Sewage outflow ($m^3$) | 26,852 | 24,177 | 26,586 | 25,260 | 24,597 | 22,034 | 23,161 | 23,581 | 23,979 | 26,785 | 26,189 | 27,050 | 300,251 |
| Nitrogen outflow (kg) | 1,184 | 1,190 | 1,212 | 692 | 630 | 624 | 737 | 627 | 906 | 1,157 | 1,200 | 1,212 | 11,371 |
| Phosphorus outflow (kg) | 301 | 264 | 269 | 263 | 273 | 231 | 315 | 274 | 221 | 338 | 301 | 311 | 3,361 |
| Nitrogen removed (kg) | 307 | 281 | 327 | 466 | 379 | 422 | 467 | 446 | 402 | 213 | 261 | 250 | 4,221 |
| Phosphorus removed (kg) | −22 | 0 | 13 | 38 | 49 | 67 | 72 | 55 | 19 | −19 | 9 | −16 | 265 |

system due to bioaccumulation in reed and other processes involved was also a subject of interest (Fig. 4.19). With the system removal load of 1,200 g/$m^2$ and nitrogen content in the surface reed biomass that equaled 65.2 g N/$m^2$ (Obarska-Pempkowiak 1991), it could be calculated that during a year the reed area of 22,000 $m^2$ can retain about 1,720 kg of nitrogen. In the case of phosphorus, the accumulation in biomass in the summer season was 148 kg. On the basis of the literature, it could be assumed that the reed new biomass on one hectare plantation in our climate varies from 10,000 to 15,000 kg day/ha, and concentrations of nutrients in the surface reed biomass range from 4.0 to 5.0 % for nitrogen and from 0.35 to 0.42 % for phosphorus (Granéli 1984; Gries and Garbe 1989; Wathugala et al. 1989). The upper and lower limits of nutrients accumulated in biomass collected from 1-ha area were 400 and 750 kg for nitrogen, and 35 and 63 kg for phosphorus. Thus, for the analyzed reed area (2.2 ha) these limits will be: nitrogen —880 and 650 kg, phosphorus—77 and 139 kg. Nitrogen and phosphorus assimilation by plants in the reed area in Frombork was similar to the upper values of these ranges, which means that biomass assimilated a larger quantity of these elements, which could be caused by their common accessibility. It seemingly showed the role of reed in the removal process of nutrient compounds. Taking into account the fact that after the vegetation season reed dies off, and after that biomass decomposition takes place on the bed bottom, an important role of reed in nutrient compounds balance should be examined. After the vegetation season, the re-emission of the compounds of nitrogen and phosphorus, accumulated in biomass, into an ecosystem of reed should be examined (Fig. 4.20). Similar phenomena could be observed in every natural marsh ecosystem used for wastewater treatment located in temperate climate (Richardson et al. 1978; Gries and Garbe 1989).

It can be assumed that the assimilation of nitrogen and phosphorus by reed was responsible for a small part of real pollutant removal in the TW. This was confirmed by pollutant balance calculated on the basis of the concentration of nutrient compounds calculated per unit area (Fig. 4.20).

The total amount of the nutrients removal was a sum of biomass accumulation in plant biomass, sorption in soil, and additionally, in the case of nitrogen, it was a result of the denitrification process.

On the basis of nitrogen and phosphorus contents in reed biomass during winter and summer periods (Fig. 4.20), it could be calculated that in the case of reed, harvested in October, the following loads of the nutrients could be removed from the system:

$$22{,}100\,\mathrm{m}^2 \cdot 65.2\,\mathrm{g\,N/m}^2 = 1440.9\,\mathrm{kg\,N}$$
$$22{,}100\,\mathrm{m}^2 \cdot 5.5\,\mathrm{g\,P/m}^2 = 121.6\,\mathrm{kg\,P}$$

Since reed was not harvested in the winter, the re-emission of these compounds took place in the analyzed system. The increase of the layer thickness of the necrobiosis plant, which resulted in an additional sorption surface for wastewater

(Richardson et al. 1978; Clymo 1983; Gersberg et al. 1986), was a positive aspect of this process.

The average annual load of nitrogen and phosphorous removed with the reed harvesting in Frombork equals:

$$\frac{15{,}592\ \text{kg}\ \cdot 1{,}000}{22{,}100\ \text{m}\ \cdot 365\ \text{days}} = 1.93\ \text{g}\ N_{tot}/(\text{m}^2\cdot\text{day})$$

and

$$\frac{3{,}568\ \text{kg} \cdot 1{,}000}{22{,}100\ \text{m} \cdot 365\ \text{days}} = 0.44\ \text{g}\ PO_4^{3-}{-}P/(\text{m}^2\cdot\text{day})$$

In publications concerning this subject there are no examples of a natural TW with such high surface loads. The highest acceptable loads according to Nichols (1985) are 0.235 g TN/($m^2$·day) and 0.049 g TP/($m^2$·day). The loads of nutrients removed are then $\eta_{N_{tot}} = 25\,\%$ and $\eta_{P_{tot}} = 30\,\%$ of the loads inflowing.

To ensure better pollutant removal, it is necessary to enlarge the reed area, and regulate flow so as to eliminate still zones and prolong the retention time of wastewater in the soil. The regulation of flow will enable the uniform loading of all the reed area, surrounded with a dyke, that is A = 76,780 $m^2$, for an effective treatment process (Fig. 4.18).

Reed harvested from the area of about 35 ha, used for energetic purposes, will allow for the annual savings of about 300 t of coal (Obarska-Pempkowiak 1992). Simultaneously, the load of nutrients discharged to the Vistula Lagoon will be reduced by about 11.5 t (TN) and 3.4 t (TP), only thanks to the accumulation of these compounds in reed tissues. Taking into account mineralization as well, it would allow for the primary production, reduction of about 180 t/week in summer (Kajak 1979).

## 4.4 Treatment Wetland Systems Applied as the 3rd Stage of Wastewater Treatment

### *4.4.1 SSF Systems*

A lot of information about the systems applied at the third stage of wastewater treatment is currently available from the Severn Trent Water's data base, and also from data bases of other British Waterworks Companies (Cooper 1990). Green and Upton (1995) described the quality of outflow water and removal efficiency of BOD, suspended solids, ammonium nitrogen, organic nitrogen and total nitrogen in the 290 monitored facilities. The research results confirmed that SSF systems with unit surface 1 $m^2$/pe ensured the concentration of organic matter equal to or below 5 mg $BOD_5$/l and suspended solids equal to 10 mg/l in the outflow. Effective nitrification occurred in many facilities. At Severn Trent Water, for the systems applied at the 3rd stage of wastewater treatment, 0.7 $m^2$ pe is usually taken as the designing standard (Green and Upton 1995). A smaller unit surface is designed for a short period of operation and supplementary facilities.

The decision about building the first reed system of new generation in Leek Wootton, Great Britain was taken in 1989. The system was applied at the 3rd stage of wastewater treatment. Initially, these systems were ment as a pilot program, and then they became the standard 3rd stage of wastewater treatment systems working for 1,500 inhabitants (in 1990 the number of inhabitants increased to 2,000). Leek Wootton was chosen because of a suitable number of inhabitants and a sufficient area assigned for building treatment wetland systems. The system serves two villages: Leek Wootton and Hill Wootton with 900 inhabitants altogether. There are also two hotels (guest houses), a golf club and a school. During the modernization of the existing local sewage treatment plant, the pump station in the wastewater inflow was removed, the existing soil filter was equipped with new distribution systems and divided into sections separated with concrete walls. A new settling tank with an automatic system for sludge discharge, and HSSF reed plantations in 3rd stage of wastewater treatment with necessary equipment were built. Monitoring results from Leek Wootton confirmed the capacity of reed beds for removing ammonium nitrogen and total nitrogen (Table 4.31). The concentration of nitrogen compounds in the outflow of the reed bed, during the whole monitoring period, were not much different from the average value of 19.1 mg/l. The total removal efficiency of ammonium nitrogen was 88.0 %, whereas for the sum of nitrates and nitrites it was 37.0 %.

Treatment wetland systems designed by Severn Trent after 1990 have gravel filling (grain diameter 5–10 mm). A special species of reed—*Phragmites australis*, growing in flowerpots, was used for the purpose of uniform planting.

The gravel filling is usually bounded by walls covered by insulating, impermeable material. The proper conditions for the treatment process are met if uniform wastewater flows through the bed. It was found out that particles of humic substances settled mainly near the inflow.

In the United Kingdom, reed beds are now considered as the best solution for the third stage of wastewater treatment in resorts with up to 3,000 inhabitants. The ESE systems have also been introduced in towns with over 11,000 inhabitants (Green and Verhoeven 1999). The results of monitoring the systems confirmed that the unit surface of these systems (0.7 $m^2$/pe) can ensure continuous operation for about 20 years. Then a necessity of gravel exchange will arise (Cooper and de Maeseneer 1996).

In the United States, monitoring results from four treatment wetland systems supplied with wastewater after the 2nd or higher stages of treatment were described by NADP (1993), and Kadlec and Knight (1996). The achieved results for both SSF and SF systems are presented in Table 4.32.

**Table 4.31** Average annual concentrations of pollutants in the inflow and outflow from the HSSF beds applied in 3rd stage of treatment at Leek Wootton, England (Cooper and de Maeseneer 1996)

| Year | $BOD_5$, mg/l | | COD, mg/l | | TSS, mg/l | | $NH_4^+$-N, mg/l | | NOx-N[a], mg/l | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow | Inflow | Outflow |
| 1990/1991 | 11.6 | 4.8 | 75.7 | 32.1 | 27.6 | 5.1 | 7.6 | 5.8 | 32.8 | 23.4 |
| 1991/1992 | 11.9 | 2.0 | 76.7 | 34.0 | 19.1 | 3.7 | 5.4 | 1.9 | 29.7 | 20.8 |
| 1192/1993 | 15.4 | 2.7 | 109.0 | 55.5 | 24.2 | 5.3 | 7.0 | 2.8 | 20.4 | 8.7 |
| 1993/1994 | 9.1 | 1.5 | 93.8 | 48.3 | 16.3 | 4.4 | 7.2 | 3.0 | 25.6 | 16.8 |
| 1994/1995 | 9.1 | 1.0 | 82.1 | 46.6 | 18.4 | 4.5 | 6.6 | 1.9 | 25.7 | 18.4 |

[a] *OCN* Oxygen compounds of nitrogen ($NO_2^-$ -N + $NO_3^-$ -N)

**Table 4.32** Average concentrations of pollutants in the inflow and outflow from TWs in North America (Kadlec and Knight 1996; NADP 1993)

| Parameter | Type | Average concentration, mg/l | | | | Surface load, kg/(ha day) | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Infl. | Outfl. | Treatment effic. (%) | No. of samples | Supplied load | Removed load | Treatment effic. (%) | No. of samples |
| $BOD_5$ | SF | 30.3 | 8.0 | 74 | 182 | 7.2 | 5.1 | 71 | 133 |
| | SSF | 27.5 | 8.6 | 69 | 34 | 29.2 | 18.4 | 63 | 29 |
| | Both | 29.8 | 8.1 | 73 | 216 | 10.9 | 7.5 | 68 | 162 |
| TSS | SF | 45.6 | 13.5 | 70 | 198 | 10.4 | 7.0 | 68 | 139 |
| | SSF | 48.2 | 10.3 | 79 | 34 | 48.1 | 35.3 | 74 | 29 |
| | Both | 46.0 | 13.0 | 72 | 232 | 16.8 | 11.9 | 71 | 168 |
| $NH_4^+$-N | SF | 4.88 | 2.23 | 54 | 220 | 0.93 | 0.35 | 38 | 141 |
| | SSF | 5.98 | 4.51 | 25 | 19 | 7.02 | 0.62 | 9 | 15 |
| | Both | 4.97 | 2.41 | 52 | 239 | 1.46 | 0.38 | 26 | 156 |
| $NO_i$-N + $NO_3^-$-N | SF | 5.56 | 2.15 | 61 | 187 | 0.80 | 0.40 | 51 | 125 |
| | SSF | 4.40 | 1.35 | 69 | 13 | 3.10 | 1.89 | 61 | 13 |
| | Both | 5.49 | 2.10 | 62 | 200 | 0.99 | 0.54 | 55 | 138 |
| Org-N | SF | 3.45 | 1.85 | 46 | 118 | 0.90 | 0.51 | 56 | 76 |
| | SSF | 10.11 | 4.03 | 60 | 11 | 7.28 | 4.05 | 56 | 11 |
| | Both | 4.01 | 2.03 | 49 | 129 | 1.71 | 0.95 | 56 | 87 |
| $N_K$ | SF | 7.60 | 4.31 | 43 | 144 | 2.20 | 1.03 | 47 | 94 |
| | SSF | 14.21 | 7.16 | 50 | 12 | 9.30 | 3.25 | 35 | 12 |
| | Both | 8.11 | 4.53 | 44 | 156 | 2.99 | 1.29 | 43 | 106 |
| TN | SF | 9.03 | 4.27 | 53 | 175 | 1.94 | 1.06 | 55 | 114 |
| | SSF | 18.92 | 8.41 | 56 | 12 | 13.19 | 5.85 | 44 | 12 |
| | Both | 9.67 | 4.53 | 53 | 187 | 2.98 | 1.52 | 51 | 126 |

(continued)

**Table 4.32** (continued)

| Parameter | Type | Average concentration, mg/l | | | | Surface load, kg/(ha day) | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Infl. | Outfl. | Treatment effic. (%) | No. of samples | Supplied load | Removed load | Treatment effic. (%) | No. of samples |
| $PO_4^{3-}$-N | SF | 1.75 | 1.11 | 37 | 148 | 0.29 | 0.12 | 41 | 112 |
| | SSF | n.c. | n.c. | n.c. | – | n.c. | n.c. | n.c. | – |
| | Both | 1.75 | 1.11 | 37 | 138 | 0.29 | 0.12 | 41 | 112 |
| TP | SF | 3.78 | 1.62 | 57 | 191 | 0.50 | 0.17 | 34 | 134 |
| | SSF | 4.41 | 2.97 | 32 | 8 | 5.14 | 1.14 | 22 | 8 |
| | Both | 3.80 | 1.68 | 56 | 199 | 0.73 | 0.22 | 31 | 142 |

### 4.4.2 SF Systems

SF systems applied in North America are usually supplied with domestic wastewater after the 2nd or a higher stage treatment. In the USA there are hundreds of SF systems, which are used for wastewater treatment after the 2nd or 3rd stage treatment (NADP 1993; Kadlec and Knight 1996). SF systems are beneficial because of the high effectiveness of treatment. According to Knight et al. (1993) and Richardson and Nicholas (1985), the proper retention time of wastewater in the system and TN concentration in the inflow are the factors that condition effective pollutant removal.

The research carried out by Richardson and Craft (1993) in Everglades, south Florida, in 1979–1988 aimed at defining the removal degree of phosphorus compounds that were flowing down from agricultural areas. Investigations were carried out in the marsh ecosystem of Water Conservation Areas (WCA), whose surface (3,500 $km^2$) is planted with reed, *Typha* and other plants. The concentration of phosphorus in soil was one to five times higher in the areas dominated by reed and *Typha* than in the areas with other plants. It was proven that the maximum degree of phosphorus accumulation was in 1979–1988 and equalled 0.56 g/($m^2 \cdot$year) (Richardson and Craft 1993).

Wastewater treatment in s system that consisted of the treatment wetland and the natural marsh ecosystem with a surface flow, in Palmico City, North Carolina, USA, investigated in 1989, was described by House et al. (1996). The role of the bed was to ensure aerobic conditions for the nitrification process, and to create proper conditions for decreasing the concentration of phosphorus compounds in wastewater. The nitrogen load in wastewater discharged to the bed was: $NH_4^+$-N 35.8 mg/l and organic nitrogen 8.6 mg/l. After passing the bed, the nitrogen present in wastewater was in the form of nitrate ions: $NO_3^-$-N 90.4 mg/l). According to House et al. (1996) the removal of TN was equal to 64.1 % (from 44.4 to 16.0 mg/l), and was caused by the nitrification processes, bioaccumulation in plant biomass, diffusion to the atmosphere and by diluting with precipitation water. Phosphorus removal from wastewater took place first of all after passing the natural marsh ecosystem, where phosphorus concentration decreased from 4.4 to 0.6 mg/l (House et al. 1996).

### 4.4.3 Treatment Wetland for Tertiary Wastewater Treatment at Wieżyca

The wastewater treatment system at Wieżyca has been constructed for several recreation centres located in the Kashubian Lake District, in the upper part of the Radunia river catchment area in Poland (Fig. 4.21). Since the Radunia river is the source of drinking water for the city of Gdansk, requirements for the outflow from wastewater treatment systems have become the strictest in this region. The design of a

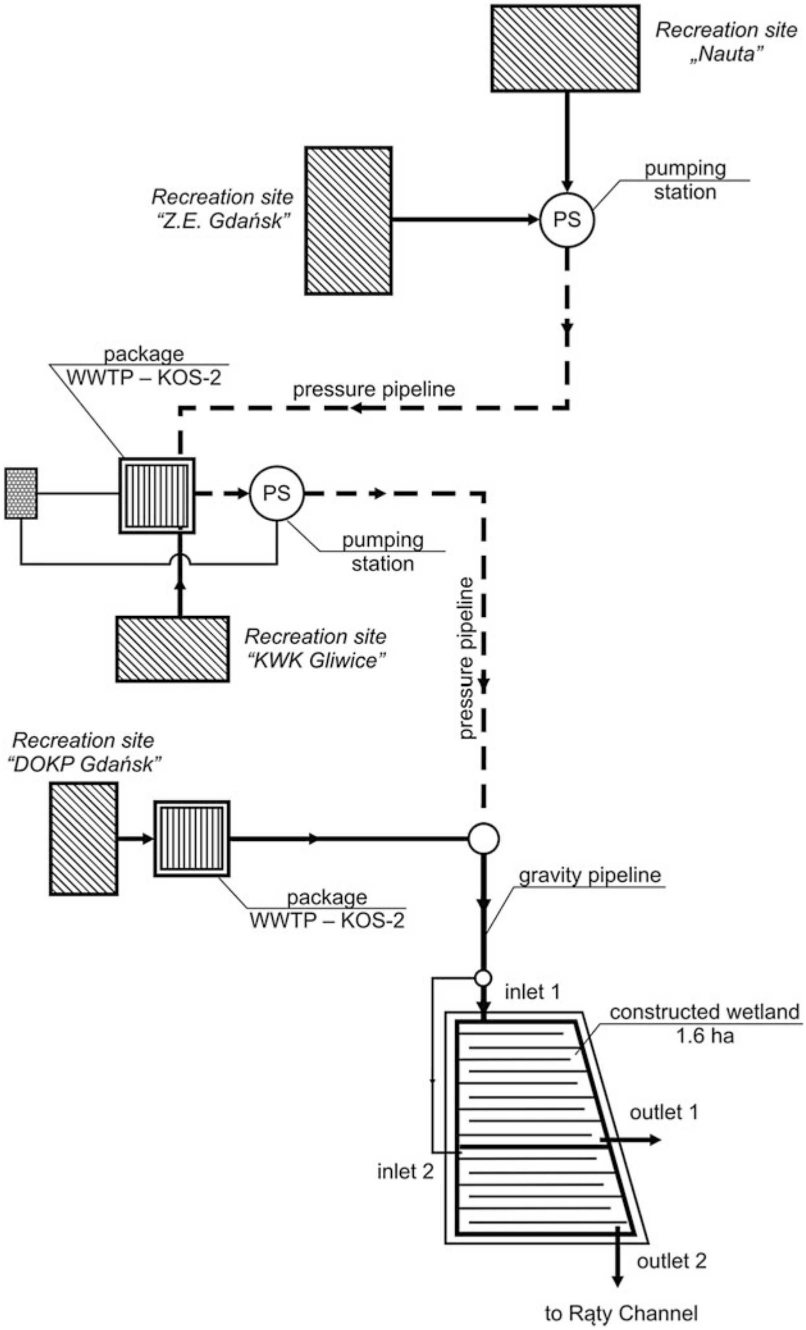


**Fig. 4.21** Scheme of the sewerage and wastewater treatment system at Wieżyca in Poland

wastewater treatment system prepared by an architectural design firm, included four individual secondary wastewater treatment systems for several recreation centres situated nearby. These were KOS-2 container mechanical-biological treatment facilities consisting of rotary trickling filters with primary aeration and sewage recirculation. The outflow parameters achieved by applying this technology, however, did not meet the requirements for the Radunia river catchment area. Thus, an upgrading of wastewater treatment was necessary. The tertiary wastewater treatment was achieved in a constructed surface flow wetland system.

The treatment wetland was designed to provide additional pollutant removal for the secondary outflow of domestic wastewater, after KOS-2 treatment facilities, characterized by the following concentration of pollutants (Kowalik et al. 1995): TSS—25–40 mg/l, $BOD_5$—30 mg/l, $NH_4^+$-N—8–12 mg/l, $NO_3^-$-N—10–23 mg/l, TN—30 mg/l and $PO_4^{3-}$—10 mg/l. The system was calculated for a population equivalent of 650 persons, with a peak during the summer, and with an average flow of 120 $m^3$/day from the secondary facilities designed. The permitted limit requirements for the wetland system outflow have to comply with 2nd class surface water quality of the following parameters: TSS ≤ 30 mg/l, $BOD_5$ ≤ 8 mg/l, $NH_4^+$-N ≤ < 3.0 mg/l, N-$NO_2^-$ ≤ 0.03 mg/l, N-$NO_3^-$ ≤ 7.0 mg/l, TN ≤ 10.0 mg/l and $PO_4^{3-}$ ≤ 0.6 mg/l.

Wetlands all over the world have shown a capacity to remove BOD, TSS, to nitrify and to denitrify. However, the research carried out all over the world shows different treatment capacity. There is still a considerable debate over how these processes occur and to what extent they take place. Universally acceptable design criteria are non-existent. At present, there are few and varied design criteria available, which can be used for the design of treatment wetlands, especially of the surface flow type. The main design criteria taken into account and accepted for the system are as follows (Nichols 1983; Watson et al. 1987):

- area requirement (given range 2.5–23 $m^2$/pe)—about 20 $m^2$/pe,
- retention time (range 10–20 days)—min. 20 days,
- length/width ratio of flow cell (L/W > 30)—>60,
- depth (0.15–0.60 m)—average 0.3 m,
- hydraulic loading rates (0.8–62 cm/day)—1.5 cm/day.

Surface flow systems are usually less loaded. Surface flow wetlands are considered to be less efficient than a subsurface system because of smaller quantities of attached microorganisms taking part in the treatment process. The area used as a treatment wetland is a low-lying land with a very small slope, which was once used as a natural swampy meadow. Taking into account the site conditions, topography, substrate soils, elevation of land etc., an emergent macrophyte-based system planted with common reed (*Phragmites australis*), and with a surface flow was selected. One of the main advantages of this system was its relatively low capital cost as well as low maintenance cost. This treatment wetland system has a form of a pond with a total area of about 1.6 ha divided into two compartments for better operation control (Fig. 4.22). In the compartments, the finger dikes were designed to extend the flow path, to minimize short-circuiting, and to maximize wastewater contact with the entire area. Thus, the water surface between the dikes consists of sequences of

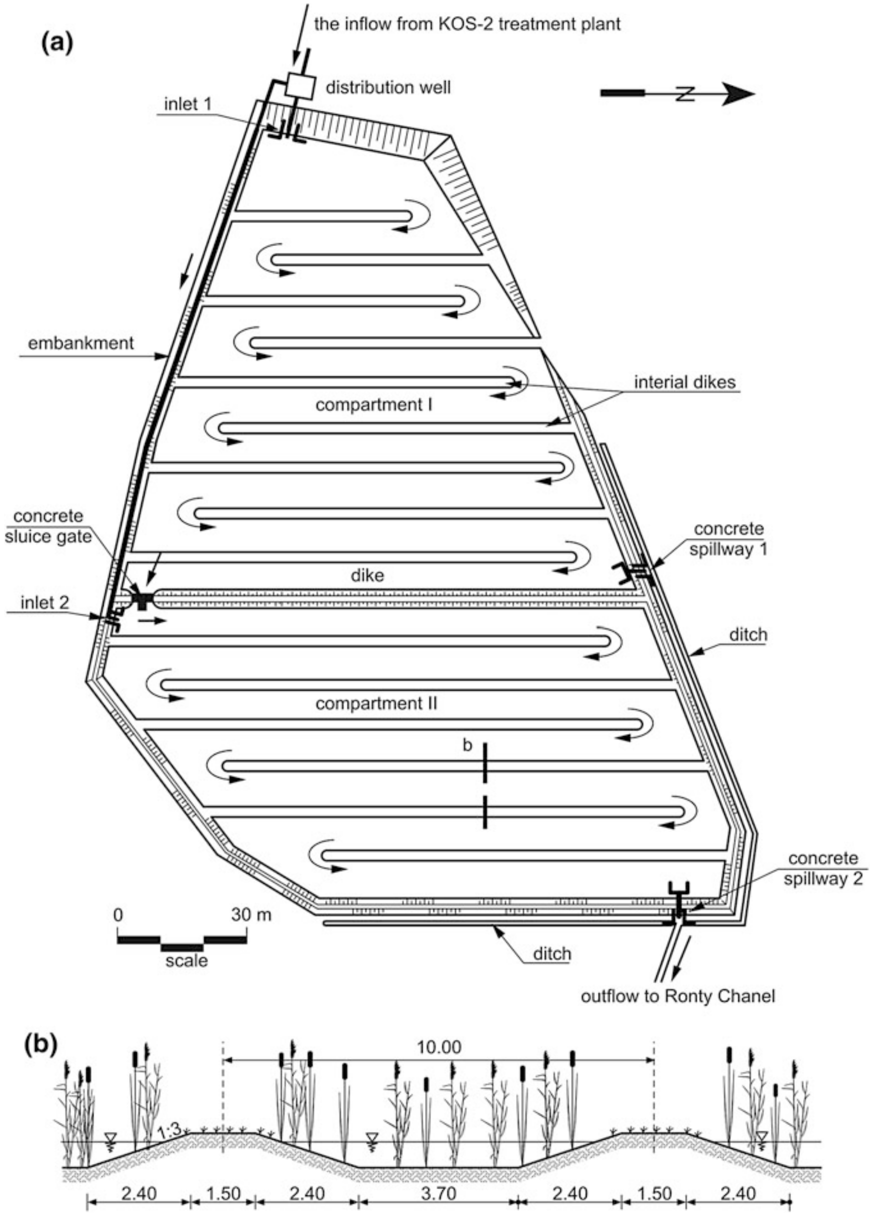


**Fig. 4.22** Plan view of the treatment wetland at Wieżyca and cross section of finger dikes (Obarska-Pempkowiak et al. 1994)

60–130 m long, and 4–7.3 m wide ditches of shallow water—0–0.60 m deep. Low flow speed and the presence of plant stalks and leaves regulate the flow in long narrow ditches and ensure plug flow conditions. Moreover, the inner dikes were thought to enlarge the contact of wastewater with soil material, which means that the dikes are saturated with wastewater due to capillary potential and soil suction between the bottom of the ditches and the evaporating surface of the dikes, especially during dry summer periods.

The upper layer of the wetland system consists of hydrogenic peaty and muddy soils, lying on relatively impermeable loamy soils. The treatment wetland makes use of natural topography. Native excavated material is used to build embankments and inner dikes. The wetland vegetation (*Phragmites australis*) has been planted on natural filling. The top width and side slope of the dikes were determined by the soil material. The design of the external pond dams was based on standard engineering practices. The ponds were divided into two compartments, which enabled the operational use of the pond as a whole. Thus, both compartments were used simultaneously or alternatively, if necessary. The run-off was routed around the pond with drainage ditches and a stream flowing near the pond.

All the construction work was carried out between March and May. The bottoms of the wetland ditches were planted with clumps of *Phragmites australis*, at a density of 4 clumps per $m^2$, between May and June. The tops and side slopes of all the dikes were sown with grass. The plants were kept wet and allowed to become established for several months before secondary wastewater was introduced. Some quantity of a fertilizer was added to the wetland within 1 week of completing the planting process to ensure that the reeds stood a good chance of survival. Reed grew well, and at the end of the growing season they reached a height of about 1.5 m.

The first secondary treatment facility KOS-2 was put into operation in September 1991, and since then the treatment wetland system has been fed with wastewater at inlet 1, and the pond has been operating as designed. The hydraulic loads have been fluctuating from 2 $m^3$/day (0.25 mm/day) to 19 $m^3$/day (2.7 mm/day), in wintertime and summertime respectively.

The initial intention was to use the pond as a whole. However, after the first year of operation there was a clearly significant gradient of plant biomass in the direction from the inlet to the outlet, and poor reed growth in the ditches at the far end of compartment 2, near the outlet. This was probably due to a shortage of nutrients, and low water depth (close to zero) towards the outlet. In this period, it was observed that there was no outflow from the wetland. Thus, from the summer of 1992, wastewater was discharged into the second compartment and thereafter both compartments were loaded by turns.

The growth of reed was quite satisfactory, but in the ditches near the inlets, and at some stances in the wetland, self-sown cattail (*Typha latifolia*) was the dominant vegetation. Besides, a thick mat of duckweed (*Lemna minor*) was slowly developing in the open-water regions of the pond. In the upper part of the second compartment, also numerous tufts of rush (*Juncus*) and sedge (*Carex*) appeared.

In July 1993 the next secondary treatment plant was put into operation. Since then the total hydraulic load of the wetland has increased from about 4 $m^3$/day (0.6 mm/day) to 65 $m^3$/day (8 mm/day), in the winter and the summer respectively.

Thus, the hydraulic load of the wetland pond was very unstable, and still lower than designed. However, wastewater flowing into the pond was periodically much more concentrated than designed, which was due to problems with the launching and operation of the secondary treatment facility KOS-2, especially in the winter time. Some initial results of the monitoring of wastewater treatment at the treatment wetland are given in Table 4.33. Samples for the analysis of wastewater were taken at two measuring points; at the inflow to the operating compartment and at the outflow from the pond.

The results show that removal efficiencies for $BOD_5$ ranged between 71–95 %, for TSS to 4–99 %, for $NH_4^+$-N to 41–98 %, for TN to 83–88 %, and for $PO_4^{3-}$ to 47–87 %. It can be concluded that the treatment wetland with flow system surface achieved a satisfactory reduction of pollutants after 2.5 years' operation. The outflow from the wetland met the class 2 requirements for surface water quality (except for phosphorus) imposed on treated outflows. The removal of phosphorus was generally lower in a surface flow wetland, because of a limited contact of wastewater with the soil and root zone. The phosphorus removal varied, although the

**Table 4.33** Treatment results of the SF system at Wieżyca, Poland (Obarska-Pempkowiak et al. 1994)

| Date | TSS, mg/l (% rem) | $BOD_5$, mg/l (% rem) | $NH_4^+$-N, mg/l (% rem) | $NO_3^-$-N, mg/l (% rem) | $NO_2^-$-N, mg/l (% rem) | TN, mg/l (% rem) | $PO_4^{3-}$, g/l (% rem) |
|---|---|---|---|---|---|---|---|
| *Inflow* | | | | | | | |
| August | 48.0 | 35.2 | 18.2 | 2.4 | 13.0 | 32.9 | 15.2 |
| September | – | 325.7 | 1.4 | 0.07 | 0.8 | 21.2 | 10.3 |
| October | 520.0 | 315.7 | 8.1 | 0.2 | 1.3 | 50.4 | 17.5 |
| February | 14.0 | – | 5.0 | 0.5 | 5.6 | 22.2 | 8.5 |
| Designed secondary outflow[a] | 25–40 | 30 | 8–12 | | 10–23 | 30 | 10 |
| *Outflow* | | | | | | | |
| August | 4.0 (91) | 9.2 (74) | 0.31 (98) | 0.05 (98) | 1.9 (−46) | 3.8 (84) | 2.0 (87) |
| September | – | 19.1 (94) | 0.19 (86) | 0.003 (96) | 0.6 (23) | 2.5 (88) | 0.3 (97) |
| October | 22.0 (95) | 1.4 (99) | 4.8 (41) | 0.014 (92) | 0.5 (57) | 7.4 (85) | Trace (99) |
| February | 4.0 (71) | – | 0.5 (90) | 0.03 (94) | 1.7 (69) | 3.8 (83) | 4.5 (47) |
| Mean value | 10.0 (85) | 9.9 (89) | 1.45 (78) | 0.097 (95) | 1.2 (26) | 4.4 (85) | 1.7 (82) |
| Permitted requirements[b] | ≤30.0 | ≤8.0 | ≤3.0 | ≤0.03 | ≤7.0 | ≤10.0 | ≤0.6 |

[a] According to the Producer of KOS-2—instruction card
[b] According to class 2 surface water qualify. Act no. 503 from 5.11.1991

system with a native soil substrate should be relatively effective for phosphorus removal. Furthermore, the pilot plant wetland had low loads because the input from the secondary treatment plant was only about 50 % of the hydraulic load originally planned, as a result the wetland was periodically fed with raw wastewater instead of secondary outflow.

Both surface flow wetland systems were effective in removing suspended solids and organic substances. However, high removal efficiency for nitrogen and phosphorus was achieved in a low loaded treatment wetland at Wieżyca.

Up to July 1994 the pond had been supplied with sewage from two treatment facilities KOS-2. During 3 years' operation time, problems with starting up one of the plants, and its maintenance (especially when the summer season finished, mainly in the winter) occurred. Supplied wastewater was untreated and even raw. This situation was the main reason for shutting down one improperly working treatment facility KOS-2.

In 1994 water meters were installed in recreation houses, which allowed for determining the actual water consumption and the quantity of the wastewater produced. On the basis of monthly water consumption (water consumption measurement results were obtained during the years 1995–1999), it was calculated that mean annual water consumption was app. 6,500 $m^3$/year with day fluctuation from min. 9.0 $m^3$/day in February to max. 39 $m^3$/day in August.

It was calculated that mean quantity of wastewater flowing into the reed pond was 6,520 $m^3$/year, which was app. 26 % of the designed quantity (25,800 $m^3$/year) (Table 4.34).

Traditional solutions i.e. mechanical-biological sewage treatment, do not provide a satisfactory removal effectiveness of nutrients (nitrogen and phosphorus). Therefore, the three-stage treatment process took place at the treatment facility for Krzeszna-Wieżyca Recreation Centre.

For water balance calculation, it was assumed that mean monthly and annual values of precipitation collected during several years of operation were equal to $P = 640$ mm.

The evaporation level from the pond area was calculated as a function of terrain evaporation from free water surface. Free water surface evaporation ($E_w$) was assumed on the basis of the literature. The value of evaporation factor ($\eta$) was estimated on the basis of observation and the measurement result analysis of water losses for the transpiration of natural marsh ecosystems with reed and other macrophytes. On the basis of the available literature, it could be indicated that evapotranspiration exceeds by several times (from 1.3–1.5 to 3) vaporization from free water surface in the area planted with hydrophytes.

In treatment wetland systems with low sewage hydraulic loads, evapotranspiration can be higher than wastewater inflow and precipitation, even in temperate climate regions, especially in dry and mean years.

This happened in the case of the reed pond analysed here. For the first time no outflow was observed in August, 1994—that was after 3 years of operation, which was associated with an intensive development of plants growing in the pond. Calculations of water balance, made on the basis of the assumed meteorological

**Table 4.34** Balance of water in the SF system irrigated with wastewater at Wieżyca, Poland

| Balance components | Unit | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | IV-IX | X-III | Year |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| *Sources* | | | | | | | | | | | | | | | | |
| Quantity of supplied wastewater mean values from 1995–1999, Q | $m^3$ | 478.64 | 376.04 | 340.07 | 400.00 | 412.30 | 618.30 | 1102.36 | 1173.65 | 531.00 | 366.42 | 331.80 | 388.43 | 4237.60 | 2281.40 | 6519.00 |
| Pond hydraulic load $F = 18{,}370\ m^2$, $S = Q/F$ | mm | 26.00 | 20.10 | 18.60 | 22.30 | 22.70 | 33.60 | 60.00 | 63.70 | 28.70 | 19.70 | 18.20 | 21.10 | 231.00 | 123.70 | 354.70 |
| Sum of precipitation, mean value from several years, P | mm | 37.00 | 39.00 | 32.00 | 39.00 | 50.00 | 58.00 | 93.00 | 84.00 | 63.00 | 56.00 | 42.00 | 47.00 | 387.00 | 253.00 | 640.00 |
| *Sinks and stock* | | | | | | | | | | | | | | | | |
| Free water surface evaporation, mean value from several years, $E_w$ | mm | 15.00 | 14.00 | 18.00 | 30.00 | 60.00 | 95.00 | 98.00 | 90.00 | 69.00 | 34.00 | 21.00 | 16.00 | 442.00 | 118.00 | 560.00 |
| Evapotranspiration factor for reed pond, η | – | 1.0 | 1.0 | 1.0 | 1.3 | 1.6 | 2.1 | 2.5 | 2.4 | 1.5 | 1.1 | 1.0 | 1.0 | – | – | – |
| Evapotranspiration of reed pond $E_T = E_w \cdot \eta$ | mm | 15.00 | 14.00 | 18.00 | 39.00 | 96.00 | 200.00 | 245.00 | 216.00 | 102.00 | 38.00 | 21.00 | 16.00 | 898.00 | 122.00 | 1020.00 |
| Water reserve in pond, $\Delta R^a$ (initial reserve $\Delta R = 150$ mm) | mm | 198.00 | 243.00 | 276.00 | 298.00 | 275.00 | 166.00 | 74.00 | 6.00 | 16.70 | 54.00 | 94.00 | 146.00 | – | – | – |
| Maximal retention water level in pond, max $\Delta R^b$ | mm | 600.00 | 600.00 | 600.00 | 600.00 | 600.00 | 600.00 | 600.00 | 600.00 | 600.00 | 600.00 | 600.00 | 600.00 | – | – | – |

[a] Actual retention in pond $\Delta R \leq 600$ mm, no water outflow
[b] Maximal water retention level related to gate backwater level in the outflow from pond (riv. 178.80 m n.p.m.)

data for a typical year, show that the reed pond with rampant water vegetation and low actual hydraulic loads (average 355 mm/year, ~1 mm/day) works as an evaporation system, without outflow into surface water and as an accumulation system for pollutants.

Water supply from wastewater inflow and precipitation is balanced by water loss connected to evapotranspiration from actual vegetation.

In 1999 measurements of characteristic concentrations of pollutants which flew from the KOS-2 treatment facility to the reed pond were made. Research on the following physical-chemical factors: pH, total suspended solids, $BOD_5$, COD, nitrogen species (ammonia nitrogen, organic nitrogen, nitrate and nitrite) organic phosphor and phosphates was carried out.

On the basis of the measurement results, it could be concluded that $BOD_5$, COD, nitrogen and phosphorus compounds (with some exceptions) in the outflow fulfilled the criteria for class 3 water quality only in the summer season. When the summer season finished concentrations were higher: TSS = 108.0 mg/l, $BOD_5$ = 100 mg $O_2$/l, $COD_5$ = 240 mg $O_2$/l, TP = 15.3 mg/l, TN = 90.6 mg/l, pH = 7.5.

Taking into account the measurements results of total nitrogen and total phosphorus concentration and the average monthly quantities of wastewater, the average loads inflowing to the reed pond were calculated: 106 kg N/ha and 11.7 kg P/ha. These values were lower than the values of the annual doses of chemical fertilizers recommended for areas of intensive agricultural cultivations, meadows and grazing lands: 200–300 kg N/(ha year) and 20–45 kg P/(ha year) respectively.

Since 1995, because of shutting down the KOS-2 treatment facility, the quantity of wastewater has decreased, so has the hydraulic load of the pond. It was estimated that the pond was working as an accumulation-evaporation system, without the outflow of treated wastewater.

In the pond, there was an accumulation of peated vegetal mass, like in natural marsh ecosystems. The pond worked as a retention system for pollutants, which was advantageous from the ecological point of view.

The analysis of pollutants removal effectiveness in treatment wetlands made by Kadlec and Knight (1996), and NADB (1993) proved that the effectiveness of the removal of phosphorus compounds was low (Tables 4.33 and 4.34). If the concentrations of phosphorus compounds in the inflow were not high, they could be reduced, but in the case of big phosphorus loads, it was necessary to have a large system surface. That is why, the primary removal of phosphorus compounds was planned, which was profitable from the economic point of view. Iron (III) or aluminium (III) salts dosages were usually applied. In the case of SF systems, the dosage must be applied before the system inflow, because there is no chance to ensure mixing inside the system. SSF systems have an advantage over SF systems, because their filling can be mixed with chemical compounds containing ions, which allows for creating an insoluble salt with phosphates ions. The oxidization of ammonia nitrogen is more effective when diffusion and the supply of oxygen to dissolved or absorbed nitrogen is easy. From that point of view, both HSSF and SF systems are not a good solution, because they are characterized by insignificant redox conditions. On the other hand the tendency of the appearance of anoxic conditions is advantageous from the point of view of denitrification. The highest nitrogen removal effectiveness is achievable when treatment in a treatment wetland system is preceded by primary nitrification. Nitrification can take place in forest soil filters (inhabited with plants or no) or in VSSF beds.

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# Chapter 5
# The Quality of the Outflow from Conventional WWTPs and Treatment Wetlands

Until recently the humic substances were considered to have no negative impact on human health. Therefore elimination of these substances from potable water was only performed due to aesthetic reasons. In the recent years, however, it was found out that humic substances may cause unacceptable smell of water or cumulate toxic substances by complexing metal ions and adsorption of persistent organic pollutants such as pesticides, PCB or phthalanes. Moreover, it was indicated that humic acids are precursors of mutagenic trihalomethanes (THMs) (Dojlido 1995).

Humic substances impose substantial effect on the aquatic environment as well as the water organisms and human health. Among other features, the humic acids determine organoleptic properties of surface waters. Therefore explanation of the origin and properties of humic acids in surface waters is one of the priorities of water technology (Gjessing et al. 1998).

Recently it was pointed out that in the wastewater treatment process organic compounds characterized by properties similar to humic acids are formed (Obarska-Pempkowiak et al. 2006). The chemical features of wastewater indicate that they can influence the form of trace metals, the salts concentrations as well as concentrations of organic substances, including xenobiotics. The concentrations of these substances are higher in treated sewage than in surface waters. Moreover, the complexing abilities of the humic acids originating in wastewater treatment process are stronger than those of the naturally occurring humic substances (Nissinen et al. 2001). Thus humic acids formed in wastewater treatment process can be also responsible for trace metals transport and increase of their bioavailability in the ecosystem (Pempkowiak et al. 1999).

Usually, the performance of WWTP is evaluated on the basis of organics and nutrients elimination efficiency. The influence of wastewater treatment technology on the presence of resistant to biodegradation organic complexes has not been investigated yet. The sewage treated both in conventional WWTPs and TWs contains mostly organic substances resistant to biochemical transformation, however there is hardly any information available about the properties of these substances.



H. Obarska-Pempkowiak et al., *Treatment Wetlands for Environmental Pollution Control*, GeoPlanet: Earth and Planetary Sciences,
DOI 10.1007/978-3-319-13794-0_5

## 5.1 Definition of Humic Substances

Content and function of organic matter in soil have been a subject of interest for many years (Gassemi and Christman 1968). Despite this no consensus have been reached as regards definitions and nomenclature of particular organic matter fractions. Considering the influence on soil properties and participation in processes occurring in soil a particular role is played by humic substances. The most often used definition indicates humic substances as a fraction of soil organic matter that can be separated by means of extraction with alkaline aqueous solutions (Matcher et al. 1983). When the alkaline extract is acidified, humic acids precipitate out of solution, while fulvic acids remain dissolved. A schematic of humic substances extraction and separation into fractions is presented in Fig. 5.1. The presented approach to isolation and separation of humic substances produces fractions that are more homogenous than the row extract (Kononowa 1968). It has been established that the obtained fractions exhibit continuous changes of properties, as presented in Fig. 5.2. It must be stressed that no general mechanism of humic substances formation has been

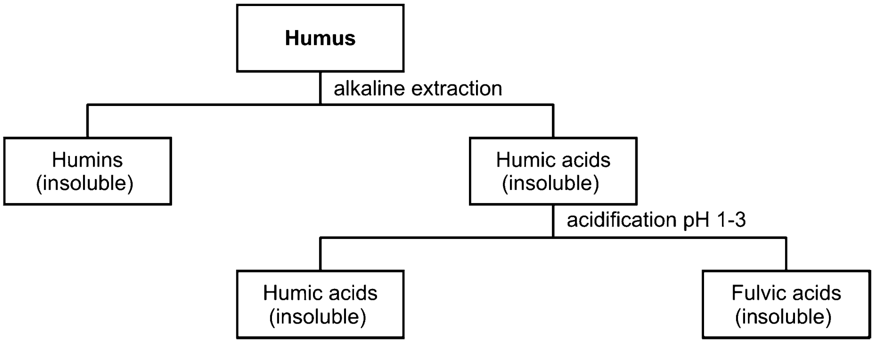


**Fig. 5.1** Schematic of humic substances isolation and fractionation

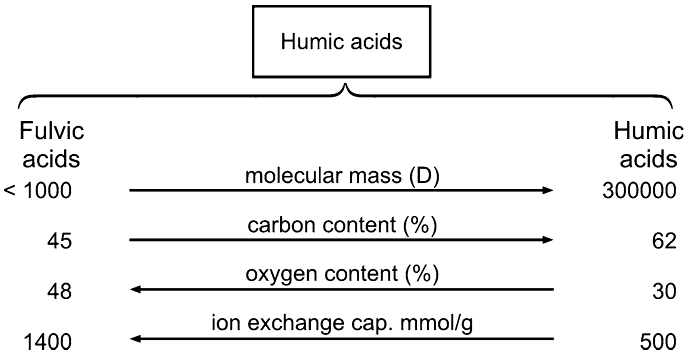


**Fig. 5.2** Drift of humic acids fractions properties

established. However, it is obvious that both local environmental conditions (temperature, moisture, type of soil materials), and quality of organic substances that are precursors of humic substances are important. It is often stressed that the number of simple organic substances, humic substances originate from, is substantial. Thus the number of macromolecules that originate from condensation reactions of simple organic substances is astronomical. A possibility that the same substances are condensed in the same order is so remote that very few molecules of humic substances will have similar structure. Therefore it could be safely said that humic substances are a mixture of a great number of organic substances mostly of acidic nature (due to the presence of carboxyl and phenolic groups) (Ertel and Hedges 1983; Otsuki and Hanya 1967).

Single components of the mixture differ with molecular mass, and size. Aqueous solution of humic substances display yellow to brown colour. An important property of soil humic substances is ability to form stable complexes with cations of metals, and other organic substances including xenobiotics. Results of research of organic substances isolated from fresh water, lacustrine and marine sediments, and sea water indicate that substantial proportion constitute organic substances that exhibit properties similar to properties of soil humic substances (Florence and Batley 1976; Pempkowiak and Kupryszewski 1980).

## 5.2 Humic Substances in Surface Fresh Water

Terms humic acids and fulvic acids originate from soil chemistry and denote the group of organic substances that are isolated from soil by means of alkaline extraction. Humic acids are insoluble in acidic aqueous solutions, contrary the fulvic acids- that are soluble. This definitions were extended to fractions of organic substances isolated from marine and lacustrine sediments, and dry residue, of both fresh water and sea-water. Some decades ago also fractions of organic substances isolated from fresh water by means of other methods (extraction, sorption, precipitation) were named with the same terms (Ertel and Hedges 1983; Szpakowska et al. 1986).

Methods of humic substances isolation from sediments do not differ from those used by soil chemists. Isolation of humic substances from fresh- and sea-water is more complex. At the beginning precipitation at the water-organic solvent interface was used. It was replaced by sorption of aquatic humic substances on macroreticular resins. It is quite obvious that the operational procedure used as a definition of aquatic humic substances differ greatly from the operational procedure used as a definition of soil and sedimentary humic substances. Nevertheless, methods used to characterise properties of soil humic substances were then applied to aqueous humic substances. Thus it was established that aquatic humic substances are composed of molecules in the 500–10,000 D range, comprising sugars, aminoacids, and aromatic rings. The presence of phenolic and carboxylic groups was also established (Pempkowiak 1989).

## 5.3 Isolation of Humic Substances from Water

Humic substances are a class of biogenic, coloured, organic substances that are ubiquitous in various compartments of the environment including surface waters. After a period of primarily academic interest humic substances have become a subject of interest of practitioners, including water purification technologists. This is due to the proven influence of humic substances on the water properties (water colour, surface tension, gas exchange rate between water and the atmosphere), and processes occurring in water (migration of nutrients, bioavailability of metals, direct and indirect influence on biota). Moreover, humic substances constitute a menace to water technologists as they release chlorinated hydrocarbons on water disinfection with chlorine, and are known to decrease iron removal efficiency due to complexation of iron (III) ions. Of these features humic substances interactions with biologically active substances are of significant interest as the interactions modify ecological impact of the latter. When dealing with humic substances dissolved in surface and ground water concentration and isolation is the first and the most important step. The usual procedure, applied also in studies on humic substances dissolved in the effluents from WWTPs, is based on the effluent samples filtration, sorption of humic substances onto XAD-8 resin at pH 2, desorption of the adsorbed humic substances using 0.2 mol/dm$^3$ sodium hydroxide aqueous solution, followed by separation of fulvic acids from humic acids, desalting, hydrogen saturation on cation exchange resin, and finally water removal to obtain dry fulvic acids (Mantoura and Riley 1975). The so obtained fulvic and humic acids are then characterized by a battery of physical and chemical analytical methods including NNR, IR, US, NIS, stable carbon isotopes ($\delta^{13}$C), molecular weight fractionations and others (Zepp and Schlotzhaner 1981).

Dissolved humic substances are most often just a fraction of dissolved organic matter (DOM) in water. In the temperate climate humic acids comprise often some 5 % the DOM, while fulvic acids constitute as much as 45 % of DOM. The ratio of fulvic acids to humic acids is thus equal to 9:1. However, in highly coloured waters humic substances may constitute as much as 80 % of DOM, while the fulvic acids to humic acids ratio is 4:1.

## 5.4 Methods of Humic Substances Characterization

Aquatic humic substances are a class of dissolved organic substances that is defined according to the procedure of the substances isolation. Thus the definition is operational. Therefore strict adhering to rules regarding the substances isolation is a prerequisite to compare humic substances isolated from aquatic environments world-wide.

The isolated substances are characterized by a variety of physical and chemical methods.

*Elemental composition*. Basic elements the substances are composed of are: carbon (C), hydrogen (H), nitrogen (N), oxygen (O), and sulphur (S). Not only elemental composition is used to characterize humic substances. Ratios C/H and C/N are often reported as they may indicate structure (aliphatic-lower C/H, vs. aromatic-higher C/H), or the origin of substances (lignins-higher C/N, vs. plankton-lower C/N). Both elemental and molecular ratios are used.

Aquatic humic substances comprise 45.0–48.0 % of carbon, 5.5–6.5 % of hydrogen, 0.0–5.0 % of nitrogen, 28.0–35.0 % of oxygen, and <1.0 % of sulphur. If aquatic humic substances are separated into humic acids and fulvic acids the latter comprise—most often less carbon, equal percentage of hydrogen, and less nitrogen.

Elemental composition characterizes humic substances in a rather unspecific way. Nevertheless it is used to document differents between the substances isolated from comparable environments (e.g. aquatic, sedimentary) or fractions of humic substances obtained from the same "parent" sample.

*Molecular mass*. Aquatic humic substances show smaller molecular mass (<10,000 D), as composed with sedimentary humic substances (<2,000,000 D). It has been proven that high molecular weight fractions of humic substances precipitate under increasing concentration of di-and tri-valent metal ions, for example in the mixing zone of fresh and saline water masses in estuaries.

*Absorption spectra—1H NMR* exhibit few absorption ranges. These are assigned to the following moleties methyl ($\sim$0.8 ppm), methylene and methine ($\sim$1.2 ppm), polyhydroxyl ($\sim$3.5 ppm), and aromatic rings (6.5–7.5 ppm).

*Absorption spectra—*$^{13}$*C NMR* are, by for, more specific. Absorption peaks are often found in the aliphatic hydrocarbon chain region (0–40 ppm), aminoacids ($\sim$40 ppm), ethers and esters (53–56 ppm), carbohydrates (63 ppm), aliphatic (74 ppm) and aromatic rings (105–145 ppm).

IR desorption spectra are used to characterize humic substances in a qualitative manner. Major absorption bonds are assigned to the following moieties: hydroxyl (3,300–3,100 cm$^{-1}$), methyl and methylene (2,850 cm$^{-1}$), carbonyl (1,720 cm$^{-1}$), aromatic rings (1,630 cm$^{-1}$).

*Absorption spectra in the UV and VIS ranges*. Aquatic solution of humic substances exhibit yellow to brown colour depending on the concentration and origin. This is caused by increasing light absorption at decreasing wave length of the light. The absorption increase is monotonic, however substances isolated from various sources may show different dA/dλ (A—absorption, λ—wave length) and A/C (C—concentration) values. The former can be expressed quantitatively as absorption ratio at selected wave length. Rations at 425 and 665 nm (A4/6), and 280/365 (A2/3) are frequently used.

*Complexing properties of humic substances* are caused by the presence of donor functional groups in molecules of humic substances. Direct measurement of complexing capacity by means of anodic stripping voltammetry show complexing capacity of aquatic humic substances in the ranges: 0.15–0.29 ng/mg (Cd); 1.9–4.6 ng/mg (Cu); and 0.9–2.3 ng/mg (Pb).

*Influence of humic substances on the algal growth*. It has been found that when humic substances are added to batch cultures of all tested phytoplankton species

(C*oscinodiscus granii Chlorella vulgaris*, *Scenedesmus microspira*) increase in the range 120–390 % of biomass was observed. This was enhanced to 1,300 % when the cultures were supplemented with humic substances and iron (III).

## 5.5 Experimental

### *5.5.1 WWTP Studied*

The objective of this chapter 5.5.1–5.5.4 is quantitative and qualitative comparison of organic substance resistant to biodegradation formed during conventional and natural (TW) wastewater treatment to biochemically resistant fraction of organic substances present in surface water. The naturally occurring persistent organic substances are often referred to as humic substances. The quality of outflow from three highly-effective WWTPs (Jamno near Koszalin, Unieście and "Wschód" Gdańsk and two TWs (Darżlubie and Wiklino) were investigated and compared to the quality of humic acids present in the water of the Vistula river (in Kiezmark, close to the Vistula mouth). As the same method was used for isolating organic substances from treated sewage and humic substances from river water, the former will be called humic substances as well. Humic substances were isolated from treated sewage in order to analyze their properties: elemental composition, absorption spectra (visible VIS, ultraviolet UV, infrared IR, NMR) and complexing properties toward selected heavy metals ions).

### *5.5.2 Experimental Procedures*

The investigations were performed at three conventional and two natural WWTPs in Poland. The characteristics of the WWTPs is presented in Table 5.1. The Jamno WWTP treats wastewater from Koszalin, while Unieście WWTP receives additionally sewage from cesspools from the area of Mielno community and recreation places.

**Table 5.1** Characteristics of investigated WWTPs in Poland

| WWTP | Sewage flow ($m^3$/day) | Configuration of biological treatment stage |
|---|---|---|
| Wschód | 84,000 | 6 bioreactors—MUCT system |
| Jamno | 30,000 | 4 bioreactors, dephosphatation-denitrification—denitrification/nitrification–nitrification chambers |
| Unieście | ~1,780 | 2 bioreactors—denitrification-nitrification chambers |
| Wiklino | 20.5 | Beds: HSSF—VSSF—HSSF |
| Darżlubie | 56.7 | Beds: HSSF I—cascade—HSSF II—VSSF—HSSF III |

Mechanical treatment at both WWTPs consists of screens, sand traps and settling tanks, while biological treatment is performed at multiphase bioreactors.

The TWs were located at Wiklino near Słupsk and Darżlubie near Puck in Poland. Both facilities provided biological treatment for sewage after primary settling tanks. Both TWs represented the so called hybrid treatment wetland systems (HTWSs) and consisted of 3 (Wiklino) and 5 (Darżlubie) beds with alternately horizontal subsurface (HSSF) and vertical subsurface (VSSF) flow regime.

The samples of wastewater were collected from the outflows in the period from March 2006 to December 2007 with the frequency once per 3 months.

The following analyses of the samples were performed: total suspended solids (TSS), $BOD_5$ (in order to determine the concentration of easily-biodegradable organic matter) and COD (in order to determine the total concentration of organic matter). Additionally the concentrations of ammonia nitrogen, nitrogen III and nitrogen V, Kjeldahl nitrogen and total phosphorus were determined.

For analyses of humic acids concentration and properties 250 ml of treated sewage was collected in April. After filtration through GF/C gloss fibre filters (Whatman) the solutions were acidified with concentrated HCl to pH 2 and then passed through a glass column (∅ 12 × 50 mm) filled with Amberlit XAD-2 (Serva). The flow rate was 1 $dm^3/h$. The adsorbed humic acids (HA) were eluated continuously with 250 ml of 0.1 mol/l $NH_3 \cdot H_2O$ during a 4 days long period. The excess $NH_3 \cdot H_2O$ in the eluat was evaporated, at 50 °C under vacuum to the final volume of 50 ml. The HA concentration in the so obtained extract was determined by weight (the determinations of dry residue and loss on ignition). The UV-VIS spectra of HA aqueous solution were analyzed in a spectrophotometer while the IR spectra were measured in a Carlo Erba model IR 05 spectrophotometer. The elemental composition (C, H, N, S) was determined in the Carlo Erba E A 12 apparatus.

### *5.5.3 Results and Discussion*

#### 5.5.3.1 The Outflow Quality

The investigated WWTPs effectively removed organics and TSS. The plants equipped with the multiphase bioreactors also ensured very good effectiveness of nutrients removal. The average daily concentrations in the outflow fulfilled the requirements given in the Regulation of the Polish Environment Ministry from 24th July 2006.

Among the investigated TWs, higher effectiveness of organics removal was observed in Wiklino (95.55 % COD and 95.9 % $BOD_5$) than in Darżlubie (74.9 % COD and 82.1 % $BOD_5$). Also the total nitrogen removal effectiveness was higher in Wiklino (79.2 %) than in Darżlubie (67.9 %). The TW in Darżlubie, despite its quite effective performance, failed to fulfill the requirements for the outflow quality given in the Regulation (Table 5.2).

Characteristics of the outflow quality in the investigated WWTPs are presented in Table 5.2.

**Table 5.2** Characteristics of the outflow quality in the investigated WWTPs

| Parameter | Unit | Jamno | Unieście | Gdańsk | Wiklino | Darżlubie |
|---|---|---|---|---|---|---|
| COD | mg $O_2$/l | 32.8 | 32.2 | 43.1 | 31.5 | 210.5 |
| $BOD_5$ | mg $O_2$/l | 5.7 | 6.7 | 5.7 | 10.9 | 72.0 |
| TSS | mg/l | 14.0 | 47.3 | 15.0 | 36.3 | 92.0 |
| TN | mg N/l | 13.2 | 17.9 | 10.8 | 21.7 | 56.5 |
| TP | mg P-$PO_4^3$/l | 0.44 | 0.93 | 0.75 | 7.2 | 6.7 |

## 5.5.4 The Concentration of Isolated Humic Acids

The concentrations of humic acids in surface waters vary from 1 to 9 mg/l (Kononowa 1968; Dojlido 1995; Pempkowiak 1989)—on average approximately 3 mg/l (Table 5.3). The investigations indicated that concentration of humic acids in the outflow is equal to: Jamno—2.8 mg/l; Unieście—3.2 mg/l; Gdańsk—3.3 mg/l. The corresponding concentrations in outflows from treatment wetlands were higher: 4.2 mg/l for Wiklino and 4.9 mg/l for Darżlubie. It should be however pointed out, that research was performed only at selected wastewater treatment plants. Therefore claiming that humic acids concentration in treated wastewater is app. 3 mg/l may be premature. Concentration of humic substances in the outflow depends, among other factors, on wetland size, composition of raw wastewater and treatment technology.

Assuming the quantity of outflow as 50,000 $m^3$/day and concentration of humic acids as 3 g/$m^3$, the daily discharge of humic acids will be 150 kg. In case of smaller WWTPs, with the same as previous humic acids concentrations, the loads will be much lower. Humic acid concentrations in surface water in Poland (Pempkowiak 1989) are thus comparable to concentrations measured in treated wastewater. It can be concluded that discharge of treated wastewater into recipient can significantly influence the surface water quality. However, proportion of humic acids from wastewater to natural humic acids in surface water depends on several

**Table 5.3** Comparison of properties of humic acids isolated from the outflows of investigated WWTPs

| Humic acids | $A_{2/3}$ | $A_{4/6}$ | Con. (mg/l) | C/H | C/N |
|---|---|---|---|---|---|
| Jamno | 1.7 | 7.9 | 2.8 | 6.6 | 7.6 |
| Unieście | 1.7 | 6.6 | 3.2 | 6.6 | 8.0 |
| Gdańsk | 1.7 | 6.9 | 3.3 | 6.8 | 8.0 |
| Wiklino | 2.3 | 9.7 | 4.2 | 8.3 | 8.6 |
| Darżlubie | 2.2 | 10.4 | 4.9 | 8.6 | 9.1 |
| Wisła | 2.3 | 8.9 | 6.2 | 6.6 | 7.4 |

factors such as: inflows, water quality and quantity in recipient, precipitation, season etc. (Górniak 1998; Kowalski 1988). The smaller catchment area and flow rate in the recipient is, the more important influence will have the load of discharged humic acids on organic matter composition (Szpakowska 1999).

### 5.5.5 *Ultraviolet (UV) and Visible (VIS) Light Absorption Spectra*

In Figs. 5.3 and 5.4 examples of VIS and UV absorption spectra of solutions of humic substances isolated from the outflow from Jamno WWTP are presented.

Absorption curves of isolated substances in VIS range are showing typical for humic acids, monotonic growth of light absorbance together with shortening of wave length. Solution absorbance values for humic acids increase with higher carbon content, with atomic mass growth, with progress of aromatic rings condensation and with growth of quotient of aromatic carbon content to aliphatic carbon content (Enev et al. 2014; Gjessing 1997).

For qualitative characteristics of humic acids the quotient of absorbance value by wavelength 465 and 665 nm is often used. Value of this quotient is defined as $A_{4/6}$.

It $A_{4/6}$ does not depend on humic acids concentration, but it results from chemical structure of molecules. Quotient $A_{4/6}$ is often used as empirical indicator of humic acids molecules genesis or/and structure (Obarska-Pempkowiak et al. 2006; Pempkowiak 1989).

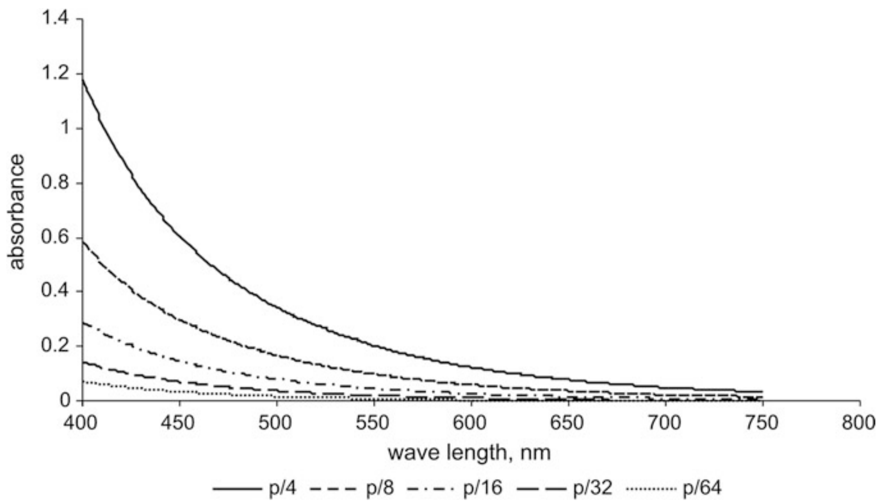


**Fig. 5.3** Absorption spectra (VIS) of humic acids solutions for the outflow of Unieście WWTP near Koszalin in Poland

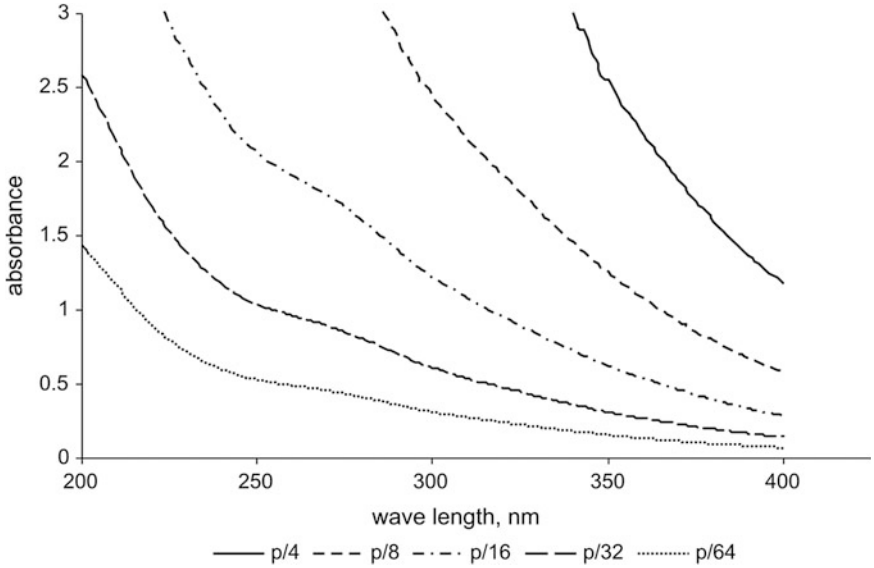


**Fig. 5.4** Absorption spectra (UV) of humic acids solutions for the outflow of Jamno WWTP near Koszalin in Poland

Value of quotation $A_{4/6}$ should be used for comparison of humic acids isolated from the same environments (Kononowa 1968). Analyzed samples fulfill this criterion. Quotients of analysed humic acids, presented in Table 5.4, differ from each other. It indicates differences in chemical structure of humic acids molecules. The highest value of $A_{4/6}$ quotient was obtained for humic acids isolated from the water of the Vistula River. Thus it should be expected that molecules of these acids contain the lowest amount of carbon atoms bonded in aromatic rings. The $A_{4/6}$ quotients of other acids (Table 5.4) vary in range 6–8, which suggests higher level of aromatisation of the analyzed substances.

Absorption spectra of solutions of substances isolated from treated sewage in visible light and ultraviolet, are characterised by lack of inflexion points. For this reason it can be stated that molecules of these substances have no vegetable dye fragments (Kononowa 1968; Pempkowiak 1989).

**Table 5.4** Spectrum characteristics of humic acids isolated from the analysed WWPTs outflow

| Humic acids origin | UV and VIS spectra characteristics | |
|---|---|---|
| | A465/A665 | A260/A320 |
| Jamno | 6.5 ± 0.7 | 1.76 ± 0.09 |
| Unieście | 7. 2 ± 0.6 | 1.72 ± 0.08 |
| Gdańsk | 6.9 ± 0.6 | 1.73 ± 0.06 |
| Gdańsk | 10.4 ± 0.4 | 2.15 ± 0.17 |
| Darżlubie | 9. 7 ± 0.9 | 2.31 ± 0.14 |
| Wisła | 8.9 ± 0.11 | 2.30 ± 0.15 |

$A_{4/6}$ quotient values (Table 5.4) for humic acids differ from each other, which result most likely from different wastewater treatment technologies. The $A_{4/6}$ quotient for humic acids isolated from Jamno WWTP is higher than $A_{4/6}$ for humic acids isolated from Unieście WWTP. Since $A_{4/6}$ values decrease with decrease of molecular mass of humic acids (Gjessing 1970; Imai et al. 2002), it can be assumed that the humic acids isolated from WWTP Jamno outflow have the smallest molecular mass and the humic acids from Wiklino—the highest molecular mass among the investigated HA. However observed differences are not significant. According to Świderska-Bróż (1984) filtration process can influence the molecule mass of humic acids due to aggregation and removal of the substances.

The measurements of absorbance in the UV range were conducted only for diluted solutions (Fig. 5.4). Recorded absorption spectra in UV range are characterised by increase of absorbance with wave shortening. The inflexion point at the curve that appears with 270 mm of wavelength indicates aromatic rings presence in humic acids molecules. In case of absorption spectra in UV range for qualitative characteristic of humic acids the quotient $A_{2/3}$ is used, that is the quotient of absorbance value at the wave length of 260 and 320 nm (Pempkowiak 1989).

The values of $A_{2/3}$ quotient, presented in Table 5.4, are similar to each other, and the average values for solutions isolated from conventional treatment plants are equal to 1.7. The mean values from treatment wetlands were similar to the $A_{2/3}$ quotient for Vistula river. $A_{2/3}$ quotient values in literature are almost equal to the values for humic acids, although the investigated HA originated from different environments (Pempkowiak 1989). Thus analysis of absorption spectra in UV range have limited application.

### *5.5.6 Infra-red Absorption Spectra*

Absorption spectra in IR pieces of range, that is for wave length from 2,000 to 14,000 nm, brings many important information about humic acids structure (Pempkowiak 1989). Absorption bands at selected wave length prove the presence of certain structural features. Due to complicated structure of humic acids molecules and the fact that humic acids are mixtures of compounds with structure and properties varying in wide range, absorption bands are usually fuzzy. Thus their qualitative interpretation is more difficult and quantitative interpretation is most often impossible (Świderska-Bróż 1984).

Absorption spectra in infra-red for substances (humic acids) isolated from treated wastewater are presented in Fig. 5.5.

The similarity of presented absorption spectra is noticeable, for instance appearance of absorption bands by the same frequency. On this basis it can be reckoned, that HAs of Jamno and Unieście outflows contain similar structural features. Appearance of a wide absorption band in 3,500–3,000 nm range is attributed to absorption by hydroxyl groups or by amines. Absorption by hydroxyl groups is more probable since the oxygen content is higher than nitrogen content in

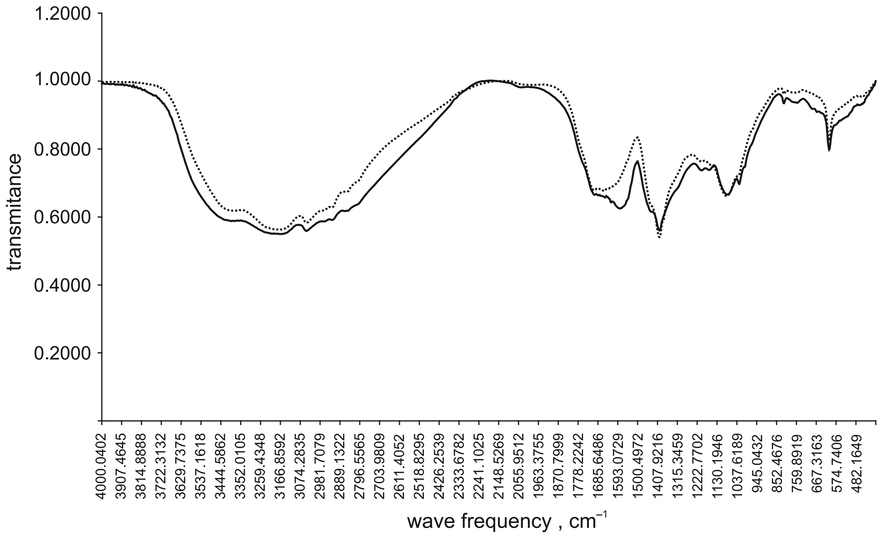


**Fig. 5.5** Absorption spectra (IR) of humic acids from the outflows of Jamno (*solid*) and Unieście (*dashed*) WWTPs

the analysed humic acids (Pempkowiak 1989). Appearance of several peaks in 3,000–2,800 range may be caused by stretching vibrations of C–H bonds in methyl and methylene groups. Very strong peak at wavelength 1,500 $cm^{-1}$ is caused by asymmetric vibrations of carboxylic groups ($COO^{-}$) and II order amines (C-NHR). In Jamno the bond is much more intensive, which may prove higher content of that functional group. However there are some noticeable differences as well. For wave number 1,430 $cm^{-1}$ there appears a peak, (in Jamno it is much stronger), which is caused by deformation vibrations –OH in phenols. On the other hand, the bond in 1,680–1,650 $cm^{-1}$ range may be assigned to stretching vibrations C=0 of carboxyl groups, aldehyde, ketone and C=C bonds in aliphatic chains and aromatic rings. The absorption in 1,000–700 $cm^{-1}$ range can be caused by mineral-organic complexes (Pempkowiak 1989).

### *5.5.7 Elemental Composition of Analysed Humic Acids*

The basic elements constituting humic acids are: carbon, hydrogen, nitrogen and oxygen. The oxygen content is usually calculated as a difference.

The analysed humic acids contain significant quantity of nitrogen (app. 6 %). This may result from significant content of aminoacids in the humic acids molecules. The high sulphur content (exceeding 2 %) is an interesting fact. One of possible explanations of high sulphur content in humic acids samples isolated from

**Table 5.5** Elemental composition of humic substances isolated from outflows of the analysed WWTPs

| Sample | Concentration calculated as DM content (%) | | | | | C/H | C/N |
|---|---|---|---|---|---|---|---|
| | C | N | H | S | 0 | | |
| Jamno | 47.2 | 6.2 | 7.1 | 2.1 | 37.4 | 6.6 | 7.6 |
| Unieście | 48.3 | 6.0 | 7.3 | 2.2 | 36.2 | 6.6 | 8.0 |
| Gdańsk | 47.4 | 5.9 | 7.0 | 2.1 | 37.6 | 6.8 | 8.0 |
| Wiklino | 49.3 | 5.7 | 5.9 | 1.7 | 37.4 | 8.3 | 8.6 |
| Darżlubie | 48.7 | 5.4 | 5.7 | 1.6 | 38.6 | 8.6 | 9.1 |
| Wisła | 46.3 | 6.2 | 7.0 | 1.4 | 39.1 | 6.6 | 7.4 |

WWPTs outflows is the presence of sulphates in wastewater which may originate from industry in the WWTP's catchment area.

According to (Dojlido 1995) the carbon content of soil-originating humic acids should vary in the range from 50 to 60 %. The lower carbon content in analysed samples is similar to the carbon content in humic acids from surface waters (Pempkowiak 1989). The relatively high content of oxygen in humic acids isolated from treated wastewater indicates presence of functional groups: carboxyl, carbonyl, metoxyl, phenolic, hydroxyl, ether or ester in molecules of analysed substances.

C/N ratio is used as an indicator of organic matter origin. In case of analysed humic acids the C/N ratio is from 7.6 for conventional WWTPs to 9.0 for treatment wetland systems, which confinns the autogenic origin of organic matter in treated wastewater (Kononowa 1968; Imai et al. 2002). The difference between C/N ratios for Vistula river may be due to different nitrogen removal effectiveness in conventional and natural WWTPs.

C/H ratio is used as an indicator of aromatisation and condensation of humic acids. It was found out that the C/N ratio value in humic acids isolated from marine environment is below 10, while for land humus it is above 10 (Pempkowiak 1989). Despite the fact that above statement was proved for marine environment, this thesis may be applied for origin evaluation of humic acids isolated from treated wastewater. In case of analysed humic acids the C/H ratio was from 6.6 to 8.5 in wastewater from treatment wetlands (Table 5.5), which indicated that these acids comprise smaller number of aromatic rings.

## 5.6 Conclusions

Humic acids, regardless of their origin, are mixtures of compounds with wide range of molecule masses. They have many different functional groups containing oxygen and nitrogen. They are characterised by similar absorption spectra and have properties of surfactants and ion exchange substances.

Analysed humic acids isolated from treated wastewater have similar elemental composition and absorption spectra in IR range, although this does not prove similar molecular structure.

$A_{4/6}$ and C/H ratios bring useful information, and are used as empirical indicators of humic acids molecules origin and structure. The values of these indicators for humic acids isolated from WWTPs outflow and for water from Vistula river differ due to aromatic rings presence in treated wastewater. This fact indicates, that humic acids from conventional and natural WWTP outflows may have various functions in the environment.

Humic acids concentration in treated sewage is similar to their concentration in surface water. It indicates that wastewater are potential source of humic acids, especially in case of smaller streams and rivers.

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# Chapter 6
# Storm Water Treatment in TWs

Storm water runoff usually carries quite large load of different pollutants. In the urbanized areas storm water runoff contains suspended solids, oils, PAHs and heavy metals from petroleum spills, de-freezing salts, detergents, pesticides and herbicides as well as organic matter. Field runoff outside the cities washes out fertilizers, pesticides and herbicides and may also be polluted with leaking manure or domestic wastewater. Another problem is associated with peaking flows of storm water during serious rain events that result in floods, especially in the urbanized or lowland areas.

In Poland the cities usually have separate sewerage systems. As a consequence, most of urban drainage systems discharge storm water runoff directly to the receivers without any treatment, which negatively affects the surface water bodies, especially small streams flowing in the urbanized area. Also the problem of urban flooding during summer rainfalls is growing in the recent years. There is a serious need to look for and implement solutions for sustainable storm water management, including retention, treatment and protection of surface water bodies against pollution with field and urban runoff. Treatment wetlands perfectly fit in this role assuring storm water retention and treatment. They can also be used as buffer zones to protect the streams and rivers against pollution with surface runoff. In this chapter case studies of treatment wetlands for treatment of storm water runoff in the Pommerania Region in Poland are discussed.

In the 90-ties a rapid deterioration of the quality of near-shore seawater occurred in the Bay of Gdańsk. This was attributed first of all to insufficiently purified wastewater and polluted rivers. The situation was caused by inadequate purification resulting from the lack of, or poor, maintenance of purification plants. Since then, in the Gdańsk region, the Municipal Sanitation Inspection has been closing most beaches not only for swimming but also for sun-bathing and even for walking. This is especially true for beaches in Gdańsk-Jelitkowo, a district of hotels and popular recreation areas.

One of the major streams actually draining water to the Bay of Gdańsk in Jelitkowo is the Jelitkowski Stream in Poland. Its main tributary is the Rynaszewski Stream which, in the middle part, passes through the zoo in Oliwa. A series of treatment wetland aimed at limiting the load of organic matter discharged to the stream from the zoo was constructed in 1992. The performance of the system is described in this chapter.



H. Obarska-Pempkowiak et al., *Treatment Wetlands for Environmental Pollution Control*, GeoPlanet: Earth and Planetary Sciences,
DOI 10.1007/978-3-319-13794-0_6

## 6.1 Situation Before Installation of Hydrophyte Treatment Wetlands

Measurements of concentrations and loads of pollution carried out in the period from 1989 to 1991 identified a substantial increase of pollution in the zoo sector of the stream, mostly organic nitrogen and coli index.

Additional investigations led to the identification of major point and surface sources of pollution. In Table 6.1 these sources are listed together with estimated loads of organic nitrogen discharged to the stream from each of them.

The following areas are considered of particular interest: Large Pond, Small Pond, Oval Pond, Seals Pond, Hippopotamus Pond and exercise areas for animals located along the stream. Major surface sources of pollutants were identified as exercise areas for deers, goats and cows, while point sources are ponds inhabited with waterfowl and/or seals and hippopotamus.

The measurements of water quality were carried out in 1991 before construction of the treatment wetland and in the period 1992–1994. Sampling stations were situated in places indicated in Fig. 6.1 in such a way that loads of pollutants originating from various point and surface sources can be assessed. For example, sampling points 5 and 11 allowed calculation of loads coming from cages of beasts of prey.

In all samples, parameters characterizing concentration of organic matter ($BOD_5$, $COD_{Mn}$) suspended solids, nutrients ($PO_4^{3-}$, various forms of nitrogen) and coli index were measured. Samples were collected once a month. The collected samples were averaged over a 5 h period. Water flow was also measured in order to assess loads of pollution originating from various sources.

In Table 6.2, average concentrations of measured pollutants are listed along with values allowable in surface waters according to regulations in Poland. It can be easily noticed that the concentrations of organic nitrogen and the coli index are critical factors for quality of water in the Rynaszewski Stream.

**Table 6.1** Characteristic sources of pollution to the Rynaszewski stream in the zoo area in Gdańsk in Poland

| Location (km) | Source of pollution | Load of total nitrogen (kg/day) | Type of source |
|---|---|---|---|
| 1.2 | Exercise area for deers | 2.5 | Surface |
| Inflow to large pond | Small pond, water-fowl and exercise area for goats | 6.0 | Point |
| 1.9 | Oval pond | 42.0 | Point |
| Inflow to seals pond | Pond + seals pond | 5.0 | Point |
| 0.7 | Hippopotamus pond | 13.5 | Point |
| 0.6 | Exercise area for cows (Zebu) | 47.0 | Surface |
| 0.4 | Rynaszewski stream outflowing from the zoo in Gdańsk | 60.5 | Point |

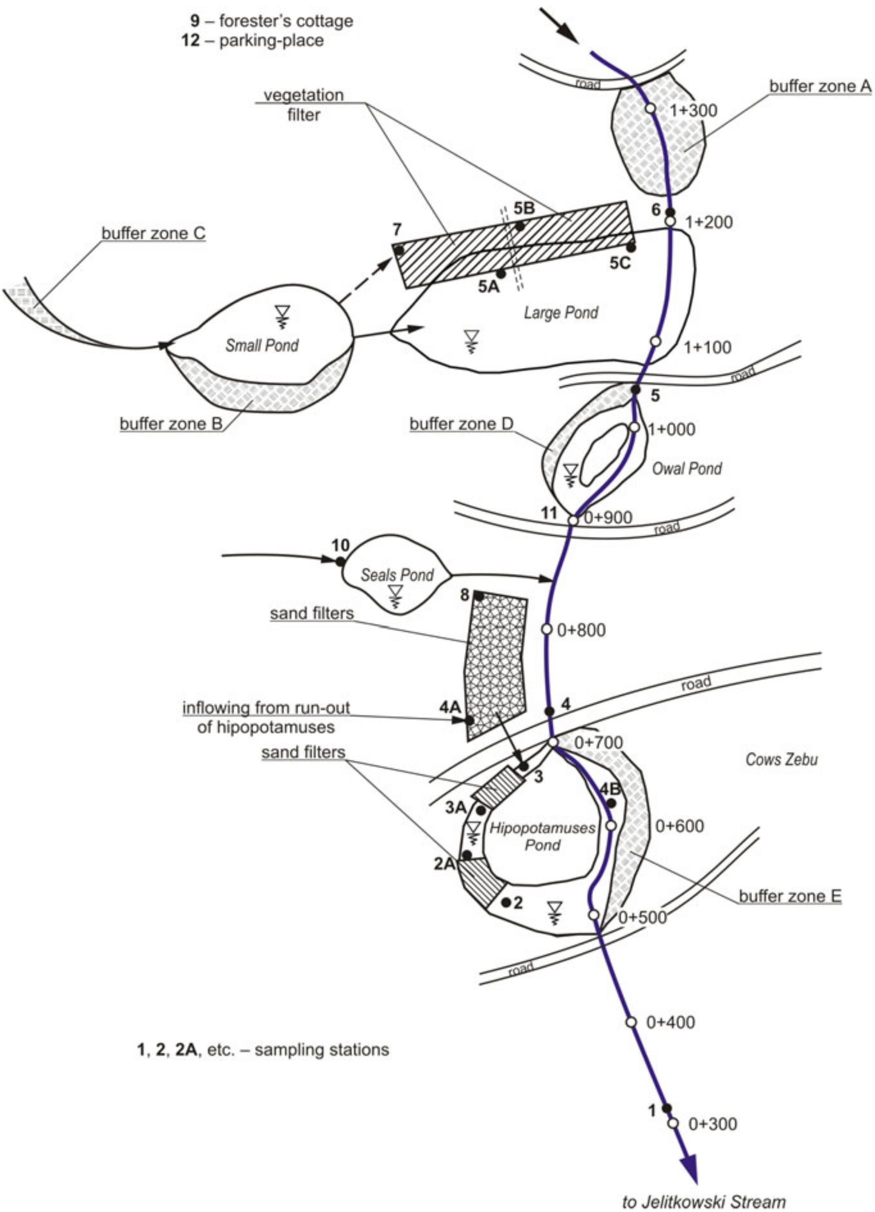



**Fig. 6.1** Configuration of the hydrophyte system and localization of sampling stations along the Rynaszewski stream

**Table 6.2** Average concentrations of organic substances, nutrients and coli index in water outflowing from the zoo area in 1991 compared to regulations

| Parameter | Unit | Outflow from the zoo | Classes of water cleanliness[a] | | |
|---|---|---|---|---|---|
| | | | I | II | III |
| TN | $g/m^3$ | 10.1 | ≤5 | ≤10 | ≤15 |
| $NH\text{-}N_4^+$ | $g/m^3$ | 1.6 | ≤1 | ≤3 | ≤6 |
| Org-N | $g/m^3$ | 8.0 | | | |
| $NO_3^-\text{-}N$ | $g/m^3$ | 0.4 | ≤5 | ≤7 | ≤15 |
| $NO_2^-\text{-}N$ | $g/m^3$ | tr | ≤0.02 | ≤0.03 | ≤0.06 |
| $PO_4^{3-}$ | $g/m^3$ | 0.6 | ≤0.2 | ≤0.6 | ≤1 |
| $COD_{Mn}$ | $g\ O_2/m^3$ | 10.0 | ≤10 | ≤20 | ≤30 |
| $BOD_5$ | $g\ O_2/m^3$ | 3.5 | ≤4 | ≤8 | ≤12 |
| Coli index | | 0.06 | 1≥ | 0.1≥ | 0.01≥ |

[a] I—clean, II—rather clean, III—polluted

Based on these findings, the following approach was adopted for protecting stream water from pollution originating in the zoo:

(a) pollutants, foremost organic nitrogen, should be retained as close to their source as possible; this would prevent dilution which makes their removal much more difficult,
(b) retention times of wastewater should be extended; this came from the observation that high concentrations of organic nitrogen were accompanied by low $BOD_5$ values—apparently biochemically stable nitrogen containing organic substances caused this situation,
(c) treatment of wastewater coming from various sources should be carried out in such a way that the landscape was not disfigured.

The implementation of the principles was carried out in the following way:

1. In order to reduce loads of pollution from point sources, treatment wetlands (natural vegetation: alder-trees, willow, reed), sand filters and vegetation filters were constructed in areas marked in Fig. 6.2. In order to further reduce organic nitrogen, the natural ponds localized along the stream within the zoo were

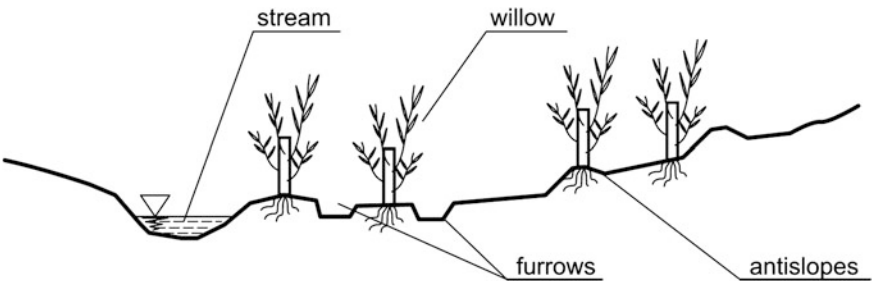



**Fig. 6.2** A buffer zone

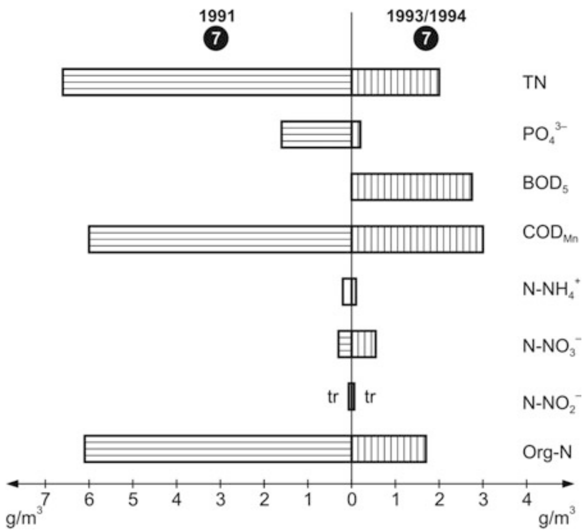
converted into ecological ponds with increased contents of aquatic plants and fish (Obarska-Pemkowiak et al. 1992).

2. In order to reduce loads originating from surface sources, a series of buffer zones inhabited with willow was constructed at areas indicated in Fig. 6.1. There are five buffer zones: A, B, C, D and E with the total area of 6,650 $m^2$. Buffer zones are defined as strips of land situated parallel to the stream, planted with willow (*Salix sp*) (Perttu 1994). They separate surface sources of pollutants from the stream. The strips are cut with furrows and antislopes (Mander et al. 1991). Both furrows and antislopes are designed to increase the retention volume and retention time of wastewater. A schematic illustration of a buffer zone is shown in Fig. 6.2.

Along the Small Pond two buffer zones were situated. One of them was intended to separate the pond from the exercise area for the deer, the other was constructed in a small valley inhabited by various species of birds. The reduction of pollution can be assessed from Fig. 6.3. A substantial reduction can be noticed. It is interesting that organic nitrogen was by far the most abundant fraction of nitrogen. In 1991, organic nitrogen constituted 92 % of the total nitrogen. In 1993/1994, the percentage of organic nitrogen decreased to 80 %. Still it is evident that in the Small Pond specific organic compounds were produced. Due to the installation of buffer zones, the retention time increased and part of the organic compounds was oxidized.

Water samples collected at sampling station no. 6, located downstream from the deer enclosure, showed substantial concentrations of nutrients and organic matter in 1991 (Fig. 6.4). The concentrations were, on average, three times higher than those measured upstream from the enclosure. After construction of buffer zone A and a vegetation filter, concentrations of contaminants decreased by a factor of 3. Again concentrations of organic nitrogen were comparable with concentrations of $BOD_5$,

**Fig. 6.3** Concentration of contaminants after the buffer zone situated along the small pond



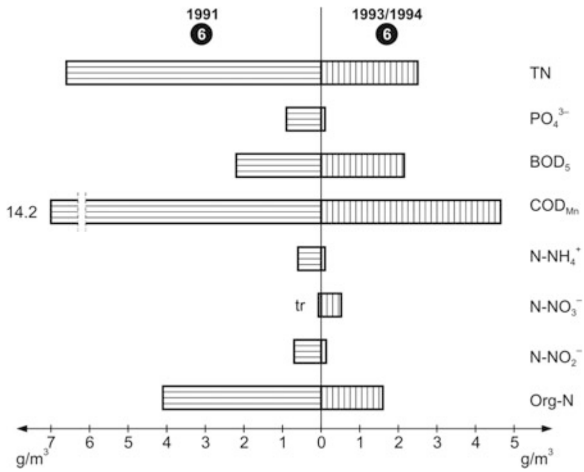



**Fig. 6.4** Concentration of contaminants at the entrance to the large pond (sampling station no. 6) downstream of a buffer zone situated along the exercise area for the deer

indicating that specific organic compounds characterized by high content of nitrogen and resistance to biochemical oxidation are present in water.

In Fig. 6.5, a diagram is presented that reflects concentrations of various forms of pollutants in water inflowing to and outflowing from the Oval Pond situated close to cages of beasts of prey. The diagram shows inflowing concentrations on the right, whereas on the left the concentrations refer to outflowing water before and after construction of a buffer zone separating cages and the pond. As can be seen,

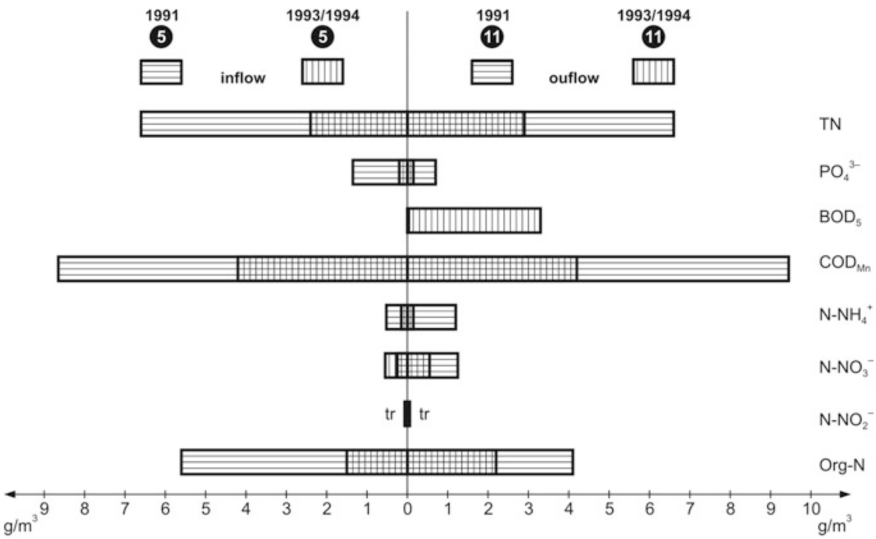



**Fig. 6.5** Concentration of contaminants upstream (sampling station no. 5) and downstream (sampling station no. 11) from the oval pond in 1991 and 1993/1994

**Table 6.3** Average yearly concentrations of organic substances and nutrients in water inflowing and outflowing from the zoo area

| Parameter | Concentration ($g/m^3$) | |
|---|---|---|
| | 1991 | 1993/1994 |
| | inflow / outflow | inflow $\pm \delta$ / outflow $\pm \delta$ |
| TN | 3.1 / 10.1 | $21 \pm 0.7$ / $24 \pm 0.6$ |
| $N\text{-}NO_4^{+}$ | 0.0 / 16 | $0.20 \pm 0.08$ / $0.09 \pm 0.04$ |
| Org-N | 3.1 / 8.0 | $1.26 \pm 0.51$ / $1.77 \pm 0.72$ |
| $N\text{-}NO_3^{-}$ | 0.0 / 0.4 | $0.58 \pm 0.21$ / $0.53 \pm 0.14$ |
| $N\text{-}NO_2^{-}$ | tr / tr | $0.013 \pm 0.01$ / $0.011 \pm 0.01$ |
| $PO_4^{3-}$ | 0.42 / 0.60 | $0.21 \pm 0.04$ / $0.13 \pm 0.03$ |
| $COD_{Mn}$ | 29 / 100 | $7.4 \pm 1.8$ / $4.8 \pm 1.2$ |
| $BOD_5$ | 2.5 / 3.5 | $2.5 \pm 1.2$ / $2.8 \pm 1.3$ |
| Coli index | x[a] / 0.06 | x[a] / 0.8 |

[a] Not determined

concentrations in the outflowing water in 1993/1994 are, on average, half the concentration in 1991. This is attributed to the improved waste management upstream of the Oval Pond, to the installation of the buffer zone planted with willow, and also due to the increased amount of duck-weed growing in the pond in 1993 and 1994. The fact that concentrations of pollutants in the inflowing water were smaller can be taken as proof that treatment wetland situated upstream of the Oval Pond worked well. No increase in concentrations in the Oval Pond was found, indicating that the buffer zone located along the pond worked well too.

Table 6.3 lists average yearly concentrations in inflowing and outflowing water from the zoo in 1991 (the first column) and in 1993/1994 (the second column). Assuming comparable flows of water in 1991 and in 1993/1994, equal to 70 l/s, the load of organic nitrogen retained by the protection measures is 46.6 kg/day (17 t/year).

## 6.2 Conclusions

1. The Rynaszewski Stream, a tributary of the Jelitkowski Stream which flows into the Bay of Gdańsk in poland, gains a load of organic nitrogen equal to ~5,600 PE in its stretch passing through the zoo in Oliwa.

2. Construction of a series of treatment wetlands led to a substantial reduction of the load, together with the improved hygienic status of the stream.
3. The treatment wetlands consisted of five buffer zones and four filters of various types.
4. The performances and characteristic features of three buffer zones inhabited with willow are described in the paper.

## 6.3 Surface Water Protection—TW System in Bielkowo for Agricultural Areas

In order to protect the surface water intake at the Goszyn Lake (Straszyn Reservoir), a treatment wetland system was constructed at the Stream receiving the waters from Bielkowo village, which directly inflows to the Lake. The wetland system was designed as a reservoir surrounded with ground slopes, consolidated with fascine and turf. The total area of the reservoir was 6,200 $m^2$; the volume was equal to 5,000 $m^3$. Inside the reservoir a set of filtration dykes was constructed. The system consisted of two sections (Fig. 6.6):

- wet section (pond) filled with water all the time (retention time 24 h and water flow 32 l/s)
- dry section designed for storm water (maximal flow 640 l/s and retention time 0.5 h).

In the periods of dry weather the level of water in the pond decreases. The sediments of the dry section emerge and become a meadow on such occasions. After heavy rainfall events the water level increases until the water overflows the dams and outflows to the stream below. Meanwhile, the first and probably the most contaminated wave of storm water is safely retained in the system.

Investigation results proved that the treatment wetland ensured decreasing the concentrations of total suspended solids, total nitrogen and total phosphorus (Table 6.4). The $BOD_5$ and COD removal effectiveness was lower (Obarska-Pempkowiak et al. 2002; Wojciechowska et al. 2004).

During the first 2 years of operation, due to mass algae blooming and lack of roots cultivating the ground, the surface of the dams was covered with a thick mat of biomass. This resulted in clogging of the dams and flooding of the “dry” section with water.

During the visits to the facility and collecting the samples of water it was observed that the water level was above overflow crest for the whole time. This means that the “dry” section was covered with water all the time and there was no retention volume for storm water run-off.

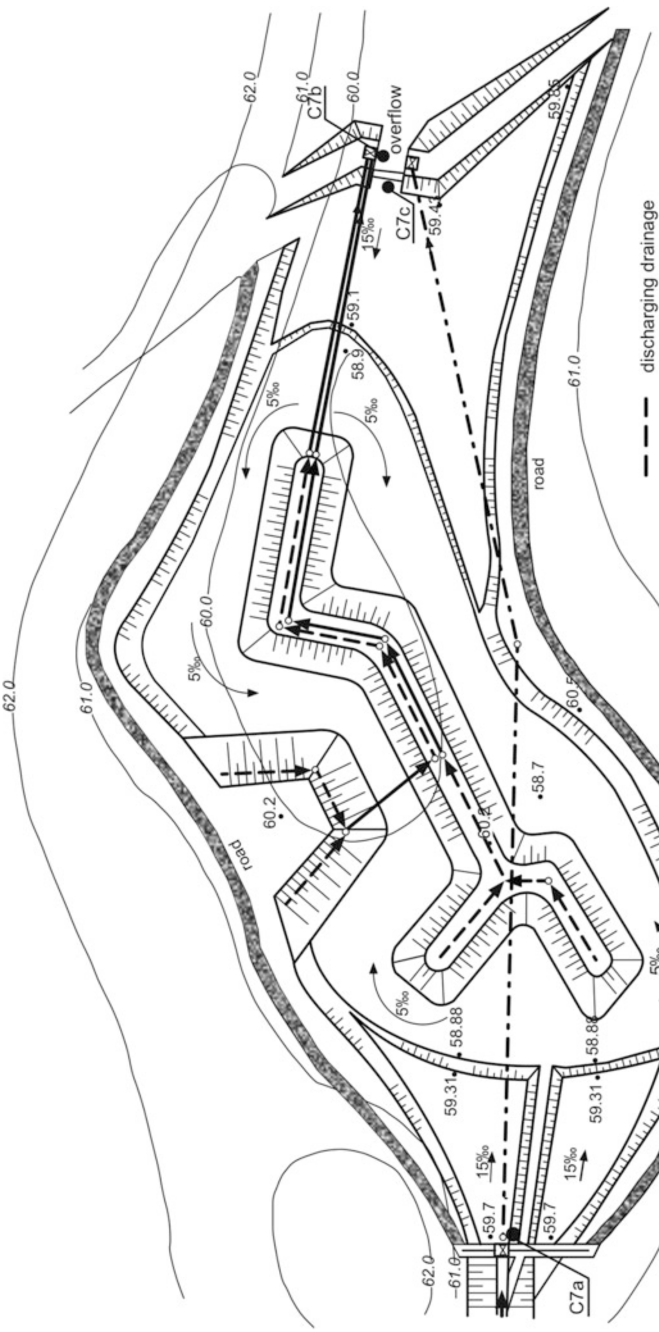



**Fig. 6.6** The scheme of Bielkowo TW for agricultural run off

**Table 6.4** Mean efficiency of contamination removal in Bielkowo, in %

| Parameter | $\eta_d$ | | $\eta_w$ | | $\eta_T$ | |
|---|---|---|---|---|---|---|
| | 2000 | 2001 | 2000 | 2001 | 2000 | 2001 |
| TSS | 39.3 | 66.6 | 81.4 | 95.6 | 20.4 | 62.2 |
| $BOD_5$ | 35.3 | 34.5 | 75.3 | 82.3 | 10.0 | 16.1 |
| COD | 33.0 | 27.2 | 76.0 | 78.3 | 8.4 | 5.9 |
| TN | 34.4 | 47.5 | 75.6 | 86.9 | 10.0 | 31.3 |
| TP | 38.8 | 38.7 | 77.4 | 73.8 | 16.2 | 12.5 |
| $O_2$ | 26.4 | 33.2 | 73.8 | 78.3 | 24.3 | 11.5 |

$\eta_d$—efficiency removal in dry section, $\eta_w$—efficiency removal in wet section, $\eta_T$—efficiency removal of the entire system

## 6.4 Storm Water Treatment in TWs

Rain events and in consequence storm water are characterized by very high fluctuations in time and unpredictability. Prosperities of storm water depends of many factors and are changing in time of the events. Generally the most polluted are storm water generated during rains evens after long dry period in big urban areas. They could have different compositions but they contain suspended solids, organics and biogenic compounds, heavy metals, oils contaminations as well as persistent organic pollutants (Garbarczyk and Gwoździej-Mazur 2005; Magill and Sansalone 2010).

Most of European countries have common sewer and storm water systems. In many of them like France and German treatment wetlands are used to treat the over flow securing both mechanical and biological treatment and moreover ensuring wave flatting and retention water in the catchment. In Fig. 6.7 the treatment wetland with vertical subsurface flow for CSO (combined sewer overflow) is shown. Such systems are very popular in France and are designed to ensure as much as possible retention volume for storm events.

Treatment wetlands could be a good alternative solutions for treatment of storm water from high—roads too (Revitt et al. 2004). In Norway it is a common solutions for treatment of tunnel wash (water used for maintenance) and storm water events like it is presented in Fig. 6.8. Investigations done by Paruch and Roseth (2008) in the FWS TW confirmed very effective removal of heavy metals and persistent organic pollutants.

According to Shutes et al. (1999) treatment wetlands for highway runoff should be preceded by an oil separator. The system itself should consist of a hydrophyte pond and a reed bed. It is also suggested to add a final polishing sedimentation tank before discharge of treated storm water to the receiver. According to British experiences the area of a pond should be equal to 2–3 % of the catchment area while minimal retention volume is 100 $m^3$/ha of catchment area. Pre-treatment of the runoff before discharge to treatment wetland is advised to remove fine solids that could cause clogging of subsurface flow beds. In cases when larger land areas are available, especially in rural and sub-urban areas, the treatment wetland should

**Fig. 6.7** The view of VFTW for combine sewer over flow in France near Lion (*Photo* M. Gajewska)

**Fig. 6.8** Free water surface wetland for treatment of tunnel and road wash in Norway (*Photo* A. Paruch)

be dimensioned to retain the flood flows. Otherwise it is only designed to retain the first flush while the excessive flow is by-passed to the receiver. In such a case the minimum retention time for the subsurface flow system is 30 min for the design rainfall (usually the rain with 100 % probability). Longer retention times result in better treatment efficiency. The optimum retention time would be several hours, preferring 24 h. The maximum hydraulic load should not exceed 1 $m^3/m^2$ day. The inlet velocity in the range 0.3–0.5 m/s is recommended since velocities exceeding 0.7 m/s may damage the plants (Shutes et al. 1999). The final sedimentation tank with minimal retention volume equal to 50 $m^3$ is recommended to remove fine solids.

The treatment efficiencies over 80–90 % for ammonia nitrogen and total suspended solids were reported (Carleton et al. 2001; Revitt et al. 2004a, b). There are several reports in the literature confirming effective removal of heavy metals and BTEX from highway runoff (Mungur et al. 1995; Thurston 1999; Tromp et al. 2012).

The subsurface flow beds are also used for treatment of the storm water runoff in the airports, contaminated with de-freezing substances, usually ethylene glycol. In many airports the systems of de-freezers recovery are applied, however the maximal efficiency is 60 %, which leaves large quantities of ethylene glycol remaining in the runoff. The concentrations of ethylene glycol can be as high as 1,400 mg/l. The $BOD_5$ concentrations can reach 15,000 mg $O_2$/l (Wallace and Liner 2010). Treatment wetlands are used to treat airport runoff in Edmonton (Canada), Heathrow (Great Britain) and Buffalo (USA) (http://naturallywallace.com/).

In Poland, storm water collected by drainage systems (separately from sewer system) is usually discharged directly to the receiver, without treatment. This practice has a significant impact on surface waters quality, especially in case of smaller streams flowing through urbanized area (Obarska-Pempkowiak et al. 2010). Very special situations occurs in Pomerania Region where there are numerous short streams which end up in the Baltic Sea. Very often they are becoming the only possible recipient of storm water discharged during rain events. It is estimated that Babilonski Strem in the Gdańsk Region is supplemented with 23.5 kg TSS/day and 7.8 kg TN/day during rain events (Materials of City Hall 2011). Thus there is an urgent need to treat the overflow of storm water.

Treatment wetland system for treatment of urban runoff was constructed on Swelina Stream in Sopot in 1994, in order to protect the Stream against pollution. The Swelina Stream discharges its waters directly to the Gulf of Gdańsk, near popular bathing areas. The Stream receives drainage waters from the surrounding area. The system consists of sedimentation-retention tank and horizontal gravel-filled bed planted with common reed (*P. australis*) (Fig. 6.9).

The treated water is collected by drainage pipes, outflows to a control well and then it is discharged back to the stream. During intensive rainfall, the first, most polluted part of drainage is collected in a retention reservoir, while the rest of water is discharged through an overflow to the stream (without treatment). The system was built in order to remove the nutrients, mainly phosphorus, and faecal bacteria discharged with drainage. After the TW was constructed, a significant improvement

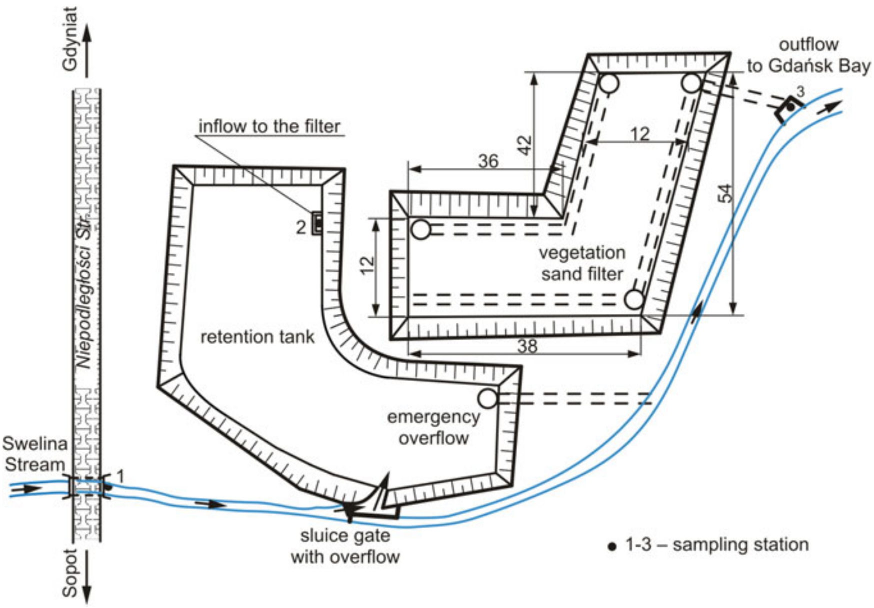



Fig. 6.9 The scheme of TW at Swelina stream

of the Stream quality was observed (Obarska-Pemkowiak et al. 2011a, b). The analyses carried out by the Regional Inspection of Environment Protection in Gdańsk indicated that Swelina Stream waters fulfill criteria of the first class waters.

Monitoring of Swelina Stream quality downstream and upstream of the TW system during rainfall events was performed within the research project "Innovative resources and effective methods of safety improvement and durability of buildings and transport infrastructure in the sustainable development" financed by the European Union from the European Fund of Regional Development based on the Operational Program of the Innovative Economy. The content and compositions of suspended solids and organics in discharged storm water as well as after subsequent stage of treatment were fluctuating and depends on weather conditions (Figs. 6.10 and 6.11).

Urban drainage contains high concentrations of TSS. The size of solids is a crucial parameter, determining sorption abilities and the way the solids settle. Smaller fractions, which are difficult to remove during conventional treatment processes, are responsible for migration of pollutants in aquatic environment, since they act as carriers of hydrophobic organic micropollutants, nitrogen and phosphorus compounds and heavy metals. Within the project, the granulometry of TSS present at the inflow and at the outflow of TW system on Swelina Stream was investigated to find the ability of the system to retain different fractions of suspended solids.

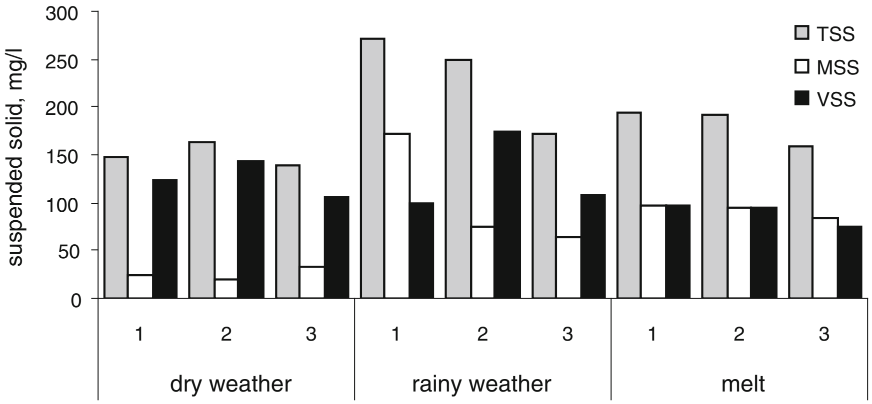



**Fig. 6.10** The content of suspended solids after subsequent stages of treatment in Swelina TW during different weather conditions (1, 2, 3—sampling station)

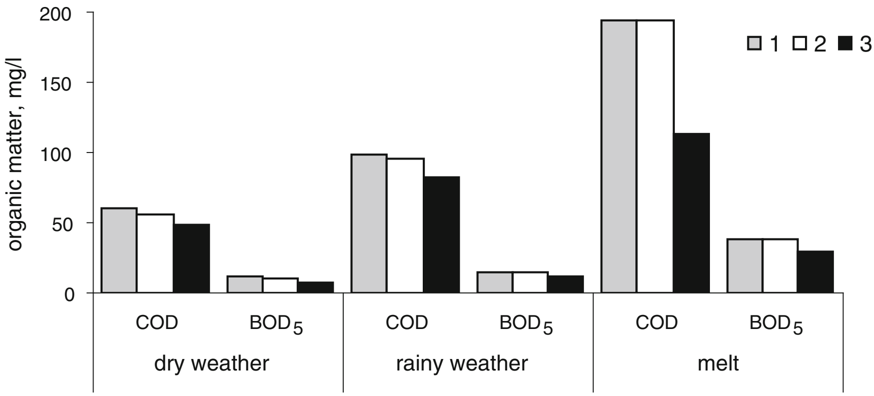



**Fig. 6.11** The content of organic matter (COD and $BOD_5$) after subsequent stages of treatment in Swelina TW during different weather conditions (1, 2, 3—sampling station)

Based on carried out long-term investigations it could be concluded:

- Concentrations of pollutants delivered together with storm water and waters of the Swelina Stream could be potential risk for quality of water in Gulf of Gdańsk.
- Pollutants delivered together with storm water and waters of the Swelina Stream were characterized by wide range of equivalent diameters typical for both, colloidal pollutants and finer suspended solids.
- The applied treatment system consisting of the pond and the vegetated bed (SSHF) was efficient in removing suspended solids.

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# Chapter 7
# Reject Water from Digested Sludge Centrifugation Treatment in HTW

Since EU Directive imposed the limit on total nitrogen concentration in treated outflow, not more than 10 mg TN/l, at WWTPs above 100,000 pe in the year 2010, local authorities and WWTP operators are still trying to improve the treatment processes towards minimizing concentration of pollutants in the inflow. One of the activities which could easily decrease final nitrogen concentration in treated wastewater is minimizing the impact of reject water return flow. Sewage sludge is a by-product of wastewater treatment and usually sludge processing at WWTP (over 100,000 pe) comprises a digestion process with biogas production and then mechanical dewatering, which generates filtrate containing high concentration of solids, both dissolved and suspended (also called reject water). The reject water is characterized by a very high concentration of nitrogen, mostly in the form of $NH_4^+$-N and organic matter (expressed as Chemical Oxygen Demand—COD) as well as total suspended solids (TSS). The most promising way of handling reject water is pre-treatment before it returns to the first stage of treatment in WWTP. High-tech solutions such as unconventional methods (Anammox, Sharon, etc.) are usually applied, despite high costs. An alternative solution could be the application of treatment wetlands (TWs). TWs are successfully used to either treat or polish landfill leachate (which has similar properties as reject water). These systems are inexpensive, simple in operation and have potential to remove not only organic substances and nitrogen compounds, but xenobiotics and heavy metals as well.

The objectives of the investigation was, among other, to consider the application of Hybrid Treatment Wetland (HTW) to treat highly polluted wastewater, namely reject water from centrifuge (RWC) after sewage sludge dewatering.

## 7.1 The Composition of Raw Wastewater and Reject Water

The object of the investigation was municipal wastewater treatment plant in Gdańsk. The WWTP is supplied with wastewater from Gdańsk city and its region. The share of industrial wastewater in the total wastewater inflow is 10 %. The



H. Obarska-Pempkowiak et al., *Treatment Wetlands for Environmental Pollution Control*, GeoPlanet: Earth and Planetary Sciences,
DOI 10.1007/978-3-319-13794-0_7

**Table 7.1** Comparison of the quantity of wastewater and generated reject water (Gajewska and Obarska-Pempkowiak 2008)

| Year | Raw wastewater (RW) | Reject water from sludge press (RWP) | | Reject water from sludge centrifuge (RWC) | |
|---|---|---|---|---|---|
| | $m^3$/day | $m^3$/day | %[a] | $m^3$/day | %[a] |
| 1st | 88,000 (±17,250) | 2,100 (±190.7) | 2.38 | 644.0 (±120.8) | 0.73 |
| 2nd | 85,034 (±16,850) | 1,980 (±181.1) | 2.33 | 603.0 (±190.2) | 0.71 |
| 3rd | 83,764 (±15,346) | 1,880 (±183.7) | 2.24 | 616.0 (±110.6) | 0.74 |

[a] Percentage share of reject water in raw wastewater

wastewater processing at Gdańsk WWTP consists of mechanical and enhanced biological treatment.

The reject waters generated during mechanical thickening and dewatering of sewage sludge are directed to the technological line of sewage treatment (after the screens). In Table 7.1 the quantity of generated reject water from sludge press (RWP) and reject water from sludge centrifuge (RWC) were compared to flow of raw wastewater.

During the wastewater treatment process two types of sludge are generated: raw sludge from primary settling tanks (average d.m. content—3.5 %) and excess secondary sludge from secondary settling tanks (average d.m. content 5.8 % after mechanical thickening). Primary and secondary sludge are mixed in a separate chamber of 50 $m^3$ volume, thickened and pumped into two digestion chambers with working volume of 7,000 $m^3$ each. After this process (recovery of biogas) the sludge is dewatered by centrifuge wafter supplementing with polymers to incense dewatering effects. The final content of dry mass is equal to 24.0 %. The daily inflow of sewage and daily amount of generated sludge for atypical year is given in Table 7.2.

**Table 7.2** The average daily wastewater inflow to the WWTP, the amount of generated sludge and reject water from sludge thickening and dewatering (Gajewska and Obarska-Pempkowiak 2008)

| Type of wastewater[a] or sludge | Average daily quantity of sewage/sludge ($m^3$/day) | Standard deviation |
|---|---|---|
| Raw wastewater | 88,000 | ±4.8 |
| Reject water from sludge press (RWP) | 2,100 | ±190.7 |
| Reject water from sludge centrifuge (RWC) | 644 | ±120.8 |
| Primary sludge | 500 | ±95.2 |
| Excess sludge | 2,500 | ±1203.4 |
| Thickened excess sludge | 270 | ±101.4 |
| Digested sludge | 770 | ±139.5 |
| Dewatered sludge | 140 | ±31.7 |

[a] Volume of wastewater after mechanical and biological treatment corresponds to the volume of raw wastewater and reject water from presses and centrifuges

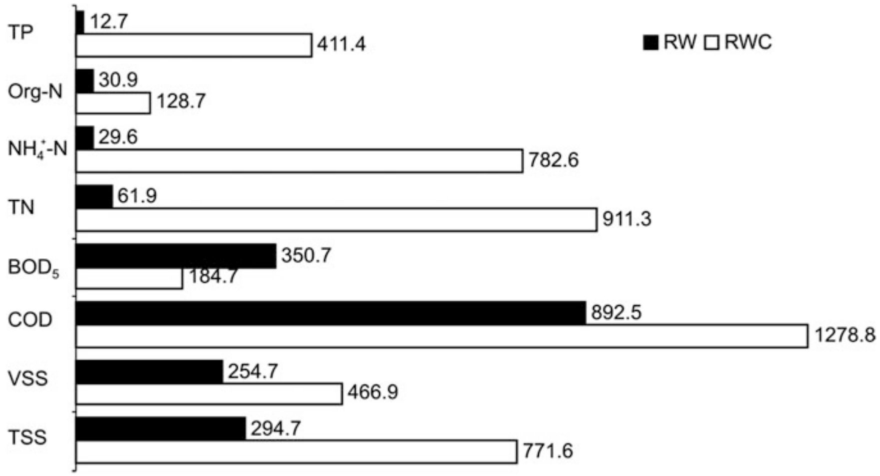
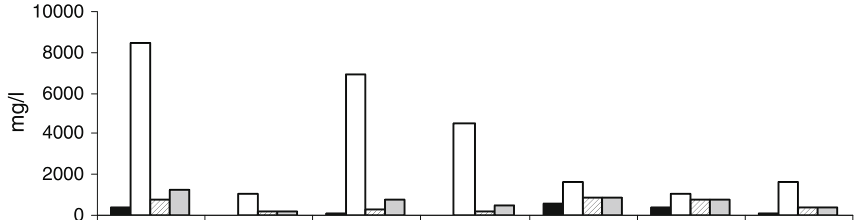



**Fig. 7.1** The characteristic of RWC pollutants in comparison to RW pollutants in WWTP in Gdańsk (concentration are given in mg/l)

The reject water generated during thickening of waste sludge (RWP) was characterized by high fluctuation of pollutants concentration in time. The maximum values could be even 50 times higher than the minimum values but they were still similar to the pollutants concentrations in raw wastewater (Gajewska and Obarska-Pempkowiak 2008). In consequence the return flow of RWP had no negative impact on WWTP operation and final pollutant concentration in treated sewage (Gajewska and Obarska-Pempkowiak 2008). The characteristics of the second type of reject water generated during mechanical dewatering of digested sludge on centrifuge (RWC) in comparison to the raw sewage influent to the WWTP Gdańsk is given in Fig. 7.1.

The TSS concentration in RWC was over two times higher in comparison to the corresponding concentration in RW. The share of VSS in the TSS was equal to 60.0 % for RWC and 91.1 % for raw sewage. This indicates that some 40 % of TSS in RWC is mineral. Such large amount of mineral suspended solids is a useless ballast during treatment and could caused problems during biological processes. The $BOD_5$/COD ratio was as follows: RWC—0.14 and RW—0.4. The $BOD_5$/COD ratio of RWC was surprisingly low in comparison to the values given for mature landfills reported in literature (Lo 1996; Klimiuk et al. 2007). Ratios obtained in this study were slightly lower than the one given by Fux et al. (2006), which was equal to 0.2 for the reject water from the WWTP in Minworth, Great Britain. Furthermore, such a low $BOD_5$/COD ratio reflects low degradability of the organic compounds where the easily biodegradable organics ($BOD_5$) have been already consumed (Surmacz-Górska 2001; Kjeldsen et al. 2002; Bulc 2006; Wojciechowska et al. 2010).




| | COD | $BOD_5$ | TSS | VSS | TN | $NH_4^+$-N | TP |
|---|---|---|---|---|---|---|---|
| min | 400 | 7,6 | 64 | 27 | 600 | 353 | 56 |
| max | 8493 | 1078 | 6954 | 4542 | 1600 | 1100 | 1595 |
| median | 813 | 187 | 279 | 193,5 | 900 | 734 | 393 |
| average | 1278,8 | 184,7 | 771,6 | 466,9 | 911,3 | 782,6 | 411 |

**Fig. 7.2** The pollutants concentration characteristics in RWC in WWTP in Gdańsk

The $BOD_5/COD_f$ ratio was significantly higher: RWC—0.52 and RW—1.1. Low values of the $BOD_5$/COD ratio and high $COD_f$ concentrations in the filtered samples indicated that organics were mostly present in the form of hardly decomposable suspension.

The TN concentration in RWC was over ten times higher than in raw sewage. The share of two dominating forms of nitrogen ($NH_4^+$-N and Org-N) in the TN concentration was equal to: 81.6 % $NH_4^+$-N and 13.8 % Org-N (Fig. 7.2) (Gajewska and Obarska-Pempkowiak 2008).

The RWC was characterized by a very high concentration of nitrogen, mainly in the form of ammonium nitrogen, and organic matter as well as TSS. Another problem with reject water management was connected with its irregular generation and a huge fluctuation of pollutant concentrations (Fig. 7.2). Maximal concentrations were often over 10 times higher than mean values, which substantially affects the average values and standard deviations. The analysed RWC exposed similar properties as described in literature. Fux et al. (2002, 2006) indicated that ammonia nitrogen and TSS concentrations in the reject water from the WWTP in Luggage Poit, Australia, varied from 943 to 1,710 $NH_4^+$-N mg/l and from 95 to 6,132 mg TSS/l. In the RW from WWTP in Minworth, Great Britain, the ammonia nitrogen concentrations changed from 450 to 750 mg/l, and TSS from 220 to 2,340 mg/l (Fux et al. 2003). At two WWTPs in Switzerland, the reject water from the sludge fermentation process was similar, and contained 657 (±56) mg/l and 619 (±21) mg/l (ammonium nitrogen), and 344 (±112) mg/l and 384 ± 137 mg/l (TSS) according to Hans et al. (1997) and Jeavons et al. (1998).

## 7.2 Estimation of RWC Return Flow Impact on WWTP Operation

The quality of raw wastewater, treatment technology as well as sludge processing and mechanical dewatering have an influence on RWC composition. According to Karvelas et al. (2003) almost 50 % of Cr, Pb, Ni, Cd and Zn daily input to WWTP ends up in the sludge and another 50 % is released with the final outflow stream. In the analyzed WWTP, the concentrations of Cd, Cu Pb and Zn in the digested sludge were 11–68 % higher than corresponding concentrations in undigested sludge. Similar large differences in heavy metals concentrations before and after digestion were described by Chipasa (2003). These higher concentrations of heavy metals in digested sludge were caused by weight loss of fresh sludge during anaerobic digestion, whereby loss of degradable organic and inorganic matter (Obarska-Pempkowiak et al. 2007). However, due to the mechanical dewatering, the load of heavy metals in final product (dewatered sludge) were lower as compared to the loads in primary or excessive sludge. Additionally, high concentrations of heavy metals in RWC, especially in aqueous phase, confirmed that during mechanical dewatering the significant amount of heavy metals was released to the aqueous phase. Thus, the return flow of RWC would contribute to the increase of heavy metals concentrations in the wastewater. Obtained results confirm that highly effective biological methods of wastewater treatment lead to an increase of heavy metals accumulation in sewage sludge, especially in digested sludge. Biological treatment process caused change in speciation of lead. In wastewater after mechanical treatment the concentration of lead was equal to 33 % of discharged loads while after biological treatment the concentration increased to 66 %.

Speciation of heavy metals differed along the treatment process. Four basic forms—species were analysed is suspension, extendable, carbonates organic and residual (Obarska-Pempkowiak et al. 2007). The average percentage share of each extractable fraction of the analyzed heavy metals in wastewater and reject water is presented in Fig. 7.3. Sequential extractable procedure was done according to wide accepted scheme given by Tessier et al. (1997) and guidelines given by Alonso et al. (2000). Application of the four–stage sequential extraction procedure proposed by the European Community Bureau of References (BCR) yields four fractions of analyzed heavy metals (Cu, Pb, Cd and Zn): exchangeable (I), reducible (II), organic (III) and residual (IV). The analyses of extractable fraction distribution of heavy metals in wastewater indicated that in raw wastewater heavy metals were present mainly in organic (III) and residual (IV) fractions except for Zn. Zinc was mostly found in a form of exchangeable fraction (I) (Fig. 7.3). In reject water, heavy metals were present in all four fractions. Reject water from sludge press was a significant source of Zn, Cd and Pb in exchangeable fraction (I) (26.5, 9.5, 7.8 %—respectively) and reducible fractions (II) (19.4, 22.6, 21.7 %—respectively).

In a consequence, after mechanical stage of treatment, significant increase of heavy metals concentration in mobile fraction (even up to 6 % for Cd in both I and II fraction and 7.5 % for Pb I fraction) was observed. During biological treatment in

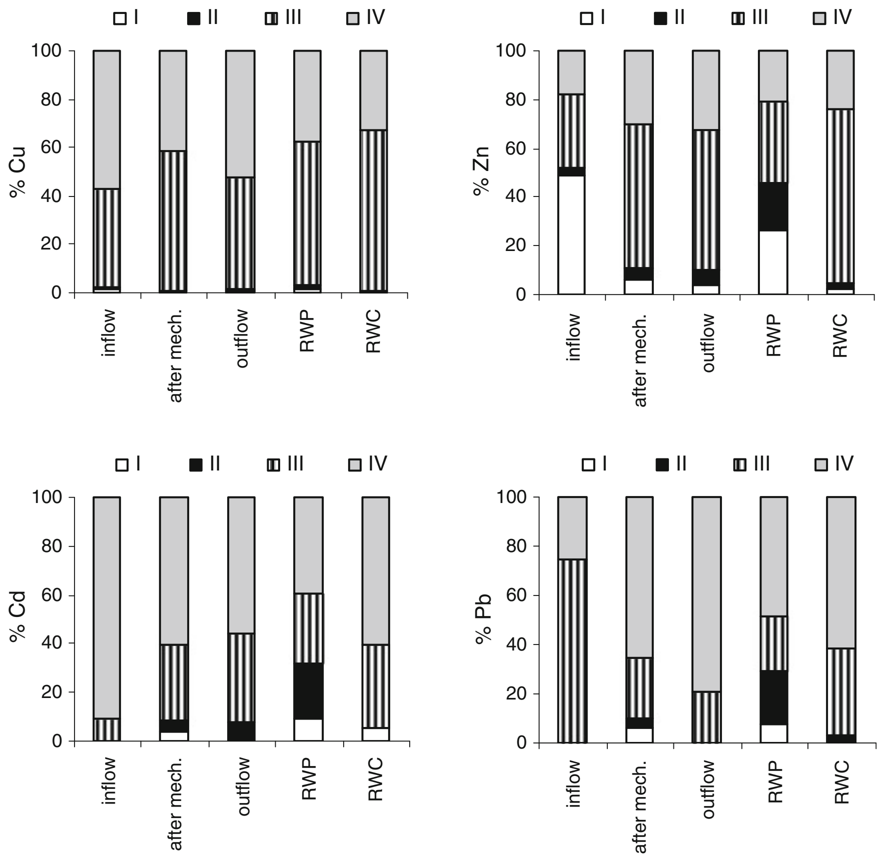



**Fig. 7.3** The percentage share of each extractable fraction of heavy metals in wastewater and RWP and RWC (*I*—easily extracted, *II*—carbonates, *III*—organic, *IV*—residual), % (Obarska-Pempkowiak et al. 2007; Obarska-Pempkowiak and Gajewska 2008)

multistage bioreactor, some portion of heavy metals present in labile fraction created stable bindings (organic and residual). Although a part of heavy metals in reducible fraction (which is bioavailable) was discharged with treated wastewater (Cu = 1.5, Zn = 110.0, Cd = 3.8, Pb = 2.5 μg/g d.m.) and create potential risk for the recipient (Obarska-Pempkowiak et al. 2007).

Based on carried out investigation in WWTP in Gdańsk it was indicated that the higher concentration of heavy metals in reject waters (from 2 to 10 times) in comparison with wastewater indicated, that metals were accumulated in sludge and that some part of them were released to reject water (RWP & RWC) during thickening and dewatering processes. Return flow of reject water at the beginning of treatment caused an increase of labile heavy metals fraction (exchangeable and reducible).

Although, return flow of RWC consisted only 0.7 % of daily flow of raw wastewater discharged to the WWTP in Gdańsk (Table 7.1), the load of nutrient was significant and could alter the activated sludge process in a conventional WWTP. It was assumed that together with RWC from 7.4 to 9.7 % of TN and from 20.5 to 25.1 % of TP was returned to the plant (Gajewska and Obarska-Pempkowiak 2008). In WWTPs with sewage sludge digestion, 15–20 % of the nitrogen load is usually redirected with the reject water (Fux et al. 2006). While the remaining COD after anaerobic digestion is generally quite low and poorly biodegradable, a separate treatment of the high nitrogen content in this stream can considerably reduce the total nitrogen concentration in the final outflow from WWTPs (Wett and Alex 2003; Laurich and Gunner 2003).

## 7.3 Characteristic and Dimensioning of Pilot Plant for RWC Treatment

It was assumed that the pilot treatment plant would consist of mechanical and biological part. According to technological laboratory analyses carried out on RWC, aeration or pH control did not improve significantly the quality of RWC. Thus, only sedimentation could lower the content of pollutants from 20 to 40 % (TSS, COD and TN). It was assumed that the mechanical treatment will consist of two 1 $m^3$ chambers working in series, where wastewater would be collected and equalized.

Since the characteristic of RWC is similar to land file leachate and Treatment Wetlands (TWs) have been successfully applied for landfill leachate treatment in the USA and Europe, including the temperate and sub-polar climate regions (Maehlum 1995; Peverly et al. 1995; Kowalik et al. 1996; Bulc et al. 1997; Martin et al. 1999; Cheng et al. 2002; Johansson Westholm 2003; Kadlec 2003; Randerson and Slater 2005; Bulc 2006; Rustige and Nolde 2006; Kinsley et al. 2006; Wojciechowska and Obarska-Pempkowiak 2008) the idea of new application for RWC treatment has arisen.

Both surface and sub-surface flow TWs (usually horizontal) as well as plants which consist of a several stages with varying flow conditions are applied. Treatment wetlands create the environment for hydrophytes growth, where both aerobic and anaerobic decomposition processes are enhanced. These processes, supported by sorption, sedimentation and assimilation, are responsible for pollutant removal. It was proved that in TWs inhabited by *Phragmites australis*, the redox potential is changing from +200 to −300 mV, which means that during the decay processes, $NO_3^-$, $SO_4^{2-}$ and other ions can act as electron acceptors while organic compounds are electrons donors (Kadlec 1995; Reddy and D’Angelo 1996; Vymazal et al. 1998; Vymazal 2001; Obarska-Pempkowiak et al. 2010b). Such fluctuating conditions together with a long retention time favors the degradation of many—often toxic—compounds such as THM, detergents or PAH (Van der Hoek et al. 1999). Especially good environment for the mineralization of organic compounds and oxidation of nitrogen compounds develops in TWs with a vertical flow of wastewater. Such beds are intermittently loaded with wastewater, which results in better aeration. During resting periods, the accumulated organic matter is decomposed, which protects the beds against clogging (Kayser et al. 2001; Gajewska et al. 2004).

In France, vertical subsurface flow beds are applied for the treatment of raw wastewater (without mechanical pre-treatment) (Boutin et al. 1997; Molle et al. 2004). According to Molle et al. (2004) two vertical subsurface flow beds (VSSF) working in hydraulic batch mode, provide very effective wastewater treatment. The unit area of the first VSSF bed should be equal to 1.5 $m^2$/pe (person equivalent), while the unit area of the second bed is only 1.0 $m^2$/pe. This configuration of VSSF beds allows for reduction of pollutant concentrations to the following level: COD—60 mg/l, TSS—15 mg/l, Kjeldahl nitrogen—8.0 mg/l. Molle et al. (2004) recommend that the hydraulic loading of the beds working in batch mode should be below 600 mm/d. Such operating conditions, the beds provide good and stable pollutant removal efficiencies. According to Molle et al. (2004) the average pollutant removal efficiencies for the first bed (VSSF I) were as follows: 82 % COD, 89 % TSS and 60 % Kjeldahl nitrogen. For the second bed (VSSF II), the following efficiencies were reported: 60 % COD, 72 % TSS and 78 % Kjeldahl nitrogen. According to Molle et al. (2004), the layer of sediments deposited on the surface of the first bed not only does not interfere the treatment process, but it even enhances the overall treatment process.

In biological part the hybrid treatment wetland (HTW) was designed. HTW consisted of three beds working in series: VSSF I → VSSF II → HSSF (horizontal subsurface flow) (Fig. 7.4).

Since the significant fluctuation in pollutants concentration was observed for RWC, further calculations of both pollutant removal effectiveness and the dimensioning of pilot treatment wetlands were based on medians. The analyses of RWC composition indicate that the major pollutant is nitrogen, present mainly in the form of Kjeldahl nitrogen (ammonia + organic). Therefore, the assumption for the design of pilot treatment wetlands was that ammonia and organic nitrogen should be effectively removed.

In order to calculate pilot VSSF operating in a batch mode, it was assumed that the beds will treat the load of wastewater corresponding to 5 pe (person equivalent). The unit area of 2.5 $m^2$/pe and the daily pollutant loads of 120 g COD/pe d, 60 g TSS/pe d and 12 g TN/pe d were assumed (Gajewska 2012).

The total area of the first and the second stage VSSF beds is equal to:

$$F = 2.5\ \mathrm{m^2/pe} \cdot 5\,\mathrm{pe} = 12.5\ \mathrm{m^2}$$

The area of the first stage bed corresponds to 60 % of the total bed area:

$$F_I = 12.5\ \mathrm{m^2} \cdot 0.6 = 7.5\ \mathrm{m^2}$$

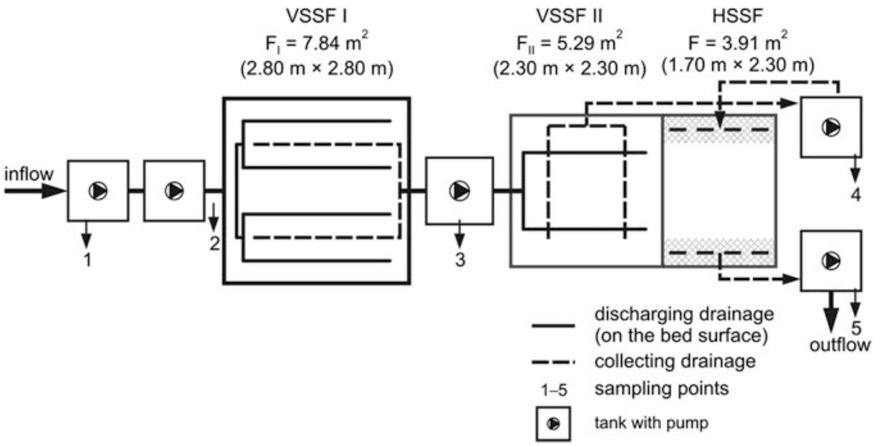



**Fig. 7.4** The pilot HTW for RWC treatment with sampling points locations (Gajewska and Obarska-Pempkowiak 2011)

The area of the second stage bed corresponds to 40 % of the total bed area:

$$F_{II} = 12.5\ m^2 \cdot 0.4 = 5\,m^2$$

The daily loads of pollutants from 5 pe are as follows:

$$COD = 120\ g\,COD/(pe \cdot day)\ \cdot\ 5\ pe = 600\ g\,COD/day$$
$$TSS = 60\ g\,TSS/(pe \cdot day) \cdot 5\ pe = \ 300\ g\,TSS/day$$
$$TN = 12\ g\,TN/(pe \cdot day)\ \cdot\ 5\ pe = \ 60\ g\,TN/day$$

Since nitrification (which has so far been considered as the main process responsible for ammonia nitrogen transformations) is the most sensitive process, the calculations of the pilot beds were based on the daily load of total nitrogen from 5 pe equal to 60 g TN/day. The assumed load unit of nitrogen (area dependent) is equal to:

$$N_{tot} = \frac{60\ g/d}{12.5\,m^2} = 4.8\,g\,/(m^2 \cdot d)$$

Assuming the median concentration of total nitrogen (900.0 mg/l for RWC) and the decrease of pollutants in the equalizing tanks (0.6 × 900.0 = 540 mg/l), the one batch for each section of the bed is equal to:

$$V_{RWC} = \frac{60\ \ g\ \ TN/d}{540.0\ \ g\ \ TN/m^3} = 0.1111\ m^3/d\ \cong 111.1\ \ l/d$$

Single hydraulic batch load for both beds (transpiration process was not taken into account) can be expressed as following:

- for the first stage bed (area $F_I = 7.5\ m^2$)

$$HL_{I_{RWC}} = \frac{V}{F_I} = \frac{0.111\ m^3/d}{7.5\ m^2} = 0.0148\ m/d = 15\ mm/d$$

- for the second stage bed (area $F_{II} = 5\ m^2$)

$$HL_{II_{RWC}} = \frac{V}{F_{II}} = \frac{0.111\ m^3/d}{5\ m^2} = 0.022\ m/d = 22\,mm/d$$

The assumed hydraulic loadings given by Molle et al. (2004) are substantially lower than 600 mm/d. Platzer and Mauch (1996) indicated that effective nitrification and nitrogen removal at VSSF beds takes place when hydraulic loading is below 300 mm/d.

The dimensioning of the last HSSF bed was done according to Cooper et al. (1997). The unit area of HSSF beds located in polishing step (third stage) in a course of treatment should be between 0.7 and 1.0 $m^2$/pe. In this case it was assumed 0.8 $m^2$/pe.

The filtration bed media was washed gravel with grain size between 4 and 8 mm and hydraulic conductivity of $4.2 \times 10^{-2}$ m/s.

Common reed was selected for the pilot wetland RWC treatment due to the good toleration of elevated concentrations of chlorides and iron. The beds were planted, in 2008, with *Phragmites australis* with 5 clumps per $m^2$ which was delivered by a specialized plantation (with well developed root zone system) to shorten the start–up period (Obarska-Pempkowiak et al. 2010a).

The water balance calculations (for the date from year 2008) showed that evapotranspiration should not have a significant impact on RWC treatment processes. Rainfall values in the vegetation season are substantially higher than the transpiration capacity of reed thus water loss due to evapotranspiration will be compensated by rainfall (Obarska-Pempkowiak et al. 2010b).

The working conditions of the pilot HTW are presented in (Table 7.3).

**Table 7.3** The operation conditions of the pilot HTW treating RWC

| Type of bed | Area ($m^{2)}$ | Hydraulic loading (mm/day) | Organics loading (g COD $m^2$/day) | $N_{Kjeldahl}$ loading (g $m^2$/day) | Batch volume (l/day) |
|---|---|---|---|---|---|
| VSSF I | 7.5 | 15.0 | 12.04 | 8.0 | 111.1 |
| VSSF II | 5.0 | 22.0 | 3.2 | 4.8 | 111.1 |
| HSSF | 3.9 | 28.5 | 1.6 | 1.4 | 111.1 |

## 7.4 Evaluation of MTW Operation

### *7.4.1 Quality of the Inflow RWC*

The concentrations of pollutants in the raw and treated RWC (also after the subsequent stages of treatment) in the pilot HTW were presented in Tables 7.4 and 7.5. In order to better characterize RWC, means with standard deviation and medians as well as the range with regard to studied pollutants were shown for 2009 and 2010, separately.

In case of the data presented above, the mean and median values did not vary significantly. Additionally, the standard deviation was less than 30 % of the mean values, which suggests that the data were normally distributed. Thus, the mean values (for both years calculated separately) were taken for further consideration. The reason for such a small difference in pollutant concentration in the inflow might be the operation of the first tank. The equal amount of RWC was added to the first tank every day, to what was pumped to the pilot plant. The working condition of the first tank ensured good mixing and equalizing the quality of the wastewater. Then wastewater (from the top of the tank I) was pumped into the second tank. The role of this stage of the treatment was both averaging and, most importantly, trapping the particulate during sedimentation (Fig. 7.5).

The efficiency removal at “the sedimentation stage” was equal to: 23.0 and 33.0 % for TSS, and 23.0 and 44.1 % for VSS. The most significant was the Org-N removal: 81 % in 2009 and 80 % in 2010. The organic matter removal was relatively low and equal to 24.2 % for COD in 2010 (Gajewska and Obarska-Pempkowiak 2011).

The $BOD_5$/COD and $BOD_5$/TN ratios bring information about biodegradability, and they decrease when the decomposition process is progressing. Additionally, in these studies $BOD_5/COD_f$ is presented as an indicator of easy degradable dissolved organic matter (Paggilla et al. 2007). The wastewater discharged to the first VSSF was characterized by 0.25–0.3 $BOD_5$/COD ratio, which is characteristic for mature landfill leachate and slightly higher than the one given by Fux et al. (2006), which was equal to 0.2 for the reject water from the WWTP in Minworth, Great Britain (Lo 1996; Klimiuk et al. 2007).

## 7.5 Subsequent Stages Efficiency Removal

Despite very inconvenient composition of discharged RWC a quite effective removal of pollutants was observed in the first biological stage of treatment: VSSF I (Fig. 7.6a). When comparing the efficiency of pollutant removal in 2009 and 2010, only small improvements can be observed (up to 5 %) in 2010 (Fig. 7.6b).

Both TSS and VSS were removed with effectiveness over 65.0 % and $BOD_5$ with over 70.0 %, which were the highest during all stages of the treatment. Total

**Table 7.4** The characteristics of RWC in 2009, mg/l (Gajewska and Obarska-Pempkowiak 2011)

| Parameter | Inflow (tank I) | After (tank II) | After (VSSF I) | After (VSSF II) | Outflow (after HSSF) |
|---|---|---|---|---|---|
| **2009** (n = 10) | | | | | |
| TSS | $\frac{591.0/592.1 \pm 71.2}{578.5 \div 729.8}$ | $\frac{445.1/452.1 \pm 48.2}{390.0 \div 560.0}$ | $\frac{162.1/164.2 \pm 10.7}{148.0 \div 180.0}$ | $\frac{55.0/56.1 \pm 7.4}{47.9 \div 67.0}$ | $\frac{21.2/21.5 \pm 6.3}{16.0 \div 35.0}$ |
| VSS | $\frac{486.3/467.2 \pm 67.4}{391.0 \div 640.3}$ | $\frac{360.1/356.0 \pm 56.3}{320.8 \div 530.6}$ | $\frac{122.0/120.5 \pm 22.4}{99.5 \div 190.6}$ | $\frac{37.0/38.4 \pm 4.8}{28.9 \div 59.3}$ | $\frac{14.8/14.5 \pm 5.5}{12.3 \div 18.9}$ |
| TN | $\frac{953.3/923.0 \pm 120.3}{824.6 \div 1309.1}$ | $\frac{742.3/719.3 \pm 99.3}{519.3 \div 850.6}$ | $\frac{405.3/401.4 \pm 58.3}{342.4 \div 573.2}$ | $\frac{254.0/256.2 \pm 36.4}{170.2 \div 296.2}$ | $\frac{151.3/150.6 \pm 22.6}{79.9 \div 201.3}$ |
| $NH_4^+$-N | $\frac{866.3/822.0 \pm 112.6}{691.7 \div 1329.6}$ | $\frac{704.2/699.8 \pm 106.3}{500 \div 906.6}$ | $\frac{384.1/382.7 \pm 56.3}{282.6 \div 548.5}$ | $\frac{240.1/236.5 \pm 40.4}{156.7 \div 337.4}$ | $\frac{132.2/134.9 \pm 26.6}{99.6 \div 198.6}$ |
| Org-N | $\frac{95.4/99.2 \pm 11.2}{69.2 \div 132.9}$ | $\frac{17.2/18.4 + 3.2}{11.2 \div 31.2}$ | $\frac{14.0/16.3 \pm 3.6}{10.4 \div 22.9}$ | $\frac{13.9/16.1 \pm 3.2}{9.8 \div 20.6}$ | $\frac{10.8/10.2 \pm 1.8}{8.3 \div 12.6}$ |
| $NO_3^-$-N | $\frac{0.35/0.4 \pm 0.2}{0.2 \div 0.5}$ | $\frac{1.4/1.1 \pm 0.1}{0.8 \div 1.6}$ | $\frac{2.45/2.4 \pm 0.2}{1.5 \div 3.4}$ | $\frac{3.4/3.6 \pm 0.2}{2.3 \div 4.5}$ | $\frac{4.2/5.5 \pm 0.3}{3.7 \div 6.1}$ |
| COD | $\frac{1160.5/1183.1 \pm 138.6}{980.5 \div 1420.6}$ | $\frac{1055/1167.05 \pm 113.9}{880.0 \div 1260.0}$ | $\frac{715.0/742.5 \pm 94.1}{605.4 \div 890.0}$ | $\frac{535.4/540.1 \pm 43.9}{470.0 \div 610.0}$ | $\frac{284.0/284.8 \pm 39}{250.5 \div 340.0}$ |
| $COD_f$ | $\frac{734.9/720.3 \pm 101.2}{671.4 \div 801.2}$ | $\frac{690.2/709.1 \pm 99.3}{559.2 \div 780.2}$ | $\frac{443.7/421.4 \pm 67.4}{356.9 \div 549.7}$ | $\frac{369.0/360.5 \pm 61.3}{278.2 \div 401.6}$ | $\frac{209.2/210.4 \pm 31.4}{129.5 \div 267.9}$ |
| $BOD_5$ | $\frac{360.3/345.6 \pm 65.3}{310.4 \div 445.8}$ | $\frac{295.2/298.1 \pm 59.2}{244.8 \div 378.6}$ | $\frac{85.2/86.4 \pm 18.2}{63.7 \div 79.0}$ | $\frac{51.0/50.2 \pm 9.7}{45.2 \div 79.0}$ | $\frac{24.9/26.3 \pm 3.7}{19.4 \div 31.4}$ |

Inflow—$p = 0.12$ for COD and $p = 0.14$ for TKN

After chamber II $p = 0.33$ for COD and $p = 0.38$ for TKN (both higher than significance level $p = 0.05$)

$\frac{\text{median/mean} \pm \text{standard deviation}}{\text{minimum} \div \text{maximum}}$

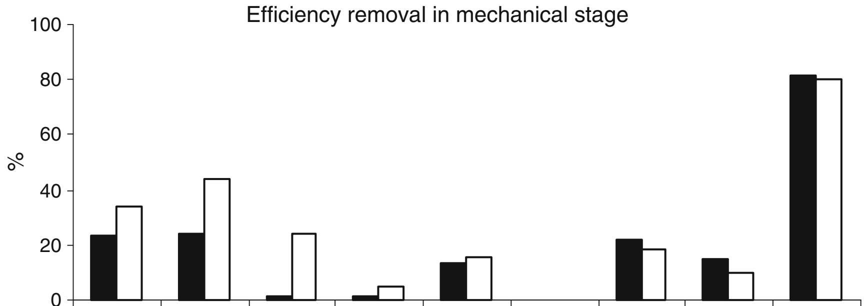
**Table 7.5** The characteristics of RWC in 2010, mg/l (Gajewska and Obarska-Pempkowiak 2011)

| Parameter | Inflow (chamber I) | After (chamber II) | After (VSSF I) | After (VSSF II) | Outflow (after HSSF) |
|---|---|---|---|---|---|
| **2010** (n = 10) | | | | | |
| TSS | 546.1/526.4 ± 84.3<br>496 ÷ 610 | 383.9/368.1 ± 67.4<br>197.3 ÷ 420 | 161.6/151.2 ± 28.8<br>119.5 ÷ 186.2 | 58.9/59.7/ ± 10.3<br>49.0 ÷ 76.2 | 26.2/24.9 ± 6.7<br>16.5 ÷ 35.2 |
| VSS | 358/355.7 ± 66.4<br>246.3 ÷ 420.3 | 193.3/194.8 ± 42.9<br>117 ÷ 252 | 113.9/109.9 ± 26.6<br>80 ÷ 152.6 | 40.7/44.3 ± 8.8<br>34 ÷ 60.2 | 18.6/17.8 ± 7.7<br>14 ÷ 30.5 |
| TN | 802.6/790.6 ± 51.1<br>710.4 ÷ 843.2 | 647.7/643.9 ± 51.3<br>547.5 ÷ 700.2 | 375.3/383.4 ± 42.5<br>323.6 ÷ 458.8 | 269.6/268.1 ± 34.9<br>235.2 ÷ 310.4 | 173/168.8 ± 21.1<br>150.3 ÷ 187.9 |
| $NH_4^+$-N | 706.7/705.5 ± 33.7<br>640.6 ÷ 730.5 | 573.2/586.54 ± 63.3<br>509.3 ÷ 675.4 | 312.8/325.5 ± 39.8<br>270.4 ÷ 389.3 | 217.2/210.6 ± 29.5<br>165.3 ÷ 247.8 | 130.2/125.5 ± 20.2<br>100.2 ÷ 154.6 |
| Org-N | 95.5/93.1 ± 25.1<br>54.9 ÷ 122.5 | 62.6/63.9 ± 21.7<br>21.5 ÷ 88.8 | 51.4/55.2 ± 20.6<br>18.5 ÷ 84.2 | 42.2/41.4 ± 17.1<br>15.1 ÷ 73.9 | 31.5/34.8 ± 15.4<br>13.2 ÷ 66.9 |
| $NO_3^-$-N | 0.3/0.4 ± 0.2<br>0.2 ÷ 0.4 | 1.2/1.1 ± 0.2<br>0.9 ÷ 1.5 | 2.4/2.4 ± 0.2<br>1.5 ÷ 3.3 | 4.0/3.8 ± 0.2<br>3.0 ÷ 5.0 | 6.0/5.5 ± 0.3<br>4.2 ÷ 8.3 |
| COD | 1213.3/1233.9 ± 87.4<br>1093.9 ÷ 1400 | 987.5/978.3 ± 33.5<br>899.2 ÷ 1008.2 | 571.9/582.1 ± 33.9<br>528.4 ÷ 640.4 | 421.6/440.7 ± 69.1<br>348 ÷ 500.2 | 245.1/242.9 ± 29.5<br>183.6 ÷ 267.3 |
| $COD_f$ | 690.5/684.9 ± 47.3<br>601.8 ÷ 712.6 | 578.2/584.5 ± 49.8<br>521.5 ÷ 670.6 | 409.4/396.9 ± 28.1<br>350 ÷ 458.2 | 188.6/186.5 ± 14.7<br>170.3 ÷ 223.7 | 109.4/110.4 ± 12.6<br>90 ÷ 134.6 |
| $BOD_5$ | 435.7/429.1 ± 57.3<br>320.4 ÷ 500.7 | 321.7/316.2 ± 29.4<br>270.8 ÷ 346.2 | 50.3/54.7 ± 13.3<br>40.2 ÷ 65.2 | 25.2/27.4 ± 7.2<br>21.3 ÷ 32.1 | 18.6/18.0 ± 4.0<br>10.6 ÷ 25.4 |

Inflow—$p = 0.36$ for COD and $p = 0.44$ for TKN

After chamber II $p = 0.66$ for COD and $p = 0.58$ for TKN (both higher than significance level $p = 0.05$)

$\frac{\text{median/mean} \pm \text{standard deviation}}{\text{minimum} \div \text{maximum}}$




| | TSS | VSS | COD | $COD_f$ | $BOD_5$ | | TN | $NH_4^+$-N | TP |
|---|---|---|---|---|---|---|---|---|---|
| ■ 2009 | 23.6 | 23.8 | 1.4 | 1.6 | 13.7 | | 22.1 | 14.9 | 81.5 |
| □ 2010 | 33.8 | 44.1 | 24.2 | 4.7 | 15.5 | | 18.3 | 10.1 | 80.2 |

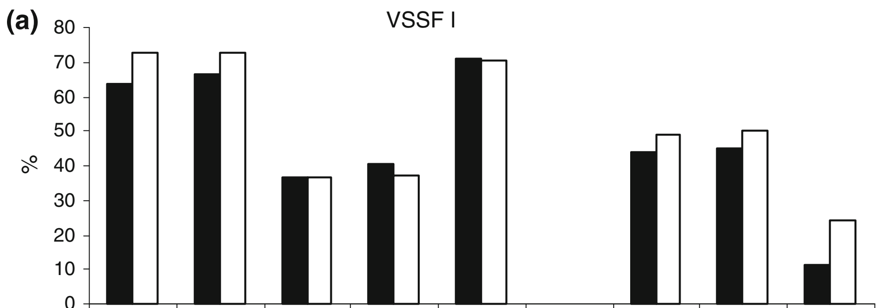
**Fig. 7.5** Efficiency of pollutant removal in the mechanical part of pilot MTW for RWC treatment

nitrogen and ammonium nitrogen were removed with a similar efficiency: 44.1 and 45.0 % 2009, and a bit higher in next year of operation (48.0 and 50.0 % respectively). During the treatment in VSSF I, the analysed rations of $BOD_5$/COD, $BOD_5$/TN and $BOD_5$/$COD_f$ decreased rapidly up to 50.0 % of their initial values, and were equal to: 0.11–0.13, 0.21–0.22 and 0.18–0.2 respectively.

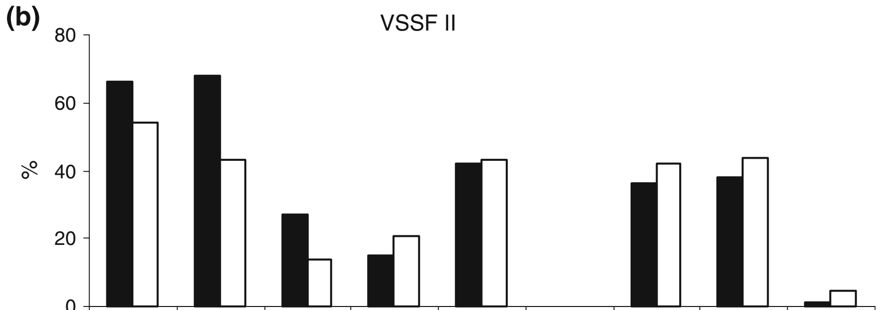
In 2009, the efficiency removal in VSSF II was very similar to the efficiency removal in VSSF I. In 2010, the effectiveness of VSSF II was about 20.1 % smaller than the one of VSSF I in case of TSS, VSS and COD removal but higher as for nitrogen compounds concerned (Fig. 7.6b). The effectiveness of TN and ammonium nitrogen were similar to those presented by the VSSF I at all the treatment stages during both years. After VSSF II, the analyzed ratios of $BOD_5$/COD, $BOD_5$/TN and $BOD_5$/$COD_f$ were similar to the initial ones, which can suggest that both organic matter and nitrogen were consumed proportionally.

In the case of HSSF the efficiency removal of Org-N was the highest and equal to 36.0 % in 2009, and 33.0 % in 2010, which was accomplished with over 40.0 % efficiency removal of TN at this stage of the treatment (Fig. 7.7a).

The achieved results confirmed that horizontal flow beds are designated for suspended solids removal—over 60.0 % in 2009 and over 70.1 % in 2010. The long retention time (about 10 days) also favours the decomposition of organic matter, even in the form of hardly degradable ones. The efficiency removal of the investigated organic fraction was equal to: 44.2–47.2 % for COD, 41.6–43.6 % for $COD_f$, 47.6–55.3 % for BOD.




| | TSS | VSS | COD | $COD_f$ | $BOD_5$ | | TN | $NH_4^+$-N | Org-N |
|---|---|---|---|---|---|---|---|---|---|
| ■ 2009 | 63.7 | 66.2 | 36.4 | 40.6 | 71.1 | | 44.2 | 45.3 | 11.4 |
| □ 2010 | 72.7 | 72.7 | 36.6 | 37.3 | 70.5 | | 48.8 | 50 | 24 |

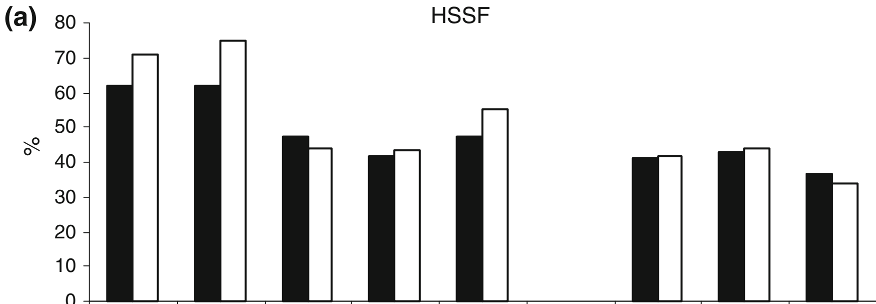



| | TSS | VSS | COD | $COD_f$ | $BOD_5$ | | TN | $NH_4^+$-N | Org-N |
|---|---|---|---|---|---|---|---|---|---|
| ■ 2009 | 66 | 68 | 27 | 15 | 42 | | 36 | 38 | 1.2 |
| □ 2010 | 54 | 43 | 14 | 21 | 43 | | 42 | 44 | 4.6 |

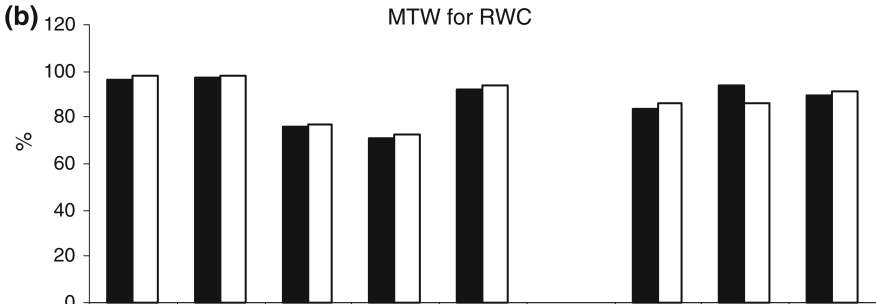
**Fig. 7.6** The efficiency of selected pollutants removal in VSSF I (**a**) and VSSF II (**b**)

## 7.6 Total Efficiency of Pollutants Removal and Quality of Outflow

The pilot HTW for RWC showed very high pollutant removal, which was over 96.0 % for TSS and over 70.0 % for COD (Fig. 7.7b). Since the $BOD_5$ removal efficiency was over 90.2 % and $COD_f$ removal was slightly lower than the COD removal, it can be assumed that part of particulate COD was transformed into easy biodegradable organic matter (Gajewska and Obarska-Pempkowiak 2011). During the treatment in the facility, the analyzed ratios of $BOD_5$/COD, $BOD_5$/TN and $BOD_5$/$COD_f$ decreased significantly and were equal to 0.09, 0.17 and 0.1 respectively. Such low ratios indicate the presence of organic matter in a hardly decomposable form, and the presence of still very high nitrogen concentration. The main




| | TSS | VSS | COD | $COD_f$ | $BOD_5$ | | TN | $NH_4^+$-N | Org-N |
|---|---|---|---|---|---|---|---|---|---|
| ■ 2009 | 61.7 | 62.2 | 47.2 | 41.6 | 47.6 | | 41.2 | 42.9 | 36.6 |
| □ 2010 | 71.2 | 75.1 | 44.2 | 43.6 | 55.3 | | 41.9 | 44.1 | 33.8 |



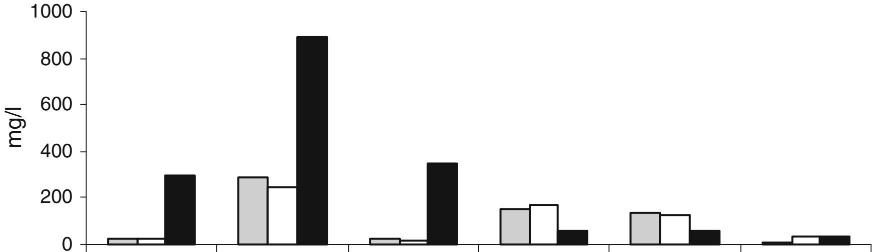

| | TSS | VSS | COD | $COD_f$ | $BOD_5$ | | TN | $NH_4^+$-N | Org-N |
|---|---|---|---|---|---|---|---|---|---|
| ■ 2009 | 96 | 97 | 76 | 71 | 92 | | 84 | 94 | 90 |
| □ 2010 | 98 | 98 | 77 | 73 | 94 | | 86 | 86 | 91 |

**Fig. 7.7** Efficiency of selected pollutants removal in HSSF part (**a**) and for the entire MTW for RWC (**b**)

form of TN in the outflow is ammonium nitrogen, which concentration varied from 100 to 189 mg/l with mean values around 130 mg/l (Fig. 7.8).

The final pollutants concentration achieved in this investigation were much higher in comparison to the assumed during designing pollutant concentrations. This lead to the conclusions, that there is no possibility of direct transfer of treatment efficiencies reported by other authors into this investigation, since the composition of wastewater differs significantly.

Although the concentration of ammonium nitrogen in the raw wastewater discharged to WWTP usually did not exceed 30 mg/l, the return flow of RWC treated in the pilot MTW should not cause any impact on the WWTP operation and final outflow quality.




| | TSS | COD | $BOD_5$ | TN | $NH_4^+$-N | Org-N |
|---|---|---|---|---|---|---|
| 2009 | 21.5 | 284.8 | 26.3 | 150.6 | 134.9 | 10.2 |
| 2010 | 26.2 | 245.1 | 18 | 168.8 | 125.5 | 34.8 |
| RW | 294.7 | 892.5 | 350.7 | 61.9 | 61.9 | 30.9 |

**Fig. 7.8** The comparison of pollutants concentration of treated RWC in HTW with pollutants concentration in raw wastewater (RW)

## 7.7 The Role of Each Stage of Treatment and Design Recommendation

The application of a HTW for concentrated wastewater treatment, especially reject water from mechanical dewatering (after sludge stabilization process) is quite a new attempt. Achieved results are compared with the appropriate data for wastewater and leachate treatment.

Very important stage of treatment took place in mechanical part (two tanks working in series). In this part, not only mixing and equalization took place but more important trapping of suspended solids in sedimentation process, which lasted 10 days. Both TSS and VSS were removed effectively in this stage but—even more important—Org-N was also removed effectively (77.1 % in 2009 and 30.0 % in 2010), which improved the efficiency removal in the whole treatment system and protected it against clogging.

In the pilot HTW plant, the VSSF stages were very effective for TSS removal, which was approx. 90 % up to this treatment stage, and further HSSF treatment stage improved it only to 97 %. The contribution of VSSF in nitrogen removal was the most crucial and the efficiency was over 72.1, and 85.0 % after HSSF. About 50.0 % of the discharged COD and about 85.2 % of the $BOD_5$ were removed in the sequential VSSF beds. This findings are in accordance with data given by Maehlum (1995, 1998) and Wojciechowska et al. (2010). While they are much better when comparing with removal efficiency reported by Bulc (2006) for TW with similar configuration working for landfill leachate in Slovenia.

In order to dimension the vertical stage, an assumption of 2.5 $m^2$/pe (after the load recalculation) was attempted. In the first stage, 60.0 % of the total area was calculated and 40 % in the second one. According to Cooper et al. (1997), the unit area of VSSF (designed for organic matter removal only) should be above 1.0 $m^2$/pe, whereas for efficient nitrification it should be over 2.0 $m^2$/pe. According to Langergraber (2007), outflow of one-stage VSSF beds with a unit area equal to 4 $m^2$/pe and organic matter load equal to 20 g/($m^2$·d) can meet rigorous Austrian outflow standards (below 90 mg/l COD and 25 mg/l $BOD_5$), regardless of the season of a year and air temperature. For Molle et al. (2004), two sequential VSSF beds, periodically supplied with raw sewage, provide effective treatment to the following level: COD—60 mg/l, TSS—15 mg/l, Kjeldahl nitrogen—8.0 mg/l. The treatment efficiency of the analyzed facilities working in the same way was very high: over 91.0 % for COD, 95 % for TSS and 85 % for Kjeldahl nitrogen.

In the applied configuration, the last stage of the treatment was carried out in HSSF, which ensured the most effective removal of Org-N (almost 40.0 %), and nearly 50 % of COD, which at this stage of the treatment were present in a hardly degradable form. The operation efficiency of HSSF bed for landfill leachate is strongly dependant on hydraulic regime as it was indicated by Wojciechowska and Gajewska (2006), Wojciechowska et al. (2010). Although working with the same medium, twin HSSF beds showed different efficiency removal of pollutants since one of them was exposed to surface runoff. In the contrast to the findings presented by Bulc (2006), addition of surface runoff due to atmospheric precipitation caused a decrease in removal efficiency down to 30.0 % for organics and down to 15.2 % for TN (in case of HSSF beds investigated in Poland).

According to Cooper et al. (1997), the unit area of HSSF beds located in such a position (third stage) in a course of treatment should be between 0.7 and 1.0 $m^2$/pe. According to Birkedal et al. (1993), if a single VSSF bed with sewage recirculation is applied after a HSSF bed, an effective removal of total nitrogen takes place, and allows decreasing the unit area of the total facilities to 10.0 $m^2$/pe.

Important parameter in designing and operation of TW facilities is mass removal rate of predominant pollutants such as organic matter (usually COD, $BOD_5$ and TN) from unit area.

Dependence between discharged and removed load of organic matter (COD) and TN is shown in Fig. 7.9a, b. However, a wide range of loading was applied from 14.0 to 22.0 g COD/($m^2$·d) and from 10 to 24 g TN/($m^2$·d), where the maximum allowable loadings given in the literature 40 g COD/($m^2$·d) and 20 g TN/($m^2$·d)—Langergraber et al. (2007), Sardon et al. (2006) were not exceeded. Studies in Spain with domestic wastewater treated in HSSF beds with BOD load ranging from 0.8 to 23.0 g/($m^2$·d) and on VSSF beds from 12.8 to 29.8 g/($m^2$·d) showed 80 and 95 % $BOD_5$ removal, respectively (Puigagut et al. 2007).

In the Polish multistage treatment wetlands organic matter loading ranged from 0.8 to 10.7 g/($m^2$·d), while removal efficiency ranged from 78 to 95 % (Gajewska and Obarska-Pempkowiak 2011).

The analyzed HTW facility, consisted of three hydrophyte beds characterized by very high MRR for both COD and TN. The mean values of MRR were equal: 13.8 g COD/($m^2$·d) and 13.5 g TN/($m^2$·d). The results achieved in this study confirmed that the TW facility design according to roles and allowable loadings could be applied for sewage with very high concentration of pollutants such as COD and ammonium nitrogen.

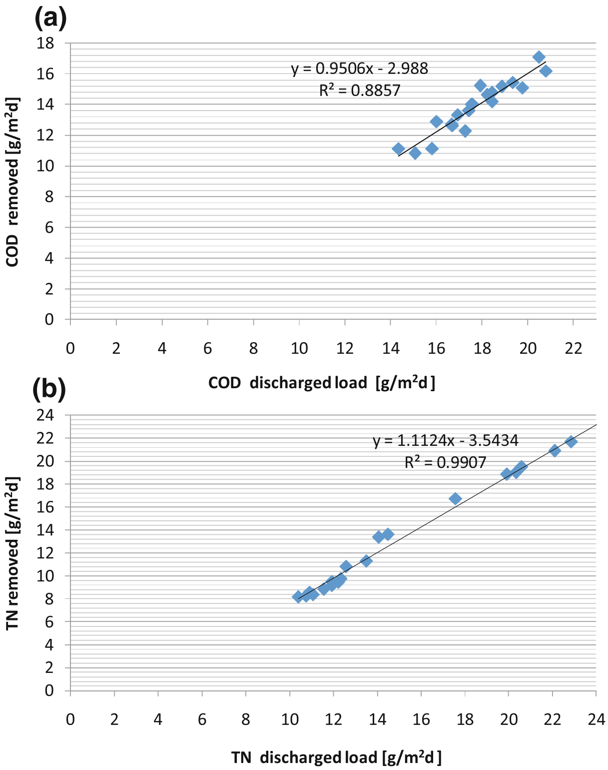



**Fig. 7.9** Organic matter and total nitrogen mass removal rate in HTW for RWC treatment

Basing on the carried out study on pilot plant with hydrophyte technology it was concluded that hybrid treatment wetlands (for reject water generated in the course of dewatering of digested sewage sludge) could be successfully applied. As a consequence, it allowed to decrease loads of pollutants returned at the beginning of treatment process, and secure stable operation of WWTP and finally decreased concentration of TN in outflow.

The full scale plant for reject water should secure retention time of about 10 days in settling tank. A biological part should be composed of two VSSF beds working in series followed by a HSSF bed. The analyses of the pilot plant operation indicated an effective removal of nitrogen compounds, especially $NH_4^+$-N in the VSSF beds, whereas it has been proven that HSSF created a good environment for the decomposition of hardly degradable Org-N and COD. The applied facility with their configuration ensured a very high removal efficiencies of major pollutants in reject water: COD—76 % and $NH_4^+$-N—93 % while the average mass removal rate were equal to: 13.8 g COD/($m^2 \cdot d$) and 13.5 g TN/($m^2 \cdot d$). There is a need for further investigation to evaluate the unit processes responsible for pollutants removal or retention in treatment wetlands for reject water.

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# Chapter 8
# Landfill Leachate Treatment in Treatment Wetlands

## 8.1 Characteristics of Leachate from Municipal Landfills

Landfill leachate is formed when rainwater percolates through the landfilled wastes, washing out organic and mineral pollutants. The leachate volume fluctuates depending on rainfall type and intensity. The leachate quantity is estimated as 1525% of the amount of landfilled wastes at well-compacted landfills and 2550% of the amount of landfilled wastes at poorly compacted landfills (Żygadło 1998; Surmacz-Górska 2001).

Leachate composition depends on landfill age and operation methods, precipitation percolating through the wastes, type of landfilled wastes and the method of wastes compaction. Also composting or waste recycling introduced at the landfill affect the leachate composition. The concentrations of pollutants in LL vary in short time periods (depending strongly on precipitation) and in long time periods due to the degradation of organic fraction of wastes. As the landfill ages four phases of degradation can be distinguished depending on how advanced are decomposition processes: aerobic phases, anaerobic phase (divided into acidic and methanogenic phase) and humic phase (also called stabilization phase) (Bozkurt et al. 2000).

Leachates from municipal landfills contain high concentrations of organic pollutants and ammonia nitrogen. Iron, chlorides and total suspended solids (TSS) concentrations may also be high (Surmacz-Grska 2001; Christensen et al. 2001; Klimiuk et al. 2007). Heavy metals and persistent organic pollutants (POPs) are also found in LL (Paxeus 2000; Schwarzbauer et al. 2001; Slack et al. 2005; Klimiuk et al. 2007). In Table8.1 the range of concentrations of some pollutants in leachates from municipal landfills, reported by Christensen et al. (2001), is presented.

A factor usually used for characterizing the biodegradability of organic matter present in the leachate is $BOD_5$/COD ratio. According to Surmacz-Grska (2001) in case of young landfills (younger than 35years) the $BOD_5$/COD ratio can be as high as 0.7, since the leachate contains high concentrations of easily biodegradable

 
H. Obarska-Pempkowiak et al., *Treatment Wetlands for Environmental Pollution Control*, GeoPlanet: Earth and Planetary Sciences,
DOI 10.1007/978-3-319-13794-0_8

**Table 8.1** Concentrations of pollutants in landfill leachate, according to Christensen et al. (2001)

| Parameter | Range |
|---|---|
| pH | 4.59 |
| TSS, mg/l | 2,00060,000 |
| $BOD_5$, mg $O_2$/l | 2057,000 |
| COD, mg $O_2$/l | 140152,000 |
| $BOD_5$/COD | 0.020.80 |
| Org-N, mg/l | 142500 |
| $NH_4^+$-N, mg/l | 50200 |
| TP, mg/l | 0.13 |
| $Cl^?$, mg/l | 1504,500 |
| $SO_4^{2?}$, mg/l | 87,750 |
| $HCO_3^?$, mg/l | 6107,320 |
| Fe, mg/l | 35,500 |
| Mn, mg/l | 0.031,400 |
| As, mg/l | 0.011 |
| Cd, mg/l | 0.00010.4 |
| Cr, mg/l | 0.021.5 |
| Co, mg/l | 0.0051.5 |
| Cu, mg/l | 0.00510 |
| Pb, mg/l | 0.0015 |
| Hg, mg/l | 0.000050.16 |
| Ni, mg/l | 0.01513 |
| Zn, mg/l | 0.031000 |

compounds. The concentrations of $BOD_5$ and COD in this phase may be on the level of 4,000mg $O_2$/l and 6,000mg $O_2$/l, accordingly, while pH indicate acidic character of LL (<6.5) due to the presence of volatile fatty acids (VFA) produced in the acidic fermentation process. In case of leachates from mature landfills (510years old), the $BOD_5$/COD ratio decreases to 0.50.3 due to mineralization of labile organic matter fraction.

The pH is neutral (6.57.5). Leachates from the landfills older than 10years have very low $BOD_5$/COD ratio (<0.1) and pH higher than 7.5. Since leachate recirculation enhances biodegradation processes at the landfill, the leachates from relatively young landfills (below 10years old) can have very low $BOD_5$/COD ratio, when recirculation takes place at the site (Reinhart and Al-Yousfi 1996). Changes of redox potential and pH (a) and composition of landfill gases (b) during aging of landfill are shown at Fig. 8.1. The organic matter transformation processes at the landfill finally produce the high-molecular compounds, mostly humic acids. Kulikowska (2009) estimates that humic substances may account for 60% of organic carbon. Calace et al. (2001) and Kang et al. (2002) compared the leachate samples from landfills of different age in France and in South Korea, respectively. Both studies concluded that older leachates contained considerably more compounds with high-weight molecules that the younger ones.

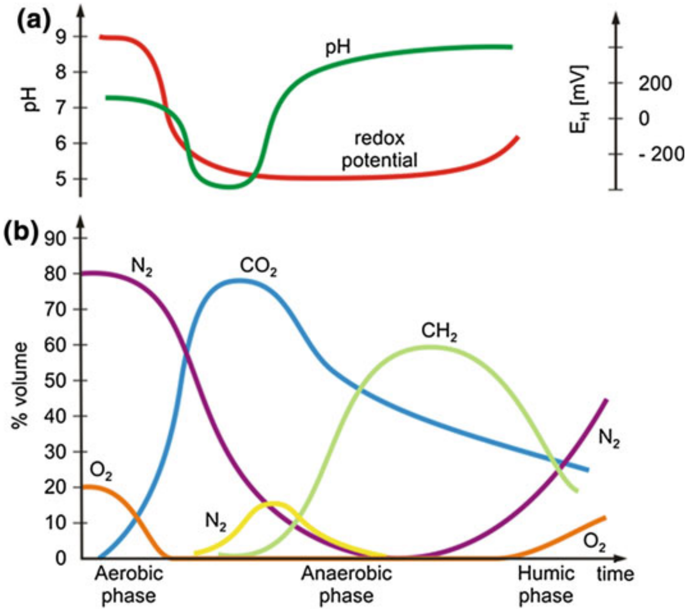


**Fig. 8.1** Changes of redox potential and pH (**a**) and composition of landfill gases (**b**) during subsequent phases of organic matter transformation, according to Bozkurt et al. (2000)

Ammonia nitrogen is the second largest (after organic matter) pollutant present in the LL. The source of ammonia nitrogen in the leachate is deamination of amino acids constituting the organic matter and hydrolysis and fermentation (in the older landfills) (Tatsi and Zoubolis 2002). The ammonia nitrogen concentrations vary from several hundreds to several thousands mg/l (Tatsi and Zoubolis 2002; Christensen et al. 2001).

Depending on type, source and composition of the landfilled wastes, the leachate can also contain heavy metals and persistent organic pollutants (POPs), for example BTEX, PAHs, phenols, pesticides, etc. (Bertanza and Pedrazzani 2007). These compounds pose a threat to the environment due to their persistence, mutagenic and carcinogenic properties and the tendency of bioaccumulation (Slack et al. 2005; Pazdro 2004). Öman and Junestedt (2008) identified over 400 different organic compounds in the leachate, while Christensen et al. (2001) found more than 1,000 organic substances in groundwater around landfill sites. Probably more compounds are present in leachates below detection level since the concentrations of organic compounds in LL are usually low. Nevertheless, there are a lot of compounds toxic even at the very low concentrations present in the LL (man and Junestedt 2008).

Due to its complex composition, co-treatment of LL at the municipal WWTP can alter the biological treatment processes there. Thus on-site LL treatment is recommended (Robinson 2005). Usually LL treatment requires a complex process and high financial expenditures (Wiszniowski et al. 2006). The methods used for leachate treatment can be physical (sedimentation, ammonia stripping, adsorption, reversed osmosis, nanofiltration), chemical (coagulation, advanced oxidation processes) or biological (SBR) (Surmacz-Grska 2001; Kulikowska 2009). In many cases a combination of methods is required to reach satisfactory treatment results.

## 8.2 Treatment Wetlands for Landfill Leachate Treatment

Due to high treatment efficiency, low treatment costs and simple operation and maintenance, treatment wetlands can be chosen as an alternative to high-tech leachate treatment methods, for instance membrane methods. According to Rew and Mulamoottil (1999) capital cost of TW system for leachate treatment are 25 times lower than other methods while operation costs are 3 times lower. Treatment wetlands can treat leachate with high enough efficiency to discharge it to surface waters, alone or in combination with other treatment methods. The letter solution also reduces the total costs of leachate treatment. TWs for leachate treatment also reduce the volume of treated leachate due to transpiration processes (Waara et al. 2008).

First applications of treatment wetlands for landfill leachate treatment took place in 1990s (Kadlec and Wallace 2009). At present treatment wetlands for LL treatment work in several countries in Europe and North America: Great Britain (Robinson 1990; Robinson et al. 1999; Kowalik et al. 1996), Slovenia (Vrhovsek et al. 2000; Bulc 2006), Germany (Rustige and Nolde 2006), Sweden and Norway (Maehlum 1995; Johansson-Westholm 2003, 2004; Waara et al. 2008) and USA and Canada (Peverly et al. 1995; Eckhardt et al. 1999; DeBusk 1999; Johnson et al. 1999; Martin et al. 1999; Rash and Liehr 1999; Kinsley et al. 2006; Nivala et al. 2007).

Both surface flow systems (SF) (Fig.8.2) and horizontal subsurface flow systems (HSSF) (Fig.8.3) are used. Application of vertical flow systems for LL treatment is in the pilot or development stage (Yalcuk and Ugurlu 2009; Lavrova and Koumanova 2010; Wojciechowska 2011, 2013a). Configurations of several treatment stages with different flow regime are also in use (Maehlum 1995; Liehr et al. 2000; Rash and Liehr 1999; Rustige and Nolde 2006; Kinsley et al. 2006). Leachate pre-treatment, for example aeration and sedimentation (Waara et al. 2008) or biological pretreatment in SBR (Johansson-Westholm 2003) can be applied before a treatment wetland.

## 8.3 Design Criteria

Specific composition of landfill leachate requires modification of standard design procedures developed for systems treating domestic and municipal sewage. Removal of suspended solids, $BOD_5$ and nutrients is not enough in this case.

**Fig. 8.2** SF wetland for landfill leachate treatment in Atleverket near rebro, Sweden (*Photo* Magdalena Gajewska)

**Fig. 8.3** HSSF beds for landfill leachate treatment in Szad?ki, Gda?sk, Poland (*Photo* A. Ostojski)

According to Kadlec and Wallace (2009) the basic design assumptions for LL treating treatment wetlands are as follows:

- TW has to remove high concentrations of iron,
- potential leachate toxicity to the plants,
- very small flows, depending on precipitation amount and transpiration,

- TW has to remove BTEX,
- TW has to remove POPs (PAH, PCB, etc.),
- TW has to remove heavy metals,
- protection of the receiver of treated leachate.

Treatment wetland has to be designed according to these overall rules, taking into account the specific site composition of the leachate. Unfortunately, some of the TWs are built without sufficient know-how, often leading to serious operation problems and malfunctions of the systems (Barr and Robinson 1999; Wojciechowska et al. 2010).

## 8.4 Treatment Mechanisms

According to Kadlec and Zmarthie (2010) iron should be removed from the leachate before it is discharged to the treatment wetland, especially when it is a SSF system, due to clogging risk. Iron is present in the leachate in the form of soluble divalent ions Fe(II). When the divalent Fe(II) is oxygenated to the trivalent form Fe (III), it starts to precipitate, clogging the pores of SSF beds. Such a situation took place in the HSSF system in Szad?ki near Gda?sk (Wojciechowska and Obarska-Pempkowiak 2008) and in Anamosa, USA (Nivala et al. 2007). In the SF systems iron sedimentates forming a layer of characteristic reddish-brow sediment, which need to be periodically removed. Therefore in case of high concentrations of iron the pre-treatment tank for iron oxygenation and precipitation is recommended (Kadlec and Zmarthie 2010). Additional role of the pre-treatment tank is averaging of the leachate quantity and composition.

Removal of high concentrations of ammonia nitrogen present in the leachate requires adequate conditions for nitrogen transformation processes: nitrification and denitrification (Faulwetter et al. 2009). Ammonia nitrogen can be removed either in a SF or HSSF system, however removal of very high ammonia concentrations requires large systems (Kadlec and Zmarthie 2010). Nitrification of ammonia takes place in the aerobic conditions. In the HSSF beds additional aeration would be necessary, which rises the capital and operation costs. VSSF beds with batch leachate discharge would be a better solution (Lavrova and Koumanova 2010; Kadlec and Wallace 2009). Denitrification of nitrates formed in the nitrification process requires anaerobic conditions in turn. Kadlec and Zmarthie (2010) recommend using a VSSF bed for nitrification followed by a SF bed for denitrification. A similar solution, with two VSSF beds for nitrification followed by a HSSF bed instead of a SF system was used in the study of Obarska-Pempkowiak et al. (2008) with very good nitrogen removal results (Wojciechowska 2011, 2013a).

In the SF systems adequate conditions exist for removal of POPs and heavy metals (Kadlec and Zmarthie 2010; Wojciechowska and Waara 2011; Wojciechowska 2013a, b).

According to Kadlec (2003) the key processes involved in POPs removal are as follows: volatilization, photochemical oxidation, sedimentation, adsorption and biodegradation. Depending on the type and properties of an organic compound, as well as TW type (surface or subsurface flow system, type of vegetation, type of soil substrate, hydraulic regime) some of the removal processes play a major role, while other are less important.

Biodegradation of POPs is strongly dependent on chemical structure of the molecules. Recalcitrant POPs usually have chloride atoms bound via atomic bonds to carbonic skeleton of their molecules (Imfeld et al. 2009). The breakage of carbonchloride bond is crucial and determines the persistence of a pollutant. Moreover, pollutants with a high molecular mass and/or higher number of chloride atoms, like PAHs, PCBs, PCDDs and PCDFs, are usually strongly sorbed by organic substances in the bottom sediments or filtration material and become unavailable for biodegradation processes. Hence, although the breakdown of carbonchloride bonds results both in better solubility and improves biodegradability of these organic compounds, dechlorination is slow in anaerobic conditions (Imfeld et al. 2009).

The parameter determining volatilization capacity of a chemical compound is Henry constant. In case of the POPs with high Henry constant (chlorinated solvents, BTEX) volatilization is a probable removal pathway, particularly in the SF systems, at water-atmosphere interface (Kadlec and Wallace 2009). In the SSF systems direct volatilization plays a minor role due to low diffusion rates in the aeration zone.

Sorption on soil substrate (SSF systems) or bottom sediments (SF systems) depends on the POP properties, organic carbon content in the substrate and its chemical structure. In the early stage of operation of a TW system sorption efficiency is high due to high sorption capacity of the substrate. POPs can be also sorbed on the surface of peat or clay particles as well as on the TSS, followed by sedimentation of TSS to sediments (Wojciechowska 2013a, b). Soprtion causes retention of a pollutant in a TW system, which enhances the probability of its biological decomposition. Many organic micropollutants (PCB, PCDD, PAHS, chlorinated benzenes) are strongly sorbed on TSS and accumulate in the sediments (Imfeld et al. 2009). Generally, sorption is the key removal process of pollutants characterized by high hydrophobicity (Wojciechowska 2013a, b).

## 8.5 Leachate Toxicity to Hydrophytes

Hydrophytes are tolerant to the high concentrations of typical pollutants present in the leachate, as well as heavy metals and PAHs (Peverly et al. 1995; Hawkins et al. 1997; Ye et al. 1997; Weis et al. 2004; Weis and Weis 2004). High leachate salinity may disturb some aquatic plants, although, according to literature reports, the plant most commonly used in the treatment wetland systems, *Phragmites australis*, can withstand relatively high $Cl^?$ concentrations (Lissner and Schierup 1997; Lissner et al. 1999; Weis et al. 2004; Choi et al. 2005). High ammonia concentrations present in raw leachate may be harmful to aquatic plants. Thus, Kadlec and Zmarthie (2010) recommend to recirculate treated leachate to dilute the ammonia concentrations at the inflow.

## 8.6 Treatment Effectiveness

Treatment efficiencies in treatment wetlands for leachate treatment depend on leachate composition, TW configuration and treatment system arrangement. In Table8.2 the efficiencies of $BOD_5$, COD, total nitrogen and ammonia nitrogen reported in the literature is presented.

The SF systems are most effective in $BOD_5$ and COD removal. In the SF systems in Perdidio (Martin et al. 1999) and Atleverket (Wojciechowska et al. 2010) the $BOD_5$ removal effectiveness was equal to 95%. At the same time, COD removal was equal to 88% in Perdido and 68% in Atleverket. In the TWs in Esval (Maehlum 1995) and Laflche (Kinsley et al. 2006), consisting of anaerobic lagoon, HSSF bed and hydrophyte pond, the $BOD_5$ removal was 91 and 9399%, respectively. In Esval also COD removal was very high (88%). The values of COD removal reported by Maehlum (1995) and Martin et al. (1999) are outstandingly high, since landfill leachate consists recalcitrant organic matter.

In Dragonja (Bulc 2006) the $BOD_5$ and COD removal efficiencies were equal to 59 and 50%, respectively. Nivala et al. (2007) reported very good $BOD_5$ removal efficiency in TW in Anamosa, while COD removal efficiency was low, especially in the period of aeration system breakdown.

The highest nitrogen removal efficiencies were again reported in the SF systems. In the SF systems in Perdido, Florida and Atleverket, Sweden nitrogen removal efficiencies were equal to 99% (Martin et al. 1999; Wojciechowska et al. 2010). In Atleverket ammonia stripping was applied before the discharge of leachate to treatment wetland, which reduced $NH_4^+$-N concentration by 68% (from 415 to 134mg/l) (Wojciechowska et al. 2010). High efficiencies of nitrogen removal were reported in the treatment plants using combination of treatment wetlands with other treatment techniques. In Istra, Sweden, 99% efficiency of $NH_4^+$-N removal (77% efficiency of total nitrogen removal) was achieved in the treatment system consisting of SBR reactor, lagoon and HSSF TW bed (Johansson-Westholm 2003). In the leachate treatment plant in Lt (Sweden), consisting of aerated lagoon, sand-gravel filter and hydrophyte pond total nitrogen was removed with 89% efficiency and ammonia nitrogenwith 99% efficiency (Johansson-Westholm 2004). In Esval, Norway (anaerobic lagoon, 2 HSSF beds and hydrophyte pond) total nitrogen was removed with 84% efficiency (Maehlum 1995). In Laflche (Ontario, Canada) in a system consisting of a peat filter, HSSF beds and polishing pond, efficiencies of ammonia nitrogen removal and total nitrogen removal were equal to 9799 and 9094%, respectively (Kinsley et al. 2006). Both in Esval and Laflche nitrates remained in leachate after HSSF, indicating incomplete denitrification. In both cases polishing pond vegetated with hydrophytes was used in the last stage of treatment to enable denitrification of remaining nitrates. Plant detritus produced in the pond provided organic carbon for denitrification.

In leachate treatment systems consisting of HSSF beds alone, the efficiencies of nitrogen removal were significantly lower. According to Bulc (2006) ammonia nitrogen removal efficiency in Dragonja (Slovenia), consisting of 2 parallel HSSF

**Table 8.2** Efficiencies of pollutants removal in TWs treating landfill leachate, based on literature reports

| Name of object, country | Treatment system configuration | Removal efficiency, % | | | | | Literature source |
|---|---|---|---|---|---|---|---|
| | | $BOD_5$ | COD | TN | $NH_4^+$-N | Fe | |
| Esval, Norway | (1) Anaerobic lagoon | 91 | 88 | 83 | | 88 | Maehlum (1995) |
| | (2) Aerated lagoon | | | | | | |
| | (3) 2 parallel HSSF beds | | | | | | |
| | (4) Hydrophyte pond | | | | | | |
| Dragonja, Slovenia | (1) Equilibrium tank | 46 | 68 | | 81 | 80 | Bulc et al. (1997) |
| | (2) 2 parallel HSSF beds | | | | | | |
| Perdido, Florida, USA | (1) Aerated lagoon (retention time 500days) | 95 | 88 | | 99 | 99 | - |
| | (2) SSF system (serpentine ditched) | | | | | | |
| Laflche, Ontario, Canada | (1) Equilibrium tank | 9399 | | 9094 | 9799 | | Kinsley et al. (2006) |
| | (2) Peat filter | | | | | | |
| | (3) HSSF bed | | | | | | |
| | (4) Hydrophyte pond | | | | | | |
| Istra, Sweden | (1) Equilibrium tank | 82 | 40 | 77 | 99 | | Johansson-Westholm (2003) |
| | (2) SBR reactor | | | | | | |
| | (3) Equilibrium tank II | | | | | | |
| | (4) Flooding area | | | | | | |
| | (5) HSSF bed | | | | | | |
| Anamosa, Iowa, USA | (1) HSSF bed with a patented aeration system | 7581[a] 97[b] | 053[a] 3560[b] | | 1440[a] 9398[b] | | Nivala et al. (2007) |
| | (1) Aerated equilibrium tank | 97 | 82 | 94 | 99 | | Wojciechowska et al. (2010) |

(continued)

**Table 8.2** (continued)

| Name of object, country | Treatment system configuration | Removal efficiency, % | | | | | Literature source |
|---|---|---|---|---|---|---|---|
| | | $BOD_5$ | COD | TN | $NH_4^+$-N | Fe | |
| Atleverket, Sweden | (2) SSF system composed of 10 ponds | | | | | | |
| Szad?ki, Poland | (1) 2 parallel HSSF beds | 62 (27) | 35 (12) | 66 (52) | 67 (52) | 33 (42) | Wojciechowska and Obarska-Pempkowiak (2008) |
| Chlewnica, Poland | (1) Sedimentation tank | 8790 | 4859 | 6480 | 7686 | | Wojciechowska (2013a) |
| | (2) Two VSSF beds operated in batch | | | | | | |
| | (3) HSSF bed | | | | | | |

[a] Aeration system in operation
[b] Results during breakdown of aeration system

beds, were equal to 50%. In TW Szad?ki (2 parallel HSSF beds), ammonia nitrogen removal efficiencies were equal to 67% for bed 1 and 52% for bed 2 (Wojciechowska and Obarska-Pempkowiak 2008). Nivala et al. (2007) reported outstandingly high removal efficiency of ammonia nitrogen in HSSF beds (90%), which was due to additional aeration system used in the beds. During the breakdown of aeration system, the ammonia nitrogen removal efficiency drastically deteriorated and was between 14 and 43%.

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# Chapter 9
# Dewatering of Sewage Sludge Dewatering in Reed Systems

Within the last several years new methods of sewage sludge utilisation have been introduced. They may supplement or, on occasion, even replace traditional methods of sewage sludge utilisation, such as agricultural use, application to landfarming, incineration or land-filling. New technologies are especially suitable in rural areas where, for economic reasons, sewage sludge is stored in lagoons and drying beds operating in summer. In other seasons the sludge is transported to municipal landfills or to central conventional wastewater treatment plants (WWTPs). New technologies take advantage of aquatic plants' (reed, calamus, bulrush) or willow's (*Salix viminalis*) ability to grow in mineral soil periodically covered with layers of sewage sludge (Hofmann 1990; Nielsen 1993; De Maeseneer 1996; Lienard and Payrastre 1996; Pempkowiak and Obarska-Pempkowiak 2002).

In Northern Poland three macrophyte facilitiesment for sewage sludge utilization, were constructed: in Darżlubie near Gdańsk (loaded with primary sludge), in Swarzewo near Gdańsk and in Zambrów near Suwałki (loaded with secondary sludge). In this chapter the design, operation and results of sludge utilisation in the mentioned facilities are presented. The measurements were carried out in order to evaluate the impact of plants on the rate of dewatering and decomposition of organic matter.

## 9.1 Facilities in the Northern Poland

### *9.1.1 Location and Construction of Facilities*

#### 9.1.1.1 Reed Bed in Darżlubie

In the village of Darżlubie on the coast of the Bay of Puck, in the Gdańsk voyevodship, suspended solids are removed from sewage in household sedimentation tanks. Then sewage in the amount of 140 $m^3$/day is directed to an Imhoff tank. Further treatment takes place in a hybrid treatment wetland. Digested sludge of volume 36 $m^3$ and moisture content in the range 90−96 % was removed from the



H. Obarska-Pempkowiak et al., *Treatment Wetlands for Environmental Pollution Control*, GeoPlanet: Earth and Planetary Sciences,
DOI 10.1007/978-3-319-13794-0_9

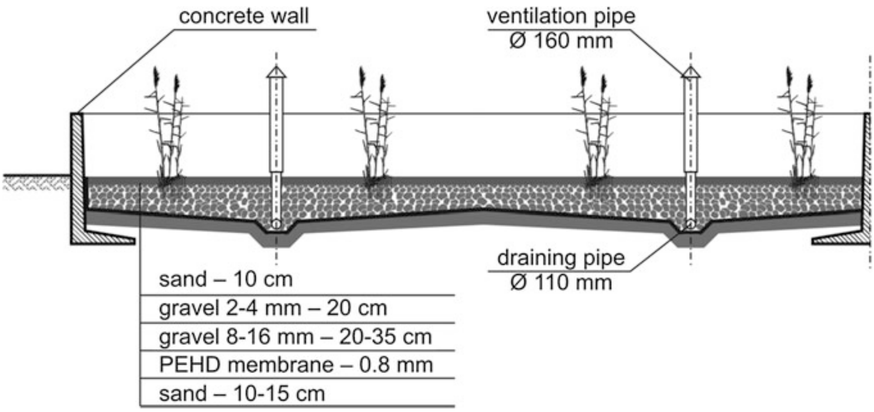


**Fig. 9.1** A cross-section diagram of the reed bed in Darżlubie

Imhoff tank eight times a year and directed into reed beds. There are two beds with a total area of 480 $m^2$ (12 × 20 m each) in the facility. The beds were constructed in 1995. Only one of the beds is in operation since the amount of sludge collected in the Imhoff tank has amounted to half of what was expected. The beds are constructed as tanks with concrete walls. The outflow is drained off through the draining pipes located in the sandy layer and the bed is aerated through ventilation chimneys. The drainage system is composed of the following layers (from the bottom to the top): coarse gravel 8/16 mm (30 cm thick), medium gravel 2/4 mm (20 cm thick) and sand 0.8 mm (10 cm thick). The gravel layer serves as a draining system while the sand provides growing medium for reed (Fig. 9.1).

The beds were planted with rhizomes of reed (*Phragmites australis*) with the density of 8 pcs/$m^2$. The sludge is discharged to the bed via a ∅ 130 mm pipe. In order to prevent the bed from being hollowed out, 4 pavement tails 50 × 50 cm are placed in the area where the sludge is discharged (Zwara and Obarska-Pempkowiak 2000). In January 1998 a small control bed (0.8 × 1.2 m), separated from contact with discharged sludge was established within the reed bed.

#### 9.1.1.2 Reed Lagoon in Swarzewo

In the mechanical—biological treatment plant in Swarzewo, 4,000 $m^3$/day domestic sewage, in winter, and 6,500 $m^3$/day, in summer, are processed. After screens and sandtraps, the sewage is directed to biological reactors with activated sludge. The excess sludge of 98 % moisture (800 $m^3$/day in winter and 1,000 $m^3$/day in summer) is stored in 32 drying beds (10 × 30 m each). Beds are flooded with sludge 3−4 times per year. Since the area for sludge drying was insufficient, the reed lagoon was constructed in autumn, 1994. The total area of the lagoon was equal to 2,500 $m^2$ (50 × 50 m). In the period from January to April 1995, reed rhizomes were planted with the density of 9−15 pcs/$m^2$ (Obarska-Pempkowiak et al. 1997).

#### 9.1.1.3 Reed Lagoon in Zambrów

In the WWTP in Zambrów (Podlaskie voyevodship), the average amount of treated domestic sewage is equal to 3,500 $m^3$/day. Domestic sewage and rainwater are collected separately. Domestic sewage undergoes treatment in screens, sandtrap and biological reactors with activated sludge. The excess secondary sludge (of the amount of 150 $m^3$/day), of 99 % moisture, is collected in two traditional lagoons in the non-vegetation period. In the vegetation season it is discharged directly to a reed lagoon of the total area of 5,500 $m^2$. The amount of sludge utilised in the reed lagoon equals 87 % of the total volume of produced sludge. The remaining part of sludge (13 %) is directed to the vermiculture beds during the summer season, and, in autumn, it is used in landfarming. The bottom of the beds is covered with a layer of clay. Above the clay layer there are draining pipes (∅ 100 mm) placed in filtration medium. The outflow collected by the draining pipes is recirculated and mixed with raw sewage inflowing to the WWTP. The reed was planted in the sandy filtration medium with the density of 4 pcs/$m^2$ (Alachamowicz and Gawkowski 2001).

### *9.1.2 Methods*

Measurements of the sludge were recorded in Darżlubie for 6 years. The bed was divided into 4 sections along the symmetry axes. The samples of sludge were collected from 4 sampling points located in the centre of each section. The samples were collected from four layers of the vertical profile of the bed (I—0 to 7 cm from the bottom of the bed, II—7 to 14 cm, III—14 to 22 cm, IV—22 to 30 cm). An average sample was obtained by mixing equal volumes of collected material. The samples were collected in the period 1995−2000 at 6 weeks intervals, following the frequency of sludge loading. In Swarzewo the thickness of the sludge layers was also measured in the period 1995–1998. The average samples of nonstratified sludge were collected once a month during the period of investigation. In Zambrów the layers of sludge discharged to the bed and remaining in the bed were measured only once a year in the period 1997−2000. The average samples of nonstratified sludge were collected once in 3 months.

The following properties of the solid medium collected were determined: moisture, organic matter, total nitrogen and total phosphorus contents, the fecal coli index, *Clostridium perfringens* index and the number of parasite ova. The analyses were carried out according to standard methods. A detailed description of analytical methods was presented elsewhere (Obarska-Pempkowiak et al. 1997; Zwara and Obarska-Pempkowiak 2000). Also, the average contents of heavy metals: Cu, Pb, Ni, Zn, Cr and Cd in non-stratified sludge stored in the bed were determined. The total contents of heavy metals were determined for sample of homogenized sludge. After digestion in 5 ml of HCl and $HNO_3$ (3:l) for 2 h at 80 °C, the mixture was centrifuged and the supernatant was evaporated to dryness. Then, the dry residue was dissolved in 0.l mol $HNO_3$. All solutions were analysed for heavy metals in a model video 11E atomic absorption spectrometer (Thermo Jarrel Ash). Both flame and electrothermal atomizations were applied. Appropriate blanks were analysed at the same time as the samples. The concentrations of $BOD_5$, COD, SS, total nitrogen and total phosphorus in the outflow collected from the draining system were measured three times in the investigation period as well.

## 9.1.3 Results and Discussion

*Darżlubie*. The results of sludge thickness measurements of sludge layers in Darżlubie are presented in Fig. 9.2. The total thickness of sludge discharged to the bed was 5.5 m and the thickness of the remaining layer of sludge was only 0.30 m. The 15 cm thick layers of primary, anaerobically stabilised sludge, were discharged to the bed once in 6 weeks. Thus, the annual amount of sludge was equal to 1.2 $m^3/(m^2 \cdot year)$. EPA suggests the following hydraulic loading of the beds: 0.78 $m^3/(m^2 \cdot year)$ for anaerobically stabilised sludges with a dry matter content of 5 %. According to De Maeseneer (1996), the wetland systems operating in Western Europe were fed with sludge 8, 16 or 24 times a year. The hydraulic loadings varied from 0.4 to 1.6 $m^3/(m^2 \cdot year)$ for anaerobically stabilised sludge. Thus, the hydraulic loading of the beds in Darżlubie was similar to the ones applied in other countries.

The inlet volume of the sludge decreased by 94.6 % due to the transformations taking place on storage. Similar results were obtained by Nielsen (1993) during investigations in Allerslev and Regstrup (90.3 % reduction). The main reason for the decrease of the sludge volume was dewatering and, to a smaller extent, biochemical decomposition (Nielsen 1993).

The average results of measurements of moisture, organic matter content, total nitrogen and total phosphorus contents for the stratified layers of sludge from the reed bed in Darżlubie are presented in Table 9.1. The lowest average moisture was measured in the layer I, lying directly on the mineral medium. Limited changes of sludge moisture along the profile were observed (Table. 9.1). It is probably due to frequent loading of sludge and atmospheric precipitation, which causes filtration of rainwater through the entire layer of sludge. The decrease of moisture in the deepest layer was probably caused by changes in the structure of sludge resulting from

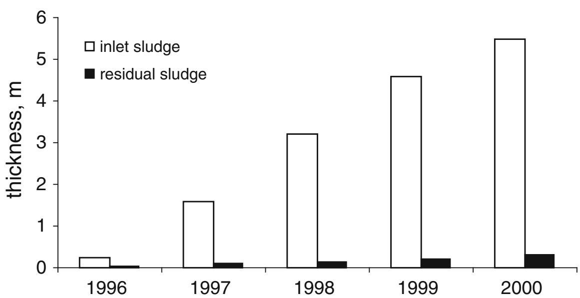


**Fig. 9.2** Thickness of discharged and residual primary sludge in reed bed in Darżlubie

**Table 9.1** The average values (± standard deviations) of physical and chemical parameters of sludge stored in the reed bed in Darżlubie

| Parameter | Layer I[a] | Layer II[a] | Layer III[a] | Layer IV[a] |
|---|---|---|---|---|
| Moisture, (%) | 43.01 ± 4.28 | 60.29 ± 5.74 | 63.91 ± 6.78 | 65.42 ± 7.83 |
| Organic matter content, (% d.m.) | 43.34 ± 4.24 | 41.62 ± 6.26 | 46.17 ± 5.26 | 50.11 ± 4.33 |
| TN, % of organic matter | 2.30 ± 0.32 | 2.26 ± 0.51 | 2.30 ± 0.57 | 2.48 ± 0.73 |
| TP, % of organic matter | 0.27 ± 0.07 | 0.23 ± 0.05 | 0.22 ± 0.1 7 | 0.24 ± 0.13 |

[a] layer I—bottom; layer IV—surface

biochemical changes of organic matter. Penetration of the sludge layer by roots and rhizomes of reed creates conditions suitable for heterotrophic microorganisms and formation of a rhizosphere.

The mean content of organic matter varied from 50.1 to 43.3 %. The lowest content of organic matter (43.3 %) was measured in layer I (the bottom layer) while the highest values were observed in the surface layer (layer IV). The average difference of organic matter content along the profile was equal to 6.8 % d.m. This indicates that approximately 14 % of organic matter loaded to the bed was decomposed.

The nutrient content (nitrogen and phosphorus) in the organic matter changed with in wide range. The standard deviations varied from 18 to 30 % for nitrogen and from 22 to 45 % for phosphorus. The values of standard deviations were lower for layers I and II (those were stored for a longer period of time) and the highest for the IV layer. The changes of quality of loaded sludge determined the contents of nutrients in the surface layer. The changes in nutrient content in the lower layers were caused by decomposition of organic matter and sorption of nutrients dissolved in the drainage waters on the filtration medium. The average contents of nitrogen and phosphorus were stable down the profile and ranged from 2.3 to 2.5 % of organic matter and 0.23−0.27 % organic matter, respectively. Only in the surface layer (layer IV) was it slightly higher (2.5 %).

The results of sanitary parameters determinations of the sludge from Darżlubie are presented in Table 9.2. In the analysed samples of sludge, the number of *Ascaris lumbricoides* ova increased from 80 to 4,500 in l kg d.m. After 8 months of storage without loading fresh layers of sludge, the numbers of invading ova of *Ascaris lumbricoides* per 1 kg d.m. slightly increased. However, bacteriological analyses showed that the coli index of the sludge decreased and pathogenic *Salmonella* bacteria were destroyed, indicating improvement of the bacteriological condition of the sludge. The average contents of heavy metals in sludge stored in Darżlubie are presented in Table 9.3. These values do not exceed the levels permissible for sludge applied for landfarming (Regulation of Environment Minister 2002).

The total volume of drainage water was only 4 $m^3$, which is 10 % of the volume of loaded sludge (36 $m^3$). The average concentrations of COD, TN and TP in drainage water were similar to concentrations in treated sewage and were lower than the corresponding values in drainage waters from typical drying beds (Nielsen 1993)

**Table 9.2** The sanitary parameters of the sludge from the WWTP in Darżlubie

| Parameter | Sludge from the Imhoff tank | Sludge discharged to the reed bed[a] | Sludge stored in the bed | Sludge after 8 months of storge[b] |
|---|---|---|---|---|
| *Coli* index | $1.0 \times 10^9$ | $1.2 \times 10^8$ | $5.0 \times 10^5$ | $5.0 \times 10^4$ |
| Pathogenic bacteria of the *Salmonella* species | *Salmonella* $C_1$ group | *Salmonella* $C_1$ group | not detected | not detected |
| Invading ova of *Ascaris Lumbricoides* (per kg d.m.) | 140 | 80 | 4,050 | 4,500 |
| *Trichocephalus trichuria* | 10 | 0 | 300 | 150 |

[a] after Imhoff tank
[b] 8 months of storage without irrigation

**Table 9.3** Average contents of heavy metals in sludge stored in Darżlubie, mg/(kg d.m.)

| Layer | Cu | Pb | Ni | Zn | Co | Cr | Cd |
|---|---|---|---|---|---|---|---|
| I | 28.56 | 26.68 | 12.36 | 748.30 | 3.16 | 18.32 | 1.40 |
| II | 28.36 | 37.28 | 18.84 | 1093.70 | 4.52 | 26.96 | 1.92 |
| III | 28.04 | 30.32 | 16.60 | 855.60 | 4.16 | 21.76 | 1.80 |
| IV | 27.84 | 31.28 | 18.96 | 779.70 | 4.48 | 22.80 | 1.64 |
| Permissible level (Polish Standard) | 800.00 | 500.00 | 100.00 | 2000.00 (2500)[a] | – | 500 | 20 (10)[a] |

[a] Proposed

**Table 9.4** Comparison of the quality of the inflow to the reed bed in Darżlubie with the quality of drainage water

| Parameter | Raw sewage | Outflow drained off |
|---|---|---|
| Flow | 140 $m^3$/d | 4 $m^3$/d |
| COD | 1,000 mg $O_2$/l | 250 mg $O_2$/l |
| TN | 100–150 mg/l | 12 mg/l |
| TP | 10–20 mg/l | 1 mg/l |

(Table 9.4), The average loads of these contaminants in outflow represented only about 1–4 % of the load of contaminants discharged with raw sewage.

*Swarzewo*. The reed lagoon in Swarzewo was in operation between May 1995 and September 1998. In this period of time it was loaded with a 10.5 m layer of secondary sludge. When the utilisation process was completed, the thickness of the dried sludge layer was equal to 1.1 m and the content of dry matter in residual sludge was 359.5 tons (Fig. 9.3). This rather large thickness of the residual sludge layer resulted from loading huge volumes of sludge, leading to destruction of reeds in several areas. It was the main reason for ceasing operation of the lagoon.

The organic matter content in the analysed time period decreased from 75 to 60 %. At the same time, moisture of sludge decreased from 92 to 86 %. The total

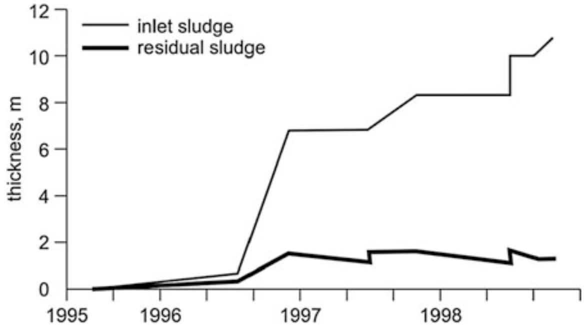


**Fig. 9.3** Loads (m) of inlet and dried sewage sludge in the reed lagoon in Swarzewo

nitrogen concentration varied over a wide range from 1 to 10 %, while the total phosphorus content changed from 0.2 to 1 % d.m. The results of microbiological investigations of secondary sludge were more variable than the corresponding results for primary sludge. The *coli* index changed from $5.9 \times 10^5$ to $2.5 \times 10^6$, the fecal *coli* index from $5.0 \times 10^5$ to $5.9 \times 10^5$ and *Clostridium perfringens* index varied from $2.5 \times 10^5$ to $2.5 \times 10^6$. The sludge from Swarzewo did not meet the standards for sludge suitable for in landfarming (Obarska-Pempkowiak et al. 1997).

*Zambrów*. The yearly average amounts of sludge loaded to the reed lagoon in Zambrów and remaining in the lagoon are presented in Fig. 9.4.

The average loading was equal to 34 kg d.m./($m^2$·year). In the operation period, the volume of drained-off outflow was approximately 100−120 $m^3$/day. Content of dry matter in residual sludge was 410.5 tons. The quality parameters of dewatered sludge are presented in Tables 9.5 and 9.6.

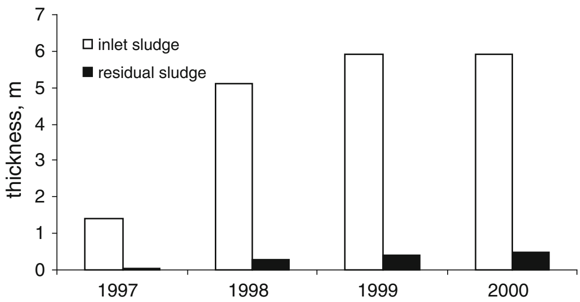


**Fig. 9.4** Loads (m) of inlet and dried of secondary sludge in reed lagoon in Zambrów

**Table 9.5** The average values of physical and chemical parameters of sludge stored in reed lagoon in Zambrów

| Moisture (%) | Organic matter (% d.m.) | pH | TN (mg/kg d.m) | TP (mg/kg d.m) |
|---|---|---|---|---|
| 82.5 | 64.4 | 7.58 | 3.9 | 0.55 |

**Table 9.6** The average contents of heavy metals stored in reed lagoon in Zambrów; mg/kg d.m

| Pb | Hg | Cu | Cd | Ni | Zn | Cr |
|---|---|---|---|---|---|---|
| 38.7 | 1.95 | 150 | 3.10 | 11.8 | 1,258 | 29.7 |

**Table 9.7** Quality the outflow of sludge dewatered in the reed lagoon in Zambrów

| Year | COD (mg/l) | TN (mg/l) | TP (mg/l) |
|---|---|---|---|
| 1999 | 436 | 172.3 | 16.15 |
| 2000 | – | – | 17.80 |
| 2001 | 270 | 85 | 19.30 |

The quality of outflow is given in Table 9.7. Similarly, as in the case of the facility in Darzlubie, the loading of contaminants in the outflow represented from 3 to 10 % of the load of contaminants discharged to the facility.

### *9.1.4 Conclusions*

Analyses of the results lead to the following conclusions:

1. Filtration of the drainage waters through the older layers of sludge significantly changes chemical and microbiological properties of sludge.
2. Small changes of the sludge moisture result from frequent sludge loading, increase of mineral substances content in lower layers and surface evaporation.
3. The most rapid changes in organic matter content in the profile of sludge layers were observed at the interface of the sludge and mineral layer.
4. Microbiological tests indicated that sanitary quality of the utilised sludge does not change during storage.
5. The number of invading ova of parasites in the sludge stored in reed beds increased. Sanitation of sludge removed from the beds will be necessary before it is used in landfarming or forestry.
6. The heavy metal concentrations did not exceed the permissible values for sludge used in landfarming.
7. The load of contaminants outflowing from the reed beds represents only a few percent of the load inflowing to the WWTP. Thus, the drainage waters can be recirculated to the plant without causing alterations of its operational parameters.

## 9.2 Facilities in Denmark

Sewage sludge dewatering and stabilization. This method uses plants which grow on mineral subsoil with overlying layers of sludge (with low content of dry matter about 0.5−1 %). In hydrophite method reed (*Phragmites australis*) is used most often (Obarska-Pempkowiak and Sobociński 2002). Reed systems are built as concrete constructions (beds) or as tight tanks placed in the ground (basins). Nicoll (1998) also tried to convert traditional sludge drying beds. Reed systems have construction similar to traditional sludge drying beds. However in reed systems draining systems which secure additional aeration are used and accumulation of sludge is conducted for 10–15 years (Kołecka and Obarska-Pempkowiak 2008; Zwara and Obarska-Pempkowiak 2000).

The cost of dewatering 1 ton of sludge in the reed systems is low, specifically only 5–10 % of the overall cost required using traditional sludge handling methods (Nielsen 2003b). Until now it has been proven that utilization of sewage sludge in reed basins results in stabilized and sanitary sludge (Kołecka and Obarska-Pempkowiak 2008; Nielsen 2003b, 2007). Therefore, it is possible to use it as a fertilizer in agriculture. Additionally, it was proven that the obtained product is safe as regards microbiological standards (Nielsen 2007).

Based on the conducted research, De Maeseneer (1996) found that the correct operation of reed systems requires determination of optimum dose of sludge: both quantity and frequency of supply is important to ensure sufficient time of rest between the subsequent supply events of sludge in time and depends on the age of plants and on type of sludge as well as content of dry matter. In the first season of operation low loads of sludge are recommended. In this period intensive propagation of root systems and development of plant occur. When plants are well rooted sludge can be supplied according to recommendations (Nielsen 2003a, b; Obarska-Pempkowiak et al. 2003). A surface load of reed systems depends on a type of sludge, a climate and available basin.

In temperate climate doses which were determined based on long standing experiences in Denmark can be applied. According to Danish experiences optimum doses are: 30–60 kg d.m./$m^2$ annum for activated sludge and 30–50 kg d.m./$m^2$ annum for digested sludge.

Based on long-term experience, Nielsen (2003a, b) recommended that reed systems should be built using several basins, namely at least 8. This makes it possible to supply raw sludge (irrigation) and ensure the time of rest (without irrigation) (Nielsen 2003a, b).

Sewage sludge after stabilization in reed systems can be used as fertilizer, because of high concentration of nitrogen and phosphorus and low concentration of heavy metals, which will be presented in the following pages.

## 9.3 Experimental Procedures

Secondary sludge stabilized in reed basins was investigated. Samples of sludge were collected from 4 reed basins located in conventional WWTPs: Rudkobing, Nakskov, Vallo and Helsinge (Denmark). The characteristic of the locations are presented in Table 9.8.

**Table 9.8** Characteristics of the analyzed sites

| Location of sites | Time of operation (years) | Number of basins | Total area ($m^2$) | Amount of sludge (t d.m./year) | pe |
|---|---|---|---|---|---|
| Rudkobing | 13 | 8 | 5,000 | 232 | 13,000 |
| Nakskov | 15 | 10 | 9,000 | 870 | 33,000 |
| Vallo | 7 | 8 | 3,867 | 300 | 9,000 |
| Helsinge | 9 | 10 | 10,500 | 630 | 40,000 |

The dry matter in sludge was determined after drying in a temperature of 105 °C to a constant weight according to the guidelines PN-78/C-04541.

The organic matter determination involved of burning dried and homogenized samples in a temperature of 450 °C for 8 h. It was assumed that loss on ignition correspond to the share of organic matter in the samples.

Kjeldahl nitrogen, the sum of organic and ammonia nitrogen, was determined in the analyzed sludge. The sludge sample was dried and homogenized. It was then alkalized using a 35 % solution of NaOH and mineralized in the presence of the catalyst $CuSO_4 + K_2SO_4$ using ammonium distillation. Sample mineralization was completed using Digestion Systems 1006 from the Swedish Company Tecator. The determination of ammonia nitrogen was carried out using the distillation method in the Kjeltec System 1026 from the Tecator company.

For determining the phosphorus concentration, the sample was dried, homogenized, and then mineralized using a mixture of the concentrated acids $HClO_4$ and $HNO_3$. In the obtained solution, $PO_4^{3-}$ ions were determined calorimetrically in the reaction with ammonia molybdate in the presence of glycerin with dissolved $SnCl_2$. Sample mineralization was completed in the Digestion System 1006 manufactured by the Swedish company Tecator. Calorimetric measurements were carried out in the Aquatec 5400—Analyzer from the company Tecator.

Six heavy metals (Cd, Cr, Cu, Ni, Pb i Zn) were determined in the analyzed sludge. The sludge sample was dried and homogenized. The mixture of two acids HCl and $HNO_3$ (in the ratio of 3 to 1) reacted on sample during 2 h in temperature of 80 °C. Obtained solution was cenrtifuged and evaporated to dryness. The rest was dissolved in 0.1 mol/l $HNO_3$. Next the heavy metal concentration was conducted used atomie absorption in spectrophotometer Thermo Jarrel Ash model 11E.

## *9.3.1 Results*

Average contents of dry matter in sludge utilized in reed basins from analyzed objects varied from 20.7 ± 2.6 % in sludge from Helsinge to 29.3 ± 3.5 in Rudkobing. In case of organic matter, average contents varied from 41.1 ± 2.9 % of d.m. in Helsinge to 46.0 ± 4.3 % of d.m. in Vallo. Table 9.9 presents average contents of dry matter and organic matter in sewage sludge utilized in analyzed reed systems.

**Table 9.9** Average contents of dry matter and organic matter in analyzed sludge

| Object | Average contents ± standard deviation (% d.m.) | |
|---|---|---|
| | dry matter | organic matter |
| Vallo | 26.1 ± 2.7 | 46.0 ± 4.3 |
| Rudkobing | 29.3 ± 3.5 | 42.0 ± 2.3 |
| Naskov | 23.6 ± 2.9 | 44.1 ± 2.0 |
| Helsinge | 20.7 ± 2.6 | 41.1 ± 2.9 |

**Table 9.10** Average concentrations of NK (Kjeldahl nitrogen) and TP (total phosphorus) in analyzed sludge, % d.m

| Object | Average concentrations + standard deviation (% d.m.) | |
|---|---|---|
| | NK | TP |
| Vallo | 2.2 ± 0.3 | 4.2 ± 0.6 |
| Rudkobing | 1.9 ± 0.2 | 4.7 ± 0.1 |
| Naskov | 2.4 ± 0.3 | 4.1 ± 0.4 |
| Helsinge | 2.0 ± 0.1 | 3.8 ± 0.2 |

Table 9.10 presents the concentrations of Kjeldahl nitrogen (NK) and total phosphorus in sewage sludge utilized in analyzed reed systems. In case of nitrogen it was noticed that it average concentrations changed from 1.9 % of d.m. in Rudkobnig to 2.4 % of d.m. in Naskov. And average concentrations of phosphorus varied from 3.8 % of d.m. in Helsinge to 4.7 % of d.m. in Rudkobing.

Table 9.11 presents average concentration of selected heavy metals and permissible values in agriculture usage Regulation of Environment Minister (2002) (Dz. U. No. 134, item 1140). Results of heavy metals contents compare well with these obtained in Poland (Sect. 9.1) except for copper and zinc (large contents were measured in Denmark). Based on research, it was found that the lowest average concentrations were in case of cadmium (about 1 mg/kg of d.m.). While the highest average concentrations were noticed for zinc. These concentrations were about 500 times higher than for cadmium.

**Table 9.11** Average content of heavy metals in sludge stored in reed basins and permissible values Regulation of Environment Minister (2002) (Dz. U. No. 134, item 1140), mg/kg d.m

| Object | Heavy metal | | | | | |
|---|---|---|---|---|---|---|
| | Cd | Pb | Cr | Ni | Cu | Zn |
| Vallo | 1.07 ± 0.13 | 10.0 ± 1.8 | 11.3 ± 1.2 | 26.3 ± 1.6 | 165.6 ± 10.3 | 520.2 ± 43.5 |
| Helsinge | 0.95 ± 0.12 | 10.7 ± 3.6 | 18.1 ± 4.5 | 20.7 ± 2.7 | 236.6 ± 41.5 | 416.0 ± 57.6 |
| Rudkobing | 0.84 ± 0.17 | 14.6 ± 2.7 | 32.1 ± 5.7 | 20.3 ± 3.4 | 218.6 ± 54.3 | 542.2 ± 95.6 |
| Nakskov | 0.74 ± 0.06 | 15.6 ± 2.3 | 17.5 ± 5.1 | 22.4 ± 2.1 | 80.8 ± 10.6 | 437.1 ± 39.1 |
| Permissible values | 10 | 500 | 500 | 100 | 800 | 2,500 |

### 9.3.2 Discussion

The dried sludge is characterized by high dry matter content. Similar dry matter content can by obtained using mechanical equipment (e.g. centrifuge or drying press) (Kołecka and Obarska-Pempkowiak 2008). So effective dewatering causes significant decrease of sludge volume which exceeds 90 %. Such efficient dewatering was caused by two factors: transpiration of water from sludge to the atmosphere by reeds and gravitational outflow of water aided by roots and rhizomes (Obarska-Pempkowiak et al. 2003). In case of organic matter it was notices that the obtained results were comparable to those obtained in pioneer reed bed in Darżlubie (Poland) (Obarska-Pempkowiak et al. 2003). Relatively low organic matter content indicates that organic matter in the sludge was biodegraded and stabilized. Similar conclusions were reached in research conducted by Nielsen (2003b) and Obarska-Pempkowiak and Sobociński (2002).

High concentrations of NK (Kjeldahl nitrogen) and TP (total phosphorus) were measured in the sludge. Especial concentrations of phosphorus were much higher than in sludge dewatered in reed beds in Darżlubie (Nielsen 2003b). So high concentrations of phosphorus can be caused by higher use of phosphorus artificial fertilizer in Denmark as well as longer time of stabilization.

Based on the obtained results it was found that sewage sludge utilized in reed basins can be used in agriculture since the analyzed heavy metal concentrations are below permissible legal values specified in the regulations. Utilization of sewage sludge in reed systems is a relatively new ecological method. This method permits long-term stabilization of sludge. Due to high dewatering, the volume of sludge decreases significantly (above 90 %). After 10−15 years sludge can be used as fertilizer in agriculture. High concentration of nitrogen and phosphorus indicate high value of dewatered in reed basins sludge as fertilizer. It is important that analyzed heavy metal concentrations are below permissible legal values for agriculture usage.

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