018530 - SWITCH

Sustainable Water Management in the City of the Future

Integrated Project
Global Change and Ecosystems

D5.3.1: Literature review on the use of natural systems in urban water management
D5.3.12: 6 PhD and 18 MSc theses on the theme of this work package

Shrestha, R. (2007) Possibilities for recycling domestic wastewater with vertical flow constructed wetlands. UNESCO-IHE MSc Thesis ES 07-44

Due date of deliverable: M60
Actual submission date: M24

Start date of project: 1 February 2006 Duration: 60 months

Organisation name of lead contractor for this deliverable: UNESCO-IHE

Revision [FINAL]

Project co-funded by the European Commission within the Sixth Framework Programme (2002-2006)		
Dissemination Level		
PU	Public	X
PP	Restricted to other programme participants (including the Commission Services)	
RE	Restricted to a group specified by the consortium (including the Commission Services)	
CO	Confidential, only for members of the consortium (including the Commission Services)	

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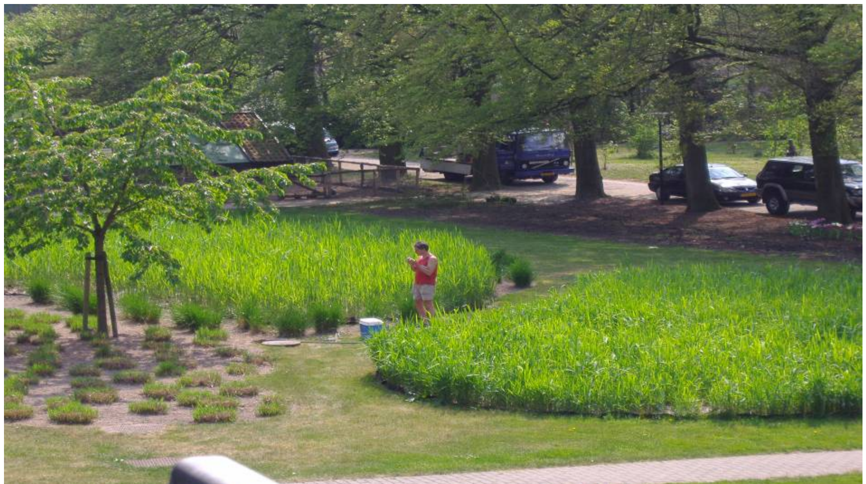

Possibilities for Recycling Domestic Wastewater with Vertical Flow Constructed Wetlands

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MSc Thesis ES 07.44
September 2007

UNESCO-IHE
Institute for Water Education

Possibilities for Recycling Domestic Wastewater with Vertical Flow Constructed Wetlands

Master of Science Thesis

by

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Delft

September 2007

Abstract

Making potable water and good sanitation accessible to all is one of the major challenges of the 21st century. Small scale onsite wastewater treatment technology can help to some extent to fight the challenge. In this context, a study was carried out to investigate the possibility of using small scale, low cost, decentralized wastewater treatment options for isolated areas. The research focused on the performance capacity and efficiency of Vertical Flow Constructed Wetlands from two sites: ZIN (a conference building) and geWooNboot (a houseboat), in treating the wastewater. Possibilities for potable and non potable reuse of treated wastewater were also explored. The effluent from the wetlands in the houseboat is further treated using a coal filter, a decalcifying filter, a fiber filter and a Reverse Osmosis system as an extra effort to make water potable. Performance of the wetlands was evaluated on the basis of concentration based removal efficiencies and mass removal rates. The effluent quality was compared with different discharge standards and reclaimed water guidelines. The results indicated that CWs were very efficient in removal of BOD, COD and were less efficient in nutrient removal. So the treated wastewater could not meet the discharge standards of sensitive areas while it could meet the standards for agricultural area. Despite of iron grit and calcium addition in the wetlands media, TP removal efficiency was still low. High pathogen concentrations in the effluent were observed, indicating the need for further disinfection before reusing the wastewater. From the literature review made during the study it can be concluded that the addition of a horizontal flow bed might help in decreasing the pollutants level. Despite of recycling treated wastewater in the houseboat, NO_3-N was not removed effectively, leading to high concentrations in the treated water. Moreover, continuous building up of pollutants was observed due to the short and closed water cycle in the houseboat. Though the water generated through Reverse Osmosis was free of bacterial contamination; certain parameters like pH, conductivity and nitrate concentration were not within the range of drinking water and thus the water might cause adverse health effects. From all this it can be drawn that generating potable water from wastewater in the houseboat was not the best option. Installation of a dry toilet and reusing only grey water can make the recycling more effective. Using rain water for producing drinking water could also be a better option; however the water volume collected from the roof was not sufficient so replacing the existing green roof by a normal roof would increase the water collection.

Key words: domestic wastewater, constructed wetlands, vertical flow constructed wetlands, and waste water reuse and recycling

Acknowledgements

My sincere gratitude goes to my supervisor Dr. Diederik Rousseau, for his invaluable comments and insightful suggestions on my work. His patience and encouragement was a great support.

I would like to extend thanks and appreciation to Mr. Frank van Dien of ECOFYT for his incredible help and providing me valuable information and making the field work interesting.

I would ALSO like to thank the entire laboratory staff for their tireless support during my lab work. Fred, Frank, Don, Peter and Lyzette all deserve the acknowledgement.

I would also like to thank The Netherlands government for the financial support of the study, and SWITCH project funded by the FP 6 programme of the European Union (no. 018530) which partially participated in funding this research.

Last but not the least my special thanks to my family and friends, Juna, Devendra, Nagendra, Binakak and Paula for their continuous help and support for the study. Also many thanks go to my friends at IHE who made me my stay in the Netherlands pleasant.

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List of Symbols, Acronyms and Abbreviations

BOD-Biological Oxygen Demand
COD-Chemical Oxygen Demand
CFU-Colony Forming Units
CW-Constructed Wetland
EC-Electrical Conductivity
EPA-Environmental Protection Agency
Et-Evapotranspiration
EU-European Union
FAO-Food and Agricultural Organisation
FC-Faecal Coliforms
HDPE-High Density PolyEthylene
HF-Horizontal Flow
IC-Ion Chromatography
IWA-International Water Association
Lpd-liter per person per day
LDPE-Low Density PolyEthylene
NA-Not Available
OTR-Oxygen Transfer Rate
PE-Person Equivalent
SSF-SubSurface Flow
TN-Total Nitrogen
TP-Total Phosphorus
TSS-Total Suspended Solids
PLC-Programmable Logic Controller
RE-Removal Efficiency
RO-Reverse Osmosis
RZE-Root Zone Effect
TC-Total Coliforms
USEPA-United States Environmental Protection Agency
UV-UltraViolet
VF-Vertical Flow
WHO-World Health Organisation

CHAPTER 1. INTRODUCTION

1.1 Background and problem statement

The world is urbanizing rapidly. At present 60% of the world's population lives in urban areas and worldwide it is estimated that by 2025 more than 90% of the population growth will take place in urban areas (Ujang and Henze, 2006). As urban population increases, the surrounding rural areas will also quickly develop. Water and sanitation for all is one of the major challenges in these areas. Finding ways to satisfy human water demand ranks among the most critical and difficult challenges of the 21st century. Most large cities are already facing water supply problems and these will only increase in the future.

Rural and non sewered areas often have to deal with the unsatisfactory performance of the conventional onsite wastewater treatment system (Steiner *et al*., 1993). Conventional onsite systems usually consist of septic tanks, leachfields and sand filters. Inadequate treatment of domestic wastewater in these areas can contribute to the pollution of surface water and ground water (Steiner and Combs, 1993). The nutrients and pathogens in human waste may impair water of streams and threaten public health. So there is a need of onsite treatment alternatives to conventional systems which can solve these potential problems, and which are practical, affordable, effective, simple, reliable, and environmentally friendly. Small scale systems have the capacity to make waste treatment more sustainable. Small scale sanitation systems keep the nutrient and water cycle small, making use of recovered waste more practical and cost effective.

There is a wide range of small scale onsite treatment technologies; constructed wetland (CW) is one of them. CWs have grown in popularity since early 1980s (Reed *et al*., 1995). Where land is available, CW systems will often provide the most cost effective and practical wastewater treatment alternative (Kadlec and Knight, 1996). The use of CWs for treatment of domestic wastewater has increased exponentially in the past decade, especially for small scale applications such as individual homes and small communities. Many urban areas around the world have already used this approach (Daniel, 1991). If well designed and maintained, their effluents can meet the high standards required for reclaiming the water (Rousseau, 2006). Reuse of wastewater for non potable use like irrigation, toilet flushing, laundry etc can reduce demand on limited fresh water resources. In Europe more than 200 water reuse project have been identified for non potable use. Only one reuse project had been identified for potable water production (Bixio *et al*., 2006).

Gleick (1996) proposed a minimum of 50 liters/person.day (lpd) as the basic water requirement for meeting the four basic human needs: drinking, human hygiene, sanitation and preparing food. Per capita domestic water consumption varies from country to country. At the uppermost is USA with an average consumption above 300-370 lpd. At the lower end are countries such as Gambia and Nigeria where consumption is in the range of 4-30 lpd. Unfortunately about 70% of the countries can only provide just less than 30 lpd (Butler and Memon, 2006).

In USA average water consumption was highest in toilet (27.6%), followed by clothes washing (21.7%), showers (16.8%), faucet (13.7%), bath (1.7%), dishwashers (1.4%) and other domestic uses (2.2%). In Sweden, the situation is slightly different. The major consumption is in bathrooms (32%) followed by kitchen use (23%), toilet (18%) and clothes

washing (13%) (Butler and Memon, 2006). In The Netherlands only 5% of all piped water used in houses is for drinking and cooking, nearly two thirds is for bathing and washing cloths and dishes, while nearly one third is to flush toilets and clean houses and streets.

It has been demonstrated that more than 60% of the drinking water could be substituted by grey water (from kitchen, laundry, and shower). However, if only domestic grey water could be re-used to water lawns and ornamental gardens, the average household potable water usage could be reduced by 30–50% (Butler and Memon, 2006).

Besides, in a country like The Netherlands where water pollution in a canal is a special problem due to the large number of houseboats – 10,000 house boats in the canals of major cities like Amsterdam, Rotterdam, Leiden and Delft without proper treatment – small scale CWs on the houseboats could solve the problem to some extent. The Center of Appropriate Technology, "De Twaalf Ambachten" in Boxtel, The Netherlands suggested constructed wetlands on houseboats for wastewater treatment (Veenstra, 1998). In this research such a constructed wetland built on a houseboat has been studied.

Constructed wetlands are very complex biological systems imitating the processes of natural wetlands. They provide effective and reliable wastewater treatment. The pollutants in such systems are removed through a combination of physical, chemical and biological processes. Constructed wetlands have many advantages over conventional systems e.g. low cost of construction and especially, maintenance; low energy requirement; simple technology system and thus can be established and run by relatively untrained personnel and the systems are usually more flexible and less susceptible to variation in loading rate than conventional treatment systems. The major disadvantage of CW treatment systems are the increased land area required, compared to conventional systems, Therefore the disposal wastewater in CW is an especially attractive alternative for small to medium sized communities.

In this research several constructed wetlands treating domestic wastewater (black and grey water) have been studied. They were located at two sites: (i) a conference building (ZIN) and (ii) a houseboat (geWooNboat). A reverse osmosis system on the houseboat to generate potable water was also considered in this research.

1.2 Goal and objectives of the research

1.2.1 Goal

The overall goal of this research is to assess the potential contribution of constructed wetlands to water reuse and to conduct an extensive literature review on Constructed wetlands for wastewater treatment as an input for the demonstration cities of the EU SWITCH project on urban water management.

1.2.2 Specific objectives

- To determine and compare the performance of two similar wetlands built in different years near the conference building of ZIN;
- To evaluate and compare the performance of constructed wetlands from two different sites (ZIN and geWooNboat);
- To calculate water budgets of the constructed wetlands;
- To calculate mass balances of nutrients of the constructed wetlands;

- To determine the technical and economical feasibility of constructed wetlands;
- To evaluate advantages and limitations of the systems;
- To suggest the suitable reuse of treated wastewater at the conference center ZIN;
- To determine the technical and economical feasibility of closed loop in a houseboat.

1.3 Hypothesis

Following assumptions were made for this study:

- Constructed wetlands are efficient for domestic wastewater treatment.
- The treated water can be easily reused for non potable purposes.

1.4 Limitation of study

Polderdrift community in Arnhem, The Netherlands was the first choice, as an example of sustainable water management. The community consists of 40 households having a rooftop rain water harvesting system and separate wastewater (black and grey) collection system with grey water treatment in a constructed wetland. Treated grey water is reused for toilet flushing whereas rain water is used for laundry. All the households were equipped with water saving devices. Unfortunately, during the study period the wetland stopped functioning and therefore the conference building and the houseboat were chosen for the study.

One of the objectives of this study is to suggest the suitable reuse of wastewater. However, due to the lack of guidelines or regulations at the European Union level, the suggestions are made on the basis of guidelines used in some other countries.

1.5 Overview of the content

The structure of this report can be briefly summarized as follows:
Firstly the literature review discusses about theories and past studies done in the field of constructed wetlands and possible reuse of treated wastewater. This will be followed by the Methodology section, which summarizes the procedure undertaken to conduct this study (site description, sample collection and analysis, data analysis). The chapter Results and Discussion is a compilation of the major results and data analyses. This will indeed try to explain the interesting findings on the use and efficiency of VF CWs. The next chapter is the Conclusion and comprises the main output of the study on the economical and technical feasibility of constructed wetlands for ZIN and the houseboat, and suggestions on the suitable reuse of wastewater. Finally recommendations on the suitable reuse of wastewater and ways to improve the effluent quality are presented.

CHAPTER 2. LITERATURE REVIEW

2.1 History and Present Scenario of Constructed Wetlands

The historical background on the use of CWs in water pollution control originates from the research pioneered by Seidel and Kickuth in Germany from 1952 onwards (Bastian and Hammer, 1993; Kadlec and Knight, 1996). Utilization in different countries started and developed at different times and rates. For example, in the Netherlands, the use of constructed wetlands started in 1967 with the experimental work using *Scirpus lacustris* in a camping site in Flevoland (de Jong, 1976). Application in USA commenced in 1967 (Kadlec and Knight, 1996) while in United Kingdom, the use of the technology started in 1985 (Cooper and Green, 1998). Extensive research works on CW technology have been undertaken but mainly in temperate regions.

Currently, over 50,000 wetland treatment systems have been constructed worldwide (Kadlec, 2004), and the number is increasing with rapid spreading of interest in wetland treatment. Using plants to purify wastewater has tremendous appeal to the general public. Within the regulatory and design community interest has been sustained because of the simplicity of treatment wetlands. Wetland systems are one of the few technologies that can produce a biologically treated effluent (to secondary or better standards) with sufficiently low pathogen content. Wetland systems can achieve this level of treatment with minimal external labor or energy input and operator support. This system can be constructed from local material and is easy to operate and maintain, so they have the potential for widespread applications in developing countries. Constructed wetlands are currently in use in a variety of countries, including Tanzania, South Africa, Peru, Colombia, Mexico, Egypt, Turkey, Nepal, Thailand, India and China (Wallace and Knight, 2006).

2.2 Types of Constructed Wetlands

CWs can be classified based on the type of macrophytes used as:

- Free-floating plants (e.g. *Eichhornia crassipes, Lemna, Spirodella, Wolffia*)
- Emergent plants (e.g. *Phragmites, Scirpus, Typha*)
- Submerged plants (e.g. *Isotes lacustris, Elodea canadensis*)

CWs can further be categorized into two major groups according to the flow regime namely:

- Free water surface (FWS) or surface flow (SF) and
- Subsurface flow wetlands (SSF).

In addition different types of constructed wetlands can be combined with each other or with conventional treatment systems in order to exploit specific advantages of the different systems (Vymazal, 2001). The classification based on the flow pattern is shown in Figure 1.

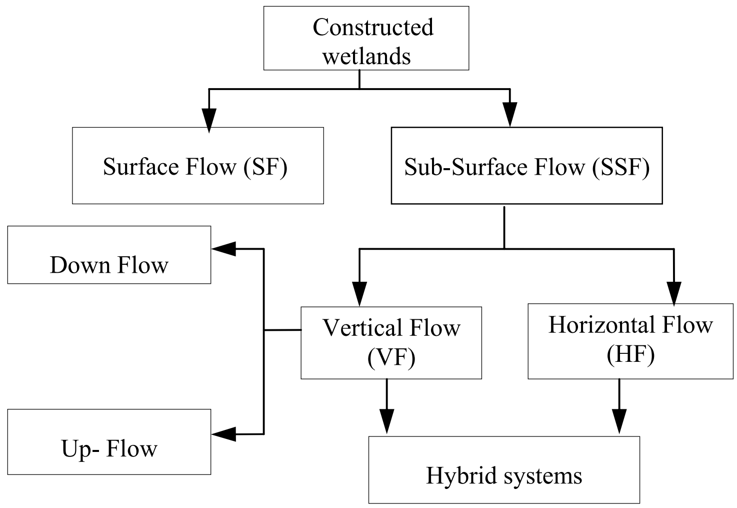


Figure 1: Classification of constructed wetlands for wastewater treatment based on flow pattern
Source: Vymazal 2001

2.2.1 Surface Flow (SF) wetlands

These wetlands contain areas of open water, floating vegetation, and emergent plants. Dikes, beams and liner are used depending on the local regulation and soil conditions in order to control flow and infiltration. As wastewater passes it is treated by the processes of sedimentation, filtration, oxidation, reduction, adsorption, and precipitation (US EPA, 2000).

SF constructed wetlands attract wildlife, namely insects, mollusks, fish, amphibians, reptiles, birds and mammals as they closely resemble natural wetlands (Knight *et al*., 1993; Kadlec and Knight, 1996). Because of the potential for human exposure to pathogens, SF wetlands are rarely used for secondary treatment (US EPA 2000). The most common application is for polishing effluents from secondary treatment process (e.g. lagoons, trickling filters, activated sludge system, etc.).

SF wetlands are more suitable in warmer climate because biological decomposition rates decrease with decreasing water temperature, and they potentially freeze in cold climate. In addition, the oxygen transfer from atmosphere decreases when ice covers open water surface, further decreasing oxygen dependent treatment process (US EPA, 2000).

2.2.2 Sub-Surface flow Systems (SSF)

In this system water flows below the surface in a media of sand, gravel or rock. Emergent aquatic plants are planted on top of the media. Existing systems of this type range from those serving single family dwellings to large scale municipal systems (Reed, 1995). Nowadays, constructed wetlands are common alternative treatment system in Europe's rural area and over 95% of these wetland systems are subsurface flow wetlands. Unlike SF wetland, in this system wastewater is not exposed during the treatment process, minimizing the risk associated with human exposure to pathogenic organisms.

SSF can be classified further into horizontal flow (HF) and vertical flow (VF) depending on the direction of flow.

Horizontal Flow (HF) Wetland Systems

Initially the main interest was horizontal flow systems because they were simple and promised low construction and operation costs. There are many fine examples of HF systems for secondary treatment and even in cold season they gave satisfactory results where standards require only BOD_5, TSS and bacteria removal (Cooper, 1999). However oxygenation of the rhizosphere in such systems is often insufficient hence incomplete nitrification causes limited nitrogen removal. This system can remove nutrients at an average efficiency of 40-60% (Haberl *et al*., 1999). In order to meet the higher requirements of small and very sensitive recipients there has been growing interest in achieving fully nitrified effluents.

Vertical Flow (VF) Wetland Systems

Originally developed by Seidel (1967), VF system has been in operation in Europe for approximately 20 years in Germany, The Netherlands, Austria and UK (Burk and Lawrence, 1990). Brix (1992) gave an overview of Seidel systems in Europe. Cooper (1996) has developed design criteria for desired nitrification in VF constructed wetlands based on the oxygen demand. There has been a growing interest in VF systems due to much greater oxygen transfer capacity resulting in good nitrification; they are considerably smaller (1-2m^2/PE) than the HF system (which needs 5-10 m^2/PE for secondary treatment); they can effectively remove BOD_5, COD and bacteria (Cooper, 1999).

Hybrid Systems

More recently there has been growing interest in the hybrid system. Various types of constructed wetlands can be combined in order to achieve high purification effect. Most hybrid systems comprise VF and HF wetlands. In these systems the advantages and disadvantages of HF and VF system are combined to complement each other. With this system it is possible to produce an effluent with low BOD, which is fully nitrified and partly denitrified and hence has much lower TN concentration (Cooper, 1999).

2.3 Scope of Constructed Wetlands

The use of CWs for sewage treatment at different levels is commonly known. However, they have also been applied for the treatment purpose of different types of wastewater. Some of these applications (Table 1) include treatment of wastewater originated from several industries, agricultural activities, landfills, surface runoff, acid mine drainage, etc.

Table 1: Distribution and percentage of studies on CWs used for several purposes (1994-2000)

Type of Treated wastewater	Number	% of Distribution
Municipal	242	36.90
Leachate	23	3.50
Acid Mine drainage	27	4.11
Surface Runoff	41	6.25
Sludge Dewatering	23	3.51
Industrial	63	9.60
Restoration and Rehabilitations, Prevention of Eutrophication	58	8.84
Agro-Industries	65	9.91
Reviews, Suggestions, Design Criteria	114	17.37
Total Number of Reviewed CW Studies	656	100.00

Reference: Korkusuz (2005)

2.3.1 Constructed wetlands for Sewage Treatment

During the early years (pre-1985) of the development of the CW technology, virtually all emphasis was on the secondary and tertiary treatment of domestic and municipal wastewater after mechanical pretreatment. Since then, there are an expanding number of applications used for sewage treatment throughout the world (Kadlec *et al*., 2000). The number of subsurface flow (SSF) CWs in operation in Europe is around 5000. In Germany alone, nearly 3500 systems are in operation (Borner *et al*., 1998). Many systems are also in operation in Denmark (200-400), Austria (around 160), Czech Republic (around 80), Poland (around 50), Slovenia (around 20) and Norway (around 10).

In general, most European SSF treatment wetlands are designed to treat domestic or municipal wastewater from sources less than 500 population equivalents (PE). However, most systems are designed for small sources of pollution (less than 50 PE) and many systems are designed for single households. Only a small number of systems were designed for large sources of pollution (>1000 PE) (Kadlec *et al*., 2000).

Since developed countries widely used wetlands technology for treatment of domestic wastewater under various conditions successfully; there is a growing interest in use of CW in developing countries as well. However, there is limited information on the level of development of wetlands technology in these countries. It appears that in some countries, basic research is being carried out, while in others, the technology has reached pilot and full scale levels for various applications (Kivaisi, 2001). Morocco, Egypt, Iran, Thailand, Uganda, Nepal, India are some developing countries where several researchers recently have conducted studies on treatment of domestic wastewater in CWs.

For example, Mandi *et al*. (1998) conducted a study on the purification of domestic wastewater under semi arid conditions in Morocco. Reed beds planted with *Phragmites australis* had a COD removal of 48-62%, TSS of 58-67% and a parasitic removal of 71-95% at a hydraulic load of 0.86-1.44 m^3/d. Further experiments were carried out to improve the removal performances of the reed beds. In Iran, a subsurface flow reed bed (*Phragmites australis*) of 150 m^2 was tested for treating municipal wastewater, at an organic loading rate of 200 kg/ha.d which is higher than previously recommended (<133 kg/ha.d) (Metcalf and Eddy, 1991), removal efficiencies of 86% COD, 90% BOD_5, 89% TSS, 34% TN, 56% TP and 99% faecal coliform bacteria were obtained. Besides, no clogging problems were experienced (Badkoubi *el al.*, 1998).

The potential of CWs for application by small communities for wastewater treatment has been examined in Nepal (Laber *et al.*, 1999). A hybrid system comprising of HF and VF beds (140m^2 HF and 120 m^2 VF) with *Phragmites karka* were tested for one year on full scale for treatment of hospital wastewater. At a hydraulic loading rate of 107 mm/d, removal efficiencies for COD, BOD_5, NH_4-N, TP, total coliforms, *Escherichia coli, Streptococcus* and TSS were 93%, 97%, 99.7%, 74%, 99.995%, 99.97% and 98%, respectively.

2.3.2 Constructed wetlands for Industrial Wastewater Treatment

The use of CWs to treat industrial wastewater has increased significantly over the past ten years. Unlike the municipal wastewater discharges, which usually have a consistent composition, industrial wastewater often have a variety of components of varying degrees of biodegradability and toxicity, thus necessitating different treatment modalities and strategies (Davies *et al.*, 1990). However, due to the toxic constituents, not all the industries can discharge their effluent into constructed wetlands (Husband *et al*., 2000).

Ongoing studies are concentrated on the removal of industry-specific contaminants unique to the generated effluent of a particular industrial operation (Husband, 2000).
Some of the constructed wetlands applications for industrial wastewater treatments are (Ferraro and Kadlec, 2000):

- Leachate control of effluent from landfills
- Water from power plants
- Abandoned and active mines
- Remediation of polluted groundwater and surface water in hazardous waste industry
- Remediation of hydrocarbon effluents of the petrochemical industry
- Remediation and control of effluent from pulp and paper industry and
- Remediation and control of effluents from food industries

The distribution of the recent studies on CWs for industrial wastewater treatment between 1994-2000 is presented in Table 2.

Table 2: Distribution of recent studies on CWs for industrial wastewater treatment

Types of industrial wastewater	Total no. of application	% of Distribution
Petro-Chemical	6	20.70
Pulp and paper	3	10.34
Heavy metal treatment	7	24.14
Radionuclide	2	6.89
Laboratory wastewater	1	3.45
Olive oil wastewater	1	3.45
Explosives	1	3.45
Smelter Effluent	1	3.45
Textile	1	3.45
Sugar Beet	2	6.89
Potato Processing Industry	2	6.89
Chlorinated Alkenes	1	3.45
Chemical Manufacturing plants	1	3.45

Reference: Korkusuz (2005)

2.3.3 Constructed wetlands for Agro-Industrial Treatments

Wastewater from intensive agricultural activities has significant higher concentrations of organic matter and nutrients than municipal effluent. The high pollution loads can contribute to water management problem if waste is allowed to discharge directly to receiving water. Agriculture wastes must be treated prior to disposal and constructed wetlands in association with stabilization ponds have been suggested as a potential treatment option prior to land application (Geary and Moore, 1999).

In most cases, a SF wetland will be the most cost-effective choice for treatment of this wastewater, since the smaller land areas and other potential advantages of the SSF are not usually essential in agriculture setting. The distribution of agro–industrial wastewater treatments in constructed wetlands is given in Table 3.

Table 3: Distribution of the recent studies on CWs for agro-industrial wastewater treatment

Types of agro-industrial wastewater	Total no. of Applications	% of Distribution
Animal wastewater	12	38.70
Dairy wastewater	10	32.26
Agriculture wastewater	5	16.13
Pesticide containing wastewater	1	3.24
Slaughter house wastewater	3	9.67

Reference: Korkusuz (2005)

2.3.4 Constructed wetlands for storm water Runoff

Wetlands are an essential part of nature's storm water management system. Important wetland functions include natural restoration processes, which improve water quality while being cost effective; conveyance and storage of storm water, which dampens effects of flooding; reduction of flood flows and velocity of storm water, which reduces erosion and increases sedimentation; and modification of pollutants typically carried in storm water. Although detention ponds to reduce storm water pollution concentration in urban catchments are increasingly being used (Bavor and Mitchell, 1994), there is a great amount of interest in incorporation of natural wetlands and constructed wetlands into storm water management systems (Sakadevan and Bavor, 1998). The reason of the preference of constructed wetlands over the detention ponds is the effective removal capacity of constructed wetlands for treatment of particulate bound pollutants such as metals, nutrients and microorganisms (Bavor, 2000).

2.3.5 Constructed wetlands for highway Run-Off

Pollutants accumulated on the road surface from vehicle wear, emission, and accidental spillages are transported into runoff. These pollutants include sediments, metals, salts, hydrocarbons, pesticides and herbicides. The quality of road runoff also has an impact on receiving water bodies. Adsorbed pollutants can have acute or chronic toxic effects (Pontier *et al*., 2000).The use of CWs for the treatment of highway runoff is a relatively new technology although it has been established in US for several years (Kadlec and Knight, 1996). The focus on fulfilling the objective of Agenda 21 in Europe and the UK has encouraged the adoption of wetlands as method of sustainable environmental management.

The related monitoring studies for the treatment of highway runoff via CWs show effective removal of suspended solids, as well as heavy metals like copper, chromium, nickel, zinc, lead and iron (Sakadevan *et al*., 1999). But still, there are no established design criteria for CWs for treatment of highway runoff, since road runoff is an intermittent and highly variable feedstock (Pontier, 2000).

2.3.6 Constructed wetlands for Sludge Dewatering

Vertical flow (VF) CWs have been applied for dewatering sludge for over 30 years. Such wetlands, in which common reed is planted in soil, sand or gravel appears to offer both economic and environmental advantages over alternative methods of sludge dewatering. They are far more effective than sand bed drying; as they do not require chemical flocculation, centrifuges or belt presses (Edwards *et al*., 2000).

Reinhofer and Berghold (1994) reported that on an area of about 0.15-0.25 m^2/PE, a sludge volume can be reduced by 80-90% and dewatering capacity to about 35% dry matter is possible. Pauly (1994) succeeded in dewatering sludge even up to 60% dry matter. Moreover, he could achieve good hygienic quality of the end product. Even though most of the full scale experience on reed bed dewatering comes from Denmark (Nielsen and Willoughby, 2005), there are also both full scale and pilot scale applications for sludge dewatering in other countries like France (Lienard *et al*.,1995), US (Kim, 1997), UK (Nuttall *et al*., 1997; Cooper and Willoughby, 1999), Germany (Platzer, 2000) and Poland (Obarska-Pempkowiak, 2000).

2.4 Removal Mechanisms in Constructed Wetlands

Wetland systems reduce many contaminants, including organic matter, suspended solids, nitrogen, phosphorus, trace metals and pathogens by a complex variety of physical, chemical and biological processes (Vymazal *et al*., 1998, Hammer, 1990). A list of parameters and removal mechanism is given in Table 4.

Table 4: Removal mechanism for various parameters in CWs (Reed Beds)

Wastewater Constituent	Removal mechanism
Organic matter	• Aerobic microbial degradation • Anaerobic microbial degradation
Suspended solids	• Sedimentation • Filtration
Nitrogen	• Ammonification followed by microbial nitrification and denitrification • Plant uptake • Matrix adsorption • Ammonia volatilization
Phosphorus	• Matrix sorption • Plant uptake
Metals	• Adsorption and cation exchange • Complexation • Precipitation • Plant uptake • Microbial oxidation/reduction
Pathogens	• Sedimentation • Filtration • Natural die-off • Predation • UV radiation • Excretion of antibiotics from roots of macrophytes

Source: Cooper et al., 1996

2.4.1 Organic Matter Removal

Treatment efficiency of constructed wetlands for removal of organics (BOD_5, COD) is generally high (Kadlec and Brix, 1995; Vymazal *et al*., 1998). Settleable organic matter is removed in wetland systems under quiescent conditions by sedimentation and filtration. Organic compounds are degraded biologically both aerobically as well as anaerobically in the wetland system depending on the oxygen concentration in the bed. Nearly all biological degradation takes place within bacterial films present on solid surfaces, including sediments, soils, fill media, litter and live submerged plant parts (Kadlec, 2000).

The oxygen required for aerobic decomposition is supplied directly from the atmosphere through diffusion, convection and oxygen leakage from macrophyte roots (Vymazal *et al*., 1998, Cooper *et al*., 1996). Anaerobic degradation of organic matter will occur during periods of oxygen depletion but is much slower than aerobic degradation. However, when oxygen is limiting at high organic loading, anaerobic degradation will predominate (Vymazal *et al*., 1998,) Treatment efficiency of constructed wetlands for removal of organics is highly dependent on the oxygen concentration in the bed; the wetland design; treatment condition; characteristic of the filling media (Kadlec and Brix, 1995; Vymazal *et al*., 1998). Uptake of organic matter by the macrophytes is negligible compared to biological degradation (Cooper *et al*., 1996).

2.4.2 TSS Removal

All settleable and floatable solids are removed in wetland systems due to long hydraulic residence time. The major mechanisms are sedimentation and filtration. Non settleable and colloidal solids are removed by bacterial decomposition, adsorption to the wetland media and plant root system (Stowell *et al.*, 1981) .The extensive root system adds surface area to the wetland media, which reduces water velocity and reinforces settling and filtration in the root network. However, plant effects are usually observed after three years of establishment in many of the wetlands (Brix, 1997).

In a subsurface flow wetland, the end result of subsurface biological and vegetative activity is the build up of solids within the pore spaces of the media. This build up is large near the inlet and large near the top of the bed (Kadlec and Watson 1993). A significant portion of the pore volume can be blocked by accumulated organic matter, which can result in an increased hydraulic gradient and decreased retention times (Tanner and Sukias, 1994). Clogging can reduce the hydraulic conductivity of the media and result in surface overflow (Reed *et al.*, 1995). In many systems, to prevent clogging, the majority of settleable solids are removed in a mechanical pretreatment unit (e.g. sedimentation or Imhoff tanks) before the wastewater is discharged to the actual wetland system.

2.4.3 Nitrogen Removal

Nitrogen compounds are among the principal constituents of concern in wastewater because of their role in eutrophication. N-compounds particularly ammonia, can exert a significant oxygen demand through biological nitrification and may causes exhaustion of dissolved oxygen through concentration in receiving waters. Unionized ammonia can be toxic to aquatic organisms and can readily react with chlorine. High nitrate level in water supplies has been reported to cause methemoglobinemia in infants. Therefore, the need for N-control in wastewater effluents has generally been recognized and many treatment processes have been developed to remove N from the wastewater system (Lee and Lin, 1999).

Nitrogen in wastewater exists in the form of organic and inorganic nitrogen. Typical domestic wastewater contains 20 mg/L of organic nitrogen and 15 mg/L of inorganic nitrogen. In wetland systems, the initial removal of organic nitrogen as TSS is usually rapid. Microbial processes convert particulate organic nitrogen via decomposition into new biomass and ammonium. The removal mechanisms for N in constructed wetlands include volatilization, ammonification, nitrification/denitrification, plant uptake and matrix adsorption (Brix 1993). However, the major removal mechanism in most constructed wetlands is microbial nitrification and denitrification (Vymazal, 1998, Moshiri, 1993, Cooper *et al.*, 1996). In systems with free-floating macrophytes ammonia volatilization can significantly contribute to nitrogen reduction (Vymazal, 1998).

2.4.3.1 Ammonia Volatilization

Ammonia is a weak base and exists in equilibrium with the ammonium ions. The concentration of the species is pH dependent and significant concentrations of ammonia (NH_3) occur only at pH values above 8. Indeed, Reddy and Patrick (1984) pointed out that loss of ammonia through volatilization from flooded soils and sediments are insignificant if the pH value is below 7.5. At pH of 9.3, loss via volatilization is significant (Vymazal, 1998).

2.4.3.2 Ammonification

Ammonification (mineralization) is the biological transformation of organic N into inorganic N, especially NH_4-N (Vymazal, 2002). Mineralization rates are faster in the oxygenated zone

then in anaerobic zone. The rate of ammonification in wetlands depends on temperature, pH value, C/N ratio of the residue, available nutrients in the system, and soil conditions such as texture and structure (Reddy and Patrick 1984). The optimum pH range is between 6.5 and 8.5. The aerobic ammonification rate doubles with the increase in temperature of 10^{o}C (Reddy *et al*., 1979).

In aerobic wetland environment, the ammonia resulting from ammonification of organic nitrogen is more likely to undergo nitrification, and TN typically will be reduced through consequent denitrification (Reddy and Patrick, 1984).

2.4.3.3 Nitrification

Nitrification is a biological oxidation of ammonium to nitrate with nitrite as an intermediate in the reaction sequences. The organisms require oxygen during oxidation (Lee and Lin, 1999). According to the nitrification equation, to oxidize 1 mg/L of NH_4-N to NO_3-N, approximately 4.3 mg/L of oxygen is required; where as 8.64 mg/L of bicarbonate is utilized. Due to release of protons and alkalinity destruction during biological nitrification, there is a drop in pH value.

Vymazal (1995) summarized that nitrification is influenced by temperature, pH value, alkalinity of wastewater, inorganic carbon source, microbial population and concentration of ammonium–N and dissolved oxygen. Reported data show a wide range of optimum pH value (from 7.0 to 9.5) with maximum activity at pH of about 8.5. Below pH of 7.0, adverse effects on ammonia oxidation become pronounced (Lee and Lin, 1999).

In an intermittently loaded VF system, oxygenation in the wetland matrix is increased several folds compared to the horizontal substrate flow system (Brix and Schierup, 1990). Therefore, these systems are generally able to provide good nitrification of the wastewater (Brix, 1998). For gravel bed systems, compared to soil bed systems, the higher porosity allows more oxygen transfer to the substrate and more biomass accumulates inside the substrate, both of which are helpful for nitrification. It was reported that Root Zone Effect (RZE) might exist in vegetated constructed wetlands system, which could enhance nitrification by transferring oxygen to the root zone area (Yang *et al*., 2001). There are varying reports about the quantity of plant root zone oxygen release. Brix *et al*. (1996) found oxygen inputs of 20 mgO_2/m^2.d, whereas Gries *et al*. (1990) and Armstrong *et al*. (1990) measured oxygen transfer rates of 2-12 gO_2/m^{2}d. These differences could be explained by the differences by short term nature of the measurements. More reliable data can only be given by long term investigations (Luederitz *et al*., 2001)

2.4.3.4 Denitrification

The first oxidation process to occur after oxygen depletion is the reduction of nitrate to molecular nitrogen or nitrogen gases, which is called denitrification. Vymazal (1995) summarized the environmental factors known to influence denitrification rate including the absence of oxygen, redox potential, soil moisture, temperature, pH value, the presence of denitrifiers, soil type, organic matter and the presence of overlying water. The optimum pH for denitrification lies between pH of 7-8; however, alkalinity produced during denitrification can result in slight rise in pH. Denitrification is also strongly temperature dependent and proceeds at very slow rate, if the temperature falls below 5^{o}C (Vymazal *et al*., 1998).

2.4.3.5 Plant Uptake

The potential rate of nutrient uptake by plants is limited by its net productivity (growth rate) and the concentrations of nutrients in the plant tissue. Thus, desirable traits of plants used for nutrient assimilation and storage would include rapid growth, high nutrient content in tissue and a high biomass per unit area (Reddy and Debusk, 1987). On the other hand break down and decay of dead leaf and stem materials on the surface of reed bed, eventually leach nutrients back to the water column. Although plant uptake of nitrogen occurs, only a minor fraction can be removed by plants. Reports have indicated that under optimum conditions the amount of nitrogen removal through harvesting the plant biomass accounts for only 10-16% of the total removal nitrogen from the wastewater (Gersberg *et al*., 1985; Dusek *et al*., 1997).

2.4.3.6 Matrix Adsorption

Ionized ammonium may be removed from solution through a cation exchange adsorption reaction with detritus and inorganic sediment in wetlands. However, this removal is not considered to be a long term sink as it is bound loosely to the substrate and can be released easily when water chemistry changes.

2.4.4 Phosphorus Removal

Phosphorus is often a limiting nutrient in freshwater and it normally controls the productivity of these systems more than any other nutrient or environmental factor (Wetzel, 2001). Input and accumulation of phosphorus from external sources is recognized as the main cause of eutrophication in surface waters.

Phosphorus occurs as phosphate in organic and inorganic compounds. Free orthophosphate represents a major link between organic and inorganic phosphorus cycling in wetlands as it is the only form of phosphorus believed to be utilized directly by the algae and macrophytes (Vymazal, 1995).

Phosphorus removal in constructed wetlands is through adsorption, complexation and precipitation. Removal is high in submerged bed designs when appropriate soils are selected as the media. A significant clay content and iron, aluminum, and calcium will enhance phosphorus removal. Phosphorus is removed through precipitation by iron and aluminum in acidic conditions ($pH < 6$), and by calcium and magnesium in alkaline condition ($pH > 8$) (Polprasert and Veenstra, 2000). Removal is low in surface flow wetlands due to limited contact with soil and root zone (Hammer, 1990, Vymazal *et al*., 1998). Moshiri (1993) has compiled the removal rate for phosphorus as 20-90%.The alternate wet and dry periods enhance the fixation of phosphorus in sediments. However, sites of adsorption and precipitation are finite and are susceptible to saturation. When this stage is reached, wetlands act as sources (Lee *et al*., 1975). It has been reported that after 2-5 years the wetland may no longer be effective at removing phosphorus from the wastewater (Drizo *et al.,* 1999).

The uptake by macrophytes is low, only a small fraction of TP is removed. In order to have efficient removal of phosphorus through macrophytes it is necessary to harvest the macrophytes biomass (Vymazal *et al*., 1998). Plant uptake may be significant only in systems where the area specific loading rate is low.

2.4.5 Pathogen removal

The fundamental purpose of collecting and treating wastewater is the protection of public health. All the different wastewater treatment methods and operational procedures aim at achieving this central issue (Metcalf and Eddy, 1991). Human pathogens are typically present in untreated domestic wastewater. These include bacteria, protozoa and helminths. According to WEF, 1992; Metcalf and Eddy (1991) as cited in Kadlec and Knight (1996), conventional technology reduces these pathogens to non infective levels with the addition of necessary removal processes such as chlorination, ozonation, and ultraviolet disinfection. Chlorination has been the disinfection method of choice for many years; however negative side effects on aquatic organism have been noticed, due to residual free chlorine. Cost, operation and maintenance, and performance problems have limited the use of ozonation and ultraviolet disinfection (Metcalf and Eddy, 1991).

Constructed wetlands are known to offer a suitable combination of physical, chemical and biological factors for the removal of pathogens (Vymazal *et al*., 1998, Cooper *et al*., 1996). Wetlands are known to act as bio filter through the combination of physical, chemical and biological factors which all participate in the reduction of number of bacteria (Brix, 1993: Vincent *et al*., 1994). Physical factors include filtration, sedimentation, aggregation and UV ray action. Biological mechanisms include antibiosis, ingestion and natural die off. Chemical factors include oxidation, adsorption and exposure to toxics excreted by other microorganisms and plants (Gersberg *et al*., 1989). Constructed wetlands reduce pathogens more successfully due to longer residence time and land intensive treatment (Kadlec and Knight, 1996). At a retention time of 3 days the pathogen reduction is essentially complete (Wallace, 2006). The vegetation appears to be affect pathogen removal. Gersberg *et al*. (1987) reported that the removal of total coliforms was higher in a vegetated bed compared to an unvegetated bed. Moreover, aquatic plants (*Phragmites australis*) can kill faecal indicators (*Escherichia coli*) and pathogen bacteria (Seidel, 1976). Several regions of root are known to produce compounds which may inhibit certain microorganisms (Bowen and Rovira 1976). Gerba *et al*. (1999) reported a typical removal rate of 98-99% for total coliforms and faecal coliforms in an SSF wetland.

Measurement of human pathogenic organisms in untreated and treated wastewater is expensive and technically challenging. Due to this fact the indicator organisms that are easy to monitor and correlate with population of pathogens organism, though no perfect indicators have been found, however the coliform bacteria group has been the first choice among the indicator organisms.

Wetland systems produce non zero background concentrations of coliform bacteria as a result of flora and fauna living in the wetland. Because of this background concentration, meeting FC limit of 200 CFU/100 mL for surface water discharge, will likely require disinfection or some other pathogen reduction process (Vega *et al*., 2003).

2.5 Recycling and reuse of wastewater

The quest for more sustainable water management practices has led to an increased interest in water reuse practice worldwide. As a source of water that is relatively constant through the year and due to the opportunity of nutrient recycling, wastewater reuse can contribute to increasing the reliability of water supply and to close the nutrient cycle (Ghermandi, 2007).

Domestic wastewater contains valuable nutrients. Pure human urine contributes the largest amount of nutrients to domestic wastewater, 80% of the nitrogen and 50% of phosphorus and potassium, can be used as valuable nutrient rich liquid fertilizers. Facies has organic matter which can be anaerocically digested to produce biogas or compost to solid fertilizer. Grey water being a large fraction of total wastewater flow can readily be applied for reuse after simple treatment (Fittschen ad Hahn, 1998).

To date the major emphasis on wastewater reclamation and reuse has been for non potable applications such as agriculture and landscape irrigation, industrial cooling and in-building applications such as toilet flushing. Direct potable reuse of reclaimed wastewater is, at present, limited to extreme situations. Although a number of household reuse schemes have been developed for both black water and grey water, the only reuse scheme which is potentially cost effective is considered to be the reuse of grey water for toilet flushing (Geary, 1998).

2.6 Design Criteria for Vertical flow Constructed Wetlands

There is a limited amount of information on design of VF systems (Cooper, and Green 1998). VF constructed wetlands tend to be designed on “rule of thumb” criteria.

Pretreatment

Pretreatment prior to the wetland system is necessary to remove coarse and heavy solids. Pretreatment consist of a wide variety of elements. For small domestic sewage volumes usually a simple septic tank or settling tank is used. According to Danish guidelines for small scale CWs for domestic wastewater treatment, pretreatment in a two – or three chamber sedimentation tank (Brix and Arias, 2005) is suggested. For municipal sewage pretreatment mostly comprises of screening and Imhoff tank, when storm water runoff is also to be treated grit chamber is included. Grease filters are used when oils and grease is expected. A septic tank or settling tank properly sized, build and maintained is expected to reduce BOD load up to 20-30% (Cooper *et al*., 1996). This application also helps to minimize the possible problem of media clogging.

Area of Bed

Variation in the size required for treatment was observed in the literature. For example in The Netherlands 3-5m^2/PE is recommended or applying maximum hydraulic loading rates of 4 cm/d (Veensta, 1998). Other countries recommend: 5 m^2/PE in Denmark, 10 m^2/PE in Norway, 1 m^2/PE for BOD removal and 2 m^2/PE for BOD and nitrogen removal in the UK (Cooper and Green, 1998). In US there is no design guideline for VF, however designs tend to follow organic loading rate.

Depth

Depth is normally between 0.5 - 1m**.** In UK most of VF systems have been built 0.5 to 0.8 m deep, where as in Denmark 1 m is used for small scale domestic wastewater treatment (Cooper and Green, 1998).

Beds

Most of the VF systems are designed with layers of graded gravel as the media, topped up with coarse sand to allow even distribution of wastewater over the whole surface area and

then to let it pass. Correct selection of sand is important to prevent clogging. In addition use of large stone in the drainage layer is recommended.

Sealing of bed

Sealing of the bed is necessary in order to prevent seepage and to allow water level control. If local soil is with hydraulic conductivity of 10^{-8} m/s or less then it is likely to have high clay content and may be puddled to seal the bed. Compacted clay, synthetic membrane or concrete and sealed concrete blocks can be used as well. Impermeable plastic liner or membranes as HDPE or LDPE are also used. Recently a number of CWs have been build using liner made from bentonite and a geo-textile such as Fibertex (Cooper and Green, 1998).

Inlet flow distribution

In these systems it is essential to have an even distribution over the whole area. So for this reason some systems have used a series of pipe with holes or gutter to distribute the flow evenly. Another method is to completely flood the bed as part of intermittent dosing cycle (Cooper and Green, 1995).

Outlet pipe collector

The flow is collected by a network of agricultural drainage pipes spread across the full area of the bed, surrounded by large stones (50-200mm) (Cooper and Green, 1998).

Selection of plants

Effects of macrophytes in relation to wastewater treatment process are the physical effects (e.g. erosion control, filtration effect, provision of surface area for attached microorganisms) and metabolism of macrophytes (e.g. plant uptake, oxygen release, etc.) (Vymazal *et al*., 1998). However, the major function of macrophytes is to counteract clogging of the filter bed (Brix and Arias, 2005). The most commonly used plant species worldwide is *Phragmites australis* (common reed). This species has remarkable growth rate, root development, and tolerance to saturated soil conditions. The other commonly used species are *Scirpus spp* and *Typha spp.*

2.7 Cost estimation of Constructed Wetlands

Construction of a new wetland involves careful consideration of a number of criteria, including a realistic look at the expected cost. The amount of funding available, the period of time, and the limits and rules concerning expenditure are to be dealt with early in planning stages of a constructed wetland (Mitsch and Gosselink, 1993). System costs will obviously vary widely according to the site specific issues (Kent, 1994). The major elements to be considered are:

- Planning, engineering and legal issues
- Land acquisition
- Construction (including site preparation, excavation, trenching, placement of soil, planting, drainage and piping, inlet and outlet structures, pretreatment devices, placement of fill, riprap and crushed stone, erosion control measures, mulching and seeding, general landscape, construction supervision and inspection)
- Operation and maintenance (including monitoring, inspection, cleaning, disposal of waste material) and
- Monitoring whether the construction objectives are being met.

Due to these factors it is not possible to give a “rule of thumb” for the estimation of the costs of constructed wetlands for water quality enhancement. In this regard, variation in constructed cost may range from about 1.5 US $\$/m^2$ to over 20 US $\$/m^2$ of land (Hochheimer *et al.*, 1991).

CHAPTER 3. METHODOLOGY

3.1 Study Area

3.1.1 ZIN conference center

In Vught, 51°38’19.20” N, 5°18’22.07” E southern part of The Netherlands, basically an agricultural area, a monastery was converted into a conference building named ZIN in the year 2000. It has a total area of around 10 hectares, of which 7.5 hectares of land is occupied by the garden. The garden needs about 13m^3 of water per day for irrigation (found through oral interview). More than 1 hectare is occupied with different buildings (monk quarter, main conference building etc). In the year 2000 a kidney shaped vertical flow constructed wetland was built to treat the wastewater from the conference building. Building of constructed wetlands for treating its waste water was a better option due to its isolated location (long distance to the main sewer system) and availability of land. In the year 2004 a second VF - CW of same shape and size was built adjacent to the previous one because the conference centre appeared to be a big success and the wastewater volumes had increased. A schematic drawing of the CWs is presented in Figure 2.

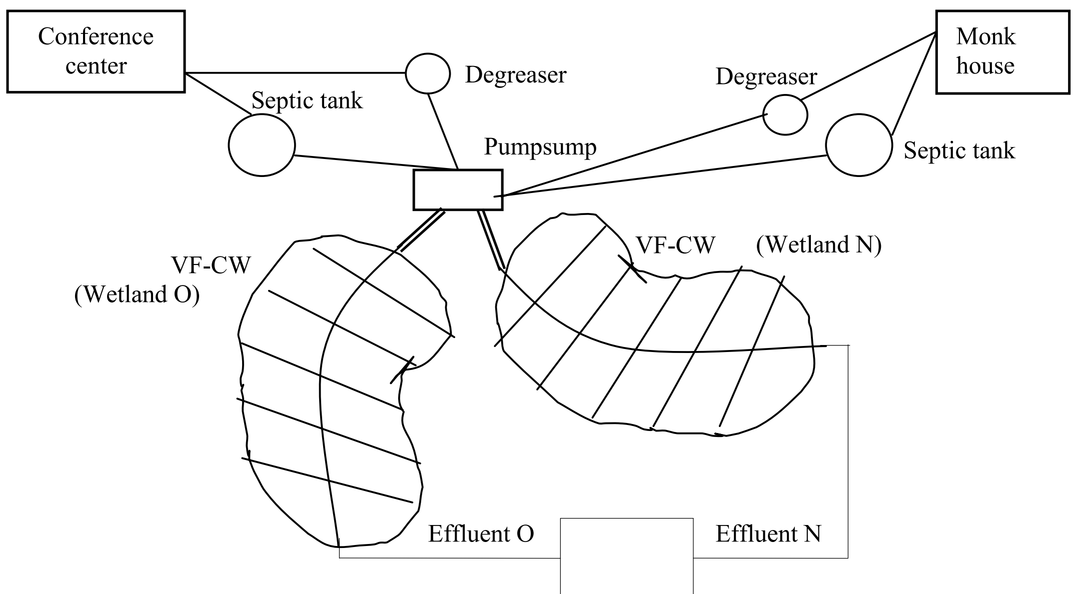


Figure 2: Schematic drawing of CWs at ZIN (O = oldest wetland, N = newer wetland)

Figure 3: (i) CW during construction with distribution network and (ii) during the operation.

Source: Frank van Dien, ECOFYT.

Design parameters of the constructed wetlands are given below. The plant was sized on the basis of 3.5 m^2/PE, with one person estimated to be producing 120-150L wastewater per day.

- Design flow rate = around 3.4m^3 /day
- Bed area 95 m^2
- Bed was planted with *Phragmites australis*

So, the two wetlands in total treat about 6.8 m^3 of wastewater per day. The total construction cost of the wetlands was € 59,873.11 (excluding VAT) and therefore an average investment cost of € 1110 /PE.

From top to bottom, the filter media consist of:

- A depth of 10cm, filled with coarse gravel of 8-32mm ø, containing a network of PVC pipes with detachable cap at end of each branch (to facilitate the maintenance) under the surface with several outlets at an equal distance of 1m permitting an optimum distribution of the inflow over the whole surface.
- 10cm coarse sand (1-2mm ø) with iron grit. Iron grit was incorporated into the media to increase phosphorus adsorption.
- 9 layers of fine sand (0.063-0.500 mm ø), each of 10 cm depth of which the lower 6 layers have straw and limestone additions. Limestone is used for the stabilization of pH and binding of phosphorus. Straw serves as a source of carbon for bacteria when the filter is young. With time plant roots and organic matter provide alternative sources of carbon.
- A drainage layer separated from the substrate with a layer of geotextile, to prevent the flushing of sand and growing of roots in the drainage layer.
- 12 cm gravel (8-16 mm ø) with 2 drains of size 80 mm ø.
- Inner lining was done with polyethylene (1 mm) to avoid ground water exchange, and the infiltration of water.

A schematic drawing of this layered structure in the CW is presented in Figure 4.

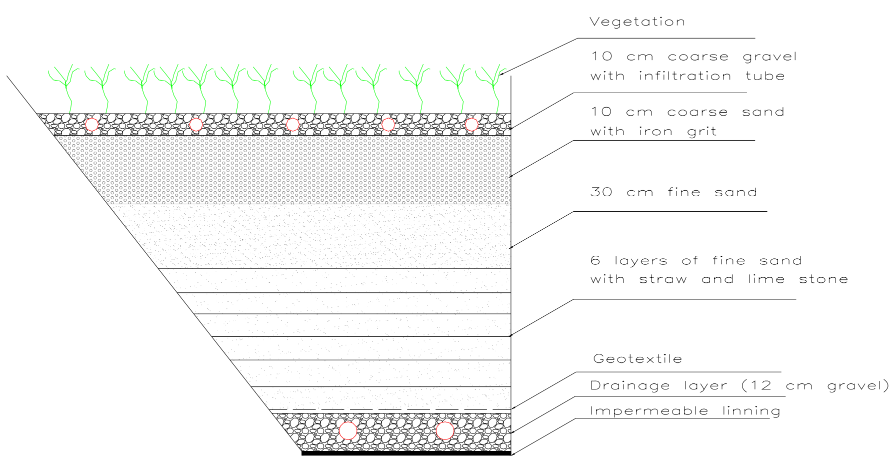


Figure 4: Schematic drawing of the VF CW system at the conference center ZIN.

Grey water and black water are collected in different tanks. Grey water (kitchen, shower, and laundry) is collected in a degreaser (size 2.1 m^3 for the conference centre, 1.7 m^3 for the monks' house) where as black water is collected in 2 septic tanks (size 12 m^3 for the conference centre and 3 m^3 for the monks' house). Wastewater in these tanks is pretreated anaerobically. Pretreated wastewater then flows to the pump sump. From the pump sump

water is pumped to both of the wetlands at a loading rate of 4cm each time, and is done approximately twice a day. In the pump sump there are four pumps which feed wastewater to the two wetlands alternately. When the water level reaches 515 mm water is pumped to the wetland. Pumps are switched off automatically when the level goes down to 145 mm. These levels can be adjusted to change the loading if necessary. A Programmable Logic Controller is used to dose the filter accurately.

3.1.2 Houseboat in city (geWooN)

Building and living in a house boat is not new. In a country like The Netherlands where almost half of the country is below sea level, and with a history of disastrous floods, a trend toward living in a houseboat that adapts to changes in water level could be an innovative solution to catastrophic problems brought by a changing climate and rising sea levels. Dutch visionaries foresee a day when entire cities might float on water. There are more than 10,000 houseboats, mainly located in major cities of the Netherlands, namely Amsterdam, Rotterdam, Leiden and Delft (Veenstra, 1998).

In such a situation to make these houseboat more sustainable and self sufficient, in the year 2006 a houseboat was built in which ECOFYT was involved in building a constructed wetland and a rain water harvesting system. Its major objective was to have wastewater treated through constructed wetland and reuse it as the only source of water (potable and non potable) after passing through Reverse Osmosis (RO).

The houseboat has an area of around 332 m^2 with two wetlands floating alongside of the boat. Wetlands were built in two steel containers, each $6m^2$. Inside the containers 36 cm thick Polystyrene cover was used against the walls. In order to keep the wetlands floating, some light weight aggregates were added inside the wetland. Inside the boat grey and black water are collected in different tanks. Grey water (kitchen, shower, laundry) is collected in a degreaser (0.48 m^3) and black water in a septic tank of 1.14 m^3 (Figure 5). Wastewater in these tanks is treated anaerobically. Pretreated wastewater from both of the tanks overflows to the primary pump sump. From this pump sump wastewater is intermittently pumped to the wetlands, a Programmable Logic Controller (PLC) is used to dose the filter accurately. Treated water is pumped back from the wetland into the boat where it is stored in a tank for further treatment by RO or directly for flushing water for the toilet.

Another tank inside the houseboat is used to collect rainwater from the vegetated roof (PP3). Rain water volumes collected from the roof surface are rather small. Also the presence of vegetation on the roof makes only part of this water available. In very wet periods, the system may get overfilled in that case either rain water or wetland effluent is discharged on the surface water.

Rain water and effluent from CWs provide water for the RO system. RO is a membrane based process. This form of water treatment is effective for removal of dissolved inorganics and organics, bacteria and particulates. RO membranes are however susceptible to fouling if the feed water is not appropriately pretreated for calcium, magnesium, iron and manganese, thus can form scales on RO membranes (Maher, 1994). During this study period, only treated effluent from CW was used for RO system, which had addition of calcium and iron coming through wetland bed (for removal of TP). This addition could be problematic for RO. In order to prevent damage of the filter and have better treated water through the RO, the RO system was extended with a coal filter, a decalcifying filter and a fiber filter. Further to prevent bacteria contamination of the treated water UV treatment system was installed at the end.

Discharge water from RO is fed back into the degreaser, then to the pump sump and to the wetland again. Though the wetlands were designed to treat 0.6 m³ of wastewater per day, the RO system began to flush more water over time so the flow was 1.2 m³ per day (0.6m³ per day per filter).

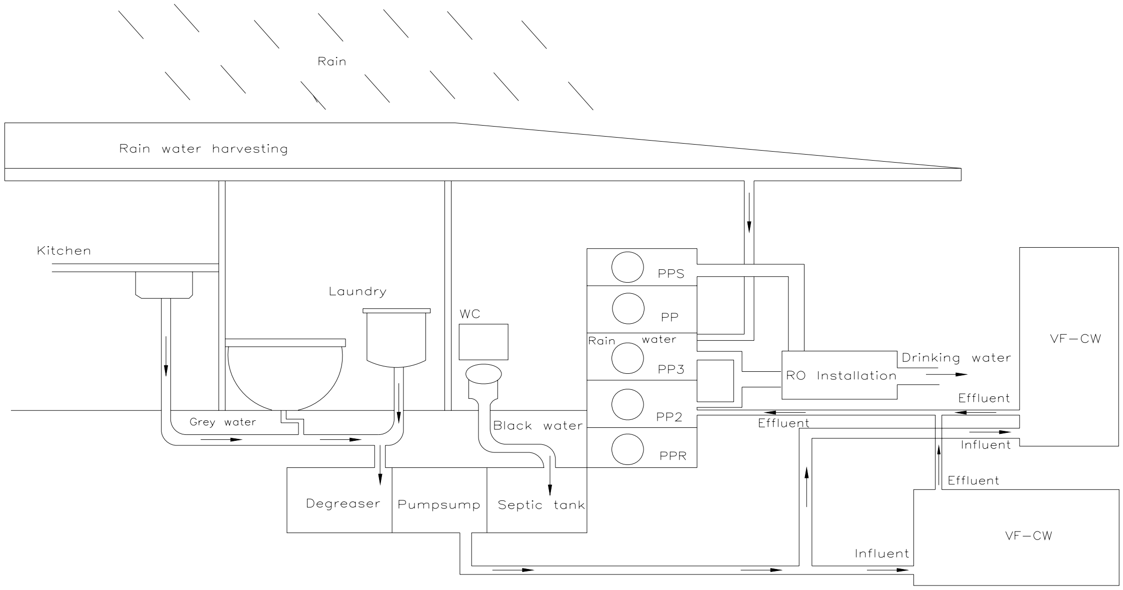


Figure 5: Schematic drawing of water cycle at houseboat with CWs, RO and rainwater harvesting system

Figure 6: Tanks, pumps, and RO system installed in the basement and CWs on either side of houseboat.
Source: Frank van Dien, ECOFYT

Design parameters of the constructed wetlands are as follows: the plant was sized on the basis of 3m^2/PE, with one person producing around 150L wastewater per day.

- Design flow rate = around 0.6m^3 /day,
- Total Bed area = 12 m^2 (3 m^2/PE)
- Both beds were planted with *Phragmites australis, Iris pseudacorus* and several species of bamboo to make it more attractive (*Fargsiar rufa, Pleoblastis hinzi, Fargesia nobusta pinglu*).

The total cost of construction of the wetlands was around €60,000 (excluding VAT) with an investment cost of €15,000 /PE.

In order to make the constructed wetland containers light, volcanic material was used in the upper part of the two steel containers (sizes: 9.20m x 1.50 m x 1.50 m). For stability reasons, the lower part was filled with sand and gravel. From top to bottom, the filter media in the houseboat wetlands consist of:

- A layer of 9 cm of volcanic gravel (8-16 mm ø), constituted by a single infiltration pipe with several outlets at an equal distance of 1m permitting an optimum distribution of the inflow over the whole surface.
- A layer of 43 cm volcanic grain (1.0-2.0 mm ø), with iron grit (10 cm) added on the top for enhanced phosphate removal.
- The rest of the substrate consists of 50 cm fine sand mixed with straw and calcium.
- A drainage layer is separated from the substrate with a layer of geotextile, to prevent the flushing of sand and growing of roots in the drainage layer.
- A layer of 2-7cm gravel (8-16 mm ø) with drainage pipes (32 mm ø).

A schematic drawing of the CW structure is shown in Figure 7.

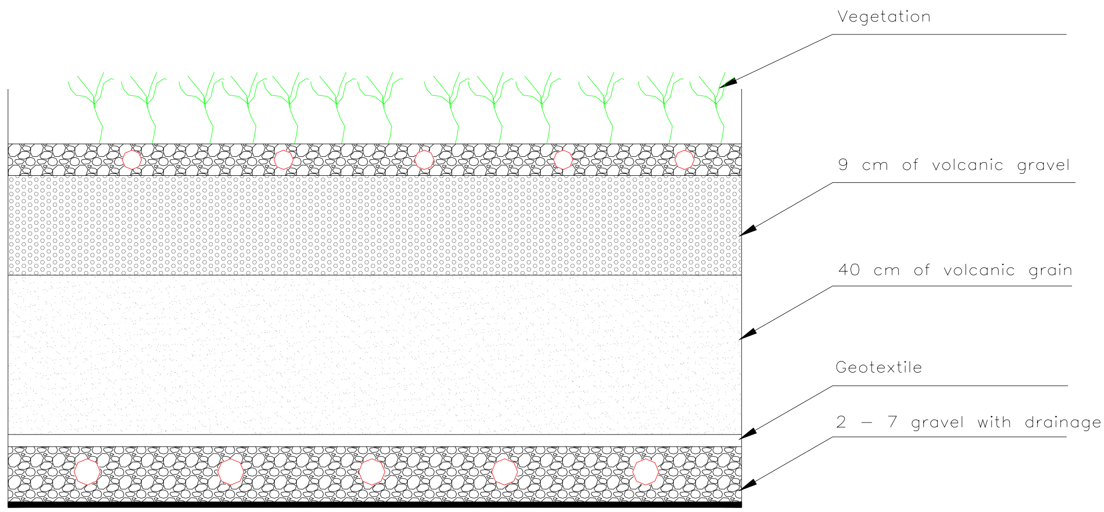


Figure 7: Schematic Drawing of the CW system at the houseboat.

3.2 Meteorological data

The meteorological data (air temperature, precipitation) were gathered from the nearest meteorological stations located at Eindhoven Airport and Twente for CWs at ZIN and Houseboat respectively. The daily average rainfall and calculated evapo-transpiration data were used for the calculation of the water budgets of the constructed wetlands whereas the ambient air temperature data were evaluated to understand the effect of temperature on the efficiency of the wetlands.

3.3 Water Budget Calculation of CWs

The water budget consists of a single state variable Water Volume. Water inputs to the wetland are from wastewater inflows and precipitation. Water loss from the wetland is due to outflow and evapo-transpiration (ET) (Wynn and Liehr, 2001). Since the beds are generally

lined, so it is assumed that there is no ground water exchange, and the infiltration of water is excluded in the water budget calculations. If the constructed wetland receives water through runoff, it should be also considered as an input term in the water budget equation, though this has not been considered for this study, so the water budget can be described as:

$Q_{out}=Q_{in}+P-ET-Q_{stored}$

Where:

Q_{out} = the quantity of treated water exiting the wetland ($m^3.d^{-1}$),

Q_{in} = the quantity of wastewater entering the wetland ($m^3.d^{-1}$),

Qstored = dWaterVolume/dt = the quantity of wastewater stored in the porous parts of the wetland substrate ($m^3.d^{-1}$),

P = the amount of water entering the wetland through precipitation ($m^3.d^{-1}$),

ET = the loss of water through evapo-transpiration ($m^3.d^{-1}$).

The inflow through precipitation can be calculated by multiplying the surface area of the wetland by the precipitation data obtained from the nearest meteorology station.

Evapo-transpiration values can be calculated according to Thornthwaite method, using the equation shown below which takes into account the average ambient air temperature. Moreover, the daily evaporation values can be approximated to the evapo-transpiration values since the evapo-transpiration rate of the reed plants is almost equal to the evaporation rate (IWA, 2000).

Thornthwaite method:

$E = 16 \times \text{day length} \times (10T/I)^a$

Where:

E= monthly potential evapotranspiration (cm)

T= mean monthly temperature (°C)

I= a heat index for a given area which is the sum of 12 monthly index values i

$i = (T/5)^{1.514}$

a= an empirically derived exponent = $6.75*10^{-7} I^3 - 7.71*10^{-5} I^2 + 1.79*10^{-2} I + 0.49$

Additional water flowing in and out of the wetland (precipitation, evapo-transpiration, groundwater infiltration etc.) affects inflow and outflow concentration of the pollutants to be treated other than the fluctuation in the wastewater. Precipitation dilutes the pollution concentration within the wetland so that the measured effluent values are lower then the actual ones. On the other hand, evaporation and evapo-transpiration cause pollutant concentration due to the decrease in water amount so that the measured effluent values are higher than the actual ones (Kadlec *et al.*, 2000). Therefore, in this study, daily average outflow discharge has been calculated by adding the difference between evapo-transpiration and precipitation values, multiplied by area of the CW to daily measured inflow values of each. The correction factors have been calculated by dividing the calculated outflow values by the measured daily inflow values.

3.4 Water Sampling

3.4.1 Conference center ZIN

Water samples of the CWs at the conference center ZIN were collected from the influent and effluents. The influent was collected from the pump sump which was primary treated in the septic tank and degreaser. Effluents of new and old wetlands were collected at Effluent N and Effluent O respectively as shown in Figure 2.

3.4.2 Houseboat (geWooNboat)

Influent and effluent water samples of the CWs were collected from pump sump and effluent tank (PP2) respectively. Rain water was collected from rain water storage tank (PP3), and treated water coming through RO was collected through the tap in the kitchen.

The samples were collected eight times from January to April 2007 at a two-weekly interval. The sampling dates were as follows:

♦ 16th January	♦ 14th February	♦ 14th March	♦ 11th April
♦ 30th January	♦ 28th February	♦ 28th March	♦ 25th April

3.5 Physical-Chemical analysis of wastewater

Analyses of water samples were carried out following Methods described in Environmental Chemistry (Kruis, 2005) unless stated otherwise. A total of 1 liter sample was collected in 3 bottles. The samples were then put in a cool box with ice cubes before being transported to the laboratory. This was to prevent any change which could occur between the time of collection and analysis in the laboratory. A portion of the water was filtered the same day using membrane filters of 0.45 µm pore size. The analyses were carried out in raw (unfiltered) and filtered water as per requirement of the analytic procedure. Parameters which were determined immediately upon arrival in the laboratory included Total coliform, E-coli, BOD_5. The samples were kept in the refrigerator at 6^oC when analysis was to be done the next day of which the cooled samples were allowed to adjust to room temperature before analysis. Procedures for the parameters that were measured in water samples are summarized below.

Physical parameters: temperature, pH, dissolved oxygen, and electrical conductivity were measured in situ using WTW probes and meters. Recorded EC were standardized at 25ºC. The probes were calibrated in the laboratory before each sampling to provide reliable measurements. In addition all supporting information was recorded before leaving the sampling station. This included the time, the weather, harvesting event, or any unusual sights or smells. These observations were recorded so as to assist in interpretation of analytical results.

Ammonia was measured in filtered samples with spectrophotometer using Dichloroisocyanurate Method.

Nitrate, nitrite, orthophosphate, were measured in IC (ion chromatography) after filtering through Whatman filter (AQUA 30/0.45 µ m).

Total phosphate, samples were digested and measured with Ascorbic Acid spectrophotometric Method.

COD was measured according to the closed reflux method.

BOD_5 was measured using DO meter and Winkler bottles. Initial and final DO readings were taken and BOD concentrations were calculated using the formula given below
BOD_5 = (B-S)*Vb/C = mg/L
Where
B = Dissolved oxygen blank (mg/L)
S = Dissolved oxygen sample (mg/L)
Vb = Volume of sample bottle (ml)
C = Volume of sample (ml)

E-coli and Total coliform. Chromocult coliform agar, a combination of two chromogenic substrates, was used for the simultaneous detection of Total Coliform and E.coli. Plates with 3 replicates, with 3 dilutions for each sample were incubated with the sample spread on the surface of plate and incubated for 24 hours at 37^oC. Finally colonies were counted.
E.coli: dark –blue to violet colonies
Total coliforms: Salmon to red colonies and dark blue to violet colonies

3.6 Treatment Performance of Constructed Wetlands

The physico-chemical data obtained from the influent and effluent water samples were presented in a table with mean concentration (Avg.), standard deviation (Std.), minimum (Min.) and maximum (Max.) concentration values. Furthermore a graph was plotted (sampling dates vs. influent and effluent concentration and concentration based removal efficiency (%) wherever possible.

Concentration based removal efficiency was calculated according to the following equation:
RE% = [Cin –Cout] * 100/Cin
Where
Cin = influent concentration (mg/L),
Cout = effluent concentration (mg/L),
RE (%) = Removal Efficiency

Loading rate of pollutants (TP, NO_3-N, NH_4-N, BOD_5, COD) were calculated using the hydraulic loading rate, influent concentration of pollutants with the equation given below. Rregressions were calculated for each pollutant and presented graphically (pollutant loading rate vs. effluent concentration, with linear regression analysis, wherever useful).

Pollutant loading rate ($g/m^2.d$) = (Cin)* (Qin/A)

Area adjusted removal rates for each of the pollutants were calculated according to the following equation:

Area-adjusted Removal Rate ($g/m^2.d$) = [(Qout*Cin) – (Qout*Cout)]/A
= [(Massin – Massout)]/A

These values were plotted against loading rates (pollution loading rates vs. area adjusted removal rates) with linear regression analysis wherever deemed necessary.

Where:

Qin	= water flow rate entering the wetland (m^3/d),
Cin	= inlet concentration (mg/L),
Qout	= flow rate of treated wastewater coming out of the wetland (m^3/d),
Cout	= outlet concentration (mg/L)
A	= surface area of the wetland (m^2)
$Mass_{in}$	= mass of the pollutant in the influent (g/d), and
$Mass_{out}$	= mass of the pollutant in the effluent (g/d).

For the above given calculation an area of 95 m^2 and 12 m^2 was taken into account for ZIN and houseboat CWs respectively and Qin of each sampling day from the meter readings was taken. In order to compare the treatment performance of these CWs with the constructed CWs of other countries, a literature review was done and the performance data of those were presented and compared. Besides the effluent quality was compared with the Dutch discharge standard which has different class of effluent required depending on the land use and sensitivity of the area. For example Class I is the agricultural area which has less strict standards where as Class III b is sensitive area requiring high quality effluents. The effluents from CWs were also compared with the European discharge standards and irrigation water quality standard (where needed). Treated water through RO was compared with WHO and EU drinking water standards.

Oxygen Transfer Rate (OTR)

Oxygen transfer rates were calculated using the formula given below (Cooper, 1999, 2001): OTR (g/m^2.day) = Q [(BOD_5in –BOD_5out)] + 4.3 (NH_4-N in –NH_4-N out)]/total area beds, and was compared with the Oxygen Transfer Rate (OTR) reported in different literatures.

3.7 Statistical Analysis

Data analysis and relevant statistical analysis was carried out using MS Excel. Average values, as well as the standard deviation were calculated for all the parameters. Linear regression analysis was also performed using above mentioned software, wherever it was deemed necessary.

CHAPTER 4. RESULTS AND DISCUSSION

4.1 ZIN conference center

4.1.1 Meteorological Data

The daily average meteorological data (temperature, precipitation) recorded on the sampling days from Eindhoven meteorological station and calculated evapo-transpiration is presented in Table 5. The daily average air temperature, precipitation for the period of January – April 2007 is illustrated in Figure 8.

Table 5: Recorded daily average meteorological data (temperature and precipitation) and calculated evapo-transpiration data from meteorological station at Eindhoven for the sampling days

Sampling Date	Mean Daily Ambient air Temperature (°C)	Mean Daily Precipitation (mm)	Mean Daily Thornthwaite's Evapotranspiration (mm)
16/1/2007	7.6	1.8	0.09
30/1/2007	7.5	0.2	0.09
14/2/2007	6.2	11.0	0.08
28/2/2007	9.6	7.1	0.08
14/3/2007	6.4	0.0	0.08
28/3/2007	11.4	0.0	0.08
11/4/2007	12.5	0.0	0.14
25/4/2007	20.5	0.0	0.14

Source: Meteorological station of Eindhoven;KONINKLIJK NEDERLANDS METEOROLOGISCH INSTITUUT (KNMI)

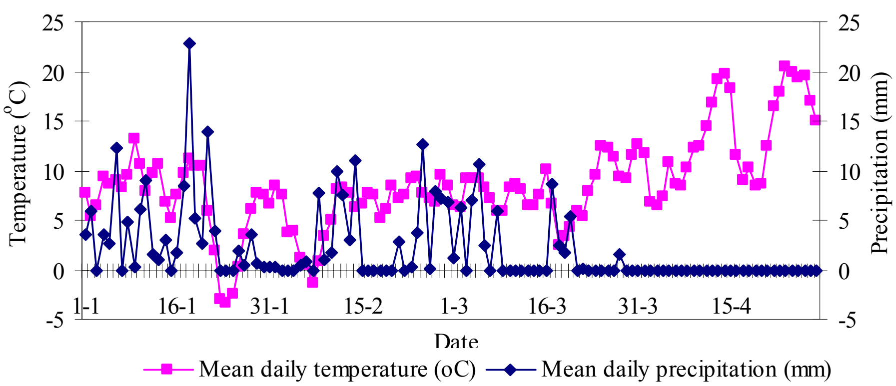


Figure 8: Mean daily temperature and precipitation recorded at Eindhoven meteorological station during the monitoring period

4.1.2 Water Budget Calculation

According to the water budget equation, the outflow discharge of the constructed wetland at ZIN was calculated using the daily average precipitation, evapo-transpiration and inflow data. Both wetlands (new and old) of ZIN have been operated at the same loading rate, there was a slight fluctuation (3%) in the loading rate between the two wetlands which could be due to not flushing of all the branches in the wetlands properly, or extra flushing in case of clogged pipe. However, high variation in loading rate was observed from one sampling day to another, this could be due to the change in number of visitors in the conference center. The inflow discharge, calculated outflow value and correction factor of both of the wetlands are presented in Table 6 below. The correction factors have been calculated by dividing the calculated outflow values by the measured daily inflow values. Unfortunately, there was not

any flow meter to measure the out discharge of the wetlands, the actual value could not be measured and in turn could not be compared with the calculated one.

Table 6: Water budget calculation and correction factor of the CWs at ZIN

Sampling Date	Wetland N Qin (m^3)	Qout (m^3)	Correction factor	Wetland O Qin (m^3)	Qout (m^3)	Correction factor
16/1/2007	3.4391	3.6016	1.0472	3.3394	3.5019	1.0486
30/1/2007	4.7350	4.7345	0.9999	4.8026	4.8021	0.9999
14/2/2007	3.7880	3.8385	1.0133	3.7204	3.7709	1.0136
28/2/2007	2.9086	2.9370	1.0097	2.9086	2.9370	1.0097
14/3/2007	4.2615	4.2610	0.9999	3.9909	3.9904	0.9999
28/3/2007	6.6290	6.6285	0.9999	5.2085	5.2080	0.9999
11/4/2007	2.7734	2.7725	0.9997	2.7734	2.7725	0.9997
25/4/2007	3.6527	3.6519	0.9998	3.4498	3.4489	0.9998

As the wetlands were lined with the water tight layer, it is believed that there was no infiltration and seepage of water from and to the wetlands. The wastewater was equally distributed on the surface area of the wetland via perforated pipes kept at equal distance of 1 meter. The wastewater passed through the entire depth of the wetland substrate in the vertical down flow direction and reached drainage pipe at the bottom.

According to the correction factor of this study, the measured outflow concentrations can vary within a range of -0.024 % to 4.9 % in new and old wetlands. However, as the treatment performances of most of the treatment wetlands have not been presented in the literature considering these factors (Kadlec *et al*., 2000), the outflow concentration measured in the laboratory have been presented without any correction in the further sections. Nevertheless, in the mass balance calculations, the corrected outflow values (Qout) were considered.

4.1.3 Physical and chemical parameters

4.1.3.1 Temperature, pH, Dissolved Oxygen and Conductivity

The physical and chemical characteristics of wastewater from ZIN changed considerably as the wastewater flowed through the wetland system. Table 7 presents the summary of the physical parameters (temperature, pH, dissolved oxygen and conductivity) measured on site of CWs at ZIN.

Table 7 : Physical-chemical parameters of wastewater: mean ± standard deviation, minimum and maximum values at influent and effluents of CW at ZIN

	Temperature (°C)			DO (mg/L)			pH			Conductivity µS/cm		
	Inf.	Effluent N	Effluent O	Inf.	Effluent N	Effluent O	Inf.	Effluent N	Effluent O	Inf.	Effluent N	Effluent O
Avg.	14.8	11.1	11.1	0.7	3.7	6.2	7.3	6.2	6.1	1225	997	913
Std.	1.9	2.2	2.1	0.5	0.6	1.0	0.4	0.1	0.1	84.8	64	112
Min	13.0	9.0	9.0	0.2	1.4	2.7	6.9	6.1	6.0	1117	896	692
Max	19.0	16.0	15.5	1.6	4.8	7.5	8.2	6.4	6.4	1332	1092	1023

Temperature difference was observed between influent and effluent during the study period. The wastewater entering the system had higher temperature as compared to that of effluent. Mean temperature of the influent was 14.8ºC ± 2 where as that of effluents were 11.1 ºC ± 2 at both (new and old) wetlands. The pH at influent ranged from 6.9 to 8.2 with mean 7.3 ±

0.4. After the treatment mean pH decreased to 6.2 ± 0.1 and 6.1 ± 0.1 at the effluent of new and old wetlands respectively. A decrease in pH could have resulted from nitrification.

Dissolved Oxygen (DO) increased radically after the treatment. Mean influent concentration 0.7 mg/L ± 0.5 increased to 3.7 mg/L ± 0.6 and 6.2 mg/L ± 1.0 at the effluent of new and old wetlands respectively. The high concentration of oxygen at the effluents could be due to the fact that the wetlands were fed intermittently (air water interfacial transfer) thus allowing mass oxygen transfer from the atmosphere. It could also be due to the Root Zone Effects (RZE) as the wetlands were vegetated, which transfer oxygen to the root zone area (Yang *et al.*, 2001; Brix, 1986). There are varying reports about the quantity of plant root zone oxygen release. Brix *et al.* (1996) found an oxygen input of 20 mg O_2/m^2.d where as Gries *et al.* (1990) and Armstrong *et al.* (1990) measured oxygen release of 2-12 gO_2/m^2.d. Higher concentration of oxygen at effluent of old wetland could be due to the fact that the wetland root has developed more compared to the new wetland.

However, in a more recent study, Brix (1998) reported that common reed species has a transfer potential of $2gO_2/m^2$.d to the root zone, which mainly is utilized by the root rhizomes themselves.

OTR were calculated for both of the wetlands at ZIN, the calculated OTR average ± Std. deviation data were, 27 ± 9.2 g/m^2.d and 25 ± 6.7 g/m^2.d which ranged between 18-47 g/m^2.d and 18-37 g/m^2.d at new and old wetland respectively. A value of about $30gO_2/m^2$.d has been recommended by a number of workers (Cooper, 2001).

Even though there is no effluent discharge standard for DO, the required DO concentration for irrigation is 5 mg/L (according to EU irrigation standard). So effluent from old wetland is suitable for irrigation in respect to DO concentration where as the treated water from new wetland need some aeration before use.

During the monitoring period, the recorded mean conductivity with standard deviation at influent was 1225 µS/cm ± 95. At effluent the conductivity was reduced to mean 997 µS/cm ± 64 and 913 µS/cm ± 112 of new and old wetlands respectively. Though there is no discharge limit for conductivity, for irrigation (according to FAO) the upper limit is 2000 µS/cm. The effluents are within this limit.

4.1.3.2 Total Suspended Solids

Mean TSS concentration of 754 mg/L (Table 8) found at effluent of primary treatment, an influent to CWs, was much higher than the typical constructed wetland influent from septic tanks that range between 44-54 mg/L (USEPA, 2000). This variation could be due to the difference in the quantity of water use. In the Netherlands, water used per day is around 123.8 L/d.PE (Kanne, 2005), where as in US the amount is at least 2 times higher then the Netherlands, which could have diluted the wastewater.

Table 8: TSS concentrations (mg/L) at influent, effluents and concentration based removal efficiencies (%) of CWs at ZIN

TSS	Influent	Effluent N	Effluent O
Avg. (mg/L)	754	49	50
Std.	405	36	28
Min (mg/L)	328	3	1
Max (mg/L)	1333	108	122
RE (%)		94	93

During the monitoring period, the inlet TSS concentration of the CW varied between 328 and 1333 mg/L. The fluctuation in influent TSS concentration could be related to changes in the sewage characteristics due to the varying water use pattern depending on the number of visitors in the conference building.

As the wastewater flows through the wetland there was a significant drop (more than 90% removal) of suspended solids in both of the wetlands. TSS dropped from 754 mg/l to 49 ± 36 mg/L and 50 ± 28 mg/L in the effluent of new and old wetlands respectively. TSS concentration in the effluent is not only function of the treatment system but also of the weather condition. After heavy rainfall or rapid snow melt, the effluent could be turbid (Borner *et al.*, 1998). Therefore, the high TSS on 14th and 28th Feb. could be due to the high rainfall (Figure 9) leading to flushing of trapped solids from the wetland media, and also due to the high concentration at the influent.

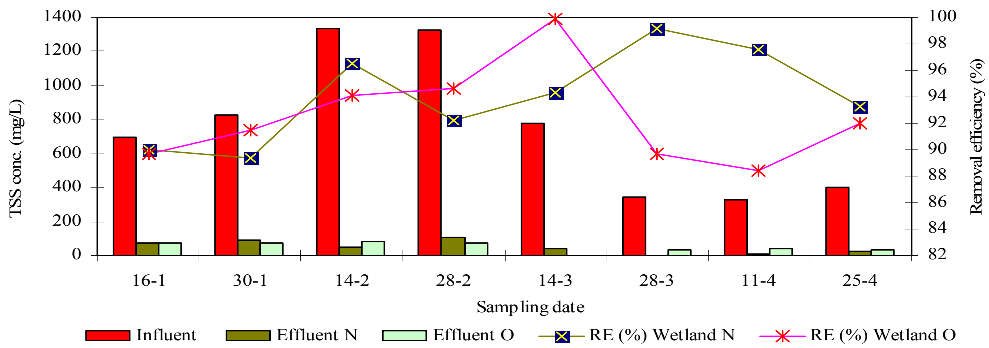


Figure 9: TSS concentration (mg/L) on sampling days at the influent, effluents and concentration based removal efficiencies (%) of CWs at ZIN

The new and old wetlands were 94 and 93 % efficient in removal of TSS. Removal efficiency of new and old wetland did not differ much; therefore it can be stated that the difference in the age (3) years did not lead to significant adverse effect on TSS removal of the wetlands.

During the monitoring period loading rates did not differ much between the two wetlands, however, there was high variation in loading rate over time. TSS loading rate varied between 9.6 and 53 $g/m^2.d$ (32 $g/m^2.d$ ± 13) and 9.6 to 52 $g/m^2.d$ (31 $g/m^2.d$ ± 14) in new and old wetlands respectively. No correlation was observed between loading and removal rate. However there was a relation between loading rate and effluent concentration (Figure 10).

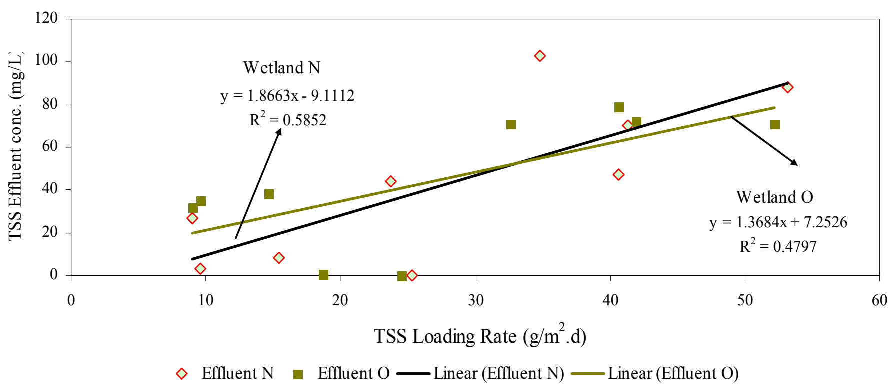


Figure 10: TSS effluent concentration (mg/L) vs. loading rate ($g/m^2.d$) of CW at ZIN.

Though the conference building of ZIN falls under class I, which has the least strict discharge standards, the wetlands were designed to meet the effluent quality of Class III b, for the sensitive area with discharge limit of <60 mg/L TSS. The average effluent concentrations were below this limit in both of the wetlands. However, the effluent concentrations did not meet the European discharge standard (30 mg/L). In order to comply with this standard the TSS has to be reduced at primary treatment level.

4.1.3.3 Nitrogen

Ammonia nitrogen

Ammonium nitrogen is the most common form of nitrogen in wastewater, due to its anaerobic nature. The pretreated wastewater pumped at ZIN CWs had average NH_4-N concentration of 87 ± 14 mg/L. This is higher than the typical constructed wetland influent from septic tanks, which ranged between 28-42 mg/L (USEPA, 2000). As wastewater was percolated through the system there was significant reduction in NH_4-N concentration with mean concentration less than 1 mg/l, at both the effluents with a reduction efficiency of around 99% (Table 9 and Figure 11). NH_4-N was probably converted by microbial nitrification and by plant uptake processes since temperature and pH was within the range that could support these processes. Ammonia volatilization could not have occurred as the pH in the system never rose above 9 (Vymazal, 2001).

Table 9: NH_4 - N concentration (mg/L) at influent, effluents and concentration based removal efficiencies (%) of the CWs at ZIN

NH_4^+- N	Influent	Effluent N	Effluent O
Avg. (mg/L)	87	0.61	0.85
Std.	14	0.7	0.8
Min (mg/L)	67	0.01	0.12
Max (mg/L)	106	2.2	2.2
RE (%)		99.3	99.0

In general, with sufficient oxygen supply in an intermittently loaded VF bed, more than 90% reduction of NH_4-N occurs (von Felde and Kunst, 1997). Moreover, in both of the systems, vegetation might have slightly increased nitrification through oxygenation of the substrate through RZE (Yang *et al.*, 2001). Establishment of a healthy root system is supposed to facilitate a rich and productive community of attached microorganisms by providing higher surface area.

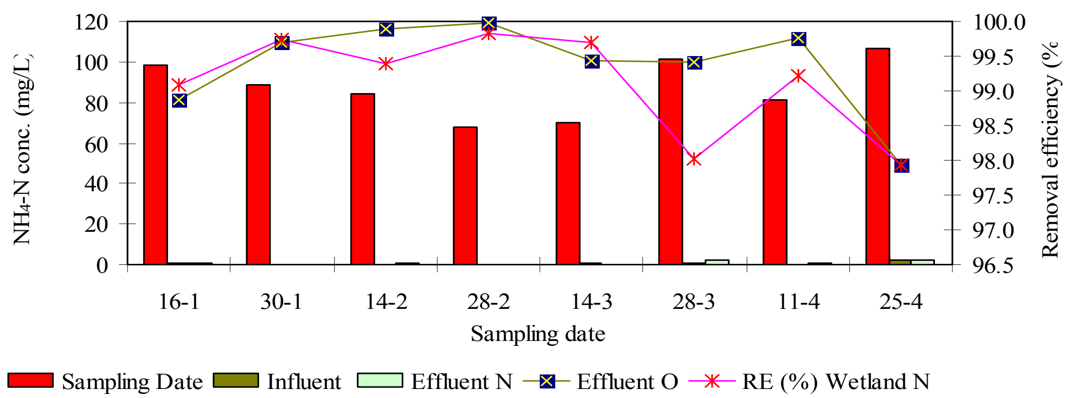


Figure 11: NH_4-N concentration (mg/L) on sampling days at the influent, effluents and concentration based removal efficiencies (%) of CW at ZIN

Loading rate of NH_4-N for the wetlands new and old changed between 2.1 to 7 g/m^{2}d (3.8 g/m^2.d ± 1.5) and 2.1 to 5.5 g/m^2.d (3.5 g/m^{2}d ± 1.1) respectively. The effluent concentrations were not linearly dependent on the NH_4-N loading rate for both of the wetlands. Nevertheless, there was a strong correlation (Figure 12) between loading and removal rate in both wetlands, R^2=0.9997 and R^2 = 0.9496 at new and old wetland respectively. With increasing loading rate the removal efficiency increased. Similar results were also reported by Korkusuz (2004) in VF constructed wetlands studied in Turkey.

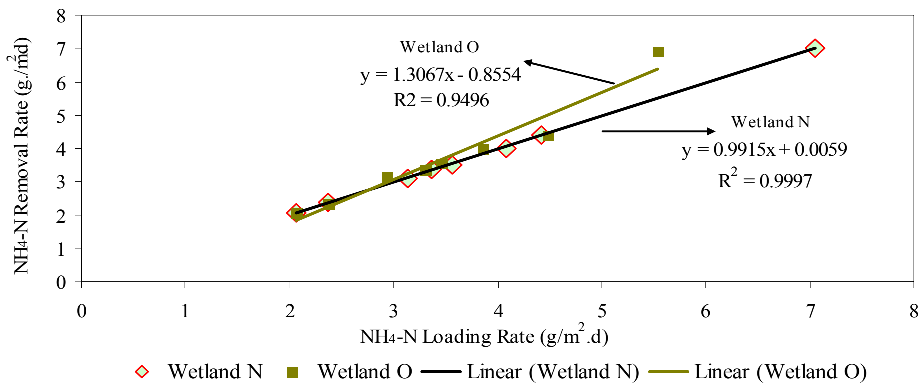


Figure 12: NH_4 - N loading rate (g/m^2.d) vs. removal rate (g/m^2.d) at CW of ZIN

Though there is no Dutch standard for NH_4-N discharge concentration for Class I, it is well known that ammonia exerts a significant amount of oxygen demand through nitrification in the receiving water body. Moreover, unionized ammonia can be toxic to aquatic organisms (Lee and Lin, 1999). Therefore it is necessary to reduce the effluent NH_4-N concentration from environmental point of view. The Dutch standard for NH_4-N discharge is <4 mg/L for class IIIa and IIIb. As the results indicated that effluent water quality from both of the wetlands were far below the required effluent standards with mean effluent NH_4-N concentration of 0.61 and 0.85 mg/L at new and old wetland respectively, it can be concluded that CWs at ZIN for removal of ammonia was a promising solution.

Nitrate - Nitrogen

The mean influent NO_3-N concentration of primary treated wastewater was 0.4 ± 0.2 mg/L and ranged between 0.04 to 0.56 mg/L. As the wastewater was passed through the system, as a result of nitrification of NH_4-N, NO_3-N mean concentration was increased to 71 mg/L ± 12 and 63 mg/L ± 15 at effluents of new and old wetlands respectively (Table 10 and Figure 13).

Table 10: NO_3^- - N concentration (mg/L) at influent, effluents and concentration based removal efficiencies (%) of CW at ZIN

NO_3^-- N	Influent	Effluent N	Effluent O
Avg. (mg/L)	0.4	71.3	63.5
Std.	0.2	11.8	15.0
Min (mg/L)	0.04	51.2	35.9
Max (mg/L)	0.56	85.4	82.5

Typical organic loads between 8-12 gBOD$_5$/m^2.d are recommended to treat secondary domestic wastewater under temperate conditions, and this is generally associated with a TSS loading of 1-10 g TSS/m^2.d (Kadlec and Knight, 1996). At high loading rates, only suspended solids and carbon removal can be obtained, whereas at low loads also nitrification can take place (IWA, 2000). However, in spite of high TSS and COD concentrations, nitrification of ammonia was found to be high which might be due to the fact that the wetlands had high

oxygen transfer rate, as the mean effluent oxygen concentrations reached around 4 and 6 mg/L at the effluent of the two wetlands (new and old) from 0.4 mg/l at influent. Also the pH was within the optimum range for nitrification.

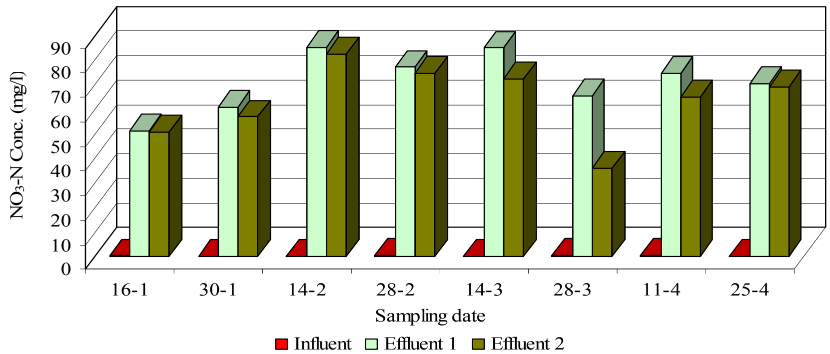


Figure 13: NO_3 - N concentrations (mg/L) on sampling days at influent, effluents of CW at ZIN

Due to the nature of nitrification process, a relation between NH_4-N influent and NO_3-N effluent is likely. Thus effluent NO_3-N concentration was plotted against influent NH_4-N concentration. However, NO_3-N effluent concentrations were not dependent on the influent NH_4-N concentration as expected. NO_3-N production and NH_4-N loading rate indicated a linear dependency (R^2= 0.4939) in new wetland (Figure 15). With increasing NH_4-N loading rate effluent concentration of NO_3-N decreased (R^2=0.5383) in the old wetland (Figure 14). These differences between the two wetlands indicate a varying rate of denitrification. The exact logic behind these results could not be explained, due to lack of TN concentration data.

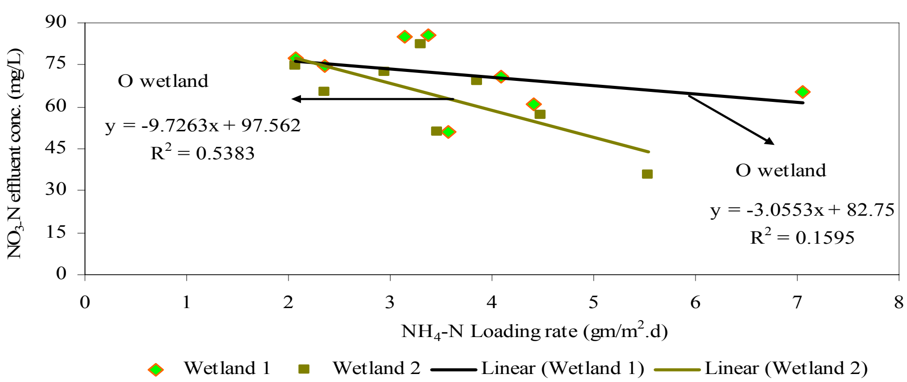


Figure 14: NH_4- N loading rate (g/m^2 d) vs. NO_3 - N effluent concentration (mg/L) of CW at ZIN

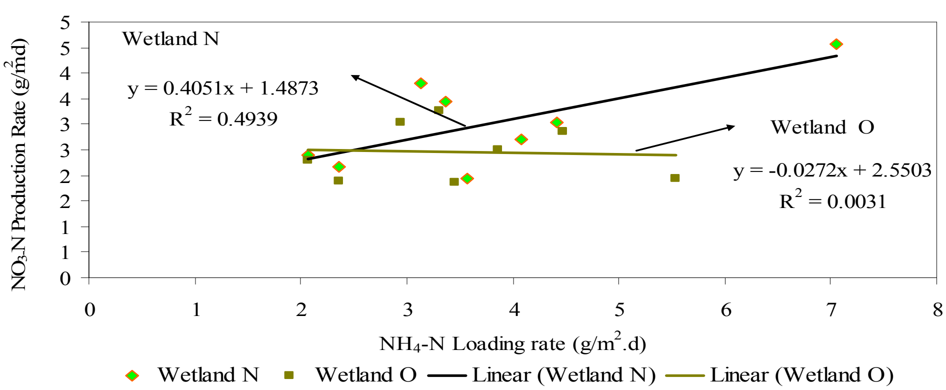


Figure 15: NH_4- N loading rate (g/m^2 d) vs. NO_3-N production rate (g/m^2 d) of CW at ZIN

There is no discharge standard for NO_3-N. However, according to the Dutch discharge standard for class IIIa and IIIb, TN concentration at effluent should be less than 60 mg/L. Nitrate concentration alone exceed this level at both of the effluents during the monitoring period. Indeed it could neither meet the more restricted European discharge limit (15mg/L for TN). Addition of a horizontal flow bed after the VF bed could benefit the removal of nitrogen by denitrification (Cooper, 2001). This will also achieve additional benefit of removing more TSS from an already oxidized effluent (low BOD/low NH_4-N effluent). This is also seen in the study by Mitterer-Reichmann (2002). The more economic option could be recycling of the nitrified water back to the feed tank where it is presumed that nitrate could be removed by biological denitrification using the carbon source present in the wastewater (Laber, 1997).

4.1.3.4 Total Phosphorus

Mean influent TP concentration of wastewater after the primary treatment in septic tank was 9.6 mg/l. The observed TP was lower than the typical constructed wetland influent from septic tanks, which ranges between 12-14 mg/L (USEPA, 2000). The mean influent concentration varied between 8 and 12 mg/L. This variation could be due to the change in the water and detergent use. After the treatment, mean effluent concentrations were 7.4 mg/L ± 1.5 and 6.6 mg/L ± 0.9 TP with removal efficiency of only 23% and 31% at effluent of new and old wetland respectively (Table 11 and Figure 16). The removal percentage at the old wetland build in 2000 was a bit higher than the new wetland build in 2004. It appears that the precipitation by iron grit and calcium in the wetlands was not very effective in removal of TP. This could be due to the fact that the pH 7 of wastewater was neither low enough to precipitate with iron nor high enough to precipitate with calcium. Lack of understanding of the most efficient pathways and also inability to predict desorption of P (Richardson *et al.*, 1996) makes it difficult to maximize long term high efficiency P retentions.

Table 11: TP concentrations (mg/L) at influent, effluents and concentration based removal efficiencies (%) of CWs at ZIN

TP	Influent	Effluent N	Effluent O
Avg. (mg/L)	9.6	7.4	6.6
Std.	1.4	1.5	0.9
Min (mg/L)	8.2	5.2	5.1
Max (mg/L)	12.1	9.0	7.9
RE (%)		23	31

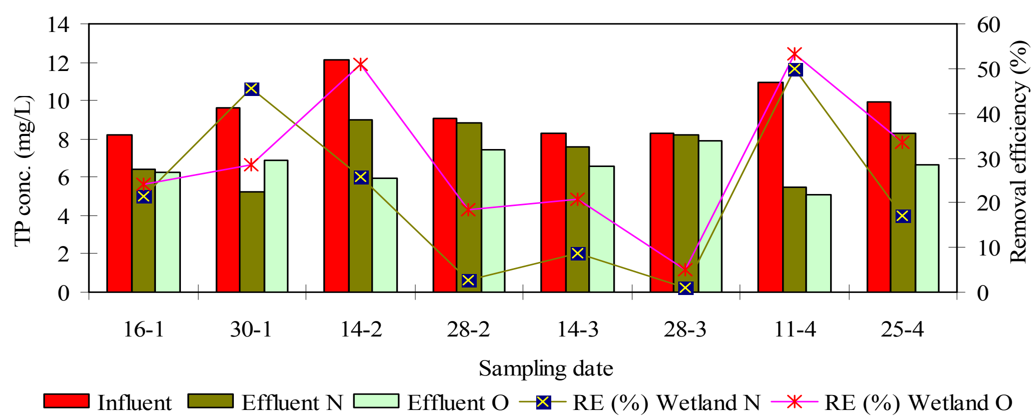


Figure 16: TP concentration (mg/L) on sampling days at influent, effluents and concentration based removal efficiencies (%) of CWs at ZIN

The average loading rates ± Std. for new and old wetland during the monitoring period were 0.4 ± 0.11 $g/m^2.d$ and 0.38 g/m^2 ± 0.08 respectively. The effluent concentration and removal rate of TP were independent of TP loading rate in both of the wetlands. Similar independency was also found by Brix *et al*. (1998) in CWs in Denmark.

Since the wetland lies in an agricultural area, it needs Class I discharge standard. According to the Dutch regulations there is no standard set for the discharge of TP for this class. However, it is well known that the presence of trace amount of phosphorus (even less than one mg/L) in the treated wastewater from municipalities and industries is often responsible for eutrophication which leads to short and long term environmental and aesthetic hazard in the receiving water body. The Dutch discharge standard for TP in effluent water is 6 mg/L for class IIIb whereas the European discharge standard is 2 mg/L. The effluent from ZIN did not meet these limits.

4.1.3.5 COD

The typical constructed wetland influent from septic tanks ranges between 310-344 mg/L (USEPA, 2000). The observed concentration of COD of 417mg/L at influent is higher than these values. This high concentration could be due to difference in quantity of water use. The inlet varied between 296 and 516 mg/L. As the wastewater percolated through the wetland there was considerable reduction of COD concentration. The COD dropped from 417 mg/l to 49 mg/L ± 13 and 51 mg/L ± 16 at effluents of new and old wetlands respectively, with a removal efficiency of 88% at both wetlands (Table 12). The effluent concentrations were affected by the influent COD concentration as well as by precipitation and evaporation. Rain affects the concentration in two ways. For example 28th Feb. (Figure 17) effluent concentrations at new and old wetlands were highest. This could be due to high influent concentration and as well due to the rain. Indeed rain might have flushed COD out of the bed increasing its concentration up to 122mg/L and 152 mg/L at effluents of two wetlands (new and old). Whereas on 14th Feb, the concentration at effluent was low despite the rain which could be due to the dilution effect of the rain and lower influent concentration.

Table 12: COD concentration (mg/L) at influent, effluents and concentration based removal efficiencies of the CWs at ZIN

TP	Influent	Effluent N	Effluent O
Average (mg/L)	417	49	51
Std.	66	36	45
Min (mg/L)	296	16	10
Max (mg/L)	516	122	152
RE(%)		88	89

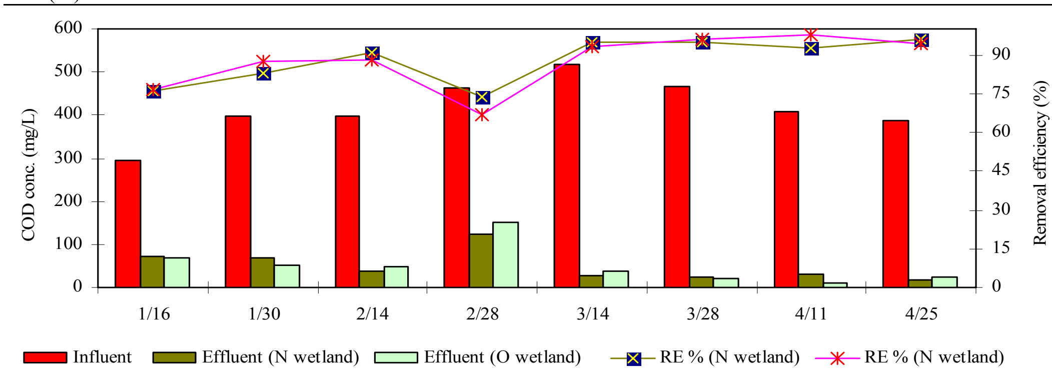


Figure 17: COD concentrations (mg/L) on sampling days at influent, effluents and concentration based removal efficiencies (%) of CWs at ZIN

The mean loading rate of the two wetlands did not vary much. The difference was 1.2 g/m^2.d. However, there was a variation of loading rate over time during the monitoring period. Mean loading rate ± Std. [min; max] for new wetland and old wetland were 18 g/m^{2}d ± 7 [11 to 33] g/m^{2}d and 17 g/m^2.d ±5.2 [10 to 26] g/m^2.d respectively.

An average COD: BOD_5 ratio of the influent wastewater of 1.5:1 was found. It can be concluded that the wastewater had a high level of biodegradability and could be classified as "high strength" wastewater. This could be due to the fact that wastewater is from a conference building with a hotel in it, where high amounts of kitchen waste are also generated which is biodegradable.

No relation was found between the loading rate and the effluent concentration, however a strong relationship between loading rate and removal rate for both of the wetlands (new and old) with R^2= 0.975 and R^2= 0.896 was found (Figure 18).

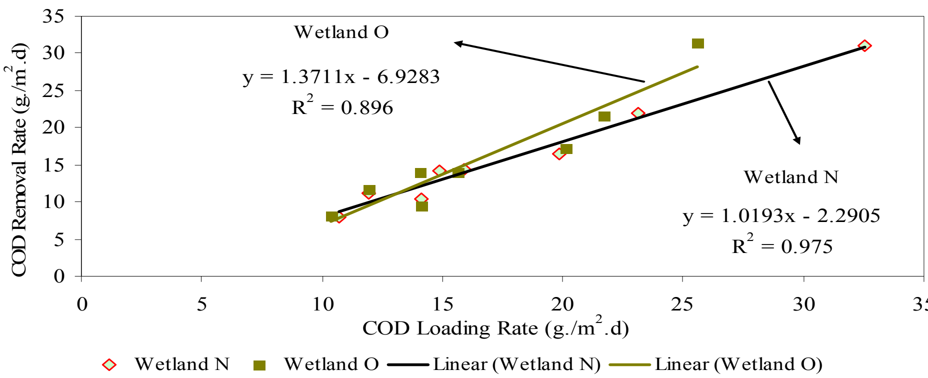


Figure 18: COD loading rate (g/m^{2}·d) vs. removal rate (g/m^{2}·d) of CW at ZIN

Throughout the monitoring period, effluent COD concentrations of both wetlands were below the European discharge limit (125 mg/L) and also complied with the class IIIb Dutch effluent standard which is <200 mg/L.

4.1.3.6 BOD_5

Typical constructed wetland influent from septic tanks ranges between 129-147 mg/L (USEPA, 2000). The observed concentration of BOD_5 of 279mg/L at influent was higher than the range value. As the wastewater flows through the wetland there was reduction (more than 95 % removal) of BOD_5 in both wetlands. The mean concentration of BOD_5 of 279 mg/l at influent drops to 13 mg/L ± 11 and 11 mg/L ± 14 at effluents of new and old wetlands respectively (Table 13 and Figure 19). Furthermore, BOD_5 removal percentage of new and old wetland did not vary much. The difference was less than 1 %. Therefore, it could be stated that the difference in age (3 years) did not lead to adverse effects on BOD_5 removal of the wetlands which received the settled wastewater with relatively high BOD_5 concentration. The high BOD_5 concentration at the effluents at the beginning of the monitoring period could be due to the low air and water temperature. BOD_5 concentration at effluents were generally lower with the increase in temperature and the removal percentage reached more than 98% in April, when the temperature was highest.

Table 13: BOD_5 concentrations (mg/L) at influent, effluents and concentration based removal efficiencies (%) of the constructed wetlands at ZIN

BOD_5	Influent	Effluent N	Effluent O
Avg. (mg/L)	279	13	11
Std.	38.8	11.4	13.5
Min (mg/L)	218.5	3.6	1.7
Max (mg/L)	334.9	34.7	36.3
RE (%)		95.2	96.0

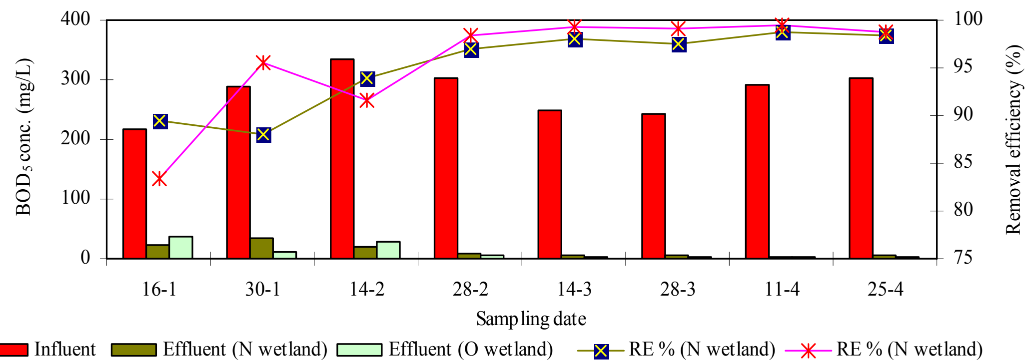


Figure 19: BOD_5 concentrations (mg/L) on sampling days at influent, effluents and concentration based removal efficiencies (%) of CWs at ZIN

The calculated average loading rate ± Std. dev [min; max] data for BOD at influents of new and old wetlands were 11.6 g/m^2.d ± 3.3 [7.9; 16.9] and 11.0 g/m^2.d ± 2.7 [7.7; 14.6] respectively. There was a strong correlation between removal rate and loading rate: at high loading rate the removal efficiency increased (Figure 20). The relation was strong in the new wetland compared to old wetland, with R^2=0.9701 and R^2=0.7998. No correlation was observed between the effluent concentration and loading rate for both of the wetlands.

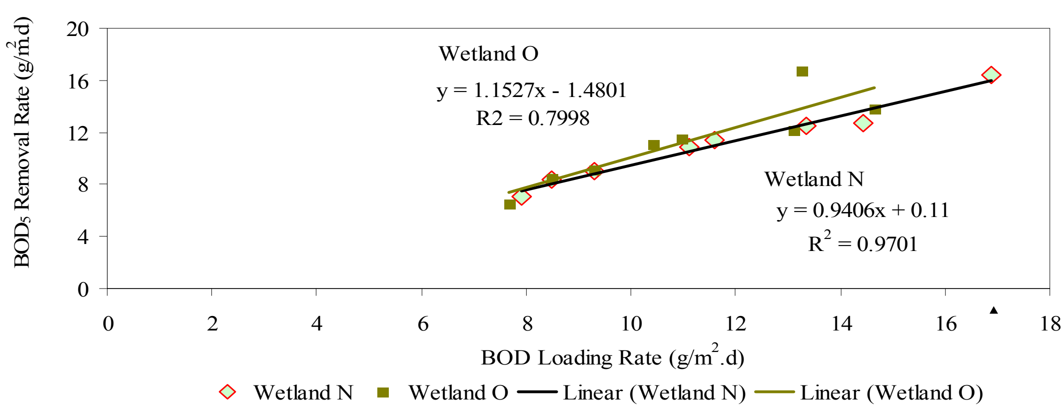


Figure 20: BOD_5 loading rate (g/m^2·d) vs. removal rate (g/m^2·d) of CWs at ZIN

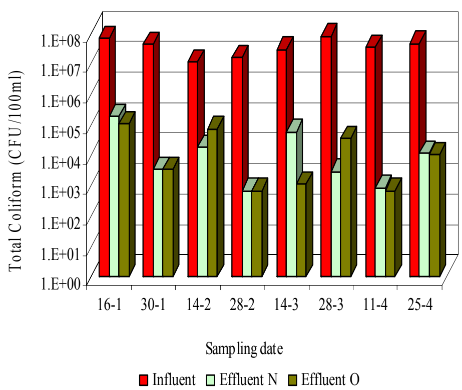
Throughout the monitoring period, effluent BOD_5 concentrations of both wetlands were below the discharge limits of class IIIb (<40 mg/L) and also below the European discharge standard (<20 mg/L).

4.1.3.7 Total coliform and E.coli

The mean concentrations of total coliform and E.coli at the effluent of pretreated wastewater/influent to the VF wetland system were 4E+07/100mL and 1E+06/100mL respectively. As the wastewater flows through the wetland there was a significant drop. Total coliform concentration 3.47E+04/100mL and 2.78E+04/100mL at effluents of new and old wetlands respectively were found. The mean effluent E.coli concentration at effluent of new and old wetlands were 376/100mL and 667 /100mL with reduction of about 3.5 and 3.2 log units at new and old wetlands respectively (Table 14 and Figure 21).

Table 14: Total Coliform and E.coli (CFU/100mL) at influent, effluents and removal efficiencies (log) of the constructed wetlands at ZIN

Pathogens	Total Coliform			E.coli		
	Influent	Effluent N	Effluent O	Influent	Effluent N	Effluent O
Avg. (CFU/100mL)	3.93E+07	3.47E+04	2.78E+04	1.12E+06	3.76E+02	6.67E+02
Std.	2.26E+07	6.35E+04	3.89E+04	6.13E+05	8.25E+02	1.75E+03
Min . (CFU/100mL)	1.10E+07	6.67E+02	6.67E+02	3.33E+05	0.00E+00	0.00E+00
Max . (CFU/100mL)	7.55E+07	1.85E+05	1.03E+05	2.00E+06	2.33E+03	5.00E+03
Log reduction		3.1	3.2		3.5	3.2



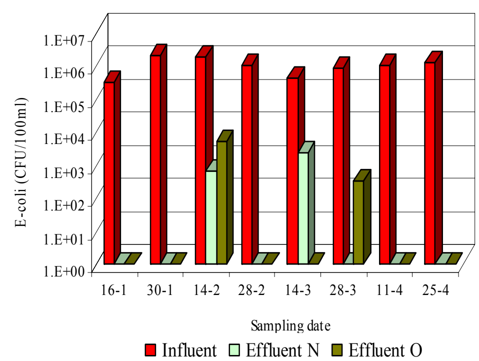


Figure 21: Total Coliform and E.coli concentration (CFU/100mL) on sampling days at influent and effluents of CW at ZIN.

Finally the effluent quality at both of the wetlands (new and old) could meet the WHO effluent guidelines for safe us in agriculture or aquaculture (<3 log units/100mL E.coli) as well as the Dutch guidelines for direct discharge into recreational sensitive surface water (<3 log units/100 ml E.coli). Thus it can be concluded that the CW at ZIN is a promising solution for the treatment of bacteria.

However, total coliform were still high to meet any standard. According to WHO Total coliform or Faecal coliform for irrigation of crops likely to be eaten uncooked, sports field, public park need to have <1000/100 ml total coliform, which the effluent from ZIN could not meet.

4.2 Houseboat in city of Zwolle

4.2.1 Meteorological Data

The daily average meteorological data (temperature, precipitation) recorded on the sampling days from the Twente meteorological station and calculated evapo-transpiration data are presented below in Table 15. Daily average air temperature, precipitation for the period of January – April 2007 is illustrated in Figure 22.

Table 15: Recorded daily average meteorological data (temperature and precipitation) and calculated evapo-transpiration data from meteorological station at Twente for the sampling days

Sampling Date	Mean Daily Ambient air Temperature (°C)	Mean Daily Precipitation (mm)	Mean Daily Thornthwaite's Evapotranspiration (mm)
30/1/2007	7.5	0.0	0.076
14/2/2007	5.6	8.5	0.070
28/2/2007	9.5	4.8	0.070
14/3/2007	5.1	0.0	0.070
28/3/2007	11.4	0.0	0.070
11/4/2007	11.9	0.0	0.132
25/4/2007	19.8	0.0	0.132

Source: Meteorological station of Twente; KONINKLIJK NEDERLANDS METEOROLOGISCH INSTITUUT (KNMI)

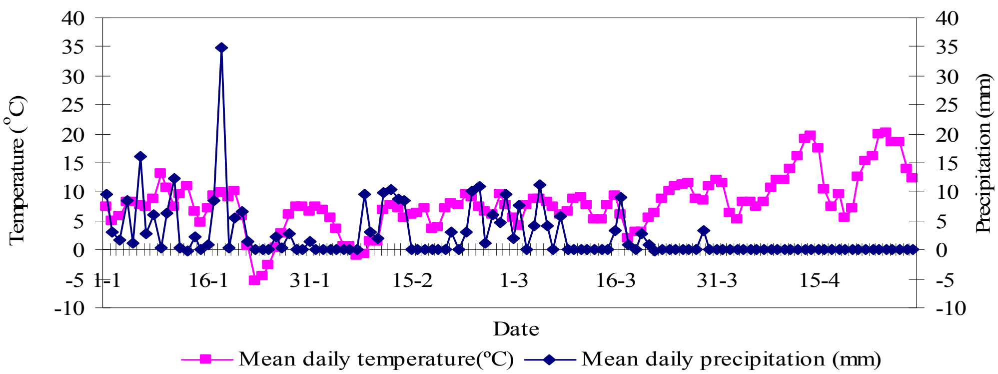


Figure 22: Mean daily temperature and precipitation recorded at Twente meteorological station during the monitoring period

4.2.2 Water Budget Calculation

Though the wetlands were designed to treat 0.6m^3/day, there was a fluctuation in amount of wastewater generated, which reached as high as 1.3 m^3/day due to addition of wastewater from the RO. The inflow discharge and calculated outflow values of the wetlands along with the correction factors are presented in Table 16. Correction factors have been calculated by dividing the calculated outflow values by the measured daily inflow values. Unfortunately, there was not any flow meter to measure the out discharge of the wetland, the actual values could not be measured and in turn could not be compared with the calculated ones.

As the wetlands were built in a steel container lined with a water tight layer, it is believed that there was no leakage of wastewater from wetlands to the environment. The wastewater was equally distributed on the surface area of the wetland via pipes with perforation kept at equal distance. The wastewater passed through the entire depth of the wetland substrate in the vertical down flow direction and reached the drainage pipe at the bottom.

Table 16: Water budget calculation and correction factor of the CW at houseboat

Sampling Date	Qin (m^3)	Qout (m^3)	Correction factor
30/1/2007	0.3420	0.3411	0.9973
14/2/2007	1.0470	1.1486	1.0970
28/2/2007	1.0020	1.0592	1.0571
14/3/2007	1.0500	1.0496	0.9996
28/3/2007	1.0630	1.0626	0.9996
11/4/2007	1.3070	1.3062	0.9994
25/4/2007	1.1970	1.1962	0.9993

According to the correction factors of this study, the measured outflow concentration can vary within a range of -0.27 % to 9.7 %. However, the outflow concentration measured in the laboratory has been presented without any correction in the further sections. Nevertheless, in the mass balance calculations, the corrected outflow values (Qout) were considered.

4.2.3 Physical and chemical parameters

4.2.3.1 Temperature, pH, Dissolved Oxygen and Conductivity

The physical and chemical characteristics of wastewater from the houseboat changed considerably as the wastewater flowed through the wetland, and RO system. Summary data of the physical parameters (temperature, pH, dissolved oxygen and conductivity) measured on site on the sampling dates, including the rain water are presented in Table 17.

Table 17: Physical-chemical parameters of water: mean ± standard deviation, minimum and maximum values from CW, RO and rain water at Houseboat.

	Temperature °C				pH				DO mg/L				Conductivity μs/cm			
	Inf.	Eff.	RO	Rain	Inf.	Eff.	RO	Rain	Inf.	Eff.	RO	Rain	Inf.	Eff.	RO	Rain
Avg.	16.1	13.7	19.8	14.3	7.4	6.8	9.1	7.7	0.6	2.6	6.1	7.3	1929	1779	436	169
Std.	3.6	3.2	2.8	2.5	0.3	0.1	0.2	0.3	0.3	0.5	0.6	0.5	352	345	79	28
Min	11.7	9.7	14.5	11.0	7.1	6.7	8..9	7.4	0.2	2.1	5.2	6.5	1373	1265	344	140
Max	21.0	18.7	22.1	16.8	8.0	7.0	9..5	7.9	1.0	3.5	6.7	7.7	2420	2230	581	210

The wastewater entering the wetland had a higher mean temperature of 16.1ºC ± 3.6 compared to the effluent temperature of 13.7 ºC ± 3.2. Temperature of RO was the highest with mean 19.8º C ± 2.8, where as the mean temperature of rain water was 14.3ºC ± 2.5.

Wetland water chemistry and biology are affected by pH. Influent pH of pretreated wastewater ranged from 7.1 to 8.0. Mean effluent pH dropped to 6.8 ± 0.1 after the treatment. Treated water from RO had very high pH with a mean value of 9.1 ± 0.2 and ranging from 8.9 to 9.5. During the monitoring period the pH even reached the higher limit of the EU's drinking water guideline which is ≤ 9.5. Drinking water with a pH level above 8.5 could indicate that the water is hard. Hardness does not pose a health risk, but can cause aesthetic problems, such as an alkaline taste to the water that makes coffee taste bitter; build-up of scale on pipes and fixtures than can lead to lower water pressure; build-up of deposits on dishes, utensils and laundry basins; difficulty in getting soap and detergent to foam; and lowered efficiency of electric water heaters. Addition of calcium to the wetland bed for

removing TP might have increased the water hardness. Recirculation of the same water time after time probably resulted in build up of hardness in the water.

Normally rainwater is slightly acidic because it is exposed to the carbon dioxide in the atmosphere. However the rain water collected at houseboat had a mean pH of 7.7 ± 0.3 which can be explained by the presence of the green roof.

Pretreated wastewater influent to the CW of the house boat had low DO, which is due to the anaerobic treatment. Its mean DO was 0.6 mg/L ± 0.3 and ranged between 0.2 to 1.0 mg/L. After the treatment of wastewater through intermittently fed VF constructed wetlands, the mean DO increased to 2.6 mg/L ± 0.5.

DO in drinking water is important as a high DO level makes drinking water taste better. There was a significant rise in DO concentration after treatment through RO with mean DO concentration of 6.1 mg/L ± 0.6. However, rain water had the highest DO concentration which reached as high at 7.7 mg/L during the monitoring period with an average concentration of 7.3 mg/L ± 0.5.

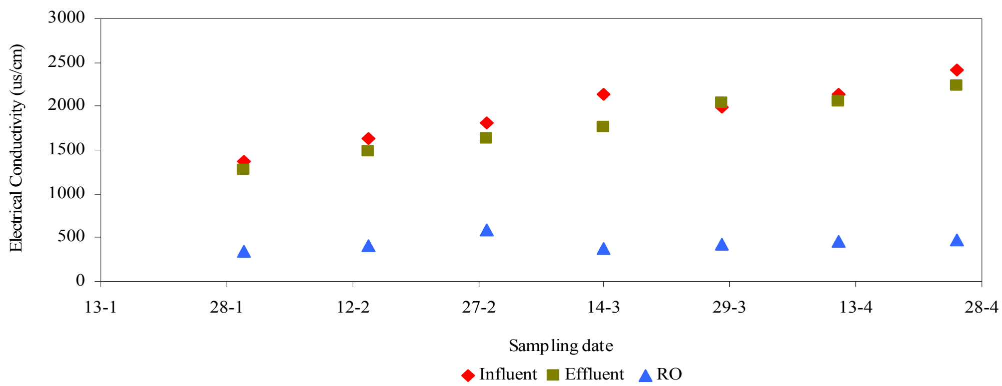


Figure 23: Conductivity (μc/cm) measured on sampling days at influent, effluent of CW and treated water through RO system.

Average electrical conductivity (EC) at influent was 1929 μS/cm ± 352. After the treatment through CW, EC was not as low as expected with mean effluent value of 1779 μS/cm ± 354.This could be due to the recirculation of discharge wastewater from RO to the wetland, with build-up of EC. This is further proved by more or less continuous increase in EC at influent, effluent and treated RO (Figure 23). Mean conductivity of water coming out of RO was 435μS/cm, where as that of rain water was 169 μS/cm. WHO and EU guidelines for conductivity in drinking water are 250μS/cm; mean conductivity of treated water through RO could not meet this limit.

4.2.3.2 Total Suspended Solids

Mean TSS concentration during the monitoring period at influent was 338 mg/L, higher than the typical CW influent from septic tanks, which ranges between 44-54 mg/L (USEPA, 2000). The difference could be due to the nature of the wastewater, the treatment system etc.

According to monitoring results, the inlet TSS concentration of CW varied between 208 and 565 mg/L. The highest TSS concentration on 11th April could be due to the increased number of visitors coming to the boat, as there were educational tours (Figure 24). The quantity of wastewater flowing into the wetlands was also high on this day. As the wastewater percolated

through the wetland, the observed removal efficiency was 75% (Table 18) with mean effluent concentration of 83.7 mg/L ± 40.5.

Table 18: TSS concentrations (mg/L) at influent, effluent and concentration based removal efficiencies (%) of the CWs at Houseboat

TSS	Influent	Effluent	RO	Rain
Avg (mg/L)	334.9	83.7	NA	61
Std.	138.1	40.5	NA	50
Min (mg/L)	208	11	NA	17
Max (mg/L)	565	128	NA	133
RE (%)		75	NA	NA

NA-Not Available

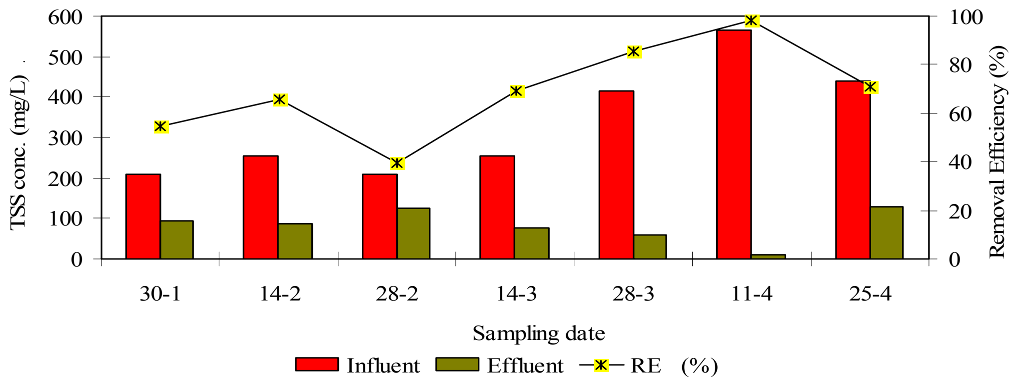


Figure 24: TSS concentrations (mg/L) on sampling days at the influent, effluent and concentration based removal efficiencies (%) of CWs at houseboat.

A high variation in the loading rate was observed within the monitoring period. High loading rate could be due to increasing number of visitors. Loading rate was highest in April. TSS loading rate varied between 5.9 and 61.5 g/m^2.d (30 ± 18.6).

Unlike in subsurface horizontal flow CW, where most of the suspended solids are eliminated at the inlet of the bed (Vymazal *et al*., 1998), the total surface area of the vegetated bed plays an important role in the removal of TSS in VF CW. As the loading rate increased the removal rate also increased, with R^2= 0.9548 (Figure 25). However, no correlation was found between effluent concentration and loading rate.

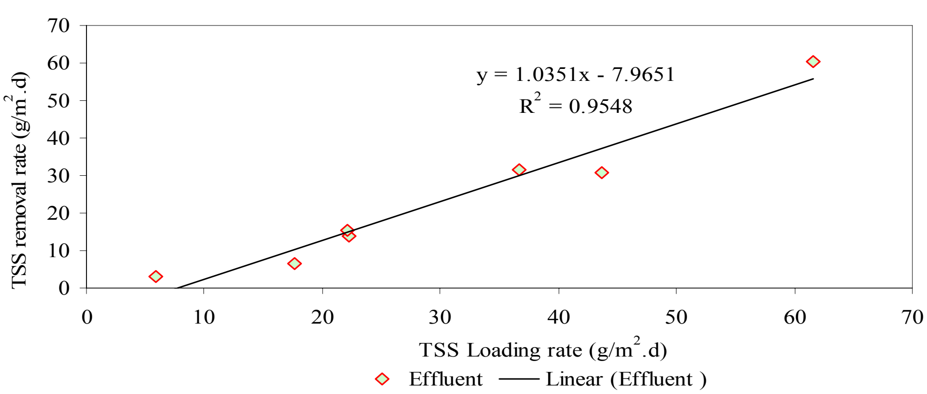


Figure 25: TSS loading rate (g/m^2·d) vs. removal rate (g/m^2·d) of the CWs at houseboat

With 75% removal efficiency, the average effluent concentration could not meet the EU discharge limit which is 30mg/L, nor the Dutch effluent standard of any class. The latter one varies between 75 to 30 mg/l, depending on the class. As this boat is moving from one place to the other the discharge standard required will also change from place to place.

TSS of rain water was also measured during the study. Average TSS ± Std. dev was, 61 mg/L ± 50 [14; 133], n=4. Due to the lack of rainfall only 4 samples could be collected during the monitoring period. TSS of rain water without contact with roof surface was found to be 1.75 mg/L (Senden, 2003). The higher concentration of TSS at the houseboat is due to the green roof. However, the concentration was comparable to the TSS concentration of 60 mg/L found in harvested rain in Bangor, India as mentioned by Sharma *(*2006).

Nitrogen

Ammonium nitrogen

The pretreated wastewater, diluted by RO discharge, pumped into CW of houseboat had an average NH_4-N concentration value of 32.18 ± 13.5 mg/L which is within the range typical for CW influent from septic tanks, i.e. 28-42 mg/L (USEPA, 2000). As wastewater passed through the system there was a significant reduction in NH_4-N concentration: mean effluent concentration was 1 mg/l ± 0.9 with removal efficiency of 97% (Table 19 and Figure 26).The variation at the effluent could be due to the variation of influent concentration, rain and evapotranspiration. NH_4-N was probably converted by microbial nitrification processes since temperature and pH were within the range that could support this process. Ammonia volatilization could not have occurred as the pH in the system never rose above 9 (Vymazal, 2001).

Table 19: NH_4 - N concentration (mg/L) of CW (influent and effluent), RO treated water, rain water and concentration based removal efficiency (%) of constructed wetland at houseboat.

NH_4 - N	Influent	Effluent	RO	Rain
Avg. (mg/L)	32.18	1.00	0.1	0.05
Std.	13.5	0.9	0.1	0.1
Min (mg/L)	17.98	0.29	0.01	0.00
Max (mg/L)	54.52	2.70	0.31	0.22
RE (%)		97	NA	NA

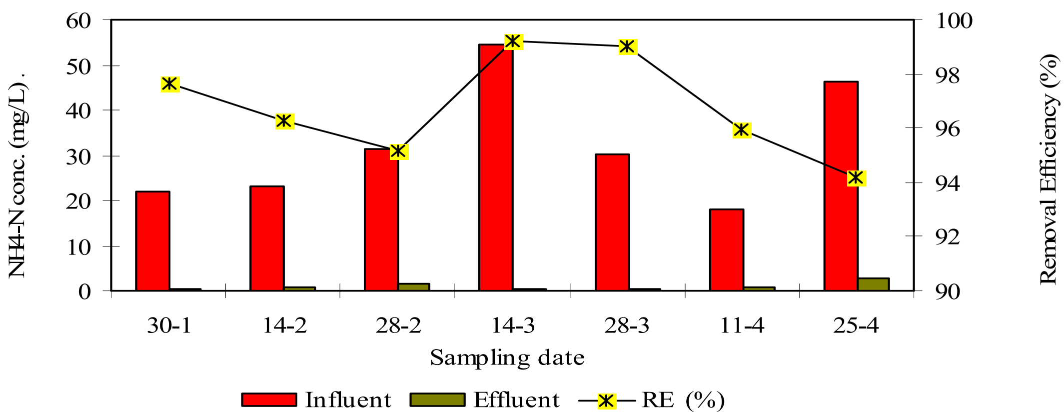


Figure 26: NH_4-N concentrations (mg/L) on sampling days at the influent, effluent and concentration based removal efficiencies (%) of CW at houseboat.

In intermittently loaded VF CWs with sufficient Oxygen supply, more than 90% reduction of NH_4-N can occur (von Felde and Kunst, 1997). The removal efficiency might further increase with the development of vegetation in the coming years as they help nitrification through oxygenation of the substrate by RZE (Yang *et al*., 2001), and facilitate growth of a rich and productive community of attached microorganisms by providing higher surface area.

The loading rate of NH_4- N changed between 0.621 and 4.770 $g/m^2 \cdot d$ (2.68 $g/m^2 \cdot d$ ± 1.5). The effluent concentrations were not dependent on the NH_4-N loading rate, however removal rates were strongly dependent on the loading rate (R^2=0.9972) (Figure 27): as loading rate increased the removal rate also increased.

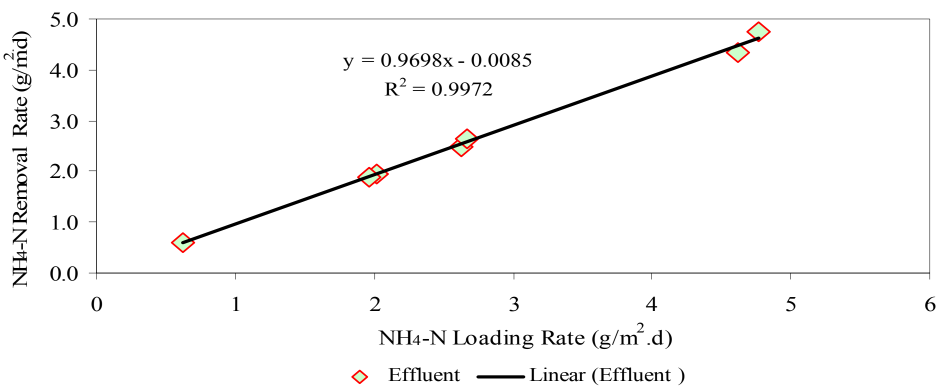


Figure 27: NH_4- N loading rate ($g/m^2 \cdot d$) vs. removal rate ($g/m^2 \cdot d$) of CW at houseboat

Dutch effluent standard for NH_4-N discharge is < 4 mg/L for class IIIa and III b. As the results indicate, the mean effluent water quality was mostly below the effluent standards, with the exception of one exceedance on 25th of April.

The mean NH_4-N concentration from RO was 0.11 mg/L ± 0.1 and varied between 0.01 and 0.31 mg/L. According to the EU drinking water standard, the upper limit is 0.50 mg/L for NH_4. During the monitoring period the NH_4 concentration treated through RO was within the standard, so is drinkable with respect to NH_4-N.

Rain water without contact on any surface was found to have NH_4-N concentrations of 1.10 mg/L (Senden, 2003). Rain water collected from the roof of the houseboat had even a lower mean ammonia-N concentration of 0.05 mg/L ± 0.1.

Nitrate - Nitrogen

Throughout the monitoring period, the primary treated wastewater had a mean NO_3-N concentration of 27.8 ± 11.1 mg/L at influent. The observed concentration of NO_3-N was very high compared to typical constructed wetland influent from septic tanks, which ranged from 0.0 to 0.9 mg/L (USEPA, 2000). Due to the anaerobic nature of the influent wastewater, NO_3-N concentration was expected to be low. High NO_3-N concentration at influent could be due to the addition of the discharge wastewater from the RO to the degreaser, which is high in oxygen and nitrate concentration. After passing the wastewater through the wetland system, due to nitrification of NH_4-N, the NO_3-N concentration increased, its mean concentration reached 70.3 mg/L ± 8.8 (Table 20 and Figure 28).

Table 20: NO_3 - N concentration (mg/L) of the CWs (influent and effluent), RO treated water, rain water and concentration based removal efficiency (%) of constructed wetland at houseboat.

NO_3- N	Influent	Effluent	RO	Rain
Avg. (mg/L)	27.8	70.3	17.9	0.9
Std.	11.1	8.8	2.5	0.6
Min (mg/L)	11	54	15	0.5
Max (mg/L)	41	81	23	2

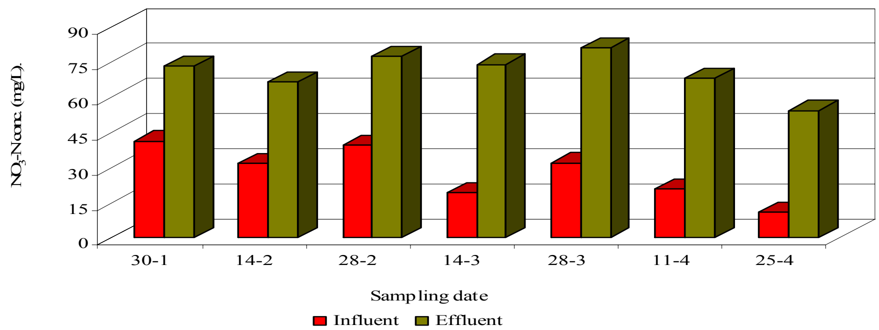


Figure 28: NO_3-N concentrations (mg/L) on sampling days at influent and effluents of CW at houseboat

According to the Dutch standard, TN of effluent wastewater should be less then 60 mg/L for both class IIIa and IIIb. Even if a nitrate single concentration exceeds this level, it could not meet the effluent standard. Obviously it could not meet the European discharge standard which is even stricter and requires an effluent concentration lower than 15 mg/L for TN. Moreover, the water treated through RO had high NO_3-N concentration, its mean concentration was 18 mg/L ± 36, and reached as high as 23 mg/L. EU's drinking water standard for NO_3 is 50 mg/L whereas RO treated water had mean NO_3 concentration of 80 mg/L and reached up to 102 mg/L during the monitoring period. Rain water had low NO_3-N concentration with mean concentration of 1 mg/L.

In this system discharge from RO was fed back into the degreaser. Treated water through RO had mean NO_3-N concentration of 18.0 mg/L, so the discharge water through RO might have even higher concentration. According to Cooper (2001) removal of nitrate was expected by biological denitrification using the carbon source present in the wastewater feed. However, even after the recirculation of the wastewater from RO there was no sufficient denitrification and sufficient removal of TN to meet the effluent standard at the end. This could be due to low BOD of the wastewater or too short HRT in the degreaser.

The average loading rate during the monitoring period in CWs varied between 1.1 and 3.3 $g/m^2.d$ with mean loading rate of 2.1 $g/m^2.d$ ± 0.9. Due to the nature of nitrification process, a relation between NH_4-N influent and NO_3-N effluent is expected. But no correlation was observed between influent NH_4-N and effluent NO_3-N concentration. The relation between NH_4-N loading rate and NO_3-N production rate was also plotted, a strong correlation was found with R^2 =0.858 (Figure 29).

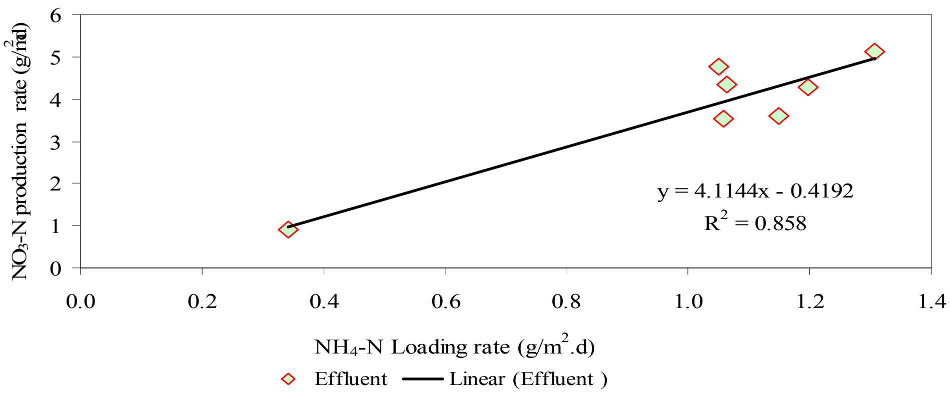


Figure 29: NH_4-N loading rate vs. NO_3-N production rate ($g/m^{2}d$) of CWs at houseboat.

Average NO_3-N concentration of rain water collected from the vegetated roof was 0.9 ± 0.6 mg/L. The concentration was less than rainwater collected without any surface contact (3.4 mg/L) (Senden, 2003).

4.2.3.3 Total Phosphorus

The mean TP concentration of wastewater after primary treatment in the septic tank was 14 mg/L ± 5.6. The observed average TP was at upper limit of the typical constructed wetland influent from septic tanks, i.e. 12-14 mg/L (USEPA, 2000). After the treatment mean effluent concentration was 10.3 mg/L ± 5.5, i.e. a reduction by 28% (Table 21 and Figure 30). As phosphorus is mostly removed by sorption the capacity is limited and the removal should decrease in the course of time. Low removal efficiency in this system could be due to the fact that the pH value was not suitable for the precipitation with calcium and iron.

Table 21: TP concentrations (mg/L) at influent, effluent and concentration based removal efficiencies (%) of the CWs at Houseboat

Phosphorus	TP		oPO_4	
	Influent	Effluent	RO	Rain
Avg. (mg/L)	14.2	10.3	0.38	1.26
Std.	5.6	5.5	0.2	0.3
Min (mg/L)	6.6	3.0	0.2	1.0
Max (mg/L)	22.1	17.1	0.6	1.7
RE (%)		28	NA	NA

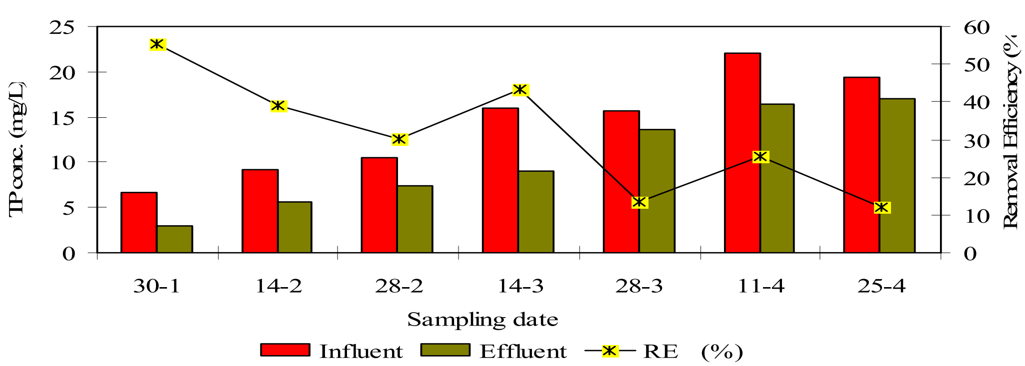


Figure 30: TP concentrations (mg/L) on sampling days at the influent, effluent and concentration based removal efficiencies (%) of CW at houseboat.

Average loading rate during the monitoring period was 1.3 $g/m^{2}d$ ± 0.7 which varied between 0.8 and 2.4 $g/m^2.d$. No correlation was observed between loading and removal rate.

However, effluent concentration was strongly dependent on the influent concentration and the loading rate. At high loading rate effluent concentration was also high (Figure 31).

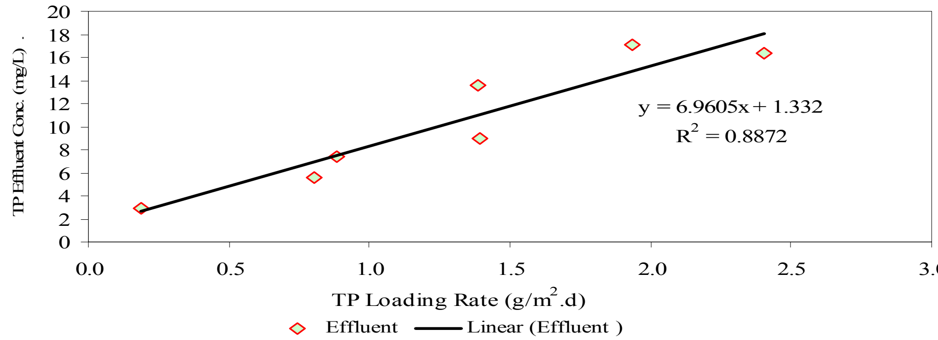


Figure 31: TP loading rate (g/m^2.d) vs. effluent concentration (mg/L) of CW at houseboat

A Dutch discharge standard for total phosphorus is available only for class IIIb area, and it is less than 4 mg/L. However, the effluent water quality at houseboat did not meet this standard nor the EU discharge limit of 2 mg/L TP.

Treated water from RO had mean orthophosphate concentrations of 0.4 ± 0.2 mg/L whereas the rain water had a bit higher concentration, with mean concentration of 1.3 mg/L ± 0.3. Rain water collected with surface contact of any surface has orthophosphate concentration of 0.15 mg/L (Senden, 2003). This could be due to the vegetated roof.

4.2.3.4 COD

The observed mean COD concentration, 140 mg/L ± 51.5 at influent is lower than the typical constructed wetland influent from septic tanks, which range between 310-344 mg/L (USEPA, 2000). This is due to the dilution of wastewater through discharge from RO. The inlet COD concentration varied between 92 and 224 mg/L. As the wastewater flows though the wetland there was a drop of 55% of COD concentration, with average effluent concentration of 63 mg/L ±18.8 (Table 22 and Figure 32).

Table 22: COD concentrations (mg/L) and removal efficiencies (%) of CW at houseboat.

COD	Influent	Effluent	RO	Effluent
Avg. (mg/L)	140	62.9	NA	NA
Std.	51.5	18.8	NA	NA
Min (mg/L)	92	38	NA	NA
Max (mg/L)	244	89	NA	NA
RE (%)		55	NA	NA

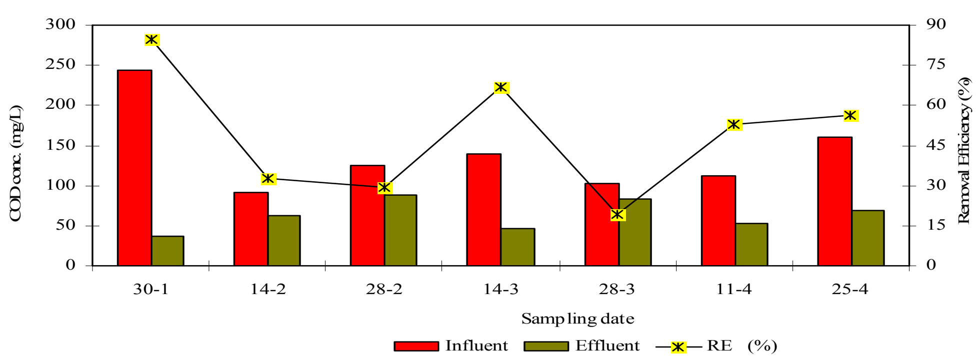


Figure 32: COD concentration (mg/L) on sampling days at the influent, effluent and removal efficiencies (%) of CW at houseboat

Moreover, the COD: BOD ratio of the influent wastewater of the houseboat was 3.1:1. It can be concluded that the wastewater has a lower biodegradability compare to the ZIN wastewater and could be classified as “low strength” wastewater. Vymazal *et al*. (1998) reported that if the COD: BOD_5 ratio of water is high, it is difficult to have effluent concentration values below 50 mg/L. This was also the case for the wetland at the houseboat.

Mean loading rate ± Std. [min; max] for the wetland was 10.7 ± 3.1 g/m^2d [7.0; 15.9]. The loading rate increased more or less continually from Jan to April. No relationship between loading rate and effluent concentration (mg/l) was observed in the wetlands. However, the removal rate was loosely dependent on the loading rate, with R^2= 0.4888 (Figure 33).

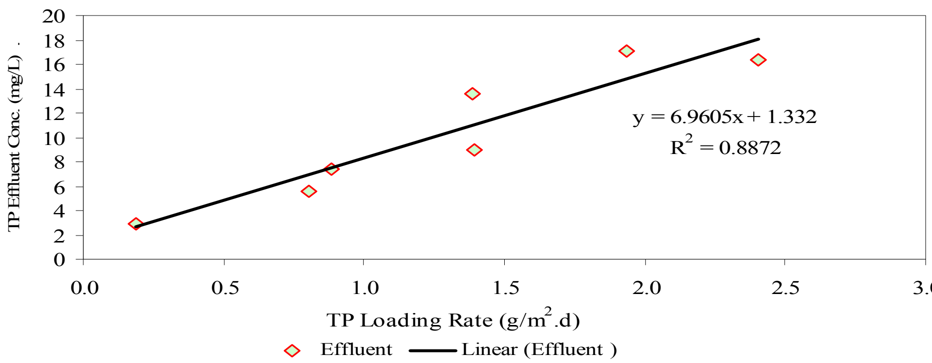


Figure 33: COD Loading rate ($g/m^{2}d$) vs. removal rate ($g/m^{2}d$) of CW at houseboat.

Though the removal efficiency of the wetland for COD was just 55%, effluent COD concentration throughout the monitoring period was below European Directive standard (<125 mg/L). This could be due to the low COD concentration at the influent. Also the effluent would comply with class IIIb of the Dutch effluent standard which is <200 mg/L, even without any treatment.

4.2.3.5 BOD_5

The observed mean concentration of BOD_5 at influent of CW was 52.4mg/L ± 26.2, the concentration was lower than the typical constructed wetland influent from septic tanks, which range between 129-147 mg/L (USEPA, 2000) and this is due to the dilution effect caused by RO discharge. Treatment efficiency of CWs for BOD_5 is usually high (e.g. Hammer, 1998; Cooper and Findlater, 1990; Kadlec and Brix, 1995). An overall 94% removal efficiency was achieved with mean final effluent concentration of 3.3 mg/L (Table 23 and Figure 34).

Table 23: BOD_5 concentrations (mg/L) and removal efficiencies (%) of CW at houseboat.

BOD_5	Influent	Effluent	RO	Rain
Avg. (mg/L)	52.4	3.3	NA	NA
Std.	26.2	1.8	NA	NA
Min (mg/L)	28	3	NA	NA
Max (mg/L)	100	32	NA	NA
RE (%)		94	NA	NA

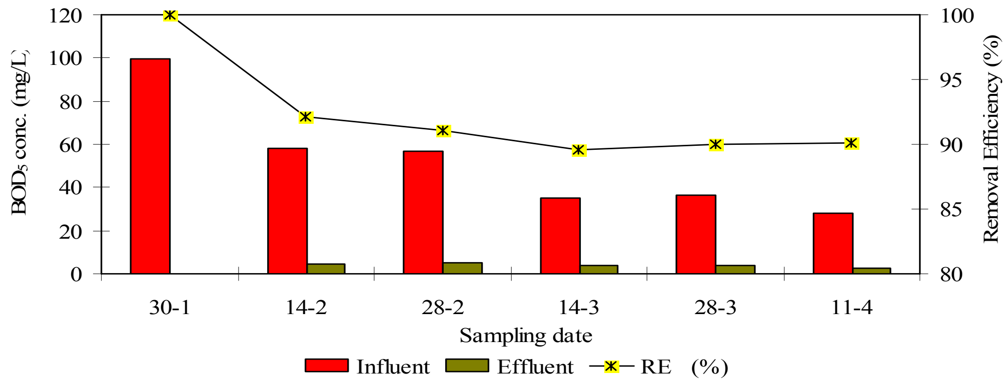


Figure 34: BOD$_5$ concentrations (mg/L) on sampling days at influent, effluent and removal efficiencies (%) of CW at houseboat.

The mean loading rate for the wetland during the monitoring period varied between 2.8 and 5.0 g/m^2.d. The calculated average loading rate ± Std. was 3.8 g/m^2.d ± 1.0. There was a strong correlation (R^2= 0.984) between loading rate and removal rate, indicating the system can perform better as loading rate increases (Figure 36). However, it should be noted that the effluent concentrations were also dependent on the loading rate, at increasing loading rate the effluent concentration increased as well (R^2= 0.6028) (Figure 35).

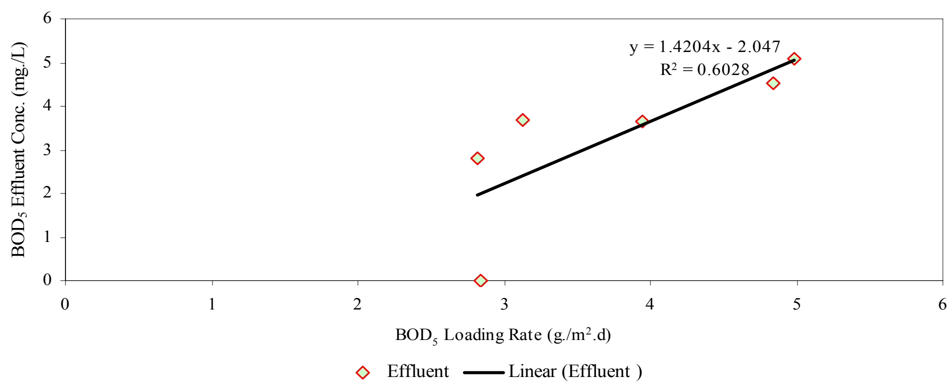


Figure 35: BOD$_5$ loading rate (g/m^2.d) vs. effluent concentrations (mg/L) of CW at houseboat.

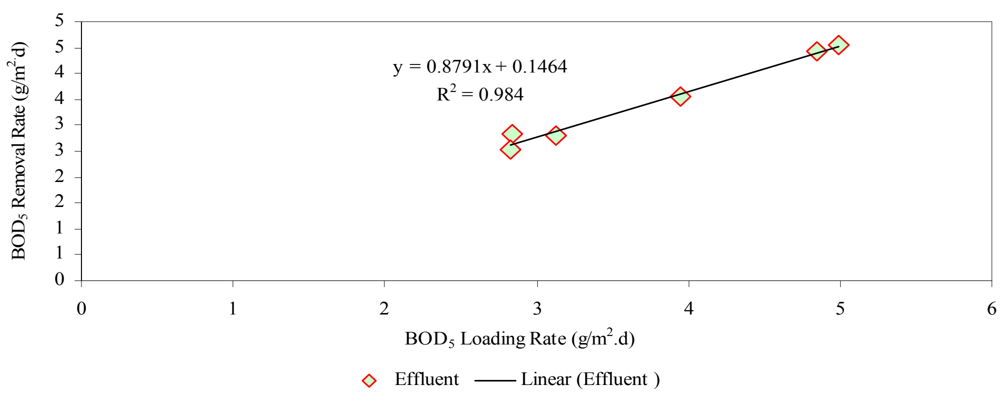


Figure 36: BOD$_5$ loading rate vs. removal rate (g/m^2.d) of CW at houseboat.

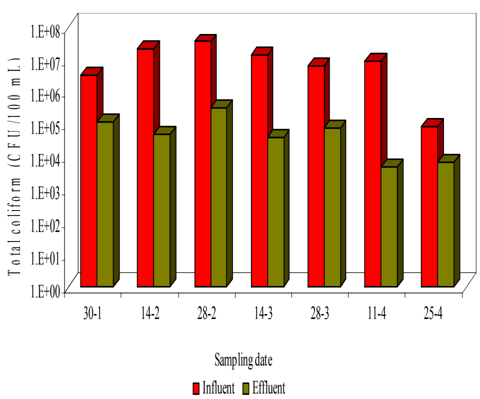
Throughout the monitoring period, effluent BOD$_5$ concentration of the wetland was below the class IIIb Dutch discharge standard of 40 mg/L and also below the European discharge standard of 20 mg/L.

4.2.3.6 E.coli and Total coliform

After the anaerobic pretreatment the influent to CW had Total Coliform and E.coli concentration of 1.28E+07/100 mL and 1.12E+04/100mL respectively (Figure 37). As the wastewater percolated through the wetland there was a significant drop in bacteria concentration. Mean concentration of E.coli found at effluent was zero where as total coliform were 8.89E+04/100mL. The log reduction of total coliform and E.coli were about 2.2 log units and 4 log units respectively (Table 24). Average effluent concentration of total coliform varied between 4.83E+03/100 mL and 3.18E+05/100mL. The fluctuation could be due to the change in influent concentration and also due to the rain and evapotranspiration. Since there was no E.coli found at effluent, the water meets all the discharge limits. However, it should be noted that the total coliform were high with >3 log units/100 mL.

Table 24: Total Coliform and E.coli (CFU/100mL) of CW (influent and effluent), Ro treated water, Rain water and concentration based removal efficiencies (%) at houseboat.

	Total Coliform				E.coli			
	Influent	Effluent	RO	Rain	Influent	Effluent	RO	Rain
Avg. (CFU/100mL)	1.28E+07	8.89E+04	0.00E+00	2.09E+04	1.12E+04	0.00E+00	0.00E+00	0.00E+00
Std.	9.03E+04	4.83E+03	0.00E+00	4.76E+04	1.19E+04	0.00E+00	0.00E+00	0.00E+00
Min (CFU/100mL)	3.53E+07	3.18E+05	0.00E+00	1.67E+02	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Max (CFU/100mL)	1.21E+07	1.09E+05	0.00E+00	1.18E+05	3.33E+04	0.00E+00	0.00E+00	0.00E+00
Log removal		2.2	NA			4.0	NA	



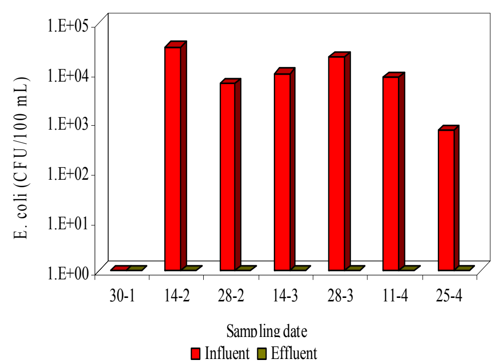


Figure 37: Total Coliform and E.coli concentration (CFU/100mL) at influent and effluent of CW at houseboat.

It is no surprise that Total coliform and E.coli were not found in the drinking water coming through RO, this form of water treatment is very effective for removal bacteria. However microbial infection of the RO module can occur, with leaky seals in the membrane. Therefore disinfection was also installed, i.e. a UV filter after the RO to ensure zero bacterial contamination in drinking water. So, as expected no bacteria were found.

Rain water collected from the vegetated roof had a mean concentration of total coliform of 2.09E+04/100 mL. E.coli was not found during the study period in rain water. Coombes *et al*. reported a zero total and faecal coliform in rain water collected from a roof. The presence of total coliform in rainwater from the house boat is probably due to the green roof.

4.3 Comparison of ZIN and Houseboat

Concentration based removal efficiencies of the pollutants from the wetlands at ZIN and the houseboat were compared with relevant literature from other countries about VF constructed wetlands for domestic wastewater treatment.

4.3.1 TSS

Influent TSS concentration at ZIN and houseboat were the highest, compared to all other countries given in Table 25. In spite of this, ZIN had highest removal efficiency of 94% and 93% (new and old wetlands) where as a removal efficiency of 75% was achieved at the houseboat. The low removal efficiency of the houseboat CW could be due to the volcanic rocks used in the media, as they are very coarse. The removal efficiency was higher than Turkey and was comparable to Poland, and Belgium removal efficiency. The differences in the removal efficiencies could be due to the variation in design and operation, climatic conditions, types of plants, substrate used etc.

Table 25: TSS influent and effluent concentration (mg/L) and concentration based removal efficiency (%) of VF constructed wetlands

Countries	Influent (mg/L)	Effluent (mg/L)	RE (%)
Austria*	121	10	92
Belgium*	60	15	75
France*	225	18	92
Sobiechy,Poland**	98	24	76
METU,Turkey***	67	29	64
ZIN Wetland N	754	49	94
ZIN Wetland O	754	50	93
Houseboat	335	84	75

*Reference: *Vymazal et al. (1998), **O' Hogain (2002); Korkusuz (2004)*

TSS removal by the CW at the houseboat was not as high as in ZIN which has well developed root system, and achieved removal of more than 90% TSS concentration. Nevertheless, the performance was better than expected as it was a very new wetland built around end of October 2006, with no proper development of roots system, which could add surface area to the wetland media and reduces water velocity and reinforces settling and filtration in the root network. Since macrophytes in these wetlands were only 2 months old, a higher removal of TSS might be expected in the following years. According to Brix (1997) and Tanner (2001) plant effects in many wetlands are usually observed only after three years of establishment. Hence, the observed TSS removal could be related to sedimentation, filtration, bacteria decomposition and adsorption to the wetland media (Stowell *et al*., 1981).

4.3.2 Nitrogen

Ammonium-Nitrogen

NH_4- N concentration at influent of CW at ZIN was much higher than the influent applied at Austria, France, Poland and Turkey. However, it was comparable to that of Germany. Even after higher concentration at influent, the removal efficiency (%) at the ZIN constructed wetlands were higher than all other countries shown in Table 26 .This could be due to the fact that the wetlands had high oxygen transfer rate, as the mean effluent oxygen concentrations reached around 4 and 6 mg/L at effluent of two wetlands (new and old) from 0.4 mg/L at influent. Vegetation might have slightly increased nitrification through oxygenation of the substrate through RZE (Yang *et al*., 2001).

Table 26: NH_4- N influent and effluent concentration (mg/L) and concentration based removal efficiency (%) of VF Constructed Wetlands

Countries	Influent (mg/L)	Effluent (mg/L)	RE (%)
Austria*	72	4.5	93.7
France*	25	18	28
Germany*	81	29	64
Sobiechy,Poland**	46	6.4	86
METU,Turkey***	26.37	11.4	55.9
ZIN Wetland N	87.2	0.6	99.3
ZIN Wetland O	87.2	0.9	99.0
Houseboat	32.18	1.00	97

*Reference: *Vymazal et al. (1998), **O' Hogain (2002); ***Korkusuz (2004).*

NH_4-N influent concentration at houseboat CW was comparable to that of France and Turkey, and lower than other countries. The removal efficiency (%) at the CW of houseboat was higher than all other countries. Despite being new, with no proper development of root system of the macrophytes, the higher removal efficiency could be due to the fact that the wetland had low BOD loading rate, thus providing sufficient oxygen for nitrification.

Nitrate-Nitrogen

The ZIN CWs had low influent NO_3-N concentration which was comparable to that of Austria, France (Table 27). But the effluent concentrations at both the wetlands were much higher than the effluents shown in the table. Since the wetlands effluent had high oxygen concentrations, and high NH_4-N concentration at influent, high NO_3-N at effluents could be due to more extensive nitrification and also could be due to less denitrification taking place in the absence of anaerobic zones in the system.

Table 27: NO_3 - N influent and effluent concentration (mg/L) and concentration based removal efficiency (%) of VF Constructed Wetlands

Countries	Influent (mg/L)	Effluent (mg/L)
Austria*	0.0	47.5
France*	0.0	5.0
Germany*	1.9	12.0
METU,Turkey***	1.47	8.38
ZIN Wetland N	0.4	71.3
ZIN Wetland O	0.4	63.5
Houseboat	27.8	70.3

*Reference: *Vymazal et al. (1998) *** Korkusuz (2004).*

The mean influent and effluent NO_3-N concentration of the CWs at the houseboat was the highest shown in Table 27. Influent concentration was also much higher than typical constructed wetland influent from septic tanks, which could be due to the recirculation of discharge wastewater from RO. Discharge wastewater from RO had high oxygen and NO_3-N concentration. This indeed leads to high NO_3-N concentration at effluent. Besides, due to the aerobic condition present in the wetland, a lower amount of denitrification might have taken place. But to no surprise, VF constructed wetland removes successfully ammonia nitrogen but very limited denitrification takes place in these systems (Vymazal, 2006). Unfortunately, total nitrogen was not analyzed during the study so the reduction of Total Nitrogen and the amount of denitrification could not be studied for both of the wetlands (ZIN and houseboat).

4.3.3 Total Phosphorus

The average TP removal efficiency from ZIN wetlands was much higher than the one found by Korkusuz (2004), in VF gravel bed. However, the performance of CWs at ZIN was less than Austria and Germany were removal efficiencies above 60% were found. Nevertheless,

the removal efficiency was comparable to the ones found in Belgium and Germany (Table 28). The low removal efficiency inspite addition of calcium and iron could be due to the unsuitable pH in the system.

Table 28: TP influent and effluent concentration (mg/L) and concentration based removal efficiency (%) of VF constructed wetlands

Countries	Influent (mg/L)	Effluent (mg/L)	RE%
Austria*	10.7	4	62.6
Belgium*	4.6	3.5	23.9
France*	8.5	5.8	31.8
Germany*	15.	4.8	69.8
METU,Turkey***	6.61	6.03	5.61
ZIN Wetland N	10	7	23
ZIN Wetland O	10	7	31
Houseboat	14.2	10.3	28

*Reference: *Vymazal et al. (1998), Korkusuz (2004).*

Mean influent concentration at the houseboat was the highest compared to influent applied in other countries present in the table, except Germany. Also the effluent concentration at the boat house was the highest. This is due to the low removal efficiency found in the CW of houseboat.

4.3.4 COD

The average influent COD concentration of 417 mg/L of ZIN constructed wetlands is comparable to Germany and was higher than that of Belgium, Austria and Turkey. Removal efficiency was higher than all other countries except Austria with a removal efficieny of 90%; this high removal efficiency could be due to the high strength of the wastewater.

The average COD influent concentration at the houseboat, 140 mg/L was lower than all countries given in Table 29. In addition, the removal efficiency was also lower than all the countries except for Turkey with removal of 49%. Lowest removal efficiency (49%) reported in Turkey could be due to the low strength of wastewater. The observed low removal efficiency could be due to the fact that the wastewater of the houseboat was of low strength and relatively low biodegradability.

Table 29: COD influent and effluent concentration (mg/L) and concentration based removal efficiency (%) of VF Constructed Wetlands

Countries	Influent (mg/L)	Effluent (mg/L)	RE%
Austria*	325	33	90
Belgium*	168	60	64
France*	495	92	81
Germany*	430	95	78
METU,Turkey***	251	143	49
Colecott,Ireland**	462	55	88
ZIN Wetland N	417	49	88
ZIN Wetland O	417	51	88
Houseboat	139.6	62.9	55

*Reference: ***Korkusuz, (2005);*O' Hogain (2002);*Vymazal et al. (1998).*

4.3.5 BOD_5

Average BOD_5 concentration at influent of ZIN CWs was comparable to Austria and was higher than France. Removal efficiency was less than that of Austria. Nevertheless it was comparable to France. Average BOD_5 influent concentration at houseboat CW was lower than the influent concentration applied in other countries shown in Table 30. Also the effluent concentration was low, except compared to the one measured in Austria, which had effluent concentration as low as 1.7 mg /L with highest removal efficiency of 99.4%. Removal

efficiency from this study was comparable to that of France, Austria VF wetland and higher than Estonia wetland with removal efficiency of only 66 %.

Table 30: BOD_5 influent and effluent concentration (mg/L) and concentration based removal efficiency (%) of VF Constructed Wetlands

Countries	Influent (mg/L)	Effluent (mg/L)	RE%
Austria*	287	1.7	99.4
France****	324	18	94.4
Kodijarve,Estonia**	143.5	49.0	65.9
ZIN Wetland N	279	13	95
ZIN Wetland O	279	11	96
Houseboat	52.4	3.3	94

Reference: Korkusuz (2005)

4.3.6 Total Coliform and E.coli

Table 31: Total Coliform and E.coli, influent and effluent concentration (CFU/100ml) and removal efficiency (%) of VF Constructed Wetlands

Countries	Influent (CFU/100mL)	Effluent (CFU/100mL)	Removal (unit Log)
Tunisia (Total Coliform)*	1.04E+07	1.53E+05	1.8
Italy (Total Coliform)**	2.9E+06	1.41E+02	4.3
ZIN Wetland N (Total Coliform)*	3.93E+07	3.47E+04	3.1
ZIN Wetland O (Total Coliform)*	3.93E+07	2.78E+04	3.2
Houseboat(Total Coliform)	1.28E+07	8.89E+04	2.2
Tunisia (E.coli)*	9.24E+06	9.25E+04	2.0
Italy (E.coli)**	1.4E+06	4.4E+01	4.5
ZIN Wetland N (E.coli)	1.12E+06	3.7E+02	3.5
ZIN Wetland O (E.coli)	1.12E+06	6.67E+02	3.2
Houseboat (E.coli)	1.12E+04	0.00	4.0

*Reference: * Ghrabi and Keffala (2005), ** Masi et al. (2007)*

High removal was found in constructed wetland in Italy with log reduction more then 4log for both total coliform and E. coli. The log removal at the ZIN and the houseboat for total coliform and E. coli were higher then found in Tunisia and the one reported by Green *et al*. (1997). Green *et al* found removals of 1.5 to 2.1 log units in two subsurface flow constructed wetlands. Ottova *et al*. (1997) and Laber *et al*. (1999) reported a removal of 99.95 % of total coli and E.coli after the treatment through the two stage (HF and VF) CW. The constructed wetlands at ZIN, which are single bed CWs, perform better with a reduction of 3.5 and 3.2 units log of E.coli at new and old wetlands respectively.

Kadlec and Knight (1996) mentioned the higher removals in constructed wetlands are due to the retention time and land intensive treatment. Vegetated wetlands appear to be more effective for pathogen removal. Gersberg *et al*. (1987) reported the removal of total coliforms was higher in a vegetated bed compared to an unvegetated bed. Moreover, aquatic plants (*Phragmites australis*) can kill faecal indicators (*Escherichia coli*) and pathogen bacteria (Seidel, 1976; Vincent *et al*., 1994). Several regions of root are known to produce compounds which may inhibit certain microorganisms (Bowen and Rovira, 1976).

CHAPTER 5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conference center ZIN

5.1.1 Conclusions

Though the two similar wetlands at ZIN were built in different years, they have similar concentration based removal efficiencies except for phosphorus. However, at the observed high loading rates of several pollutants (TP, COD, BOD_5, NH_4-N, NO_3-N) the removal rate was higher in the new wetland compared to the old one indicating that the new wetland is better in handling high loading rate. Wastewater from ZIN was found to be high strength wastewater. Wetlands were more efficient in removal of BOD, COD, TSS and E.coli, while they were less efficient in removal of nutrients.

The wetlands were designed to meet the discharge standards for sensitive areas (Class IIIb) but the treated water coming out of the wetlands could not to meet this standard as the nutrients (TP, TN) exceeded the limit. BOD, COD, TSS, NH_4-N and E.coli were however observed to be within the discharge limits.

Though the NH_4-N removal efficiency was more than 99%, and the effluent was within the limit of Class IIIb, Nitrate-N concentration in the effluents was too high (71 and 64 mg/L) to meet any discharge standard for TN.

The constructed wetlands of ZIN fall in an agricultural area which requires class I discharge standards with minimal discharge requirements for TSS, BOD and COD and no standard requirement for nutrients. The wetlands were observed to have good efficiency in removing TSS, BOD and COD hence the wastewater from the treatment plant meets the requirement of Class I discharge standard.

Total coliform was reduced by 3 log units, whereas E.coli was removed by 3.6 and 3.3 log unit at new and old wetland respectively. E.coli effluent concentration was below 1000/100 ml and met the standard required for a sensitive area. However it should be noted that the mean total coliform concentrations were higher than 3 log unit/100 ml during the study period.

Agricultural use

BOD and COD effluent concentrations could meet the water quality requirement for agricultural use. However, TSS and nutrients were higher than the guideline values. TSS <30mg/L (EPA, 1992) is needed to avoid clogging of sprinklers. High nutrients adversely affect some crops during certain growth stages (EPA, 1992). Dissolved oxygen in the effluent of the old wetland was sufficient for reusing wastewater for agriculture; however the effluent from new wetland had DO below 5 mg/L. Though E.coli concentration was below the limits for agricultural use, high concentration of total coliform indicated the need of further disinfection for agricultural use. Suggested water quality guidelines for watering gardens, parks and cemeteries are stricter and require non detectable faecal coliform /100 mL (EPA 1992). So the water needs disinfection before it could be possibly used for watering the garden and cemetery of ZIN.

Toilet flushing

Wastewater from ZIN needs disinfection to be used for flushing toilet due to the high concentration of coliform bacteria. Suggested guidelines for water used for flushing toilets are non detectable faecal coliform/100mL (EPA 1992). In Japan E. coli should be less then 10CFU/100 ml for reuse of wastewater for toilet flushing (Murakami, 1989). So, with simple disinfection technique the water can be reused for toilet flushing and this will save around 28% water supply.

Constructed wetland is good way to treat domestic wastewater but the reuse of wastewater needs simple tertiary treatment or disinfection depending on the use of wastewater.

5.1.2 Recommendation

Addition of a HF constructed wetland after the vertical flow one at ZIN could further reduce the level of pollutants. With this, additional benefit in the removal of nitrogen by denitrification can be achieved (Cooper, 2001). This will also create the additional benefit of removing TSS from an already oxidized effluent (low BOD/low NH_4-N effluent). This was also observed in the study by Mitterer-Reichmann (2002). An extra HF CW would also help reducing the bacteria concentrations. The more economic option could be the recycling of the nitrified water back to the feed tank where it is presumed that nitrate could be removed by biological denitrification using the carbon source present in the wastewater feed (Laber, 1997). Better nitrification can be expected and reduction of TN, which indeed will help to achieve the effluent criteria required for sensitive area, and reuse for irrigation and toilet flushing. For reusing the water for toilet flushing at present situation, disinfection is required.

For future study

Though the TN was analyzed for the study, but due to technical problems, the results obtained were not logical and are not presented in this report. So having TN data could provide more light on the performance of CWs for removal of nitrogen.

5.2 Houseboat

5.2.1 Conclusion

Having a constructed wetland in a houseboat is an expensive treatment method. Producing drinking water from wastewater is even more expensive. The cost of CW was 9 times higher than the one build in ZIN. The major requirements of the CW of houseboat such as the buoyancy and small area made it so costly. The CW was made from expensive light ingredients besides, some parts (especially the tanks) used in the system were especially designed for this purpose in order to make them fit in the small basement of the houseboat.

The wetlands at the houseboat achieved 97%, 94%, 55%, 75%, 28% concentration based removal efficiency for NH_4-N, BOD, COD, TSS, TP respectively. BOD, COD and NH_4-N concentrations were below the discharge limits require for the sensitive areas (Class IIIb). TSS concentration at effluent was slightly higher than this limit. These show that wetlands are not efficient enough in removal of nutrients to meet the discharge standards.

Water treated by RO was free of bacterial contamination, and NH_4-N was below the required drinking water standards. However, it had very high NO_3-N concentration, which poses risk to human health. So the treated water can not be called completely safe for drinking. Besides

the risk of fouling of RO membrane is high, due to the presence of pollutants in treated wastewater. Moreover, pH and conductivity were not within the drinking water standard range.

Continuous increase in the conductivity of the treated water was noted during the study caused by building up of pollutants due to the short water cycle in the houseboat. Moreover addition of calcium and iron in wetland for the removal of TP is creating problem in the RO system shortening its life. The treated water has higher pH and conductivity, than permitted level for drinking water. So, using wastewater to generate drinking is not a best option for the boat house.

5.2.2 Recommendations

Use of rain water for producing drinking water could be a better solution. Rain water collected has low level of pollutants compared to treated wastewater from constructed wetlands. However, the harvested rain water volume is not high enough to generate sufficient water, as some amount is lost due to soil and evapo transpiration. Though the collected rain water was turbid; pH was positively affected. So rain water harvesting without green roof could be a better option.

Other option could be installation of dry toilet and separate collection of urine (yellow water) and faeces (brown water) and treating only grey water in the constructed wetlands. Pure human urine contains more then 80% of total nitrogen and 50% of phosphorus in domestic wastewater (Fittschen and Hahn, 1998). This can be used as valuable fertilizer, faeces having high organic matter can be treated anaerobically (composting) to produce soil fertilizer. With reduction of TN and TP concentration at influent, the effluent from CW will also meet the discharge limits and the treated water will have better quality.

The cost of the CWs at the houseboat can be reduced to some extent by having one wetland instead of two wetlands for artistic view, and incorporating it on the boat instead of having it floating alongside.

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WHO-EU drinking water standards comparative table (Available at http://www.lenntech.com/WHO-EU-water-standards.htm) [Accessed on 20/3/2007].

APPENDIX

Table 32: Dutch and European discharge standards.

Parameters	Class I	Class II	Class III a	Class IIIb	EU discharge standard
BOD mg/L	<250	60	40	40	20
COD mg/L	<750	300	200	200	125
Total-N mg/L			60	60	15
NH4-N mg/L			4	4	
Total-P mg/L				6	2
SS mg/L	<70	60	60	60	30
E.coli (CFU/100 mL)					1000

Source: IBA-Systemen (1999)

Table 33: Comparison of treated wastewater through CWs with Dutch and European discharge standards.

Parameters	Class I	Class II	Class IIIa	Class IIIb	European discharge standard	Class I	Class II	Class IIIa	Class IIIb	European discharge standard	Class I	Class II	Class IIIa	Class IIIb	European discharge standard
	Effluent from wetland N					Effluent from wetland O					Effluent from CW of houseboat				
BOD	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
COD	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok
TSS	Ok	×	×	×	×	Ok	×	×	×	×	×	×	×	×	×
TP	NA	NA	NA	×	×	NA	NA	NA	×	×	NA	NA	NA	×	×
NH4-N	NA	NA	Ok	Ok	NA	NA	NA	OK	Ok	NA	NA	NA	Ok	Ok	NA
NO3-N	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
TN	NA	NA	×	×	×	NA	NA	×	×	×	NA	NA	×	×	×
E.coli	NA	NA	NA	NA	Ok	NA	NA	NA	NA	Ok	NA	NA	NA	NA	Ok

×-Exceeded
NA- not available
Ok- with in the limits

Table 34: WHO-EU drinking water standards

Parameters	Drinking water EU (1998)	WHO (1993)
Temp.		
pH	≥6.5 and <=9.5	6.5-8.5 desirable
DO	NM	NG
Conductivity µs/cm	250	250
NH_4 mg/l	0.5 mg/L	NG
NO_3 mg/l	50 mg/ (NO3)	
NO_2 mg/L	0.5	
TN	NM	50
EC	0/250 ml	
TC		
coliform bacteria	0/100 ml	

NM-Not mentioned,
NG-no guideline

Table 35: Comparison of treated water through RO system at the houseboat with WHO and EU drinking water standards.

Parameters	water treated through RO
pH	×
DO	Ok
Conductivity	Ok
Phosphorus	Ok
NH_4	Ok
NO_3	×
E.coli	Ok
Coliform bacteria	Ok

Table 36: Guidelines for interpretation of water quality for irrigation

Parameters	Degree of restriction on use		
	None	Slight to moderate	Severe
CE 25°C (dS/m)	< 0.7	0.7 - 3.0	> 3.0
TDS mg/L	< 450	450 - 2000	> 2000
NO_3-N	< 5	30-May	> 30

Reference: FAO (1985)

Table 37: Water quality guidelines for irrigation

Parameters	FAO	WHO	EU
pH	5.0-9.0	5.5-8.5	6.0-9.0
DO mg/L		5	5
Conductivity µs/cm	2000		
NH_4-N mg/L	5		
NO_3-N mg/L	10		
TP mg/L	3	3	2
TSS mg/L	45		35
BOD mg/L	30		25
COD mg/l	90		125

Table 38: Physio– chemical & microbiological parameters measure on in water sample form CWS at ZIN

pH			
Sampling Date	Influent	Effluent N	Effluent O
16-1	8.15	6.38	6.37
30-1	7.00	6.30	6.10
14-2	6.94	6.17	6.02
28-2	7.03	6.14	6.14
14-3	7.18	6.15	6.12
28-3	7.75	6.23	6.20
11-4	7.13	6.17	5.99
25-4	7.04	6.13	5.99
DO (mg/L)			
16-1	1.6	4.8	7.5
30-1	1.2	3.5	7.0
14-2	0.7	3.4	6.1
28-2	0.2	3.7	6.3
14-3	0.6	4.3	6.3
28-3	0.7	2.8	7.1
11-4	0.3	3.7	4.9
25-4	0.7	3.4	4.7
Conductivity (µs/cm)			
16-1	1199	939	843
30-1	1162	963	877
14-2	1332	1092	1010
28-2	1308	983	910
14-3	1127	1055	937
28-3	1117	896	692
11-4	1294	1020	1023
25-4	1262	1029	1019
Temperature ºC			
16-1	14	11.2	11
30-1	14	9.5	9.1
14-2	13	9	9
28-2	13.2	10	10
14-3	14.6	11	10.3
28-3	15.8	10	11.4
11-4	15	12.2	12.5
25-4	19	16	15.5
TSS (mg/L)			
16-1	698	70	72
30-1	829	88	71
14-2	1333	47	79
28-2	1326	103	71
14-3	776	44	1
28-3	340	3	35
11-4	328	8	38
25-4	402	27	32
NH4-N (mg/L)			
16-1	98.4	1.1	0.9
30-1	88.6	0.3	0.2
14-2	84.5	0.1	0.5
28-2	67.6	0	0.1
14-3	69.9	0.4	0.2
28-3	101.0	0.6	2.0
11-4	81.0	0.2	0.6
25-4	106.3	2.2	2.2

NO3-N (mg/L)			
16-1	0.39	51.2	50.9
30-1	0.37	60.9	56.9
14-2	0.04	85.3	82.5
28-2	0.46	77.2	74.8
14-3	0.38	85.0	72.5
28-3	0.56	65.4	35.9
11-4	0.52	74.6	65.2
25-4	0.34	70.6	69.2
TP (mg/L)			
16-1	8.19	6.43	6.22
30-1	9.62	5.23	6.87
14-2	12.14	9.01	5.96
28-2	9.09	8.86	7.41
14-3	8.3	7.6	6.6
28-3	8.3	8.2	7.9
11-4	11.0	5.5	5.1
25-4	10.0	8.3	6.6
COD (mg/L)			
16-1	296	70	69
30-1	398	68	51
14-2	399	36	49
28-2	462	122	152
14-3	516	26	36
28-3	466	23	20
11-4	410	30	10
25-4	386	16	23
BOD (mg/L)			
16-1	218.5	22.9	36.3
30-1	289.7	34.7	12.9
14-2	334.9	20.4	28.4
28-2	304.2	9.5	5.0
14-3	248.0	5.0	1.8
28-3	241.7	5.8	2.0
11-4	291.2	3.6	1.7
25-4	302.0	5.1	3.8
E.coli (CFU/100 ml)			
16-1	3.33E+05	1.00E+00	1.00E+00
30-1	2.00E+06	1.00E+00	1.00E+00
14-2	1.93E+06	6.67E+02	5.00E+03
28-2	1.03E+06	1.00E+00	1.00E+00
14-3	4.42E+05	2.33E+03	1.00E+00
28-3	9.00E+05	1.00E+00	3.33E+02
11-4	1.00E+06	1.00E+00	1.00E+00
25-4	1.33E+06	1.00E+00	1.00E+00
Total coliform (CFU/100 mL)			
16-1	6.55E+07	1.85E+05	1.03E+05
30-1	4.17E+07	3.33E+03	3.33E+03
14-2	1.10E+07	1.80E+04	6.97E+04
28-2	1.55E+07	6.67E+02	6.67E+02
14-3	2.74E+07	5.62E+04	1.11E+03
28-3	7.55E+07	2.83E+03	3.30E+04
11-4	3.35E+07	8.33E+02	6.67E+02
25-4	4.38E+07	1.10E+04	1.03E+04

Table 39: Physio– chemical & microbiological parameters measure on in water sample measured at the Houseboat

pH				
Sampling Date	Influent	Effluent	RO	Rain
14-2	7.28	6.75	9.07	7.9
28-2	7.31	6.74	9.45	7.42
14-3	7.95	6.94	8.91	7.93
28-3	7.33	6.83	9.14	7.81
11-4	7.14	6.81	9.15	7.4
25-4	7.4	7.0	9.2	
DO (mg/L)				
14-2	0.6	2.4	5.8	7.1
28-2	0.4	2.1	6.7	7.6
14-3	0.8	3.5	6.6	7.4
28-3	0.6	2.7	6.2	7.7
11-4	1.0	2.9	5.8	6.5
25-4	0.2	2.2	5.2	
Conductivity (µs/cm)				
30-1	1373	1265	344	2140
14-2	1629	1480	402	1762
28-2	1813	1635	581	370
14-3	2140	1762	370	140
28-3	1995	2030	422	172
11-4	2130	2050	460	192
25-4	2420	2230	475	
Temperature °C				
30-1				
14-2	11.7	9.7	20.2	11
28-2	13	11.2	14.5	11.5
14-3	15.4	13.3	19.9	16.8
28-3	16.4	13.6	22.1	15.8
11-4	19.3	15.4	22	16.4
25-4	21	18.7	20	NA
TSS (mg/L)				
30-1	208	94	NA	133
14-2	255	88	NA	43
28-2	211	127	NA	52
14-3	253	78	NA	17
28-3	414	60	NA	NA
11-4	565	11	NA	NA
25-4	438	128	NA	NA
NH4-N (mg/L)				
30-1	21.79	0.51	0.22	0.22
14-2	23.12	0.86	0.05	0.02
28-2	31.51	1.52	0.15	0.00
14-3	54.52	0.41	0.04	0.01
28-3	30.08	0.29	0.02	0.01
11-4	17.98	0.73	0.01	0.03
25-4	46.27	2.70	0.31	NA

NO3-N (mg/L)				
30-1	41	73	17	1
14-2	31	66	17	0
28-2	39	77	23	0
14-3	19	74	19	1
28-3	32	81	15	1
11-4	21	68	18	2
25-4	11	54	16	NA
TP (mg/L)				
30-1	6.6	3.0	NA	NA
14-2	9.2	5.6	NA	NA
28-2	10.6	7.4	NA	NA
14-3	15.9	9.0	NA	NA
28-3	15.7	13.5	NA	NA
11-4	22.1	16.4	NA	NA
25-4	19.4	17.1	NA	NA
COD (mg/L)				
30-1	244.3	37.7	NA	NA
14-2	92.0	62.0	NA	NA
28-2	125.3	88.7	NA	NA
14-3	139.7	46.3	NA	NA
28-3	103.0	83.0	NA	NA
11-4	113.0	53.0	NA	NA
25-4	159.7	69.7	NA	NA
BOD (mg/L)				
30-1	99.7	0.0	NA	NA
14-2	58.0	4.5	NA	NA
28-2	57.0	5.1	NA	NA
14-3	35.3	3.7	NA	NA
28-3	36.2	3.6	NA	NA
11-4	28.3	2.8	NA	NA
25-4	52.4	3.3	NA	NA
E.coli (CFU/100 ml)				
30-1	0.00E+00	0.00E+00	0.00E+00	0.00E+00
14-2	3.33E+04	0.00E+00	0.00E+00	0.00E+00
28-2	6.00E+03	0.00E+00	0.00E+00	0.00E+00
14-3	9.67E+03	0.00E+00	0.00E+00	0.00E+00
28-3	2.07E+04	0.00E+00	0.00E+00	0.00E+00
11-4	8.00E+03	0.00E+00	0.00E+00	0.00E+00
25-4	6.67E+02	0.00E+00	0.00E+00	NA
Total coliform (CFU/100 mL)				
30-1	3.47E+06	1.23E+05	0.00E+00	1.18E+05
14-2	2.06E+07	5.13E+04	0.00E+00	2.00E+03
28-2	3.53E+07	3.18E+05	0.00E+00	3.33E+02
14-3	1.45E+07	3.93E+04	0.00E+00	8.33E+02
28-3	6.12E+06	7.90E+04	0.00E+00	1.67E+02
11-4	9.21E+06	4.83E+03	0.00E+00	3.83E+03
25-4	9.03E+04	6.67E+03	0.00E+00	NA