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PERFORMANCE OF CONSTRUCTED TREATMENT WETLANDS: MODEL-BASED EVALUATION AND IMPACT OF OPERATION AND MAINTENANCE

WERKING VAN AANGELEGDE ZUIVERINGSMOERASSEN: MODELGEBASEERDE EVALUATIE EN IMPACT VAN BEDRIJFSVOERING EN ONDERHOUD

door:

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Thesis submitted in fulfillment of the requirements for the degree of Doctor (Ph.D) in Applied Biological Sciences: Environmental Technology

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Water is H_2O, hydrogen two parts, oxygen one, but there is also
a third thing, that makes it water and nobody knows what that is.
D. H. Lawrence (1885—1930), British novelist.

If there is magic on this planet, it is contained in water.
L. Eisely, The Immense Journey, 1957.

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Thesis submitted in fulfillment of the requirements
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Front: subsurface-flow constructed wetland at Butlers Marston (UK)
Back: free-water-surface constructed wetland at Põltsamaa (Estonia)

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Six years of PhD work to be summarised in a 40 minute presentation, six years of interacting and enjoying life with so many people to be summarised in just a couple of pages. The hardest part truly comes with the finish in sight.

Reaching the finish certainly has been a ride with many unexpected turns. Had the original subject I chose for my MSc thesis not been cancelled, I probably would have ended up doing research on algae. Had I not attended that specific New Year reception, I probably would have dropped out of university. And had I not been so fortunate to have the support and friendship of my promoters, family, colleagues and friends, this ride would have been a very short one!

I firstly would like to acknowledge both my promoters, Prof. dr. Niels De Pauw and Prof. dr. ir. Peter Vanrolleghem. It was a pleasure to learn from and cooperate with two experts in their respective fields although I have to admit that it was sometimes also a burden to reconcile both worlds. But that is probably what interdisciplinary research is all about. I especially appreciate their trust in me by giving me a large degree of freedom to define and carry out my research. I am also most grateful to Prof. De Pauw for the many opportunities he gave me to visit foreign countries, to establish contacts with researchers from abroad and to experience the cultural diversity in the world.

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never forget our WETPOL adventure. I sincerely hope we will find each other again for new editions.

When I was a first year Bio-engineering student, one particular phrase during the speech of the Dean got my attention: "... studying at our Bio-engineering Faculty is 50% hard work, and 50% social life". Although I am not sure I always managed to keep the balance, in essence he was right. AECO walks, laboratory Christmas parties, BIOMATH springwalks, BIOMATH parties, BIOMATH cocktails, BIOMATH weekends, ANAFYS bowling night, Academic Club barbeque and so many other activities certainly gave an extra dimension to the word colleague.

Without money no research, especially as a non-funded assistant, so I would like to express my gratitude to Aquafin for financially supporting part of this research and for their generosity in sharing data and know-how. I am also indebted to Severn Trent Water Ltd for accommodating me during a two-months research stay, it was a priceless experience. And finally my appreciation goes towards the Scientific Research Funds Flanders (FWO) for funding my research stays in the UK and Bulgaria and for several conference grants. I found it very motivating to meet and share ideas with other wetland scientists.

Which brings me seamless to my colleagues from abroad whom I met among others through NecoV and the IWA Wetlands Specialist Group. It was a real pleasure to meet the persons behind the author names and if anything encouraged me to keep going with this PhD, it were those international, intercultural nights during the conferences! I especially want to acknowledge dr. Joan García from the Technical University Barcelona for his contribution to the model part of this thesis and for the inspiring scientific and other discussions we had during his stay in Ghent. Finally also a word of appreciation for my new colleagues at UNESCO-IHE for their warm welcome and for giving me some much needed time to finish this thesis.

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Natalie, Joachim, Anke, Bram, Kevin, Abby, Isabelle and Melanie, it was a pleasure to work with you and I hope it has been a fruitful experience.

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And last, but as always certainly not least, a special word of thanks to my parents for their love and support. I now know that being a supporter is much harder and far more stressful than being a player.

This adventure ends here, new adventures can begin, or quoting from one of my favourite movies: second star to the right, and straight on till the morning ...

Diederik, 12 November 2005

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Chapter 1

An introduction to constructed wetlands technology

1.1. HISTORY AND APPLICATIONS OF CONSTRUCTED WETLANDS

Although wastewater treatment is a relatively young technology, avoiding septic conditions by evacuating wastewater from human settlements has a considerable history. Angelakis *et al*. (2005) for instance describe the advanced sewer system at Knossos (Crete), dating from the second millenium B.C. Bertrand-Krajewski (2002) elaborates on the 'Cloaca Maxima' in ancient Rome and on early sewer systems in London and Paris, while Poulussen (1987) strikingly depicts the development of water sanitation in Antwerp (Belgium). Once outside the city boundaries, the wastewater was often conveyed to nearby natural wetlands which at that time were considered as useless lands (Vymazal, 1998a; Kadlec *et al*., 2000a).

From the fifties and sixties of the past century on, however, ecologists started to realise the value of these wetlands and initiated many studies on this topic. They more or less unintendedly discovered the purification capacities of these wetlands which set off the development of constructed wetland technologies. The first relevant research seems to be the one by Dr. K. Seidel at the Max Planck Institute in Plön (Germany) as early as 1955, but it was not published in English before 1976, thus hindering dissemination of the acquired knowledge. Her research also seemed heavily criticised since the investigations and calculations were mainly aimed at nutrient removal through plant uptake which would require a regular harvesting regime and very large surface areas (Vymazal, 1998a).

Due to a growing 'green awareness' in the seventies, the practice of dumping wastewater in natural wetlands was abandoned in favour of constructed wetlands (CWs). Another positive boost was possibly due to the first energy crisis in 1973. Energy-devouring technologies all of a sudden lost their attractiveness to the advantage of the low-energy ones. Indeed, natural systems for wastewater treatment are characterised by the use of renewable, naturally occurring energies such as solar and wind energy, as opposed to conventional treatment technologies which are highly dependent on non-renewable fossil fuel energies. The above-mentioned stimuli soon outweighed the classic distrust against new technologies and, from then on, constructed wetlands development took an exponential growth. Kangas (2004) summarised this early period and called it the 'big bang model' of constructed wetlands' development (Fig. 1.1).

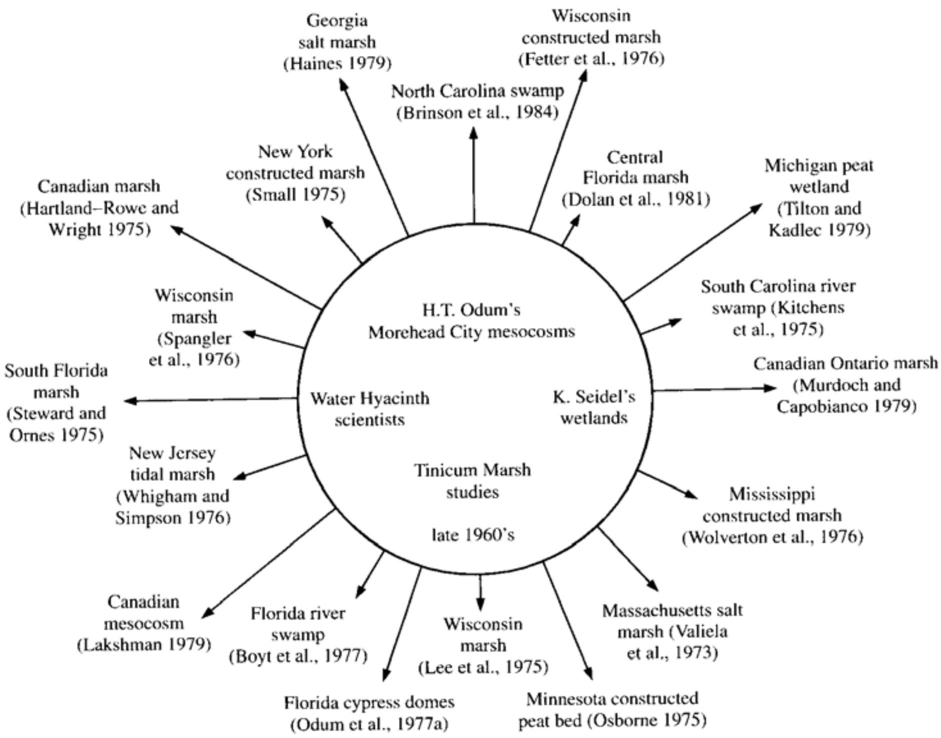

Figure 1.1. The "big bang" model of a technological explosion of early treatment wetlands projects (after Kangas, 2004).

Once past this initial period of optimism and enthusiasm (seventies), the next decade (eighties) was characterised by precaution and scepticism due to the discovery of several drawbacks of the technology and failures of some prototypes. Further research solved most of these problems and led to the maturity of the technology in the nineties. The logical last step is commercialisation which has really boosted in the latest years (Kangas, 2004).

Constructed wetlands nowadays have many applications, ranging from the secondary treatment of domestic, agricultural and industrial wastewaters to the tertiary treatment and polishing of wastewaters treated by means of activated sludge plants and even to the treatment of stormwaters. Table 1.1. summarises some specific case studies that were conducted to evaluate the potential of CWs for treating certain wastewater flows.

Table 1.1. Selected case studies with constructed wetlands.

Wastewater type	**Reference**
Domestic wastewater	
Secondary	De Wilde (2001); De Moor (2002); Story (2003)
Tertiary	Meuleman (1999); Cameron *et al*. (2003)
Domestic greywater	Dallas *et al*. (2004)
Pig manure	Hill & Sobsey (2001); Meers *et al*. (2005)
Dairy wastewater	Geary & Moore (1999); Mantovi *et al*. (2003)
Agricultural runoff	Comin *et al.* (1997)
Motorway runoff	Hares & Ward (2004); Pontier *et al*. (2004)
Wastewater from schools	Davison *et al*. (2002)
Wastewater from breweries	Billore *et al*. (2001)
Aquaculture reject water	Comeau *et al*. (2001); Schulz *et al*. (2003)
Surface water	Braskerud (2002); Coveney *et al*. (2002)
Landfill leachate	Urbanc-Bercic (1998); Rousseau *et al*. (2004a)
Acid mine drainage	Kalin (2004); Whitehead *et al*. (2005)
Stormwater	Green *et* al. (1999); Carleton *et al*. (2001)
Sludge dewatering	De Maeseneer (1997)
Abattoir wastewater	Rivera *et al*. (1997)
Heavy metal laden wastewater	Cheng *et al*. (2002a)
Pesticides and herbicides	Cheng *et al*. (2002b); Runes *et al*. (2003)
Acidic coal pile runoff	Collins *et al*. (2004)
Oil-contaminated water	Ji *et al*. (2002)
Volatile Organic Compounds	Kassenga *et al*. (2003)
Perchlorate contaminated water	Tan *et al*. (2004)
Woodwaste leachate	Tao & Hall (2004)

1.2. TYPES OF CONSTRUCTED WETLANDS AND GENERAL LAY-OUT

Wetlands can be very generally defined as *transitional environments* between dry land and open water or between terrestrial and aquatic ecosystems (Vymazal, 1998a). The different types of natural systems for wastewater treatment correspond with the different ecosystems along the land-water gradient, starting from the land-side with high-rate infiltration fields, overland flow systems, constructed wetlands and finally waste stabilisation ponds or lagoons.

The following classification only considers the middle range of ecosystems, i.e. the so-called constructed wetlands, and is based on the internationally accepted International Water Associations' Scientific and Technical Report on Constructed Wetlands for Pollution Control (Kadlec *et al*., 2000b). The various types are differentiated by water flow mode and plant species characteristics.

- Above-ground water: free-water-surface (FWS) constructed wetlands
  - with emergent macrophytes or helophytes, e.g. *Phragmites australis* (common reed), *Typha* spp. (cattails), *Scirpus* spp. (bulrushes) – Fig. 1.2 panels Ia, Ib, Ic
  - with floating-leaved, bottom-rooted macrophytes, e.g. *Nymphaea* spp. (water lilies), *Nelumbo* spp. (lotus) – Fig. 1.2. panels IId, IIe, IIf
  - with free-floating macrophytes, e.g. *Eichhornia crassipes* (water hyacinth), *Lemna* spp. (duckweed) – Fig. 1.2. panels IIg, IIh
  - with submersed macrophytes, e.g. *Elodea* spp. (waterweed), *Myriophyllum* spp. (water milfoil) – Fig. 1.2. panels IIIi, IIIj
  - with floating mats, e.g. *Phragmites australis* (common reed), *Typha* spp. (cattails), *Glyceria maxima* (giant sweetgrass) – Fig. 1.2. panel IV

- Below-ground water: subsurface-flow (SSF) constructed wetlands
  - horizontal-flow systems (HSSF), planted with emergent macrophytes or helophytes, e.g. *Phragmites australis* (common reed), *Typha* spp. (cattails), *Scirpus* spp. (bulrushes) – Fig. 1.2. panel Va
  - vertical-flow systems (VSSF), planted with emergent macrophytes or helophytes, e.g. *Phragmites australis* (common reed), *Typha* spp. (cattails), *Scirpus* spp. (bulrushes) – Fig. 1.2. panel Vb

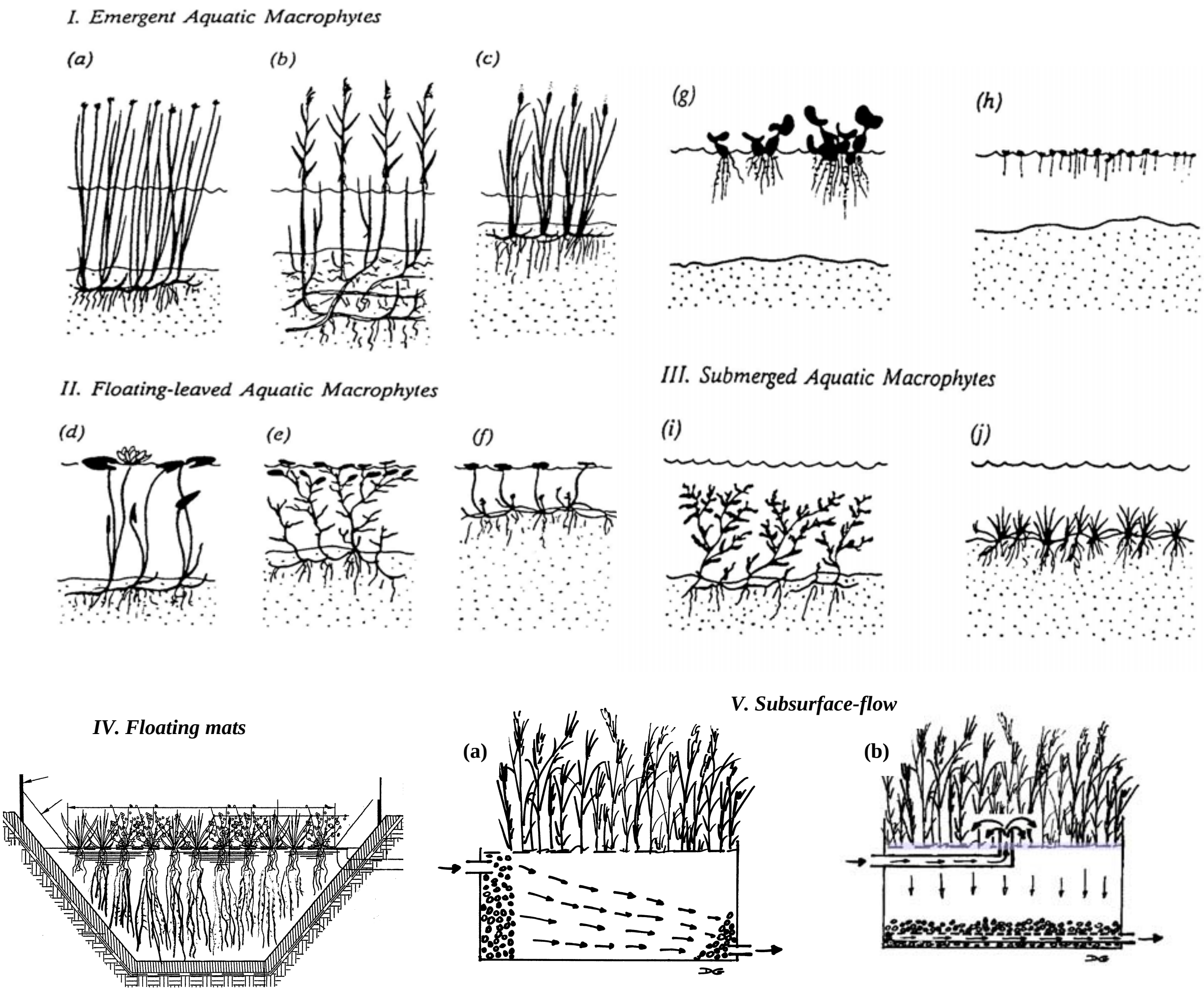

Figure 1.2. Schematic representation of different types of constructed wetlands (I, II, III after Vymazal *et al*., 1998b; IV after Van Acker *et al*., 2005; V after De Wilde and Geenens, 2003).

Generally speaking, most systems with above-ground water flow consist of a relatively shallow basin (depth between 0.3 and 1.8 meters), isolated from the groundwater by means of a plastic liner or by a local clay layer. Length-width ratios ≥ 2 are to be preferred in order to obtain near plug-flow conditions. The inlet distribution and effluent abstraction system should run along the entire width of the basin to avoid short-circuiting and the existence of dead volumes. When using free-floating macrophytes, floating barriers are often used to avoid the piling up of plants in one corner due to wind action.

Treatment wetlands with horizontal below-ground flow also consist of a shallow (0.5 – 0.8m deep) basin, isolated from the groundwater and usually filled with gravel although in some cases local soil has been used. For the inlet and outlet zone, coarser gravel is usually applied to allow a better spreading respectively collection of wastewater. The treated wastewater is evacuated by means of a drainage tube at the bottom of the wetland. An appropriate choice of filter material (c.q. hydraulic conductivity) and a correct length-width ratio are indispensable to avoid above-ground water flow, which has a detrimental effect on treatment performance and can cause odour and insect nuisances.

Finally, vertical below-ground flow systems usually consist of one or more filter layers of coarse sand and/or gravel with a total depth between 0.6 and 1.0 meter. Wastewater is preferably spread equally over the top surface, then drains through the filter layers and is collected at the bottom by means of drainage tubes. Loading often happens intermittently, i.e. batch-wise. Choosing the right filter material is a trade-off between high respectively low hydraulic conductivities, i.e. less prone to clogging versus a longer hydraulic retention time.

Obviously, these different types do not necessarily function as stand-alone treatment plants but can be combined with each other or even with other low-tech or high-tech wastewater treatment units in order to exploit the specific advantages of the different systems. The quality of the effluent appears to improve with the complexity of the facility (Vymazal *et al*., 1998b).

A further distinction is made between engineered wetlands and constructed wetlands (de-Bashan and Bashan, 2004), although these terms are often used interchangeably. A constructed wetland usually refers to passive flow systems whereas an engineered wetland is a wetland that can be changed at will, i.e. operators can manipulate process conditions and operations according to conditions of both climate and wastewater.

1.3. PROCESSES IN CONSTRUCTED WETLANDS AND INFLUENCING FACTORS

1.3.1. Processes

Constructed wetlands are capable of removing and/or converting a range of pollutants such as organic matter (BOD, COD), suspended solids, nitrogen, phosphorus, trace metals, pesticides and pathogens. This is accomplished by a vast array of processes that are complex physical, chemical and biological interactions between water, substrate, filter material, macrophytes, litter and detritus, and micro-organisms (Table 1.2.). An introductory summary is given below, adapted from the comprehensive overview in Kadlec *et al*. (2000c). For SSF systems, more information is available in Chapter 3 of this work.

Suspended solids are mainly removed by physical processes such as sedimentation and filtration. Filtration occurs by impaction of particles onto the roots and stems of the macrophytes or onto the soil/gravel particles in SSF systems. For FWS systems, most of the SS removal occurs within the first meters, giving rise to a ‘bank’ of sludge that can hinder the water flow. Subsurface-flow systems can clog when too many pores become filled with particulates.

Dissolved organic matter first diffuses into the biofilms that colonise plant stems and roots, filter particles and basin walls. Depending on the available oxygen, it is then degraded in an aerobic, anoxic or anaerobic way. *Particulate organic matter*, when biodegradable, is normally mineralised into dissolved components after sedimentation or filtration.

Table 1.2. Removal mechanisms in constructed wetlands (after Vymazal *et al*., 1998b)

Mechanism	Contaminant affected								Description
	SS	CS	BOD	N	P	HM	RO	B&V	
PHYSICAL									
sedimentation	P	S	I	I	I	I	I	I	Gravitational settling of solids
filtration	S	S	I	I	I	I	I	I	Particles filtered mechanically as water passes through substrate, roots and rhizomes or fish
adsorption		S							Interparticle attractive force (van der Waals force)
volatilisation				S					Volatilisation of NH_3 at high pH
CHEMICAL									
precipitation					P	P			Formation of or co-precipitation with insoluble compounds
adsorption					P	P	S		Adsorption on substrate and plant surfaces
decomposition							P		Decomposition or alteration of less stable compounds by phenomena such as UV irradiation, oxidation and reduction
BIOLOGICAL									
bacterial metabolism		P	P	P			P		Removal of colloidal solids and soluble organics by suspended, benthic and plant-supported bacteria. Bacterial nitrification and denitrification
plant metabolism							S	S	Metabolism of organics by plants. Root excretion may be toxic to organisms of enteric origin
plant absorption				S	S	S	S		Under proper conditions significant quantities of these contaminants will be taken up by plants
Natural die-off								P	Natural decay of organisms in an unfavourable environment

SS = settleable solids, CS = colloidal solids, HM = heavy metals, RO = refractory organics, B&V = bacteria and viruses

P = primary effect, S = secondary effect, I = incidental effect (effect occurring incidental to removal of another contaminant)

Nitrogen removal is mainly accomplished by the successive microbial pathways ammonification, nitrification and denitrification. Plant uptake and consequent harvesting is only important in low-loaded systems. Some nitrogen can be permanently stored in the recalcitrant fraction of the detritus layer. NH_3 volatilisation can occur but is only significant at high pH, i.e. above 9.

Phosphorus is biologically removed by plant uptake. Again, the amount that can be removed through harvesting of the above-ground plant parts is only significant in low-loaded systems. Periphyton and micro-organisms also take up P but most of it is released again after cell death. The main removal mechanisms are adsorption to the filter and/or soil particles, adsorption to the detritus layer and precipitation with certain metals such as Fe, Al, Ca and Mg.

Viruses seem to be effectively removed by adsorption onto the soil or detritus. Possibly the time spent outside the host organism also plays a major role. *Bacteria* are reduced by sedimentation, chemical reactions, natural die-off, predation by zooplankton, nematodes and lytic bacteria and attacks by bacteriophages. Certain wetland plants and micro-organisms are also known to synthesise antibiotics that are released into the root zone. *Parasites* such as helminth eggs can also be effectively removed through sedimentation and adsorption.

Trace metals associated with particulate matter are removed by sedimentation and filtration. Adsorption onto the matrix surface and organic material is considered the main removal mechanism for dissolved trace metals. Cation exchange with carboxyl functional groups in dead or live plant tissue is a second important removal mechanism. Another removal mechanism of trace metals being largely dependent on redox conditions, is precipitation as insoluble salts, mainly sulphides and (oxy)hydroxides. Most helophyte plant species also accumulate trace metals in their root system whereas some floating and submerged species have been described to accumulate metals to a greater extent in their harvestable plant tissue.

1.3.2. Influencing design parameters

Probably the most important design parameter is the *hydraulic retention time* (HRT). Constructed wetlands are extensive systems that entirely depend on natural energy inputs such as sunlight and wind. They therefore require a large surface area to absorb these energy fluxes and a sufficient hydraulic residence time for the processes to take place.

Isolation from the groundwater by means of a *plastic liner* or *clay layer* is absolutely necessary to prevent groundwater contamination on the one hand, and to avoid groundwater infiltration on the other hand. Both fluxes can substantially influence the hydraulic residence time and therefore the treatment performance.

The *plant species* choice is based on a range of criteria. They should firstly be able to flourish under the local climatic conditions. A high biomass production is preferable when one intends to export nutrients from the system by harvesting. The more extensive the root system, the better the filtrative capacities and the more surface is available for biofilm development. Finally, they should be able to withstand hydraulic and pollutant shock loads.

For SSF systems, an appropriate choice of the *filter material* is extremely important to avoid clogging, to ensure a sufficient hydraulic conductivity and to provide enough sorptive capacity, especially for P removal.

1.3.3. Influencing external parameters

Temperature has a major impact on microbiological process rates and obviously on plant growth as well. Especially nitrogen removal seems to be almost completely inhibited at temperatures below 4 °C. Kadlec and Knight (1996g, 1996h) use an Arrhenius equation to express temperature dependency. Temperature factors (θ) for BOD, SS, TP and FC are given as 1.0, meaning removal of these variables is not temperature dependent. This can be explained by the fact that most related processes are physical or chemical in nature and not (micro)biological. TN on the contrary has a temperature factor of 1.05, meaning that the removal efficiency is lowered by 39% when the temperature decreases from 20 °C to 10 °C.

Another important factor that affects the microbiological processes is *pH*. The optimal range fluctuates somewhat for the different processes but in general varies between 7.0 and 8.5.

Mass removal rates seem in most cases to be positively correlated with the *mass loading rates*, i.e. higher influent loads result in better treatment performance, up to a certain level of course (Ayaz and Akça, 2001). It is clear from the latter observation that the removal rates of tertiary treatment wetlands are typically lower than those of secondary treatment ones.

1.4. ECONOMIC FACTORS

Constructed wetlands are being promoted as a sustainable, low-investment and low-maintenance cost technology. Major expenses usually are land acquisition, earth moving, plastic liners to prevent groundwater contamination or infiltration and the filter material in case of SSF systems. However, after its functional life, the land can be readily made available for other purposes and therefore certain authors exclude this cost from the balance.

1.4.1. Costs

All costs given below should be interpreted with caution, for a number of reasons. Firstly, it is not always clear from the original sources which components are included, i.e. the wetland costs *sensu stricto* or also the costs for sewer construction, fencing, buildings etc. Secondly, many authors do not mention if taxes/VAT are included and at what rate. Thirdly, depreciation costs are not always clear and finally, inflation and fluctuating exchange rates can give a wrong idea about current costs.

Kadlec and Knight (1996a) summarised capital costs and operating costs, indifferent of treatment level or wastewater type, as given in Table 1.3.

Table 1.3. Range of capital and operating costs of constructed wetlands (after Kadlec and Knight, 1996a).

	Area (ha 1000m^{-3} d^{-1})	**Capital cost (1000 US$ ha^{-1})**	**Capital cost (US$ m^{-3} d^{-1})**	**O&M cost (US$ m^{-3})**	**O&M cost (US$ ha^{-1} year^{-1})**
Floating aquatic macrophytes	0.7 – 5	270	500 – 1,000	0.12 – 0.14	9,490 – 67,786
Wetlands	0.5 - 20	25 - 250	500 – 1,000	0.03 – 0.09	1,095 – 43,800

Capital costs in Table 1.3. exclude the more extreme cases, e.g. 4,741 US$ ha^{-1} for the Mt. View Marsh FWS CW (California, USA) or 1,731,936 US$ ha^{-1} for the Mandeville HSSF CW (Louisiana, USA) (Kadlec and Knight, 1996k). Indeed, capital costs are highly dependent on the local situation, i.e. soil type, groundwater table height, terrain slope, distance from settlement, discharge criteria, climate etc. Cooper and Breen (1998) state investment costs for secondary treatment wetlands between 120 – 480 € PE^{-1} whilst for tertiary treatment CWs this only amounts to 36 – 120 € PE^{-1}. Another important factor usually is the economy of scale: larger wetlands tend to be relatively cheaper per PE or per m^3 of wastewater treated. Indeed, for single-household systems, Haberl *et al*. (2003) mention an average investment cost of 1,000 € PE^{-1}, with a significant proportion made up by the primary treatment unit. One uncertainty is the 'removal' cost of the system after its functional life, now estimated around 20 years. Especially dumping or cleaning of saturated filter materials of SSF wetlands could result in a significant extra cost.

Operation and maintenance costs are rarely given in literature, but one median O&M cost for FWS CWs is mentioned in the order of 1000 US$ ha^{-1} year^{-1} (Kadlec and Knight, 1996g) whereas O&M costs for SSF CWs are estimated between 2500 and 5000 US$ ha^{-1} year^{-1} (Kadlec and Knight, 1996h). Merz (2000) reveals a scale advantage for larger wetlands: O&M costs of Australian wetlands of > 5 ha are estimated around 1500 AS$ ha^{-1} year^{-1} whereas for wetlands < 5 ha costs can be up to a factor 10 higher. This trend can also be found for very small CW as Haberl *et al.* (2003) report O&M costs in Austria of 300, 200 and 150 € PE^{-1} year^{-1} for CWs of 5, 10 and 20

PE respectively. What are the major O&M expenses? Energy consumption, if any, is usually limited to pumping and represents only a minor cost since most wetlands are designed to function gravitationally. Chemicals are rather rarely applied. Exceptions are the addition of materials with a high P-sorption capacity in SSF wetlands and the use of pesticides to eliminate plant pests such as lice or mosquitoes. Sludge production is minimal in tertiary systems. Maintenance costs are therefore mainly labour costs for site inspection, effluent sampling and control, cleaning of distribution systems and pumps, weed control, plant harvesting etc.

1.4.2. Benefits

Treated effluent can be reused for irrigation of agricultural crops, depending on its quality. Other applications are watering of gardens, golf courses, public parks etc. Merz (2000) for instance states that irrigation reuse is practised with about 30% of Australian CWs. Effluent can also be reused for flushing toilets, for cleaning purposes, as cooling water after desalination (Peng *et al*., 2004) and as a reliable water supply for natural wetlands or nature reserve areas (Worrall *et al*., 1997; Sala *et al*., 2004). A last option is to use the effluent for aquacultural purposes, with fish production for food or feed or even duck culture (Polprasert and Koottatep, 2004).

Harvested plant biomass can possibly create an extra income. Indeed, certain plant species have commercial value, some as ornamental plants, others as raw material. Mulching and composting of harvested plants can for instance yield soil additives, pulping of plants provides fibers and silaging produces livestock fodder (Polprasert and Koottatep, 2004). A pond-wetland system in Thailand generates some income by selling ornamental plants (golden torch and bird of paradise - *Heliconia* spp.) at about 0.2 US$ per flower (Shipin *et al*., 2004). El Hafiane and El Hamouri (2004) describe the use of *Arundo donax* for tomato crop production and for the creation of artisanal objects, generating an annual income of 1750 - 2900 US$ per ha per year (price of one plant about 0.007 US$). Calla lilly (*Zantedeschia aethiopica*) was demonstrated to grow well on wastewater and seems to have a high market value in Mexico (Bachand and Horne, 2000). From the above examples, it is clear that the practice of using plants for commercial purposes takes place mostly in developing countries where people try to optimise the benefits of constructed wetlands. In developing countries, a paradigm shift still needs to take place.

Cicek *et al.* (2004) investigated the possibility of using harvested plant biomass from a natural wetland to generate power. Different technologies were evaluated and yielded considerable amounts of energy. Cogeneration of heat is one possible additional benefit, greenhouse gas credits (carbon sequestration, renewable energy sources) a second one. Bolton (2004) also mentions this possibility of obtaining carbon credits from biomass and peat formation in a constructed *Melaleuca* wetland.

When combining wetlands with ponds, aquaculture can be done quite succesfully. An integrated pond-wetland system in China yearly yields between 20000 – 30000 kg fish. Unfortunately, no data are given on the area of this system. Together with large quantities of commercialisable plants like duckweed and reed, this results in significantly lower operational costs. The effluent of this system is used for irrigation during dry periods (Peng *et al*., 2004).

Another benefit includes the creation of a new habitat for flora and fauna. Knight *et al*. (2000a) summarise data from the North American treatment wetlands DataBase (NADB) concerning sightings of mammals, birds, amphibians, reptiles, fish and invertebrates and vegetation mapping surveys. Initial concerns about bioaccumulation of certain pollutants and spreading of diseases via visiting fauna seemed in most cases premature. Very few treatment wetlands have been specifically designed to contribute to wildlife conservation. According to Connor and Luczak (2002) there are indeed many obstacles like a lack of understanding of conservational needs and ecological principles among engineers, the additional costs entailed, lack of comprehensive design manuals and a lack of obviously tangible benefits to local communities. Several positive examples are summed up by Connor and Luczak (2002) as counter arguments. The Western Treatment Plant of Melbourne for example (10850 ha with lagoons, land infiltration and grass filtration) has been included in the Ramsar convention as a wetland of international importance for bird conservation. Other examples from the ornithological literature include the Aisleby sewage farm in Bulawayo, Zimbabwe, the Phakalane Sewage Ponds in Gabarone, Botswana, the Arcata wetlands, California and the Al-Ansab sewage treatment plant in Muscat, Oman.

Knight *et al*. (2000a) finally mention education (nature study), exercise activities (walking, jogging) and recreational harvest (hunting, trapping) as other positive contributions of CWs. Gearheart and Higley (1993) add picnicing, relaxing and art (photography, painting) to this list. Such additional benefits have seldomly be economically valued. Knight *et al*. (2000a) only describe for a number of wetlands the 'human use days', expressing the total amount of time spent by humans for the above-mentioned activities. The 61 ha large Arcata wetland facility in California has 5 miles of foot trails and attracts more than 130000 visitors each year (Gearheart and Higley, 1993). Carlsson *et al.* (2003) conducted a choice experiment among citizens of Southern Sweden and found that biodiversity and walking facilities are the two greatest contributors to welfare, while a fenced waterline and introduction of crayfish decrease welfare.

1.5. PROBLEM STATEMENTS AND THESIS OUTLINE

The above literature review highlights that constructed wetlands are a versatile and cost-effective technology that is suitable for removing several pollutants from different types of wastewater, at varying loading rates and under a range of climatological conditions. Chapter 2 further investigates these statements based on the available experience with CW technology in Flanders (Belgium). The relevant legislation is briefly introducted after which an overview is given of removal efficiencies and their seasonal variations for the different types of CWs in operation. Attention is also paid to investment costs, area demand and maintenance efforts. Three issues surfaced from this survey which are further treated in this thesis:

1. **Sampling frequencies seem to be irregular and generally low-frequent, thus providing very few insights in the dynamics of CWs. Chapters 3 and 4 therefore focus on process analysis.**

   Chapter 3 introduces the subject with a detailed description of processes occuring in subsurface-flow CWs. This choice was based on the fact that these technologies exhibited better results according to the overview in chapter 2. Experimental data obtained from a two-stage pilot-scale CW in Aartselaar (Belgium) are then summarised in Chapter 4 and used (i) to illustrate the different processes, (ii) to distinguish between short-term and long-term dynamics, (iii) to reveal seasonal variations and (iv) to assess the impact of different loading rates on treatment performance.

2. **Fluctuations in treatment performance can be due to a range of factors and demand for a reliable framework that is able to predict the effects of steering variables on pollutant removal efficiencies. Chapters 5 to 8 are therefore devoted to model-based design of constructed wetlands.**

   Although this 'green' wastewater treatment technology has been applied now for several decades, few quantitative research has been done on the complex web of processes inside such man-made ecosystems. Indeed, most studies adopted a black-box approach where low-frequent or seasonally-averaged data were applied to feed the empirical models, thereby largely ignoring the intrinsic variability of such treatment systems. Prominent researchers concur that unraveling the black box is one of the priorities for the future evolvement of the technology:

   R. Kadlec (in Cole, 1998): *"We've got a huge, functioning mess called wetlands out there with all sorts of interesting things going on inside it. But we do not have enough information about what goes on inside the system. We have a solid foundation of empirical understanding, but to advance our knowledge, we need to understand the internal processes that lead to the observed performance."*

R. Gearheart (in Cole, 1998): "*Basically, all we know is that they work … But if you want to be able to say, for example, what happens if you double the loading rate, we're not there yet. We can not model it.*"

Chapter 5 first reviews state-of-the-art model-based design of horizontal subsurface-flow wetlands and highlights the need for dynamic, mechanistic models. This technology was specifically chosen as it seems to be the most wide-spread one within the EU. In Chapter 6, such an existing mechanistic model is described and recalibrated by means of data from a 47 PE two-stage HSSF CW in Saxby (UK). Unsatisfactory model fits gave rise to the development of a new mechanistic model based on Activated Sludge Model N° 1 that is presented in Chapter 7. This model is consequently calibrated and validated with data from two pilot-scale horizontal subsurface flow constructed wetlands in Chapter 8.

3. **Even well-designed CWs can fail when denied adequate maintenance. Chapters 9 and 10 therefore intend to refute the 'build-and-forget' attitude and give arguments for minimum maintenance efforts.**

   Chapter 9 introduces the subject by summarising available knowledge on operation and maintenance. In Chapter 10, the results from a survey on 12 stormwater treatment wetlands are discussed and the effect of proper maintenance on the asset life of these CWs is evaluated.

Chapter 11 finally unifies the outcomes of this thesis, compares and discusses the results from the different chapters and provides some general conclusions. Some suggestions for further research are also given.

Chapter 2

Constructed wetlands in Flanders: a performance analysis

An earlier version of this chapter was published as:

ROUSSEAU D.P.L., P.A. VANROLLEGHEM and N. DE PAUW (2004). Constructed wetlands in Flanders: a performance analysis. Ecological Engineering 23(3), 151-163.

2.1. ABSTRACT

During the last decade, the number of constructed wetlands in Flanders (Belgium) increased exponentially. Extensive data collection resulted in a database of 107 constructed wetlands that was used to evaluate certain trends and treatment performances. Design sizes vary between 1 and 2,000 Population Equivalents, with the majority of reed beds having a size smaller than 500 PE. Most reed beds are used as single treatment units, although they are sometimes also combined with other reed beds or even conventional systems. The main purpose is to treat domestic and dairy wastewater. Average removal efficiencies were lowest with free-water-surface reed beds (Chemical Oxygen Demand 61%, Suspended Solids 75%, Total Nitrogen 31% and Total Phosphorus 26%). The best overall performance was obtained with vertical-flow wetlands (COD 94%, SS 98%, TN 52%, TP 70%), except for total nitrogen removal where combined reed bed systems even did better (COD 91%, SS 94%, TN 65%, TP 52%). The different types of constructed wetlands all showed a more or less pronounced seasonal performance, especially for nutrient removal. Despite the considerable removal achieved, the effluent nutrient concentrations of many systems remain too high and entail a tangible danger of eutrophication.

2.2. WATER QUALITY MANAGEMENT IN FLANDERS

Belgium, now a federal state, consists of three regions: the Flemish Region, the Walloon Region and the Region of Brussels-Capital. Flanders is the northern-most region, located in between the North Sea, the Netherlands, France, Germany and Belgium’s Walloon Region. Its total surface area is 13,522 km^2, inhabited by nearly 6 million people (Administration of Planning and Statistics, 2003).

During a number of state reforms, a number of powers and responsibilities has been transferred to the regions, among others environmental legislation and enforcement and more particularly water management. Since 1990, domestic wastewater collection and treatment in the Flemish region is mainly the responsibility of a single company named

Aquafin NV, with 51% of its shares owned by the Flemish government, 20% by Severn Trent Water International and 29% by various institutional investors (Aquafin NV, 2002). Wastewater treatment plants with a design capacity smaller than 2,000 Population Equivalents (PE) may however also be planned and constructed by several government agencies, municipalities and even private persons (if < 20 PE).

Because the EU Directive 91/271 on Urban Wastewater obliged Member States to treat the wastewater of all agglomerations larger than 10,000 inhabitants before 31/12/1998 and because in 1990 only 30% of the domestic wastewaters in Flanders were being treated, it was decided to concentrate on the large-scale projects in order to catch up as quickly as possible. Small-scale projects were not entirely neglected, but certainly had no priority.

This approach has undoubtedly been successful until now. By 2002, 57% of all domestic wastewater was being treated, resulting in a significant load reduction of organic substances and nutrients into the Flemish surface waters (Aquafin NV, 2003a). Together with other emission reduction measures, this has generally resulted in a shift from extremely bad and very bad surface water quality towards a moderate water quality, as indicated by physico-chemical as well as biological variables. However, for the majority of the monitoring sites, the water quality still does not meet the standards and has in some cases even deteriorated (MIRA-T, 2003).

At the current levels of technology and investment rates, Aquafin NV estimates that up to 20% of the Flemish population will never be connected to a large-scale wastewater treatment plant and will have to treat its wastewater by means of small-scale or even individual treatment systems (Vandaele *et al*., 2000). One of the main reasons for this is the lack of efficient town and country planning in the past, which has led to very dispersed locations of housing, resulting in extremely high investment costs for connection to centralised sewer systems.

Several small-scale wastewater treatment techniques can be applied, of which constructed wetlands are gaining popularity. Recent comparative studies between mechanical and plant-based single-household systems revealed that the latter ones are more efficient in an economical as well as an ecological way (Rausch *et al.*, 2000; al Jiroudi and Barjenbruch, 2004).

2.3. EFFLUENT STANDARDS IN FLANDERS

In the Environmental Legislation of Flanders (VLAREM II, 2005), 2004 proves to be a transition year with regard to small-scale wastewater treatment systems (20 - 2,000 PE). Indeed, before 1 January 2004, the relevant effluent standards were not stringent at all: 250, 50 and 60 mg l^{-1} for COD, BOD and SS respectively. No nutrient standards were imposed. Recently, somewhat more strict standards were issued, i.e. 125 mg l^{-1} COD, 25 mg l^{-1} BOD, and 35 mg l^{-1} SS for treatment plants with a capacity between 500 and 2000 PE or 60 mg l^{-1} SS for treatment plants with a capacity below 500 PE. Another novelty is the requirement for minimum removal efficiencies (on a yearly averaged basis): 75% for COD, 90% for BOD and 70% for SS. Still, no demands are made with regard to the nutrient levels in the effluent. Treatment plants constructed after 1 January 2004 need to fulfill these new standards immediately, older ones from 1 January 2006 onwards (VLAREM II, 2005, appendix 5.3.1). Treatment systems with plants, such as CWs, that are smaller than 500 PE were and still are even dismissed from all effluent standards if the air temperature drops below 5 °C. Systems with a capacity below 20 PE have similar standards.

Table 2.1. compares the Flemish Environmental Legislation with a selection of effluent standards in some other European countries. One can clearly see that the Flemish effluent consents from before 2004 were the most relaxed ones, only exceeded by the Dutch standards for Class I, valid in non-sensitive areas and only for existing treatment plants. The new standards are more comparable with other European ones. However, due to the omission of nutrient standards and the dismission of the standards at low temperatures, it is clear that this set of rules does not offer real protection for a small water course into which the effluent is eventually discharged. The ‘good ecological status’ as required in the European Water Framework Directive (Council of the

European Communities, 2000) seems therefore a barely attainable goal in those, usually sensitive and biologically valuable, water courses.

Besides these non-stringent standards, there is, in practice, little or no control on whether or not the effluents comply and whether or not the treatment plants are operated and properly maintained, except for the constructed wetlands operated by Aquafin NV and a few other examples. This again greatly endangers surface water quality.

2.4. EXPERIENCE WITH CONSTRUCTED WETLAND SYSTEMS IN FLANDERS

A first review on the use of CWs in Belgium was published by Cadelli *et al.* (1998), as part of a European treatment wetlands inventory (Vymazal *et al*., 1998a). At that time, only 2 FWS CWs and one combined system were described for the Flemish region. Since then, an exponential increase took place (Fig. 2.1.a.). The oldest CW is situated in Bokrijk. It is a VSSF reed bed, dating from 1986 and still in operation, although it needed some major modifications due to excessive iron deposition in the drainage pipes and consequent clogging.

Unfortunately, only those treatment plants constructed by (semi-)governmental institutions are relatively well documented. Single-household systems, CWs on farms, etc. are usually not registered with the local authorities and can therefore only be traced by newspaper articles, newsletters from agricultural associations, internet searches, etc. An extensive search through this non-scientific and some regional scientific literature (a.o. Fornoville *et al.*, 1998; Rousseau, 1999; VMM, 2001; AMINAL, 1998; AMINAL, 2002; Aquafin, 2003b; Duyck, 2003; VLM, 2003) resulted in a database about 107 wastewater treatment plants in which constructed wetland technology is being used.

Table 2.1. Effluent standards of different European countries for small-scale discharges into surface waters.

Country	Remarks	COD (mg l^{-1})	BOD (mg l^{-1})	SS (mg l^{-1})	TN (mg l^{-1})	NH_4^+-N (mg l^{-1})	TP (mg l^{-1})	Reference
Flanders, Belgium		250[a]	50[a]	60[a]				VLAREM II (1998) – before 1/1/2004
		125[a]	25[a]	35-60[a]				VLAREM II (2005) – from 1/1/2006
Germany		150	40					Börner *et al*. (1998)
The Netherlands	Class I	750	250	70				Debets (2000)
	Class II	150	30	30				
	Class IIIa	100	20	30	30	2		
	Class IIIb	100	20	30	30	2	2	
Austria	< 500 PE	90	25			10[b]		Haberl *et al*. (1998)
Poland	< 2000 m^3 day^{-1}	150	30	50	30	6	5	Kowalik & Obarska-Pempkowiak (1998)
								Kempa (2001)
Czech Republic	500 – 2000 PE[e]	125-180[d]	30-60[d]	35-70[d]				Czech Law N° 61/2003 – ch. 24 (2003)
Sweden			10[c]		15		0.3-0.5	Linde & Alsbro (2000); Sundblad (1998)

[a] for plant-based systems only if T > 5°C

[b] for plant-based systems only if T > 12 °C

[c] expressed as BOD_7

[d] mean - maximum value

[e] impact on the receiving water body may be taken into consideration and as a result discharge limits can be lower

The distribution of the design sizes expressed as Population Equivalents (PE) is shown in Fig. 2.1.b. One should be aware that these population equivalents – especially for small-scale systems - are derived from the actual number of people connected to it, and not from organic or hydraulic loading rates. Aquafin (2004) for instance found that one inhabitant in reality only produces about 40 g of BOD per day instead of the 54 g of BOD per day assumed during the design stage.

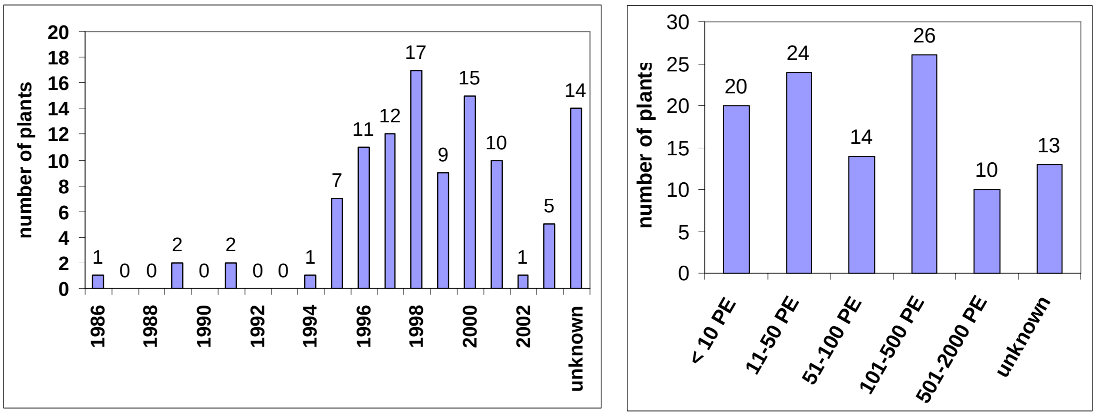

Figure 2.1. (a) Number of constructed wetlands installed in Flanders since 1986; (b) Distribution of design sizes of constructed wetlands in Flanders, expressed as Population Equivalents (PE).

Many different types of constructed wetlands are used in Flanders (Fig. 2.2), ranging from free-water-surface over horizontal subsurface-flow to vertical subsurface-flow CWs and all possible combinations thereof. It is worth noting that the majority of these wetland systems are solely planted with common reed (*Phragmites australis* (Cav.) Trin. ex Steud.). Some ecologically-oriented people, however, used their imagination to construct magnificent wetland systems in their backyard with other species of helophytes and even hydrophytes and pleustophytes. Besides these CWs *pur sang*, which serve as secondary treatment systems, a number of tertiary treatment wetlands were also installed in which natural treatment systems are combined with more conventional ones to enhance the treatment efficiency and flexibility.

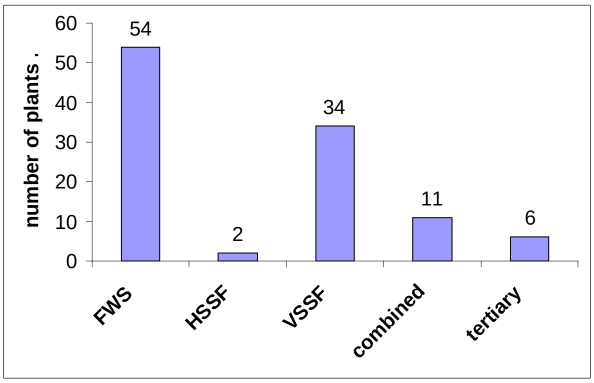

Figure 2.2. Types of constructed wetlands installed in Flanders since 1986 (FWS: free-water-surface, HSSF: horizontal subsurface-flow, VSSF: vertical subsurface-flow).

In the following sections, these different types of CWs will be further described in terms of design variables, investment costs, origin of wastewater and operation and maintenance issues. Performance will be analysed through concentration reduction efficiency and a comparison of the effluent concentrations with the Flemish standards as well as the Dutch Class IIIb standards which are imposed for new treatment plants in vulnerable regions (Table 2.1). This will allow to assess the suitability of the different systems to operate under non-stringent and stringent (Flemish resp. Dutch standards) conditions. Since the data used in this chapter were collected before 2004, the reader should be aware that the relevant effluent standards from this period were used.

2.4.1. Free-water-surface constructed wetlands

Nearly all FWS CWs in the database (52 out of 54) were ordered by the Flemish Land Agency (VLM). Most of these fit within the framework of re-allotment projects and aim at improving the local water quality (http://www.vlm.be). VLM specifically seeks out clay bottoms with a low hydraulic conductivity so that no liner is needed. This approach substantially reduces the investment costs (J. Verboven, VLM, personal communication).

A typical lay-out starts with a concrete overflow structure allowing stormwater peak discharges to bypass the treatment plant. Wastewater that is not bypassed then flows through a coarse bar screen and enters a settling pond where the majority of particulate substances can be removed before the wastewater enters the reed bed. The CW is mostly a long, narrow channel and is planted with *Phragmites australis.* Water levels are normally maintained at 40 to 50 cm (Rousseau *et al.*, 1999).

The design size of the FWS CWs in the database varies from as little as 1 PE up to 2,000 PEs with an average surface area of about 7 m^2 PE^{-1} and an average investment cost of € 392 PE^{-1}. Investment costs per PE clearly decrease as the design size increases, with a marked transition at about 100 PEs (data not shown).

Fifteen FWS CWs treat wastewater from a milking parlour, 34 treat municipal sewage, 3 systems receive a mixture of the two previous types, 1 wetland treats the wastewater of a meat processing company and a last one treats wastewater of an eel farm.

Only few of those wetlands have been monitored in some detail. Fig. 2.3 shows cumulative frequency distributions of the influent and effluent concentrations for the variables Chemical Oxygen Demand (COD), Suspended Solids (SS), Total Nitrogen (TN) and Total Phosphorus (TP). Data on flow rates are virtually non-existing and pollutant loads can thus not be calculated.

Several observations can be made from Fig. 2.3. Concerning COD, 100% resp. 98.7% of the effluent concentrations are in compliance with the 250 mg COD l^{-1} resp. 100 mg COD l^{-1} Flemish and Dutch standards. Only 3% resp. 6% of the SS effluent concentrations do not comply with the 60 mg SS l^{-1} resp. 30 mg SS l^{-1} standard but these are probably due to extreme conditions or malfunctioning since the 80-percentile equals 13 mg SS l^{-1}. As mentioned in the introduction, there are no nutrient limitations for small-scale wastewater treatment plants in Flanders. Compared with the Dutch Class IIIb standards, however, 4% of the samples has concentrations above the 30 mg TN l^{-1} standard and 28% were observed to be above the 2 mg TP l^{-1} standard.

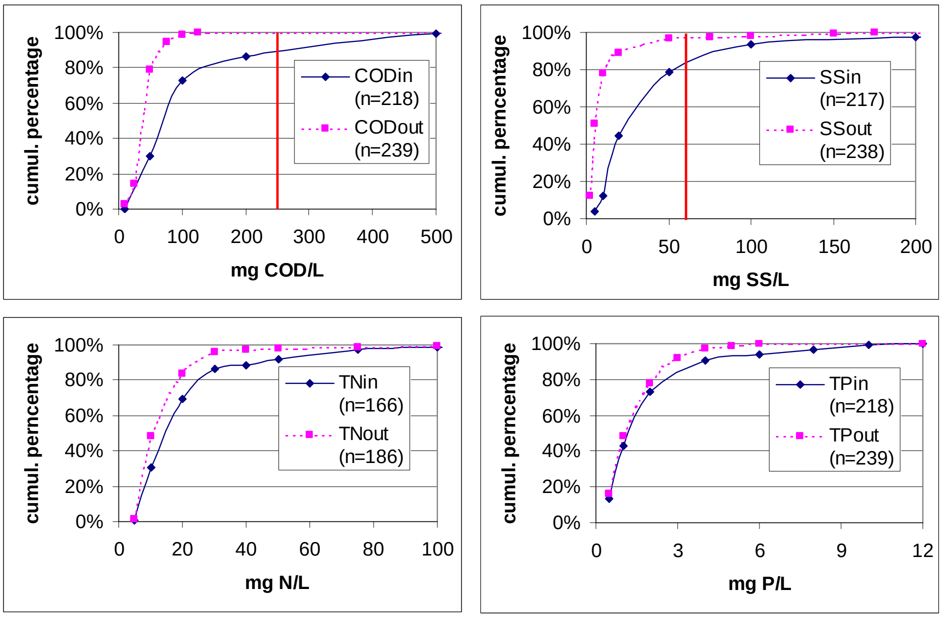

Figure 2.3. Cumulative frequency distributions of the influent and effluent concentrations of 12 free-water-surface constructed wetlands in Flanders for the variables COD, SS, TN and TP. Vertical lines indicate Flemish effluent standards for small-scale wastewater treatment plants.

Striking is that more than 80% of the influent samples are already below the Flemish COD and SS standards. This is mainly due to the combined nature of the sewer networks in Flanders and the resulting dilution by stormwater. It was also common practice in the previous decades to couple drainpipes and even ditches to the sewer system, which sometimes leads to extremely diluted wastewater.

Fig. 2.3 seems to indicate that removal of COD and suspended solids is more efficient than removal of nitrogen and phosphorus. This is confirmed by the overall concentration-based removal efficiencies: 61% and 75% for COD and SS respectively versus 31% and 26% for TN and TP respectively. The general performance is nevertheless quite low. A two-fold explanation is suggested. Low removal efficiencies are in most cases due to the stormwater and surface water discharges to the wetlandswhich result in high hydraulic and low organic loading rates. Some CWs are on the contrary organically overloaded due to the presence of local Small and Medium size Enterprises (SMEs) that produce high-strength wastewaters.

Seasonal variability of the removal performances is given in Fig. 2.4. Seasonal factors such as temperature, solar radiation and plant growth clearly affect pollutant transformations. The best performance for all variables occurs during the spring season, the worst one during autumn. During autumn, there seems to be even a slight increase of water column phosphorus concentrations, possibly correlated with plant senescence and decay. Nitrogen seasonal fluctuations could be dependent on plant uptake and leaching on the one hand, and reduced microbial activity during colder periods on the other hand. The latter factor is probably also the governing factor for variations in COD removal. Finally, suspended solids are optimally removed during spring and summer when a dense network of plant stems favours sedimentation and filtration.

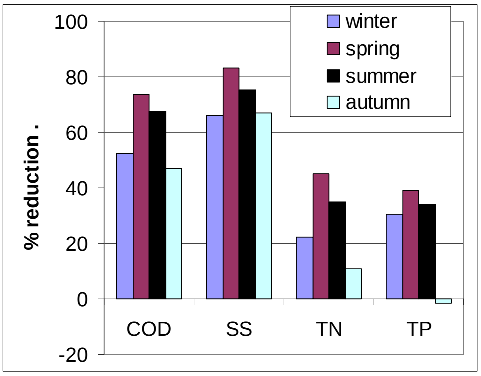

Figure 2.4. Seasonal variability of removal efficiencies of free-water-surface constructed wetlands. Reduction percentages are based on seasonal averages of the influent and effluent concentrations.

Operation and maintenance (O&M) problems are mainly related to the hydraulic constructions and to a lack of supervision. Misconceptions about the flow rates during the design phase have in some cases even caused a major part of the dry weather flow to disappear untreated over the overflow structure into the by-pass. A raise of this structure is not always possible because this would cause a backflow into the sewer system and consequently inundations of the villages during severe rainstorms. A second O&M problem results from a lack of know-how. After construction, the responsibility is generally transferred from the Flemish Land Agency (VLM) to the city council, which usually has no experience with wetlands. They also often adopt the misconception that 'natural' systems are able to manage themselves and should not be looked after anymore. Unfortunately, clogged bar screens and completely filled settling ponds are therefore frequently observed (Rousseau *et al.*, 1999).

2.4.2. Vertical subsurface-flow constructed wetlands

Vertical subsurface-flow CWs are fairly popular throughout Europe because of their reduced footprint and their good effluent quality (Haberl *et al.,* 1995). These characteristics promoted an increasing use of VSSF CWs in Flanders as well (Fig. 2.2).

The design size of the 34 VSSF reed beds in the database varies from 4 up to 2,000 PEs with an average surface area of 3.8 m^2 PE^{-1} and an average investment cost of € 507 PE^{-1}. Most reed beds (28 out of 34) however have a surface area smaller than 80 m^2. The limited data (17 CWs) again show the economy of scale, i.e. the investment cost per PE decreases as the design size of the CW increases, although large variations are noted.

Loading of the beds is in most cases intermittent to optimise re-aeration. Limited information could be found about the filter material but coarse sand seems to be most commonly applied. To enhance nutrient removal, the matrix material is sometimes mixed with one or more additions. Straw has in some cases been added as a carbon source to promote denitrification whereas iron and aluminum filings or lime are added to improve phosphorus removal.

Thirteen VSSF CWs exclusively treat domestic wastewater whereas 20 reed beds treat a mixture of domestic and dairy wastewater. One system is located at an experimental farm and treats domestic, horticultural and non-toxic laboratory wastewater.

Only 7 VF reed beds have been monitored in some detail. Fig. 2.5. shows cumulative frequency distributions of the influent and effluent concentrations for the variables COD, SS, TN and TP.

More than 99% of the COD and more than 98% of the SS effluent concentrations are in compliance with the non-stringent Flemish consents (Fig. 2.5., Table 2.1). About 97%

resp. 95% comply with the stringent Dutch Class IIIb demands for COD resp. SS. A few outliers are probably caused by system malfunctions or extreme conditions. When looking at the Dutch standards for effluent nutrient concentrations, one can observe that only 48% of the TN concentrations and 31% of the TP concentrations comply.

Compared to the FWS CWs, one can see that the influent concentrations are generally higher. Some of the systems contributing to Fig. 2.5. indeed exclusively receive wastewater since they are single-household systems in which the rainwater has been completely separated from the wastewater.

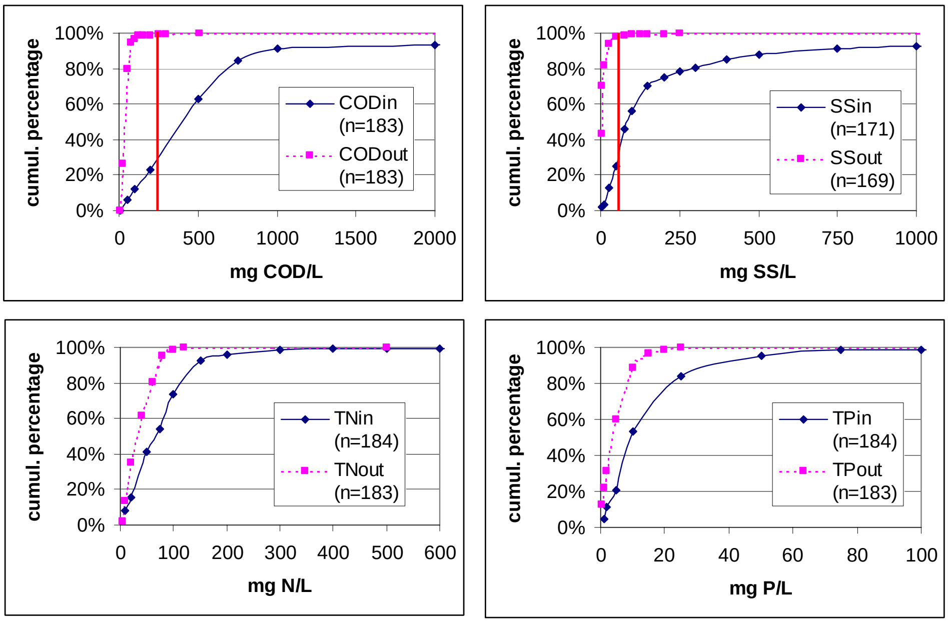

Figure 2.5. Cumulative frequency distributions of the influent and effluent concentrations of 7 vertical subsurface-flow constructed wetlands in Flanders for the variables COD, SS, TN and TP. Vertical lines indicate Flemish effluent standards for small-scale wastewater treatment plants.

Overall concentration-based removal efficiencies are fairly good and equal 94% for COD, 98% for SS, 52% for TN and 70% for TP. Vertical subsurface-flow CWs clearly perform better than the FWS CWs. TP removal is fairly high and is possibly due to the relatively young age of these wetlands, i.e. saturation of the sorption sites is not yet reached. Data were however too scarce to validate this assumption.

Fig. 2.6. gives an overview of seasonal variability within the removal efficiencies. COD and SS reduction are clearly unaffected by the season whereas nutrient removal shows seasonal variations, but substantially smaller than those of FWS CWs. Because of shorter contact times and system lay-out, microbial processes probably play a lesser role than physical-chemical processes compared to FWS wetlands, which would partly explain lower temperature dependencies. Secondly, because of smaller surface areas and relatively thick filter layers, heat losses to the environment are reduced, resulting in higher wastewater temperatures.

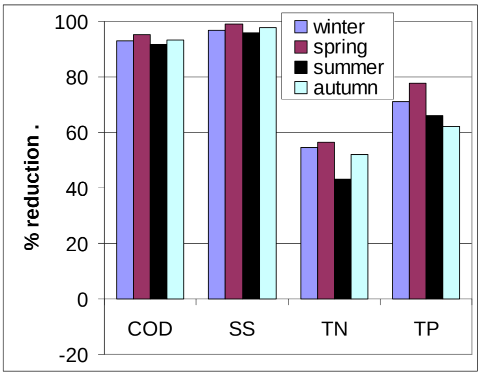

Figure 2.6. Seasonal variability of removal efficiencies of vertical subsurface-flow constructed wetlands. Reduction percentages are based on seasonal averages of the influent and effluent concentrations.

Operational problems with VSSF systems are generally related to clogging phenomena. These are for some reed beds due to the mixed nature of the sewer networks: hydraulic overloading and peak loadings of suspended solids during storm events initiate rapid pore blockage. As a result, Aquafin NV for instance has abandoned this concept until new, separated sewer systems will be constructed. Some other treatment wetlands clearly receive organic loads that are significantly above the design load and are clogging due to an insufficient degradation capability on the one hand and an excessive biofilm production on the other one. Other common causes of clogging are the use of inadequate filter materials (e.g. too finely graded sands) and an unequal distribution of wastewater on the bed surface (e.g. only one central distribution point).

2.4.3. Horizontal subsurface-flow constructed wetlands

Horizontal subsurface-flow or so-called root-zone CWs are less common as a one and only treatment step in Flanders. Due to the frequent clogging problems occuring in VSSF wetlands however, the focus is now more and more shifting towards this concept.

Two root-zone CWs could be traced and are included in the database. The treatment system in Hasselt-Kiewit was started up in 1999, has a design capacity of 152 PEs and treats domestic wastewater on a surface area of 896 m². Since 2001, a 350 PE constructed wetland in Zemst-Kesterbeek treats domestic wastewater on a surface area of 1300 m². Due to the 8 parallel beds on the one hand and some extra educational features on the other hand, the system in Hasselt-Kiewit is the most expensive one with an investment cost of € 1,636 PE^{-1}. The investment costs in Zemst-Kesterbeek were however much lower, i.e. € 879 PE^{-1}.

The Zemst-Kesterbeek system comprises a multi-chambered primary settlement tank followed by two parallel reed beds. In Hasselt-Kiewit, a primary settlement ditch is followed by eight parallel beds. Contradictory to what is commonly recommended in literature, all beds have a length/width ratio that is significantly higher than 1 and have a pulsed loading during dry weather conditions. Hasselt-Kiewit is an exception in Flanders in the sense that more than one plant species is being used. Both systems are filled with washed gravel with a diameter of 5-10 mm.

Fig. 2.7. shows cumulative frequency distributions of the influent and effluent concentrations for the variables COD, SS, TN and TP. The graphs clearly demonstrate that all COD and SS effluent concentrations are below the Flemish standards for small-scale wastewater treatment plants and 95% resp. 93% are below the Dutch class IIIb standards. 93% of the nitrogen effluent concentrations and 52% of the TP effluent concentrations comply with the Dutch class IIIb standards.

Overall concentration-based removal efficiencies equal 72% for COD, 86% for SS, 33% for TN and 48% for TP. The performance is in between the one of the vertical subsurface-flow and the free-water-surface constructed wetlands. Seasonal performance could not be reliably assessed because of a lack of data.

Maintenance problems have occurred due to clogged inlet zones and resulting overland flow and are probably caused by the high length/width ratios. The inlet zones therefore become overloaded and the pores fill up with particles. These particles probably originate from storm flow events since at higher flow rates the hydraulic retention time of the primary settling tank is insufficient.

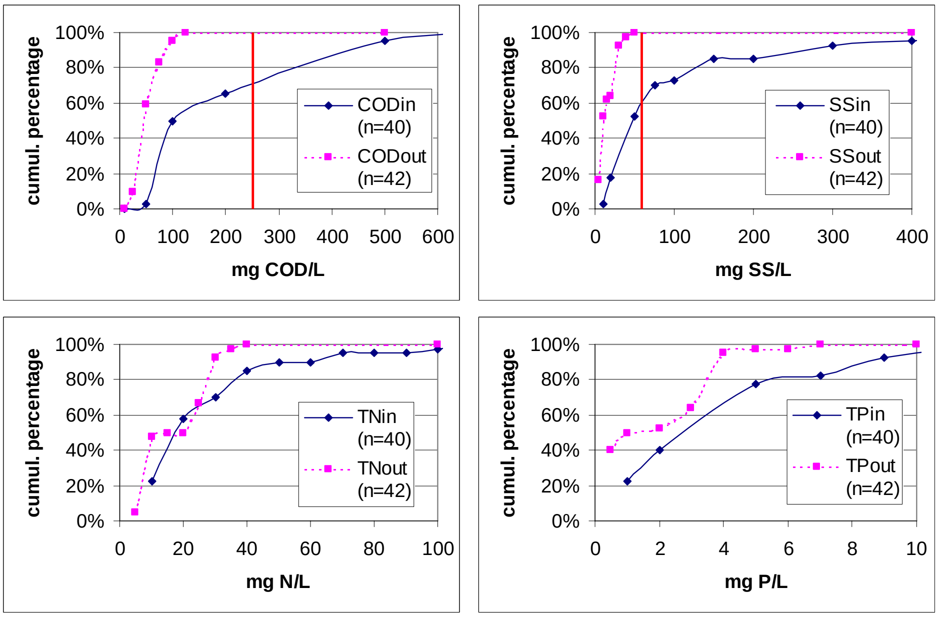

Figure 2.7. Cumulative frequency distributions of the influent and effluent concentrations of 2 horizontal subsurface-flow constructed wetlands in Flanders for the variables COD, SS, TN and TP. Vertical lines indicate Flemish effluent standards for small-scale wastewater treatment plants.

2.4.4. Combined wetlands

Several researchers have proven that a combination of different reed beds not only offers more flexibility, but also provides significantly better effluent qualities (e.g. Cooper, 1999; Cooper *et al.*, 1999; Radoux *et al.*, 2000; Gómez Cerezo *et al.*, 2001). The most popular combination in Flanders consists of one or more parallel vertical subsurface-flow reed beds followed by one or more horizontal subsurface-flow reed beds. This enhances nitrogen removal since VSSF wetlands stimulate nitrification and HSSF wetlands consequently promote denitrification.

Eleven combined systems (Fig. 2.2) were identified and included in the database. Their design size varies from 5 up to 750 PEs with an average surface area slightly exceeding 5 m^2 PE^{-1} and an average investment cost of € 919 PE^{-1}. The same trend as for the other wetland types is noted, i.e. the investment costs per PE decrease as the design size increases, with a marked shift at capacities around 200 PEs.

Nine of those combined wetland treatment systems are of the VSSF-HSSF type, one is a FWS-VSSF combination and the last one consists of two HSSF reed beds in series. Domestic wastewater is the sole source for nine systems, one treatment plant receives a mixture of domestic wastewater and rincing water from a horse stable and another one treats wastewater from a mink farm.

Cumulative frequency distributions of the influent and effluent concentrations for the variables COD, SS, TN and TP can be found in Fig. 2.8.

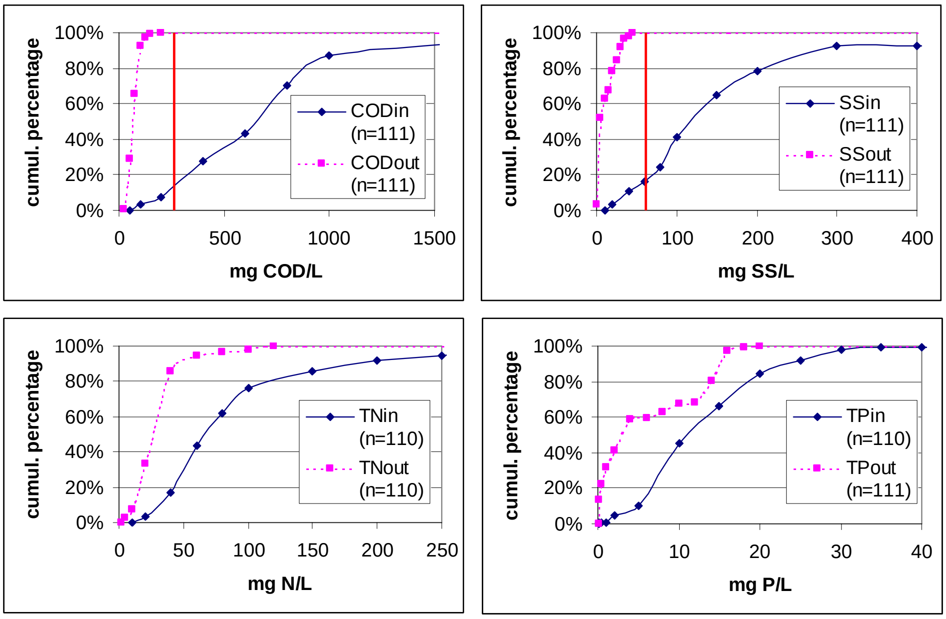

Figure 2.8. Cumulative frequency distributions of the influent and effluent concentrations of 6 combined constructed wetlands in Flanders for the variables COD, SS, TN and TP. Vertical lines indicate Flemish effluent standards for small-scale wastewater treatment plants.

All COD effluent concentrations are amply below the Flemish 250 mg COD l^{-1} consent and more than 97% comply with the more stringent Dutch class IIIb standard.

Suspended solids in the effluent reach a maximum concentration of 44 mg SS l^{-1} and thus no exceedances of the Flemish effluent standards have been noted whereas only 8% of the concentrations exceed the Dutch 30 mg SS l^{-1} standard. Concerning nutrient effluent concentrations, only 47% of the TN and 41% of the TP concentrations comply with the Dutch class IIIb standard.

Overall reductions for COD, SS, TN and TP based on average influent and effluent concentrations equal 91%, 94%, 65% and 52% respectively. Combined wetland treatment systems indeed seem to yield the highest nitrogen elimination by optimally using the strengths of each type of reed bed.

Seasonal fluctuations of treatment performance are shown in Fig. 2.9. Logically, COD and SS removal are quite stable, as was the case for VSSF beds, whereas N and P are affected by season. Especially in autumn, nutrient removal is low, possibly, as was the case for FWS wetlands, because of plant senescence and decay and associated nutrient leaching and release.

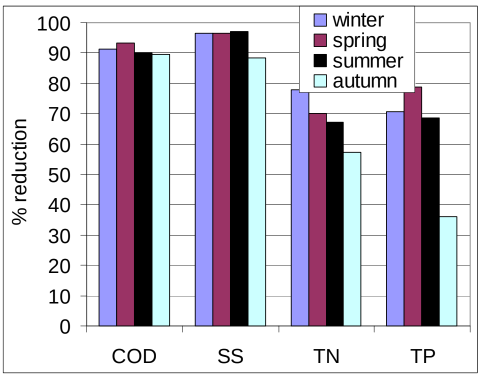

Figure 2.9. Seasonal variability of removal efficiencies of combined constructed wetlands. Reduction percentages are based on seasonal averages of the influent and effluent concentrations.

As can be expected, maintenance problems are identical to the VSSF and HSSF systems and are mainly related to clogging issues, which already have been described in the previous sections.

2.4.5. Tertiary treatment wetlands

A combination of conventional and natural systems for wastewater treatment is also fairly popular in Flanders, with the conventional ones ensuring secondary treatment and the natural ones ensuring tertiary treatment. The addition of one or more CWs greatly enhances the capacity and flexibility of the treatment process.

Six small-scale wastewater treatment plants that make use of CWs for tertiary treatment are present in the database and are described in some detail in Table 2.2. Limited data on investment costs show that the Planckendael INCOMATS^{TM} system is the most expensive one with an investment cost of € 2809 PE^{-1}, followed by the RBC-HSSF system in Aalbeke (€ 1389 PE^{-1}) and finally the RBC-HSSF system in Sint-Maria-Lierde (€ 736 PE^{-1}). Investment costs of the Planckendael INCOMATS^{TM} system are however not fully representative, since the treatment plant is located in a zoological garden and major attention was paid to educational and visual aspects.

Table 2.2. Constructed wetlands as tertiary treatment systems in Flanders, Belgium.

Site	Year	Design capacity (PE)	Area 'green' unit (m²)	Waste water origin	Lay-out
Aalbeke	1997	500	500	Domestic	2 rotating biological contactors + 1 HSSF CW
Sint-Maria-Lierde	2000	850	425	Domestic	3 rotating biological contactors + 1 HSSF CW
Planckendael INCOMATS^{TM}	1995	150	174	Domestic Restaurant Animal cages	1 activated sludge unit + 3 macrophyte beds + 1 HSSF CW
Planckendael birdcage	n.g.	1-4	20	Animal cages	1 rotating biological contactor + 2 HSSF CWs
Tielt-Winge	1994	400	?	Domestic	1 aerated lagoon + 1 duckweed pond
Lier	1995	30	100	Domestic	1 woodfilter + 1 VSSF CW

All 6 treatment plants are being monitored quite closely. Fig. 2.10. shows cumulative frequency distributions of the influent and effluent concentrations for the variables COD, SS, TN and TP. Except for one outlier, all COD and SS effluent concentrations are well below the Flemish standards. Compared with the Dutch class IIIb standard, only about 5% of the concentrations slightly exceed the required level. For the nutrients nitrogen and phosphorus, 87% of all TN and 65% of all TP effluent concentrations comply with the Dutch class IIIb standard.

Overall concentration-based removal efficiencies equal 82% for COD, 93% for SS, 49% for TN and 46% for TP. These are acceptable values but certainly not better ones than those of the previously described systems. One could therefore falsely conclude that extra energy inputs, a more controlled environment and a more labour-intensive maintenance not necessarily enhance treatment performance. Percentage reduction is however not always entirely representative, as indicated by the fact that the lowest average COD and SS effluent concentrations are produced by these combined technical-natural treatment plants.

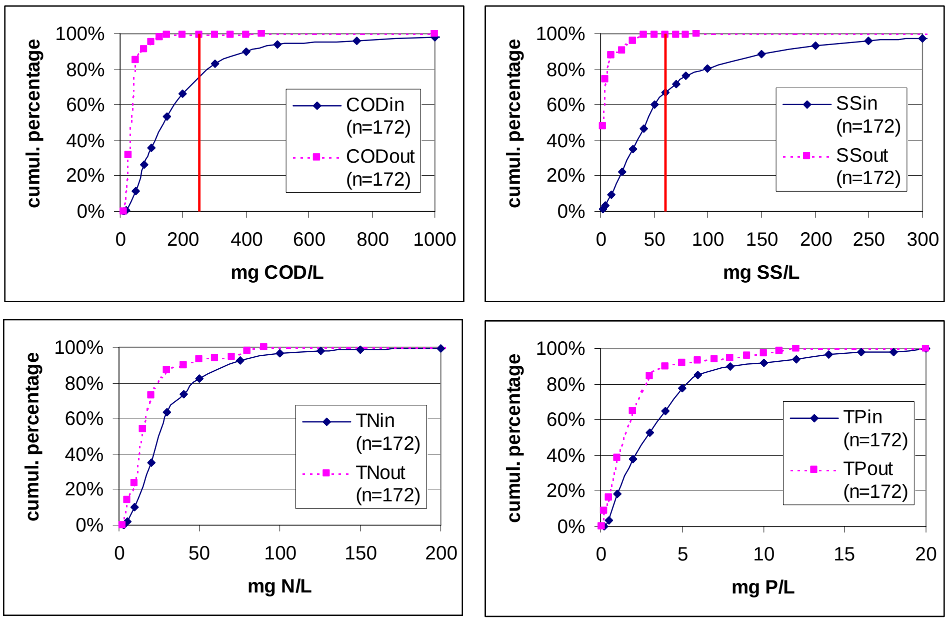

Figure 2.10. Cumulative frequency distributions of the influent and effluent concentrations of 6 small-scale wastewater treatment systems with tertiary treatment wetlands in Flanders for the variables COD, SS, TN and TP. Vertical lines indicate Flemish effluent standards for small-scale wastewater treatment plants.

The effect of season on the performance of tertiary treatment wetlands is shown in Fig. 2.11. Variations seem quite large at first sight when considering the fact that technical systems are used as preliminary treatment steps, but are possibly due to the fact that data from different technologies are lumped together. Taking into account that the influent is at least partially nitrified, lower N removal during the colder seasons probably reflects the strong temperature dependence of denitrification.

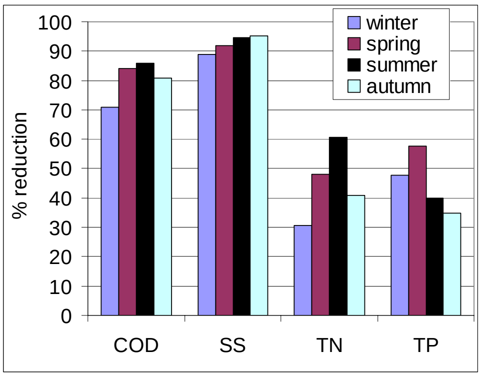

Figure 2.11. Seasonal variability of removal efficiencies of tertiary treatment systems. Reduction percentages are based on seasonal averages of the influent and effluent concentrations.

2.5. DISCUSSION

2.5.1. Organisation and legislation

Small-scale wastewater treatment remains a controversial issue in Flanders with continuing discussions about which government agency has which authority and consequent debates on the location of treatment plants, the choice of treatment technology and the organisation of maintenance and follow-up.

Two other weak points that can be identified are the non-stringent environmental legislation and the lack of enforcement. First of all, the effluent standards for small-scale wastewater treatment plants are too compliant and offer hardly any real protection for the receiving, vulnerable aquatic ecosystems. One is again referred to Table 2.1., which clearly demonstrates that the Flemish effluent consents are amongst the most relaxed ones. Fortunately, most constructed wetlands included in this study produce an effluent with a quality significantly better than the minimum required one. It nevertheless seems sensible to replace the current emission-based effluent consents with immission-based ones that take into account the carrying capacity of the receiving watercourse. Commonly known examples are the 'Total Maximum Daily Load' applied in the USA (Shanahan *et al.*, 1998) or the 'Percentile Approach' applied in the United Kingdom (www.environment-agency.gov.uk).

Secondly, there is little sense in issuing effluent standards if they are not enforced. A central registration office should firstly compile a complete inventory of natural treatment systems and adequate monitoring arrangements should consequently be made to discontinue this lack of control. At the moment, there are also ongoing discussions about certification of certain single-household treatment systems which should guarantee at least a minimum level of performance (Maes, 2000).

2.5.2. Design and investment costs

Table 2.3 resumes the average footprint, investment cost and design capacity of the different types of CWs.

Table 2.3. Average footprint (in m^2 PE^{-1}), average investment costs (in € PE^{-1}) and average design capacity of the different types of constructed wetlands in Flanders, Belgium.

	Average footprint (m^2 PE^{-1})	**Average investment cost (in € PE^{-1})**	**Average design capacity (in PE)**
Free-water-surface CWs	7.0	392	201
Vertical subsurface-flow CWs	3.8	507	158
Horizontal subsurface-flow CWs	4.8	1,258	251
Combined reed beds	5.0	919	272
Tertiary CWs	1.5*	1,645**	386

* area of 'green unit' only ** cost of full system

Free-water-surface CWs clearly require the largest area whereas the tertiary treatment systems logically occupy the lowest area per PE. The footprints of all surveyed CW types are anyhow considerably smaller than the ones reported by Boller (1997), i.e. 7-12 m^2 PE^{-1}. The largest FWS CW in Flanders comprises a total area of 1.0 ha, which compares relatively insignificant to the median value of 40 odd ha reported by Kadlec (1995) for North-American wetlands. The treatment plant at Rillaar is the biggest one in

Flanders, and consists of 4 parallel vertical subsurface-flow wetlands, jointly occupying a surface area of 1.2 ha and treating the wastewater of some 2000 PE.

Average investment costs in Table 2.3. should be interpreted with great care because data quality is highly variable throughout the database, a problem that was also reported by others like Knight *et al.* (1993a). Firstly, it was not always clear from the original sources which components are included, i.e. the wetland costs *sensu stricto* or also the costs for sewer construction, fencing, buildings etc. Secondly, several sources do not mention whether taxes/VAT are included and at what rate. Finally, inflation can give a wrong idea about current costs. Available data nevertheless indicate that FWS CWs are the cheapest ones, which is entirely due to the ease of construction and the avoidance of lining. Horizontal subsurface-flow constructed wetlands appear to be the most expensive 'green' technology, but the two available entries in the database should not be considered as fully representative. One ever-recurring fact is the economy of scale, i.e. the per capita cost decreases as the design size of the treatment plant increases. This characteristic seems to be common to all small-scale wastewater treatment plants as Boller (1997) describes a similar trend for CWs as well as for rotating biological contactors, biofilters, stabilisation ponds etc. The same author also reports a dramatic increase of per capita costs for treatment plants below a size of about 200 PEs, which is consistent with the findings of this study.

Vertical subsurface-flow systems have the lowest average design capacity. This results from the fact that they are the most popular technology for single-household systems and dairy waste treatment, which commonly have discharges below 20 or even 10 PEs. Treatment plants that combine technical and natural units exhibit the highest average design capacity as they seem to be more flexible and economically feasible for larger quantities of wastewater.

2.5.3. Systems assessment and operation

Influent concentrations of the FWS CWs are the lowest ones compared to the other types of CWs, closely followed by the ones of HSSF reed beds. This is mainly due to the fact that all FWS and HSSF CWs receive wastewater of a combined sewer system whereas at least some CWs of the other types receive undiluted wastewater. The lowest average COD and SS effluent concentrations are produced by technical systems with consequent tertiary treatment wetlands, most probably due to mechanical oxygen input and dedicated sedimentation units. Hiley (1995) indeed reports that most wetlands are oxygen limited and that performance is enhanced if extra aeration is provided. The lowest nutrient concentrations were observed in the effluents of FWS CWs which is however entirely due to the low influent concentrations.

Average removal efficiencies of FWS CWs are the lowest ones (COD 61%, SS 75%, TN 31% and TP 26%). Several reasons can be given. Firstly, due to the diluted influent, the effluent concentrations can approach the background concentrations and further removal is thus hampered. Kadlec (1995) for instance mentions background COD levels varying between 30-100 mg COD l^{-1}. A second possible reason suggested by Kadlec (1997) is the often noticed positive relation between loading rate and performance. In this case, the low influent loading rate would explain the low removal efficiencies. Finally, Verhoeven and Meuleman (1999) state that the low removal rate they observed is due to the fact that the most important processes involved occur in the sediment whereas the wastewater flows over the sediment. Dissolved nutrients thus have to transfer by diffusion, which is a fundamentally slow process.

The best overall performance was recorded for the vertical subsurface-flow wetlands (COD 94%, SS 98%, TP 70%), except for total nitrogen removal where the combined reed bed systems performed better (65%). Not considering a limited number of outliers, generally caused by extreme conditions or system malfunctions, all CWs produce an effluent with COD and SS concentrations considerably lower than the non-stringent Flemish or even stringent Dutch class IIIb standards for small-scale wastewater treatment plants. Nutrient limitations do not exist in Flanders but many treatment wetlands nevertheless demonstrate a significant removal of nitrogen (31-65%) and phosphorus (26-70%). These reductions are however in most cases not sufficient to produce an effluent that meets the demand of the Dutch class IIIb standards.

Operational problems are mainly related to clogging phenomena, a problem commonly acknowledged among wetland researchers (see a.o. Platzer and Mauch, 1997; Blazejewski and Murat-Blazejewska, 1997; Langergraber *et al.*, 2002). Next to some design changes, it looks as if this problem can only be dealt with through the construction of separate drainage systems for stormwaters and wastewaters.

Finally, maintenance really is a major issue, as evidenced by the many wetlands that are filled up to various degrees with solids, bar screens that are clogged and reed plants that are being outcompeted by a variety of weeds. Boller (1997) also reported that 'lack of trained operators is often claimed to be the major reason for malfunctioning of small plants'. Concurrent with the conclusions of Cooper *et al*. (1996) and the ones from Chapter 10 of this work, the frequent misconception that natural treatment systems are a 'build-and-forget' solution and thus do not need any attention should be dealt with. Besides, local authorities should be better informed about the nature and frequency of required maintenance tasks and be convinced of their necessity for adequate performance.

2.6. CONCLUSIONS

The number of constructed wetlands in Flanders increased exponentially during the last decade and will most likely continue to since many small-scale discharges still await adequate treatment. The oldest CW dates from the year 1986 and is still in operation, although it needed some major modifications.

Design sizes vary between 1 and 2000 PEs with the majority of CWs having a capacity smaller than 500 PEs. Nearly all of them are planted with common reed (*Phragmites australis*). Other plant species are presently rather an exception. Free-water-surface, vertical subsurface-flow as well as horizontal subsurface-flow CWs are mainly being used, usually as a single treatment unit, or sometimes combined with other CWs or even conventional systems. The CWs mainly treat domestic and dairy wastewater although they are also used for treating wastewater from animal cages, horticulture, restaurants, etc.

Average removal efficiencies of FWS CWs are the lowest ones, mainly due to the strongly diluted influent from the combined sewer systems and the limited contact with the soil or filter medium. The best overall performance was recorded for the VSSF wetlands, except for total nitrogen removal where the combined reed bed systems performed better. This proves that a combination of different wetland types can optimise nitrogen removal. Despite the considerable nutrient removal observed for many wetlands, effluent concentrations of many systems remain relatively high and entail a tangible danger of eutrophication.

To stimulate and optimise constructed wetland technology in the near future, more information about the nature and frequency of required maintenance tasks should be made readily available for owners. It seems furthermore recommendable to replace the current, too compliant emission-based effluent standards with immission-based ones that take into account the local carrying capacity of the receiving watercourses. Finally, to evaluate and enforce the previous measures, adequate monitoring arrangements should be developed.

Chapter 3

Subsurface-flow constructed wetlands: processes and influencing factors

3.1. INTRODUCTION

Wastewater treatment in subsurface-flow constructed wetlands is accomplished by an array of physical, chemical, biological and microbiological processes taking place in different compartments. As a lead up to the modelling part of this thesis, the different pollutant removal pathways in subsurface-flow constructed wetlands will be discussed by means of their respective mass balances. This overview remains restricted to the 'classical' variables such as water, suspended solids, organic matter, dissolved oxygen, nitrogen and phosphorus. Heavy metals, pathogens, pesticides etc. are outside the scope of this work. The water mass balance is valid for the wetland as a whole, all other mass balances concern substances present in the pore water of the wetland. In general, each mass balance states that the change of the components' mass in time equals influxes minus effluxes plus or minus transformations.

3.2. WATER MASS BALANCE

$$\frac{dWATER}{dt} = \text{influent} - \text{effluent} + \text{precipitation} + \text{groundwater infiltration} - \text{groundwater seepage} - \text{evaporation} - \text{transpiration}$$

A solid understanding of the water balance is of utmost importance for both data treatment and modelling. Indeed, it allows to calculate important factors such as the hydraulic residence time, the water velocity, the water depth etc. The water balance also forms the basis for all other mass balances that make up a model. Finally, it links influent and effluent and therefore allows to correlate data and draw sound conclusions (Kadlec, 1990).

Due to the relatively large specific surface areas of CWs, the water volume can significantly increase during rain storms. Many systems are also served by combined sewer systems and therefore receive high rain water flows. Evaporation in SSF CWs is however of less importance because the water flows below-ground and diffusion to the atmosphere is therefore hindered. Secondly, the plant cover reduces wind speed and temperature and therefore also limits evaporation (Brix, 1997). Transpiration on the contrary is of major importance, especially for HSSF CWs where the hydraulic residence time is in the order of days (Wood, 1995). Data from SSF CWs are scarce, but data from a natural *Phragmites australis* stand (Burba *et al.*, 1999) show evapotranspiration rates of 1.3 up to 4.0 mm per day during the early and peak growth stages, 1.8 mm per day at the beginning of senescence and near zero at the end of senescence. Herbst and Kappen (1999) found evapotranspiration rates for a reed canopy that even exceeded 10 mm per day during a hot and sunny day. Compared with the recommended hydraulic loading rates of Wood (1995) of 2 – 30 mm day^{-1}, it is clear that for low-loaded systems evapotranspiration can exceed the influent flow leading to a zero discharge.

When groundwater levels are high, groundwater can infiltrate into the CW and dilute the waste water. Inversely, at low groundwater levels, waste water can seep through the soil and pollute the aquifer. The rates of both processes depend on the local soil characteristics i.e. hydraulic conductivity. Both water flows are normally prevented by lining the basin with, for instance, clay or plastics.

3.3. SOLIDS MASS BALANCE

$$\frac{dSOLIDS}{dt} = \text{influent} - \text{effluent} - \text{filtration} + \text{plant decay} + \text{microbial growth} + \text{microbial decay} + \text{sloughing} - \text{invertebrate uptake}$$

Most HSSF CWs are filled with gravel because it has a higher hydraulic conductivity and it is less prone to clogging. Sometimes local soil is used, as is for instance common practice in Denmark (Brix, 1998). VSSF CWs are usually filled with coarse sands or fine gravels. In the water-filled pores, particles can settle in so-called micropockets or be halted by certain hydraulic conditions. Particles can also adhere to the filter material by means of several interparticle adhesive forces. The combination of all these physical processes is called filtration (Kadlec *et al.*, 2000c). Once removed from the water phase, organic particles can be hydrolysed or mineralised and converted into dissolved matter. Inorganic material either remains blocked in the pores or gets ‘resuspended’ again under certain hydraulic conditions.

Senescent plants eventually become part of the litter layer on top of the filter material, unless they are harvested during early senescence. Further degradation of this litter layer can result in small plant fragments which can migrate in the pores of the filter material. Fungi are commonly found in wetlands growing in dead and decaying plant litter. They make up a significant proportion of the carbon and nutrients available to plants and other micro-organisms (Baptista, 2003).

New solids in the wetland are also created due to bacterial growth on organic and inorganic material. Constructed wetland microorganisms tend to occur in aggregates called biofilms, i.e. thin layers of microorganisms that grow on the surfaces of the gravel or sand grains and the roots and rhizomes. When these layers become too thick and/or when flow velocities become too high, pieces of the biofilm can detach and get carried away by the water current. Also, when these bacteria die, they are partly degraded to particles and partly to dissolved substances, which are transferred to the wastewater.

In these man-made ecosystems, microscopic and macroscopic invertebrates are another important link in the food chain. They ingest pollutant particles and by grazing the biofilms they reduce the frequency of sloughing.

In general, wetlands are capable of removing between 60 and 95% of the suspended solids when applied for secondary treatment (Kemp and George, 1997). For tertiary treatment, lower efficiencies may be expected.

Obviously, when the solids balance has a continuously net positive result, the porosity and consequently the hydraulic conductivity of the filter material will be negatively affected. This can finally result in surface flow of wastewater in the case of HSSF CWs or ponding in the case of VSSF CWs, causing a number of adverse effects such as short-circuiting, algal growth, odour problems, insect nuisance etc. (Shutes, 2001; Dahab *et al.*, 2001).

3.4. DISSOLVED ORGANIC MATTER MASS BALANCE

$$\frac{dDOM}{dt} = \text{influent} - \text{effluent} + \text{rhizodeposition} + \text{litter leaching} + \text{mineralisation} - \text{microbial conversion} + \text{microbial decay} - \text{plant uptake}$$

Particulate organic matter obviously follows the pathways as described in the previous section (suspended solids mass balance). Via mineralisation, they can enter the dissolved solids mass balance.

Organic components are degraded aerobically as well as anaerobically by bacteria attached to the roots, rhizomes and filter material (Baptista, 2003). Plant uptake of organic material does occur but is reported to be negligible compared to microbiological uptake (Watson *et al.*, 1990; Cooper *et al.*, 1996, in Kadlec *et al.*, 2000c). Oxygen for aerobic microbial conversion is supplied by the influent, by direct diffusion from the atmosphere and by root oxygen release (cf. dissolved oxygen mass balance in the section below). Since oxygen demand usually exceeds oxygen supply, especially in HSSF CWs, anoxic and anaerobic degradation are important pathways in CWs (Brix, 1990, in Kadlec *et al*., 2000c). Microbial degradation of organic matter is obviously affected by its composition and by its residence time in the system. Readily biodegradable substances are effectively and quickly converted, while refractory compounds need large residence times to be even partially decomposed. Large organic molecules such as sugars, proteins and fats are usually first broken down into smaller compounds by extracellular hydrolytic enzymes. They can then penetrate the cell wall and are further converted via several pathways according to the available electron acceptors (Baptista, 2003). The presence of alternative electron acceptors, especially sulphates, is also significant. Huang *et al*. (2005) for instance indicate that influent sulphate was the major electron acceptor that contributed to the oxidation of organic matter.

Most aquatic macrophytes are known to release organic compounds from their roots, a process which is termed ‘rhizodeposition’. The chemical composition of these exudates is very diverse, e.g. sugars, vitamins, organic acids etc. Carbon loads released by rhizodeposition are only of significance in very lowly loaded systems such as acid mine drainage CW (Stottmeister *et al.*, 2003). Dead plant material which lies on the filter surface – or so-called plant litter – is also known to release dissolved organics and nutrients during the degradation process (Wynn and Liehr, 2001).

Removal efficiencies for BOD in root-zone CWs range between 0.2 - 16 g m^{-2} day^{-1} (Reed and Brown, 1995; Knight *et al.*, 1993a; Reed, 1993, in Sikora *et al.*, 1995). Kemp and George (1997) report general removal percentages of subsurface-flow wetlands between 60 and 95% for secondary treatment systems. According to many authors, BOD removal is not temperature dependent (Kadlec and Knight, 1996h). Griffin *et al.* (1999) on the contrary did find a significant difference and attributed this to the fact that they used a higher strength wastewater which makes it easier to discern the influence of temperature. Ayaz and Akça (2001) demonstrated that, regardless of plant species or matrix material, COD removal rates were linearly correlated (R^2 > 0.9) with COD loading rates, within the tested range of 0 – 75 g COD m^{-2} day^{-1}.

3.5. DISSOLVED OXYGEN MASS BALANCE

$$\frac{dOXYGEN}{dt} = \text{influent} - \text{effluent} + \text{plant root release} + \text{atmospheric input} - \text{microbial respiration} - \text{nitrification}$$

Since wetland plant roots normally grow in a water-logged substrate with a low oxygen content, they have to withdraw the necessary molecular oxygen for respiration from other sources. Oxygen is therefore actively transported from the above-ground to the below-ground plant parts through the so-called aerenchyma. Part of this oxygen is deliberately released from the roots to create an oxidised zone around the roots. The low redox potential in the substrate can indeed cause high concentrations of plant-toxic substances such as H_2S and organic acids (Brix, 1993). Aerobic microorganisms in the biofilms make use of the O_2 ‘leaks’ for oxidative decay of organic matter. The oxygen leakage rate depends mainly on the plant species and biomass and the oxygen demand. An additional source of oxygen in the root-zone is provided through air currents within dead and broken stems. Inverse fluxes of CO_2 and CH_4 also occur (Kadlec and Knight, 1996d; Beckett *et* al., 2001). The oxidised zone is only a few micrometers thick and is surrounded by anoxic and anaerobic zones. Pollutants migrate through this 'mosaic' by means of diffusion.

Lawson (1985, in Brix, 1994) calculated a potential oxygen flux through *Phragmites* roots up to 4.3 g m^{-2} day^{-1}. Other authors (in Brix, 1994) estimated the oxygen release by *Phragmites* to be respectively 0.02 g m^{-2} day^{-1} (Brix, 1990), 1 - 2 g m^{-2} day^{-1} (Gries *et al.*, 1990) and 5 to 12 g m^{-2} day^{-1} (Armstrong *et al.*, 1990). In their review, De Pauw and De Maeseneer (1992) mention oxygen fluxes between 5 and 45 g m^{-2} day^{-1}. This variation is largely due to different experimental approaches and also to seasonal variation.

Brix and Schierup (1990, in Hiley, 1995) reported a total influx of 5.86 g O_2 m^{-2} day^{-1} to a HSSF reed bed of which 3.76 g m^{-2} day^{-1} was direct atmospheric input and 2.08 g m^{-2} day^{-1} was root oxygen release whereas only 0.02 g m^{-2} day^{-1} seemed to be necessary for root respiration. Wu *et al.* (2001) report 6.01 – 7.92 g O_2 m^{-2} day^{-1} for a *Typha latifolia* HSSF CW of which only 0.023 g O_2 m^{-2} day^{-1} seemed to be released by the roots. Oxygen transfer rates were again highly dependent on oxygen demand, in this specific case mainly determined by ammonium concentrations.

Oxygen input in VSSF CWs tends to be much higher, especially when batchwise loading is applied. Indeed, when pumping large volumes of wastewater in a short period of time, the hydraulic loading rate will exceed the hydraulic conductivity and a water layer is formed on top of the reed bed. When this water layer migrates downwards, the air present in the pore spaces below is compressed and therefore dissolves easier in the water layer. Secondly, above the water layer, an underpressure is created which causes new air to be sucked in that will become available to microorganisms during the next pump phase.

3.6. NITROGEN MASS BALANCE

$\frac{dNITROGEN}{dt}$ = influent – effluent – plant uptake + rhizodeposition + leaching + plant decay – adsorption + desorption – microbial conversion – microbial uptake + microbial decay – ammonia volatilisation + nitrogen fixation + atmospheric deposition

Macrophytes take up nutrients that are present in the wastewater. For nitrogen, the preferred uptake form seems to be ammonium (Drizo *et al.*, 1997). According to Brix (1997), emergent macrophytes have a total nitrogen uptake capacity between 0.055 and 0.685 g N m^{-2} $year^{-1}$. Rogers (1985, in Wood, 1995) reports an average N-uptake capacity of *Phragmites australis* of 0.633 g N m^{-2} $year^{-1}$, Meuleman (1999) reports 0.214 g N m^{-2} day^{-1}, Kuusemets *et al.* (2002) 0.045 g N m^{-2} day^{-1} and Adcock and Ganf (1994) approximately 0.12 g N m^{-2} day^{-1}. For young plants, the highest nutrient mass seems to be located in the leaves and shoots but fully-grown macrophytes accumulate most of the nutrients in the roots and rhizomes (Mandi *et al.*, 1996). As a consequence, nutrient export through plant uptake and consequent harvesting compares relatively low to the influent load in normally-loaded systems. However, when harvesting is omitted, plants set free an initial amount of nutrients during early senescence which is called 'leaching' and mineralisation of the resulting detritus layer leads to a further release of nutrients. This detritus layer contains nevertheless a number of refractory components which act as a more or less permanent store of nitrogen.

The most important nitrogen removal mechanism is the succession of three microbial conversions, i.e. ammonification, nitrification and denitrification (Bavor *et al*., 1995).

Organic nitrogen is mineralised to (mainly) ammonium via hydrolysis and bacterial action. Ammonification rates are highest in the oxygen-rich zones and decrease from aerobic to facultative anaerobic to obligate microorganisms. Proteolysis of proteins and nucleic acids is generally carried out by bacteria under neutral or alkaline conditions while fungi take over in more acidic environments. Ammonification rates are dependent on temperature, pH, C/N- ratio, available nutrients and soil conditions such as texture and structure (Reddy and Patrick, 1984, in Kadlec *et al.*, 2000c). The optimal ammonification temperature is reported to be 40 – 60 °C (Hammer and Knight, 1994) while the optimal pH is 6.5 – 8.5 (Reddy *et al.*, 1979, in Kadlec *et al.*, 2000c). Reported ammonification rates vary from 0.004 – 0.357 g N m^{-2} day^{-1} (Reddy and D'Angelo, 1997), 0.22 – 0.53 g N m^{-2} day^{-1} (Tanner *et al*., 2002) and 0.058 g N m^{-2} day^{-1} (Senzia *et al*., 2002).

Ammonium is then further oxidised to nitrates, with nitrites as an intermediate form, by nitrifying bacteria living in the aerobic microsites. The first step, from NH_4^+ to NO_2^- is carried out by obligate chemolithotrophic and aerobic bacteria mainly of the genera *Nitrosospira, Nitrosovibrio, Nitrosolobus, Nitrosococcus* and *Nitrosomonas*. The second step, oxidation of NO_2^- to NO_3^-, is carried out by facultative chemolithotrophic bacteria such as the genera *Nitrobacter* en *Nitrocystis* (Grant and Long, 1981, in Kadlec *et al.*, 2000c; Hammer and Knight, 1994). Nitrification appears to be affected by temperature, pH, alkalinity, availability of inorganic carbon sources and ammonium and oxygen and the microbial population (Vymazal, 1995, in Kadlec *et al.*, 2000c; Merz, 2000). Optimal temperatures are between 30 – 40 °C whereas nitrification is inhibited at temperatures below 4 – 5 °C (Cooper *et al.*, 1996, in Kadlec *et al.*, 2000c). The pH optimum is situated between 7.5 – 8.6. Nitrification rates are reported to range from 0.01 – 0.161 g N m^{-2} day^{-1} (Reddy and D'Angelo, 1997), 0.56 – 2.15 g N m^{-2} day^{-1} (Tanner *et al*., 2002) and 0.20 g N m^{-2} day^{-1} (Senzia *et al*., 2002). Because the available oxygen in HSSF CWs is normally quite low, nitrification seems to be the rate limiting step in the nitrogen removal sequence (Sikora *et al.*, 1995). Incomplete nitrification can result in the production of the greenhouse gas N_2O. In one case study, Fey *et al*. (1999) however estimated that only 0.11% of the total N input was converted to N_2O.

Nitrates are finally reduced to nitrogen gas (N_2) and nitrous oxide (N_2O) by denitrifying bacteria in the anoxic wetland zones. Involved microbial genera are mainly *Pseudomonas, Aeromonas* and *Vibrio* (Grant and Long, 1981, in Kadlec *et al.*, 2000c) although *Achromobacter, Aerobacter, Alcaligenes, Azospirillum, Brevibacterium, Flavobacterium, Spirillum* and *Thiobacillus* are also capable of denitrification. This proces is affected by the redox potential, temperature, nitrate concentration, soil moisture content, pH, organic carbon content and dissolved oxygen concentrations (Meuleman, 1999; Vymazal, 1995, in Kadlec *et al.*, 2000c). Denitrification is the rate limiting step in VSSF CWs since they are mostly aerobic. Organic carbon is mostly provided from decay of senescent plants and litter (Baker, 1998). The optimal pH ranges from 7 – 8 while the optimum temperature is 25 – 65 °C (Hammer and Knight, 1994). Denitrification is inhibited at temperatures below 5 °C. According to Howard (1985, in Koerselman, 1990), the potential denitrification rate of reed beds can exceed 0.5 g N m^{-2} day^{-1}. Reported nitrate removal rates are between 0.003 – 1.02 g N m^{-2} day^{-1} (Reddy and D'Angelo, 1997), 0 – 0.00346 g N m^{-2} day^{-1} (Comin *et al*., 1997), 0.47 – 1.99 g N m^{-2} day^{-1} (Tanner *et al*., 2002) and 0.20 g N m^{-2} day^{-1} (Senzia *et al*., 2002).

Constructed wetlands optimally exploit these processes because of the presence of a mosaic of aerobic and anoxic sites (cf. oxygen balance). Nitrification and denitrification create diffusion gradients which drive the flux of nitrogen components from one site to another.

Tanner *et al*. (2002), in their review on nitrogen processing gradients in subsurface-flow wetlands, consider the possibility that in these oxygen-limited environments, nitrogen conversion may include a range of alternative and co-metabolic pathways that offer the potential of short-circuiting the classical nitrification-denitrification process. Examples of such pathways are oxygen-limited autotrophic nitrification-denitrification (OLAND), anaerobic ammonium oxidation (ANAMMOX) and heterotrophic nitrification.

Another potentially important microbial process is dissimilatory nitrate reduction. This reaction occurs under anoxic conditions and when high concentrations of easily biodegradable organic material are available, and mainly converts nitrites but also nitrates to ammonium (Stanier *et al*., 1986). The produced amount of ammonium exceeds the amount needed for cell tissue construction and is therefore partly released to the environment. Although dissimilatory nitrate reduction seems to be more energy efficient than denitrification (Meuleman, 1999), van Oostrom and Russell (1994) only found a 5% contribution of the first process to the removal of nitrates in an experiment with *Glyceria maxima*. Meuleman (1999) warns that neglecting the process of dissimilatory nitrate reduction can lead to high overestimations of the denitrification potential.

Nitrogen fixation can also be substantial. Certain heterotrophic soil bacteria, symbiotic actinomycetes and cyanobacteria are capable of synthesizing amino acids and proteins from atmospheric N_2 by means of a special enzyme called nitrogenase. Koerselman

(1990) reports an extra N-input via nitrogen fixation of a few tens to even hundreds of kg N ha^{-1} $year^{-1}$. Kadlec and Knight (1996e) mention that nitrogen fixation requires a significant amount of cellular energy which seems wasted in a nitrogen-rich environment. Fixation rates in wetlands receiving wastewater high in nitrogen are therefore probably much lower or essentially negligible compared to other nitrogen transformation processes.

Microorganisms need nitrogen as a building block for cell tissues, enzymes etc. The magnitude of this process has not been quantified for treatment wetlands (Kadlec and Knight, 1996e) but seems to be of minor importance. When microorganisms die, at least part of the cellular nitrogen is released again to the environment.

Ammonium can adsorb to the soil kation adsorption complex (Koerselman, 1990; Drizo *et al.*, 1997). This is however a reversible process and results in a balance between NH_4^+ in solution and adsorbed NH_4^+. When ammonium for instance disappears by nitrification, the balance is restored through desorption of NH_4^+. Adsorption therefore seems to play only a minor role in total nitrogen removal (Lee *et al.*, 1999) but it buffers peaks and may give rise to slow release of nitrogen.

Atmospheric deposition consists of dry and wet deposition on the one hand, and plant uptake of gaseous N compounds on the other hand. Dry and wet deposition can be easily measured via chemical analysis of the content of a pluviometer. Plant uptake can be estimated by means of a model that relates NO_x and NH_3 uptake to the leaf area (Heil *et al.*, 1988, in Koerselman, 1990). Koerselman (1990) reports atmospheric deposition rates of about 50 kg N ha^{-1} $year^{-1}$ although locally higher rates are possible, for instance around bio-industries. Kadlec and Knight (1996e) report rates for North America between 0.7 – 15.4 kg N ha^{-1} $year^{-1}$ which compares insignificant to the influent nitrogen load in most treatment wetlands.

Unionized ammonia (NH_3) is relatively volatile and can be removed from solution to the atmosphere through diffusive and advective forces, the latter ones however being quite small in subsurface-flow systems. Van Oostrom and Russell (1994) report N-losses by volatilisation of less than 0.1 kg N ha^{-1} day^{-1}, even at pH 8. Eighmy and Bishop (1989) confirm that this mechanism only plays a minor role as long as the pH remains lower than the pK_a of ammonium which is 9.3. Not only pH affects this process, also temperature, vegetation density, wind speed, water turbulence and ammonium concentration are of importance.

As for COD, Ayaz and Akça (2001) found that, indifferent of plant species or matrix material, N removal rates were linearly correlated ($R^2 > 0.85$) with N loading rates within the tested interval of 0 - 11 g N m^{-2} day^{-1}.

3.7. PHOSPHORUS MASS BALANCE

$$\frac{dPHOSPHORUS}{dt} = \text{influent} - \text{effluent} - \text{plant uptake} + \text{plant decay} + \text{leaching} - \text{adsorption} + \text{desorption} - \text{precipitation} - \text{microbial uptake} + \text{microbial decay} + \text{atmospheric deposition}$$

According to Davies and Cottingham (1993), the P-uptake capacity of aquatic macrophytes is very limited (about 6% of the influent load). Reported P-uptake capacities are quite comparable: 50 – 150 kg P ha^{-1} $year^{-1}$ (Brix, 1994a), 162 kg P ha^{-1} $year^{-1}$ (Rogers, 1985, in Wood, 1995), 55 kg P ha^{-1} $year^{-1}$ (Drizo *et al.*, 1997) and 80 kg P ha^{-1} $year^{-1}$ (Meuleman, 1999). Radoux and Kemp (1982) could export about 37 kg P ha^{-1} $year^{-1}$ through harvesting of the above-ground plant parts of an experimental constructed wetland in Viville (Belgium) whereas in a warmer climate such as Morocco, Mandi *et al.* (1996) could export 62 kg P ha^{-1} $year^{-1}$. However, when the plants are not harvested, phosphorus is released again during decay of the senescent plants or is stored in the detritus layer on the bed surface.

Phosphate adsorption by soils is mainly affected by the soil texture, its Fe content and to a lesser extent its Al content. Clay and fine sand are more effective in adsorbing P than coarse sand or gravel. This mechanism can be very important during the first years of operation of a CW. After some time however, saturation of the filter material can occur and P is no longer adsorbed or even released (Fiselier, 1990, Davies and Cottingham, 1993). Many different substrates have been evaluated for their P removal capacity, e.g. Drizo *et al.* (1997) evaluated bauxite, shale, burnt oil shale, limestone, zeolite, light expanded clay and fly ash and found that fly ash and shale had the highest P adsorption values.

Another important process is chemical precipitation (Drizo *et al.*, 1997) during which phosphates react with certain metals and form insoluble compounds. In acidic and oxidised conditions, Fe^{3+} and Al^{3+} compounds such as aluminium and ferric (hydroxy) phosphates are formed. However, under anaerobic conditions, ferric phosphate compounds dissolve again due to the reduction of Fe^{3+} to Fe^{2+} and ortho-phosphate ions are released to the water column. Under alkaline conditions a variety of Ca^{2+} and Mg^{2+} complexes prevail. Arias *et al*. (2001) screened 13 Danish sands for their P-removal capacity and found that the determining characteristic was their Ca-content rather than Fe and Al since wastewater is normally slightly alkaline. Wollastonite, a calcium metasilicate mineral mined in upstate New York was proven to be a good medium for P-sorption, provided the contact time was long enough (Brooks *et* al., 2000). Chemical substances can possibly be amended to stimulate this process. In order of effectiveness, $FeCl_3$, alum, $Ca(OH)_2$, calcite and dolomite were demonstrated to be able to substantially reduce soluble P contents (Reddy and D'Angelo, 1997; Ann *et al*., 2000).

P-uptake by bacteria is a partly reversible removal mechanism. The continuous cycle of growth, die-off and decay releases most of the initially absorbed phosphorus.

Atmospheric deposition, as for nitrogen, is relatively insignificant in most cases. Koerselman (1990) estimates it at 0.2 – 0.5 kg P ha^{-1} $year^{-1}$ for the Netherlands.

3.8. CLOSURE

Pollutant transformations and pollutant removal in SSF CWs is clearly a complex web of interacting pathways, involving both plants, microorganisms, water and filter material or soil. Which pathways dominate is determined by a range of influencing parameters, such as pH, temperature, loading rates etc. In the following chapter, three data sets from a two-stage pilot-scale CW in Aartselaar (Belgium) are examined in order to isolate dominant processes under relevant operational conditions for small-scale wastewater treatment plants in Flanders.

Chapter 4

Short and long-term dynamics in subsurface-flow constructed wetlands: a pilot-scale study

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4.1. ABSTRACT

Decades of research on constructed wetlands have revealed the need for better insight in internal processes and better design and management tools. Current research is therefore increasingly oriented towards modelling, especially dynamic modelling. These models require large and high-frequency datasets for model development on the one hand, and calibration and validation purposes on the other hand. Nevertheless, at present, little is known about the short-term behaviour of constructed wetlands. This study intensively examined a two-stage pilot-scale constructed wetland via both low-frequency and high-frequency sampling. Low-frequency sampling was conducted from Spring 1997 till Spring 2000. Two additional 10-day monitoring campaigns were conducted, one in winter (January 2001) and one in summer conditions (August 2001), during which composite samples of influent and effluent were collected at intervals no longer than 8 hours. This chapter describes the results from all three monitoring campaigns, compares the short-term data sets with the long-term one and investigates seasonal and age effects. Some modelling recommendations are deduced from the results.

4.2. INTRODUCTION

The increasing application of constructed wetlands for wastewater treatment coupled to increasingly stringent water quality standards is an incentive for the development of better design tools. Originally working with simple regression equations, most researchers and designers evolved towards the use of the well-known first-order k-C* model (Kadlec and Knight, 1996c). However, this black-box model is based on only two parameters, the first-order decay rate k, and the background concentration C*, which is an obvious oversimplification of the complex wetland processes. For a detailed overview of design equations, the reader is referred to Chapter 5 of this thesis.

More recently, several dynamic, compartmental models have been presented in literature, a.o. McBride and Tanner (2000) and Wynn and Liehr (2001), which explicitly take into account the different processes that occur in constructed wetlands. Simulation results obtained with these models seemed very promising. These detailed models however have one major drawback: they contain several dozens of parameters that have to be estimated. A sensitivity analysis can reveal those insensitive parameters that do not require a very accurate estimation. Parameters on the other hand that have a major influence on the model output have to be determined precisely (Dochain and Vanrolleghem, 2001). Since little has been published concerning the values of most of these parameters, calibration must be based on input-output data. Considering the fact that time constants of certain microbial and physical-chemical reactions range between seconds and hours, calibration probably requires large, high-frequency data sets.

A limited literature survey revealed that little data of this high detail exist. In most studies, only wastewater flow rates and some physical-chemical characteristics like dissolved oxygen, pH and temperature were monitored (semi)continuously, whereas data on BOD, COD, suspended solids, nitrogen and phosphorus were only collected biweekly, e.g. Wynn and Liehr (2001), Tanner *et al.* (1995a,b), Kozub and Liehr (1999) and Sakadevan and Bavor (1999). In some other studies, grab samples of influent and effluent were taken at monthly (Bulc *et al.*, 1997; Gómez Cerezo *et al.*, 2001) or three-monthly (Kern and Idler, 1999) intervals. Braskerud (2002) on the contrary continuously collected flow-based composite samples that were, however, only analysed approximately every 10 days, which provided interesting information about the overall mass balances, but which also masked the dynamic behaviour of the system. Bolton and Greenway (1999) took daily grab samples at the inlet, middle and outlet sampling stations with some time in between to take into account the hydraulic residence time.

This study therefore intends to investigate the influence of the data collection frequency on the manifestation of certain processes and thus indirectly on model building. An existing long-term data set from a two-stage pilot-scale constructed wetland was first examined to pinpoint the major processes that should be included in a dynamic model (Vandaele *et al.*, 2000; Rousseau *et al.*, 2001a). To differentiate between slow and fast processes, two additional monitoring campaigns have been conducted with the same pilot plant under different temperature conditions. During these 10-day campaigns, samples were collected at regular, short intervals. This chapter describes the results from these additional monitoring campaigns and compares these short-term data sets with the long-term one.

4.3. MATERIAL AND METHODS

4.3.1. Description of the experimental lay-out

This two-stage experimental constructed wetland was built in 1997 by Aquafin NV at the wastewater treatment plant of Aartselaar (54000 PE). To allow comparison, it was conceived as 2 identical, parallel two-stage reed beds that can be fed independently. Pretreated wastewater from the WWTP Aartselaar is first pumped to a vertical subsurface-flow (VSSF) CW (Fig. 4.1), drains through this bed and then flows through a horizontal subsurface-flow (HSSF) CW (Fig. 4.2) after which it is discharged again into the large WWTP.

Figure 4.1. Vertical subsurface- flow reed beds beds

Figure 4.2. Horizontal subsurface-flow reed

Both CWs are positioned on a 5 cm thick stabilizing layer that consists of Rhine sand mixed with 10% cement. This layer is covered by a geotextile to protect the above HDPE foil. In the HSSF and VSSF reed beds, several layers can be further distinguished.

Vertical subsurface-flow beds. Until August 2001, wastewater was pumped from the primary clarifier of the WWTP to the centre of the two VSSF beds, where it flowed from a bucket to the soil matrix. This method did not ensure an equal distribution of wastewater over the available surface area. Therefore, an H-shaped piping system was constructed, with one inlet in the centre of the H and 4 outlets in the 'legs' of the H, to ensure a better distribution of the wastewater. In every VSSF bed, a gravel (∅ 60 – 100 mm) layer of 30 cm was positioned on top of the HDPE foil. Two perforated PVC drainage tubes were buried in this layer to evacuate the effluent, at intervals of 50 cm. Above this drainage layer, a geotextile prevents the filter layer on top to penetrate in the drainage zone. The filter layer is a 60 cm thick 50/50 mixture of sand (d_{10} of 0.25 – 0.45 mm) and gravel (d_{10} of 2 – 4 mm). Seedlings of *Phragmites* spp. were planted in this substrate at a density of 12 plants m^{-2} (Fig. 4.3.a). Effluent is transported from the drainage tubes to a first sampling chamber (chambers A1 and A2) as shown in Figures 4.3.b and 4.4. At the end of this drainage tube, a flexible tube allows adjustment of the water level in the reed bed. Finally, the wastewater flows from chambers A1 and A2 to the HSSF beds via a 15 cm high riser in the inlet zone of the HSSF beds. A lithium tracer test by Capals (1998) yielded an estimated hydraulic retention time of 20 hours for the VSSF reedbed and also 20 hours for chambers A1 and A2.

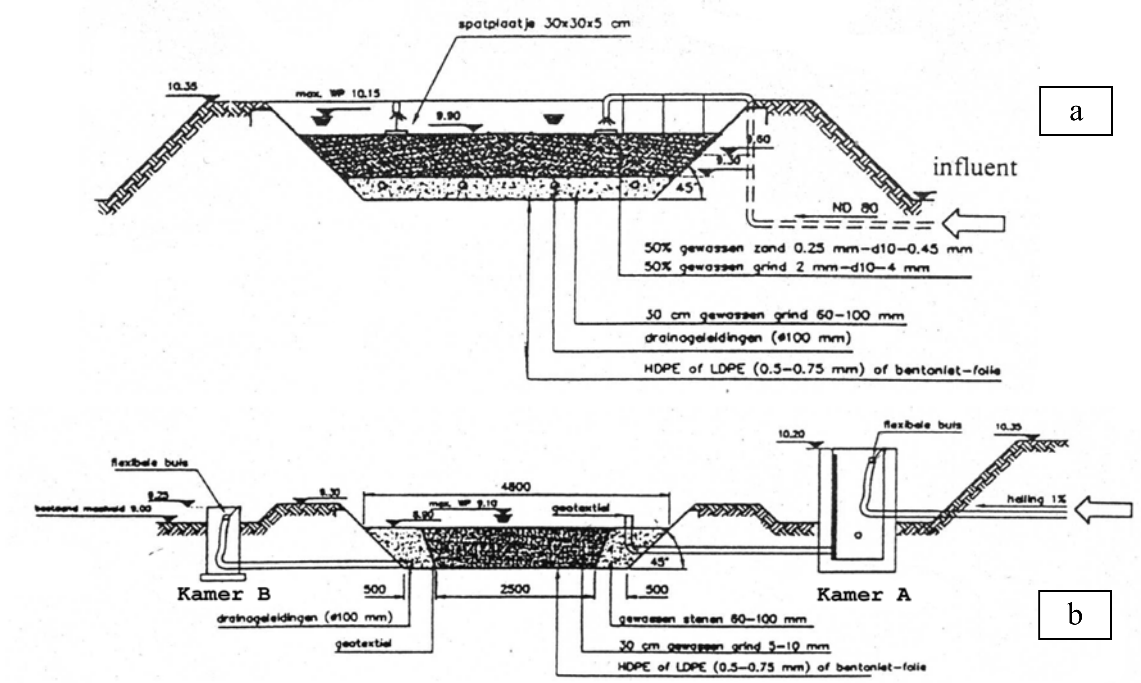

Figure 4.3. Schematic lay-out of the vertical subsurface-flow (a) and horizontal subsurface-flow (b) reed beds.

Horizontal subsurface flow beds. In the inlet zone of the HSSF beds, a stone layer of 60 cm depth and 125 cm width assures an equal distribution of wastewater over the entire bed width. Between the inlet and outlet zones and the filter layer, a water permeable geotextile prevents mixing of the matrix material of these zones. The 60 cm deep filter layer consists of coarse gravel with a d_{10} of 5-10 mm. De Wilde (2001) investigated the porosity of the upper filter layer after several years of wastewater loading and found values of about 46%. Reed seedlings were again planted at a density of 12 plants m^{-2}. In the outlet zone, one single drainage tube as long as the bed width, collects the effluent and leads it to sampling chambers B1 resp. B2. A flexible tube is connected to this drainage tube to allow adjustment of the water level in the reed bed. The hydraulic retention time of the HSSF beds was estimated at 15 hours by means of a lithium tracer test (Capals, 1998).

Surface areas. Table 4.1 gives the surface areas of the different zones in the vertical as well as the horizontal subsurface-flow reedbeds. Since the dikes have a slope of 45°, the areas are also given as mid-depth areas.

Table 4.1. Areas of the different zones in the experimental CW in Aartselaar.

Zones	**Area in lane 1 (m^2)**	**Area in lane 2 (m^2)**	**Average area (m^2)**
Total area	33.87	32.06	32.97
VSSF bed	14.84	13.24	14.04
HSSF bed	19.03	18.82	18.93
filter layer HSSF bed	9.64	9.41	9.53
inlet zone HSSF bed	4.53	4.55	4.54
outlet zone HSSF bed	4.86	4.86	4.86
VSSF bed (mid-depth)	8.67	9.01	8.84
HSSF bed (mid-depth)	8.13	7.86	8.00
Total area (mid-depth)	16.80	16.87	16.84

Hydraulic schemes and hydraulic loads. During the long-term campaign (LT), the valve between A1 and A2 was closed and samples were taken from the water in the chambers (Fig. 4.4). The first lane was operated at 1 DWF (1.3 m^3 day^{-1}) and served as a control, whereas the second lane was operated at varying hydraulic loads to estimate their influence on the performance. The VSSF beds were fed intermittently. For the short-term January 2001 campaign (STjan), two different hydraulic loading rates were used: 1 DWF (1.3 m^3 day^{-1}) from 20 till 25 January and 1.5 DWF (1.9 m^3 day^{-1}) from 25 till 29 January. The VSSF beds were fed alternately (period of 1 day) and intermittently (period of 100 min). The valve between A1 and A2 was opened, but samples were taken directly in the drainage tubes (Fig. 4.4). Both sampling procedures had the disadvantage that the volume in the sampling chambers was not negligible compared to the wetland volume, thus causing an additional residence time and a buffering influence. Therefore, during the short-term August 2001 campaign (STaug), A2 was by-passed thus forcing the effluent directly to A1, and the volume of sampling chamber A1 was also being reduced considerably (Fig. 4.4). Again, two different hydraulic loading rates were used: 1 DWF (1.3 m³ day^{-1}) from 14 till 17 August and 3 DWF (3.9 m³ day^{-1}) from 17 till 23 August. The VSSF beds were fed alternately (period of 1 day) and intermittently (period of 100 min).

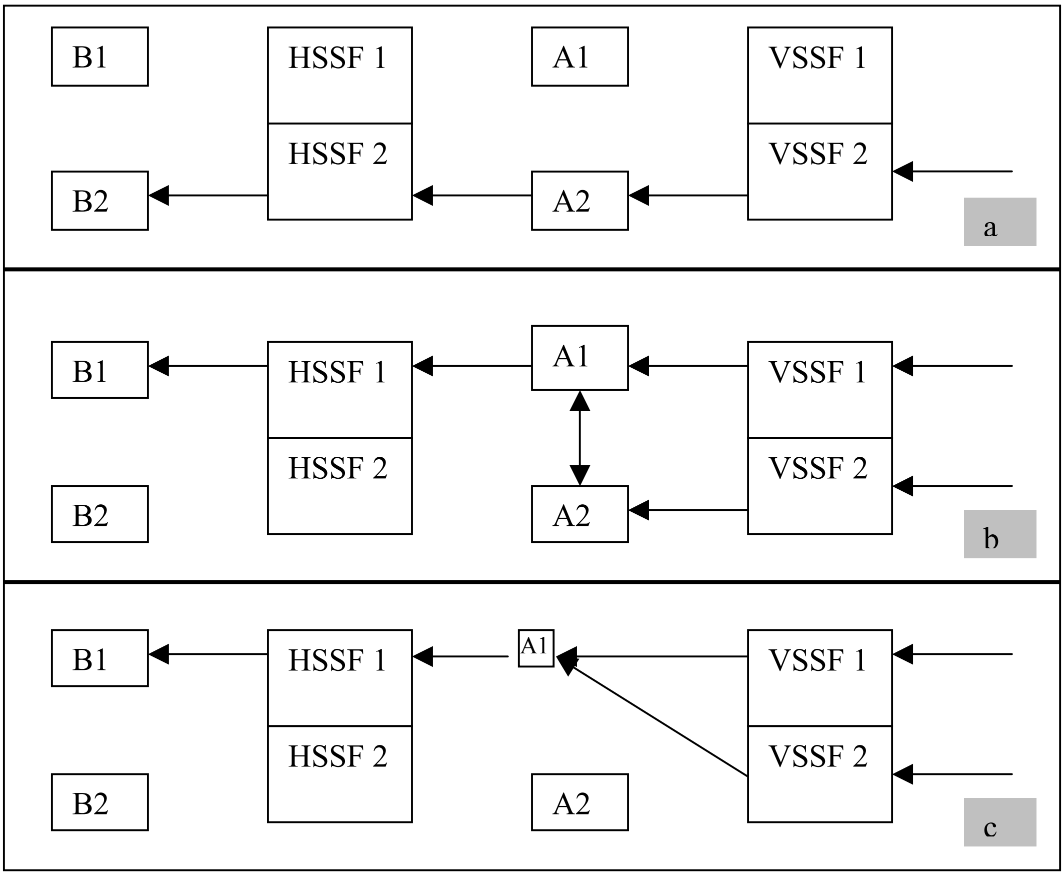

Figure 4.4. Different hydraulic lay-outs of the experimental constructed wetland in Aartselaar. (a) long-term measuring campaign (1997 – 2000), (b) short-term January 2001, (c) short-term August 2001.

4.3.2. Sampling procedures and analytical methods

Long-term, low-frequent data set (LT). Samples were usually collected biweekly from 22 April 1997 till 8 June 2000, except for February-March 1998, March-April 1999, August-September 1999 and January-February 2000. Water samples were grabbed from the end of the rectangular primary clarifier and from sampling chambers A2 and B2. They were then analysed for BOD, COD, SS, TN, TP, KJN, NH_4-N, NO_2-N, NO_3-N and o-PO_4 by the accredited laboratory of Aquafin NV according to Standard Methods (1992). Infrequent data on water temperature and precipitation were also registered on site from 17 June 1997 till 17 March 1999. Additional meteorological data were collected *a posteriori* from http://www.weeronline.be/, station Antwerpen-Deurne.

Short-term, high-frequent winter data set (STjan). Data on cold temperature performance were collected from 20 till 29 January 2001. Composite samples were taken with automated samplers at intervals specified in Table 4.2. and at the same locations as during the long-term campaign, i.e. in the primary clarifier and measuring wells A and B. These were analysed according to Standard Methods (1992) for COD, SS, NH_4-N, NO_3-N, NO_2-N, KJN, o-PO_4, TP and pH, by the Aquafin laboratory. Precipitation was measured on site via a tipping-bucket rain gauge. Additional meteorological data were again collected from http://www.weeronline.be/, station Antwerpen-Deurne.

Short-term, high-frequent summer data set (STaug). Summer performance was monitored from 14 till 23 August 2001. Composite samples were taken with automated samplers at intervals specified in Table 4.2. and at the same locations as mentioned before. The following variables were monitored on a high-frequent basis: COD, SS, NH_4-N, NO_3-N, KJN, o-PO_4, TP, pH, water temperature, air temperature, irradiance and precipitation. As before, additional meteorological data were collected from http://www.weeronline.be/, station Antwerpen-Deurne.

Table 4.2. Sampling frequencies during the winter and summer 2001 measuring campaigns at the pilot-scale constructed wetland in Aartselaar.

Sampling location	**Influent flow (m^3 day^{-1})**	**Sampling frequency (h) Winter 2001**	**Sampling frequency (h) Summer 2001**
Influent	1.3	2	2 or 3
	1.9	2	2 or 3
Effluent VSSF CW	1.3	4	6
	1.9	3	6
Effluent HSSF CW	1.3	8	8
	1.9	4	8

4.3.3. Data treatment

Whenever needed, especially for the long-term data set, detection limits are used in the graphs to represent data that were below that limit. Data that were obtained during a malfunction of the system were also deliberately omitted.

4.4. RESULTS

4.4.1. Meteorological conditions

Detailed monthly averaged precipitation and temperature data for the LT campaign are given in Figure 4.5. Belgium has a temperate climate, with average air temperatures for the period 1997-2000 around 5 °C during the winter months and around 18 °C during the summer months. September and October 1998 and December 1999 were the wettest months with respectively 211, 153 and 170 mm precipitation. February 1998 was the driest month with only 16 mm precipitation.

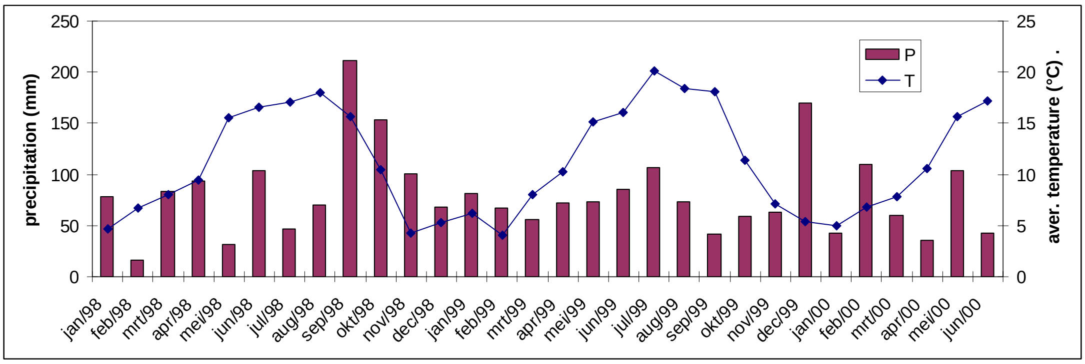

Figure 4.5. Monthly averaged air temperature and precipitation data from meteo station Deurne, located nearby the pilot-scale constructed wetland in Aartselaar.

Daily averaged meteo conditions during the STjan and STaug campaigns are summarized in Figure 4.6. January 2001 was clearly a wetter month than August 2001. Air temperatures in January varied between 0 and 10 °C whereas in August they fluctuated between 20 and 25 °C.

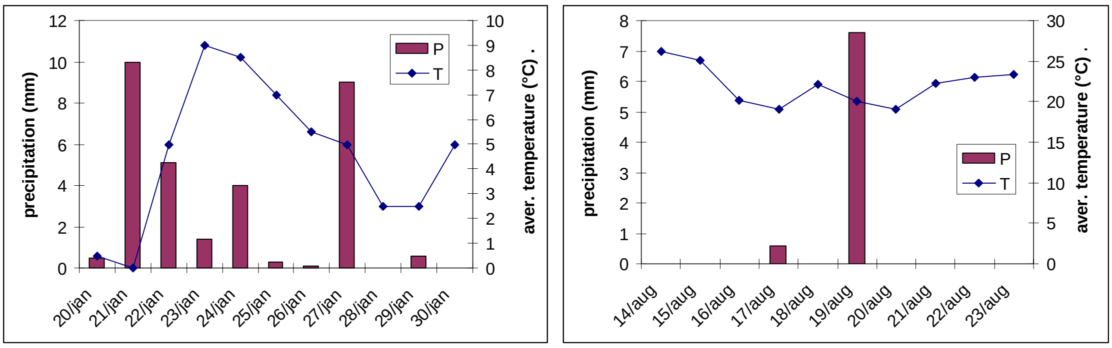

Figure 4.6. Daily averaged air temperature and precipitation data from meteo station Deurne, located nearby the pilot-scale constructed wetland at Aartselaar.

4.4.2. Chemical Oxygen Demand (COD)

Figure 4.7 shows the variations of influent and effluent COD concentrations for the different measuring campaigns. Increases in flow rate are indicated by the vertical line. Seasonal and yearly variations of concentrations and removal efficiencies are given in Table 4.3.

Table 4.3. Influent and effluent chemical oxygen demand concentrations of the VSSF and HSSF reed beds in Aartselaar and associated removal efficiencies for different seasons, years and measuring campaigns.

	Concentrations (mg COD l^{-1})			Removal efficiencies (%)		
	influent	A	B	VSSF	HSSF	TOT
spring 1997	383.8	64.4	54.1	83.2	16.0	85.9
summer 1997	360.8	25.9	24.0	92.8	7.4	93.3
autumn 1997	301.8	35.5	25.7	88.2	27.5	91.5
winter 1997-1998*	241.3	41.0	27.0	83.0	34.1	88.8
spring 1998	395.2	85.5	41.1	78.4	51.9	89.6
summer 1998	323.3	56.6	39.6	82.5	29.9	87.7
autumn 1998	100.4	25.5	22.8	74.6	10.8	77.3
winter 1998-1999	135.7	23.9	24.6	82.4	*-3.0*	81.8
spring 1999	329.4	96.6	56.1	70.7	41.9	83.0
summer 1999	261.6	27.6	27.3	89.4	1.0	89.6
autumn 1999	313.3	28.1	34.0	91.0	*-20.8*	89.1
winter 1999-2000	257.3	21.6	23.2	91.6	*-7.4*	91.0
1997*	336.2	40.4	32.8	88.0	18.8	90.2
1998	228.0	47.3	31.3	79.2	33.9	86.3
1999	301.6	50.5	39.2	83.3	22.3	87.0
springs 1997-1998-1999	369.5	82.2	50.4	77.8	38.6	86.3
summers 1997-1998-1999	315.2	36.7	30.3	88.4	17.4	90.4
autumns 1997-1998-1999	238.5	29.7	27.5	87.5	7.5	88.5
winters 1997-1998-1999*	175.8	26.0	24.6	85.2	5.4	86.0
January 2001	184.7	36.0	30.3	80.6	15.6	83.6
August 2001	426.3	64.8	62.1	84.8	4.2	85.4

* 2 outliers removed, caused by malfunctioning of primary clarifier

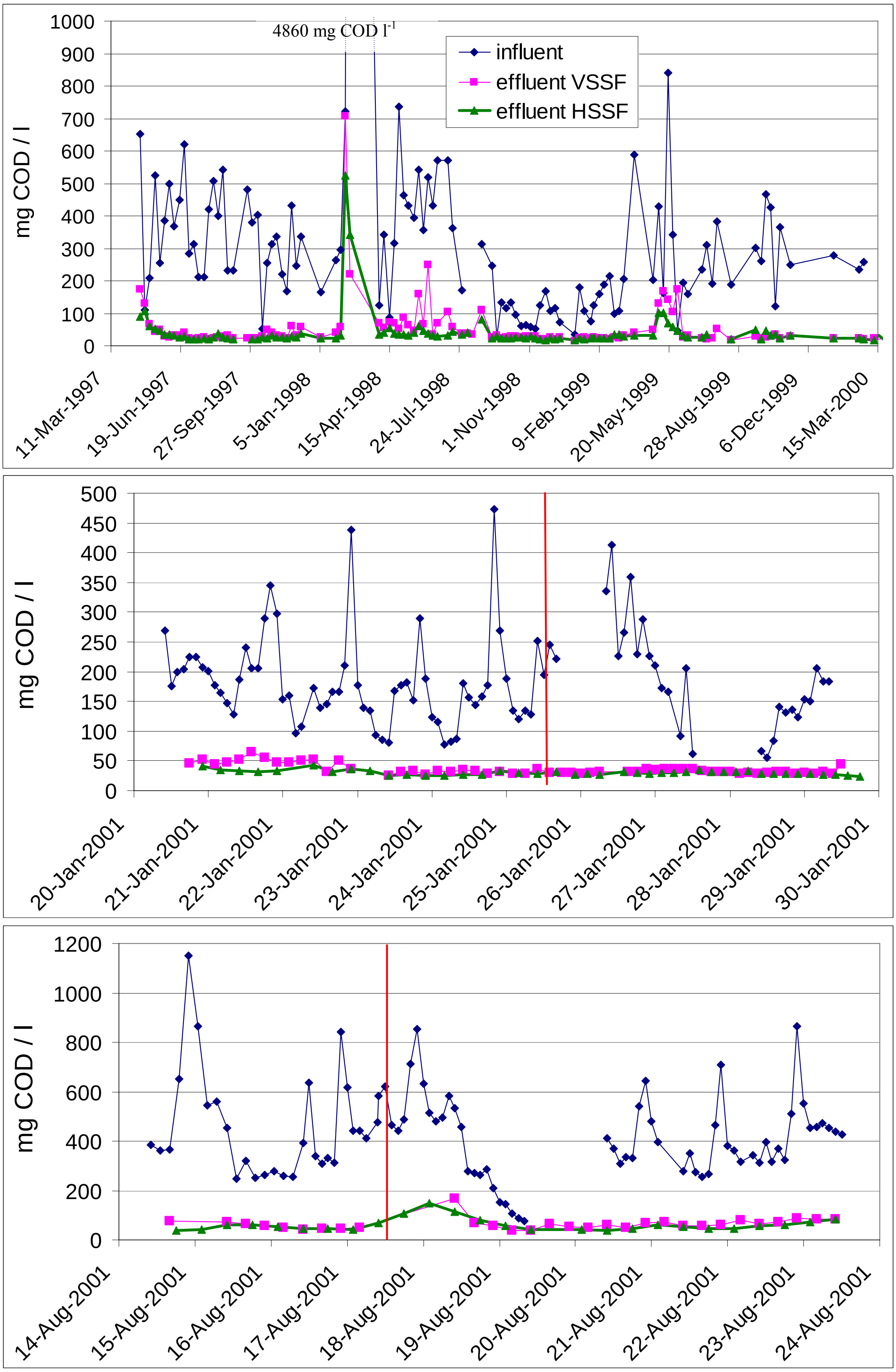

Figure 4.7. Influent and effluent COD concentrations in the pilot plant in Aartselaar for LT (upper), STjan (middle) and STaug (lower) campaigns. Vertical lines indicate the change in loading rate.

Except for a few measurements during the long-term campaign, all COD effluent concentrations largely meet the Flemish standards for small-scale wastewater treatment plants, i.e. 250 mg COD l^{-1} (cf. 2.3 Effluent standards in Flanders). These few exceedances were due to a malfunctioning of the primary clarifier, thus causing a high particle load onto the VSSF wetlands with subsequent clogging problems and reduced removal efficiencies. Stricter consents could easily be met since most of the time the effluent concentrations remained below 100 mg COD l^{-1}.

Although there is some variation in removal efficiencies of both reed beds, the HSSF reed bed seems to partly compensate for the VSSF reed bed, resulting in a relatively stable overall performance of the system, i.e. grossly between 80 and 90%. However, the major contribution to pollutant removal consistantly comes from the VSSF wetlands as it is the higher loaded system. Reversing the wetland order in this hybrid system would result in the highest removal efficiency being found in the HSSF CW. Removal efficiencies of the HSSF bed are more erratic due to the low concentration range. The lowest performance was noted during the autumn of 1998 (77%) and seems to be related to the substantially lower-than-normal influent concentration of only 100 mg COD l^{-1}. However, no significant relations could be detected between influent concentration c.q. loading and removal efficiencies. There seems to be no important decline or improvement of the performance over the years (bed maturation) or during the different seasons. Nevertheless, noticeably higher COD effluent concentrations can be seen around 21 January, which could be due to the very low temperatures on that day. However, since this occurred on the first day of monitoring and there are no data available from the previous days, this could also be an artefact of an earlier event and the relation with temperature is therefore not certain. Occasionally the HSSF constructed wetland is a minor source of COD, probably due to die-off and degradation of dead plants during autumn and winter.

Increasing the flow rate from 1.0 DWF to 1.5 DWF has no effect on COD effluent concentrations nor on removal efficiencies, i.e. 83 and 85% before and after the flow change. However, an increase to 3.0 DWF causes a short-lasting peak in COD effluent concentrations, that nevertheless disappears quickly. Removal efficiences therefore drop slightly from 89 to 83%.

4.4.3. Suspended solids (SS)

Figure 4.8 shows the variations of influent and effluent SS concentrations for the different measuring campaigns. Increases in flow rate are indicated by the vertical line. Seasonal and yearly variations of concentrations and removal efficiencies are given in Table 4.4.

Table 4.4. Influent and effluent suspended solids concentrations of the VSSF and HSSF reed beds in Aartselaar and associated removal efficiencies for different seasons, years and measuring campaigns.

	Concentrations (mg SS l^{-1})			**Removal efficiencies (%)**		
	influent	**A**	**B**	**VSSF**	**HSSF**	**TOT**
spring 1997	105.7	4.0	9.6	96.2	*-139.3*	90.9
summer 1997	99.5	2.9	2.2	97.1	25.0	97.8
autumn 1997	105.6	9.8	3.5	90.8	64.6	96.7
winter 1997-1998*	89.0	13.0	2.0	85.4	84.6	97.8
spring 1998	135.5	43.5	19.6	67.9	55.0	85.6
summer 1998	52.3	19.7	10.0	62.4	49.2	80.9
autumn 1998	30.1	2.2	7.8	92.7	*-255.4*	73.9
winter 1998-1999	56.7	4.1	2.3	92.8	44.1	96.0
spring 1999	99.1	48.3	19.4	51.2	59.8	80.4
summer 1999	65.4	3.0	6.0	95.4	*-100.0*	90.8
autumn 1999	112.7	2.2	6.8	98.1	*-215.4*	93.9
winter 1999-2000	91.0	2.0	5.2	97.8	*-160.0*	94.3
1997*	101.8	6.6	4.2	93.5	36.4	95.9
1998	70.1	16.8	10.1	76.1	40.1	85.7
1999	95.0	19.6	11.5	79.4	41.0	87.9
springs 1997-1998-1999	113.5	31.9	16.2	71.8	49.3	85.7
summers 1997-1998-1999	72.4	8.5	6.1	88.2	28.9	91.6
autumns 1997-1998-1999	82.8	4.7	6.0	94.3	*-28.4*	92.7
winters 1997-1998-1999*	68.5	4.9	3.0	92.8	38.8	95.6
January 2001	71.0	5.5	4.2	92.2	23.5	94.0
August 2001	136.4	14.1	15.0	89.6	*-6.3*	89.0

* 2 outliers removed, caused by malfunctioning of primary clarifier

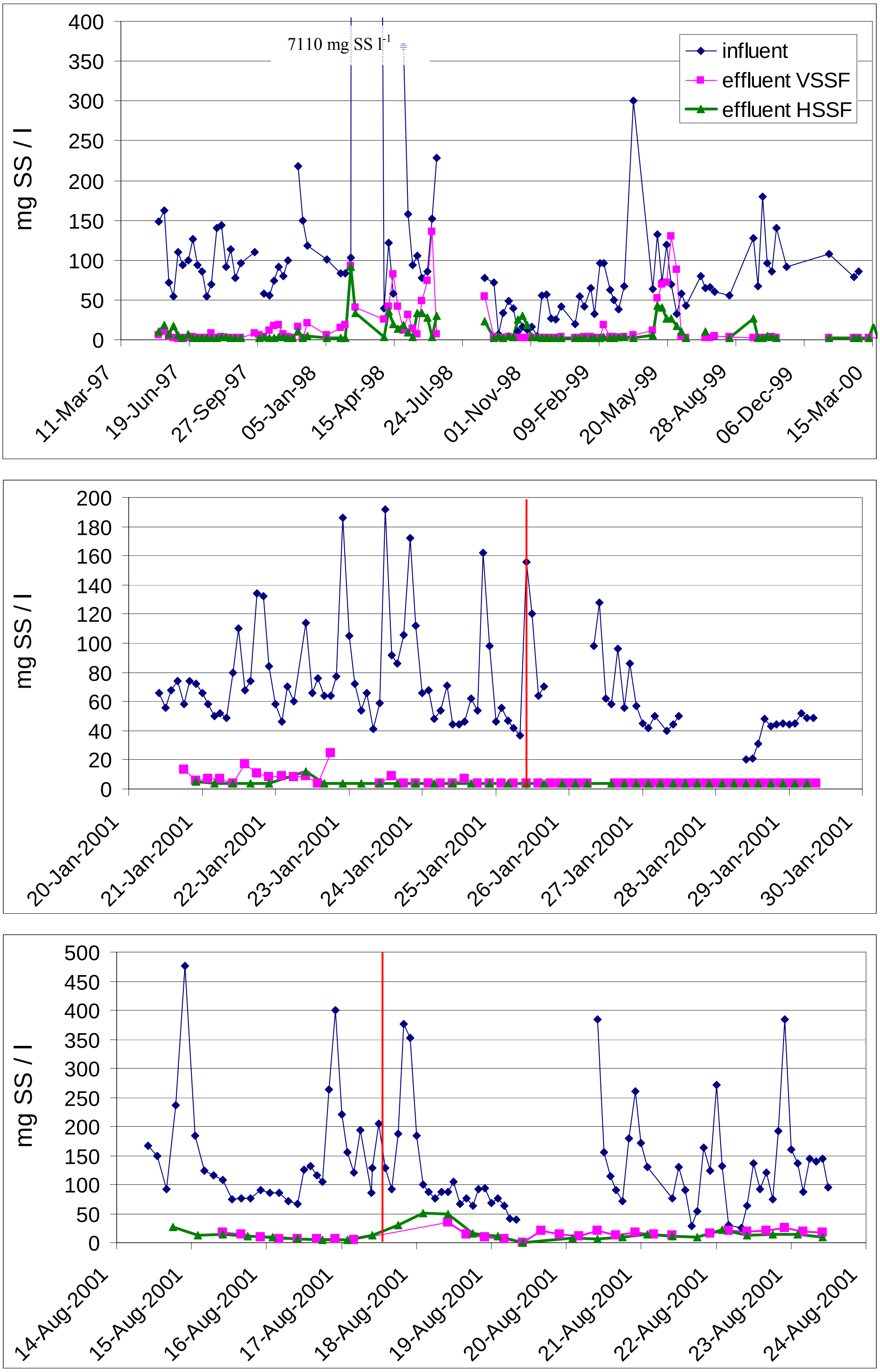

Figure 4.8. Influent and effluent SS concentrations in the pilot plant in Aartselaar for LT (upper), STjan (middle) and STaug (lower) campaigns. Vertical lines indicate the change in loading rate.

Suspended solids concentrations also largely meet the Flemish demands for small scale wastewater treatment plants, i.e. 60 mg SS l^{-1} (cf. 2.3 Effluent standards in Flanders). Even a twice as strict consent of 30 mg l^{-1} would still be respected for most of the time. A few exceptions can be noted during the long term campaign, due to a malfunctioning of the primary clarifier. This caused a high particulate load onto the VSSF reed beds with subsequent clogging problems.

The data highlight again the buffering capacity of the reed beds: the large influent variations in all data sets cannot be retraced in the effluents of both reed beds. This wastewater treatment system therefore seems to be very reliable. Stability is also partly due to the fact that the HSSF reed beds acts as a sort of backup or polishing unit for the VSSF wetlands. An increased flow rate from 1.0 DWF to 1.5 DWF has no effect on SS effluent concentrations as there is a less than 1% difference in removal efficiencies for both loading rates. On the contrary, an increase to 3.0 DWF causes a short-lasting peak that however quickly disappears again. Because the concentrations stabilize at the same level as before, the SS removal efficiency drops by only 5% from 92 to 87%.

The overal performance is consistently above 80% reduction, with one exception during autumn 1998 when only 74% of SS was removed. This appears to be correlated with the extremely low influent concentrations of particles during that period (only 30 mg SS l^{-1}).

There seems to be a slight decline in performance after the first year, mainly due to reduced efficiencies of the VSSF wetland. The HSSF reed bed occasionally seems to be a source of suspended solids, most probably due to breakdown of dead plants and litter during autumn and winter. However, data should be interpreted cautiously since SS effluent concentrations of the VSSF reed bed are already very low and a minor absolute increase of SS concentrations after the HSSF bed therefore causes a major relative increase.

4.4.4. Nitrogen species (NH_4, NO_3, orgN and TN)

Figures 4.9, 4.10 and 4.11 illustrate the variations of influent and effluent TN concentrations and the different N fractions during the various measuring campaigns. Increases in flow rate are indicated by the vertical line. Seasonal and yearly variations of nitrogen concentrations and removal efficiencies are shown in Table 4.5.

Nitrification performance of the system is consistently above 65%, resulting in ammonium effluent concentrations below 10 mg N l^{-1} most of the time. Table 4.5 clearly shows that in the first place the VSSF reed bed is responsible for nitrification, most probably because of the aerobic conditions prevailing. Indeed, De Wilde (2001) measured dissolved oxygen concentrations in the VSSF reed bed effluent during STjan between 2 and 6 mg O_2 l^{-1}. Ammonium removal is clearly hampered by clogging, as can be seen in Figure 4.9.b during March 1998. The contribution of the HSSF wetland to ammonium removal is extremely variable, i.e. from a net production with a factor 3 during the first months of operation to an additional removal of 86% during the summer of 1999. Three-year averaged results from the long-term data set clearly show reduced nitrification activity in both the VSSF and HSSF beds during low temperature periods (Table 4.5). This phenomenon could however not be demonstrated with the results of the high-frequent measuring campaigns. The explanation for this ought to be sought in the different hydraulic loading rates, with the 3.0 DWF having a negative impact on the nitrification performance, even with the higher recorded temperatures. An increase in flow rate from 1.0 to 1.5 DWF during the winter of 2001 caused no increase in effluent ammonium concentrations. During the summer of the same year, the increase in flow rate from 1.0 to 3.0 DWF on the contrary caused the effluent concentrations to peak. They dropped shortly after the event, but levelled off at a higher effluent concentration than before. Due to a competitive advantage towards oxygen, it is likely that the heterotrophic bacteria first degrade the higher organic load, thus leaving little or no oxygen for the autotrophic bacteria. A second reason could be the reduced contact time between substrate and bacteria, which would again give the nitrifiers the disadvantage. Overall data reveal no clear effect of maturation of the constructed wetlands on the nitrification capacity.

Whilst the VSSF reed bed is a consistent net nitrate producer, the HSSF wetland is a consistent net nitrate remover. However, these processes are not balanced, thus causing a net nitrate production in the entire system. As is commonly reported, influent nitrate concentrations are mostly below the detection limit. Only during periods of rainfall, elevated influent nitrate levels were measured. Table 4.5 clearly shows that denitrification is temperature dependent with the best nitrate removal efficiencies occuring during summer. Vandaele *et al.* (2000) concluded that denitrification performed better after the reed beds reached a certain level of maturity due to higher levels of available carbon. This can not really be confirmed by the year-averaged efficiencies, but is certainly confirmed by the summer performances: very low in 1997 but significantly higher in 1998 and 1999. Effluent nitrate concentrations are lower during the 3.0 DWF period. However, this is not due to a better denitrification (more anoxic conditions at higher flow rates), but due to the reduced nitrification.

At all stages, nitrite concentrations were very low and are thus no source of concern.

From Figures 4.9, 4.10 and 4.11, it is obvious that the reed beds perform very well as filters to eliminate organic nitrogen (defined as TN minus NH_4-N and NO_x-N), either by filtration of particulate organic nitrogen and/or via hydrolysis and mineralisation and further processing as ammonium.

As far as the overall nitrogen elimination capacity concerns, considerable variation could be noted during different seasons, with removal efficiencies varying between –72% and +83%. Resulting effluent concentrations are therefore also very variable, with an overall average of about 15 mg N l^{-1}. Three-year averaged results from the LT campaign and data from the high-frequent campaigns seem to support the theory that nitrogen removal is strongly limited by colder periods. However, nitrogen removal efficiency during the winter of 1997-1998 was one of the highest ones measured. One hypothesis could be that this is due to the higher loading rates c.q. influent concentrations during this first winter of operation.

Table 4.5. Influent and effluent ammonium, nitrate and total nitrogen concentrations of the VSSF and HSSF reed beds in Aartselaar and associated removal efficiencies for different seasons, years and measuring campaigns.

	NH_4						**NO_3**						**TN**					
	Conc (mg N l^{-1})			**Rem. Eff. (%)**			**Conc (mg N l^{-1})**			**Rem. Eff. (%)**			**Conc (mg N l^{-1})**			**Rem. Eff. (%)**		
	Inf	**A**	**B**	**VSSF**	**HSSF**	**TOT**	**Inf**	**A**	**B**	**VSSF**	**HSSF**	**TOT**	**Inf**	**A**	**B**	**VSSF**	**HSSF**	**TOT**
spring 1997	30.1	1.9	7.2	94	*-274*	76	0.1	26.7	11.3	*-26600*	58	*-11244*	38.2	41.9	26.3	*-10*	37	31
summer 1997	26.0	0.8	0.6	97	24	98	0.1	24.2	15.3	*-24118*	37	*-15209*	39.0	26.2	17.4	33	34	55
autumn 1997	29.1	3.5	2.1	88	41	93	0.1	25.6	16.4	*-23408*	36	*-14897*	41.6	30.6	19.5	27	36	53
winter 1997-1998*	20.8	7.0	7.2	66	*-3*	65	0.6	6.2	4.2	*-1000*	32	*-643*	42.4	18.5	14.6	56	21	66
spring 1998	24.8	8.7	6.5	65	25	74	0.1	7.3	5.5	*-7208*	25	*-5350*	37.0	21.0	14.0	43	33	62
summer 1998	15.2	4.8	3.3	68	31	78	0.6	15.1	4.9	*-2610*	67	*-787*	31.5	23.0	17.9	27	22	43
autumn 1998	10.7	1.1	0.4	90	59	96	0.4	29.7	6.3	*-7624*	79	*-1541*	15.7	31.8	7.5	*-103*	76	52
winter 1998-1999	8.0	0.5	0.3	94	44	96	2.3	11.7	11.5	*-405*	2	*-395*	15.0	13.1	12.7	13	3	15
spring 1999	29.2	11.4	5.0	61	56	83	0.1	4.5	3.5	*-4389*	23	*-3367*	33.8	16.1	9.9	52	39	71
summer 1999	14.8	1.4	0.2	90	86	99	0.1	14.8	1.8	*-14740*	88	*-1733*	21.6	17.9	3.7	17	80	83
autumn 1999	23.4	1.2	2.7	95	*-131*	88	0.2	20.8	12.4	*-13133*	41	*-7773*	33.2	23.5	17.1	29	27	49
winter 1999-2000	14.9	0.1	0.1	99	*-40*	99	1.0	53.4	21.9	*-5072*	59	*-2021*	19.5	53.5	33.5	*-175*	37	*-72*
1997*	27.0	2.7	2.6	90	4	90	0.2	22.8	13.0	*-13560*	43	*-7698*	40.0	31.2	20.2	22	35	50
1998	14.5	3.6	2.5	75	31	83	0.8	16.1	7.2	*-1848*	56	*-767*	23.8	22.6	12.2	5	46	49
1999	22.7	4.6	2.7	80	40	88	0.2	20.3	9.7	*-8593*	52	*-4059*	29.3	25.7	16.1	12	37	45
springs 1997-1998-1999	28.0	7.4	6.3	74	15	78	0.1	12.8	6.8	*-12732*	47	*-6654*	36.3	26.3	16.7	28	36	54
summers 1997-1998-1999	18.7	2.3	1.4	87	41	93	0.3	18.1	7.4	*-7053*	59	*-2817*	30.7	22.4	13.0	27	42	58
autumns 1997-1998-1999	21.1	1.9	1.8	91	9	92	0.2	25.4	11.7	*-11600*	54	*-5284*	30.2	28.6	14.7	5	49	51
winters 1997-1998-1999*	11.5	1.4	1.3	62	8	65	1.3	23.8	12.5	*-1724*	47	*-861*	20.7	25.2	18.5	-22	27	11
January 2001	10.3	2.6	2.0	75	25	81	0.7	11.9	11.8	*-1699*	0	*-1695*	17.2	16.0	15.3	7	4	11
August 2001	26.7	6.1	5.3	77	13	80	0.0	10.5	7.9	*-24909*	24	*-18837*	32.1	19.3	15.8	40	18	51

* 2 outliers removed, caused by malfunctioning of the primary clarifier

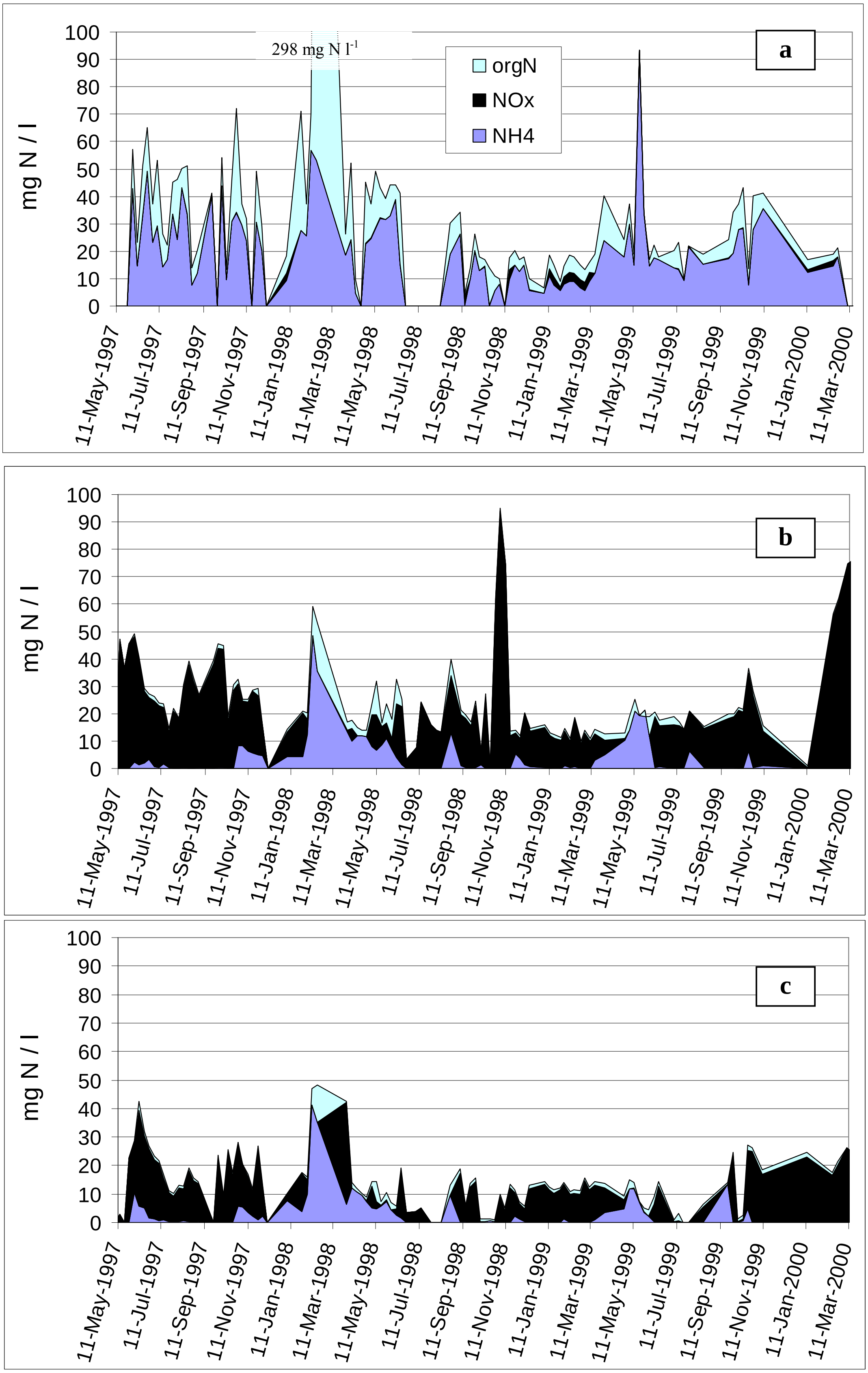

Figure 4.9. Influent and effluent nitrogen fractions in the pilot plant in Aartselaar for the long-term measuring campaign (1997-2000). a=influent, b=effluent VSSF reed bed, c=effluent HSSF reed bed.

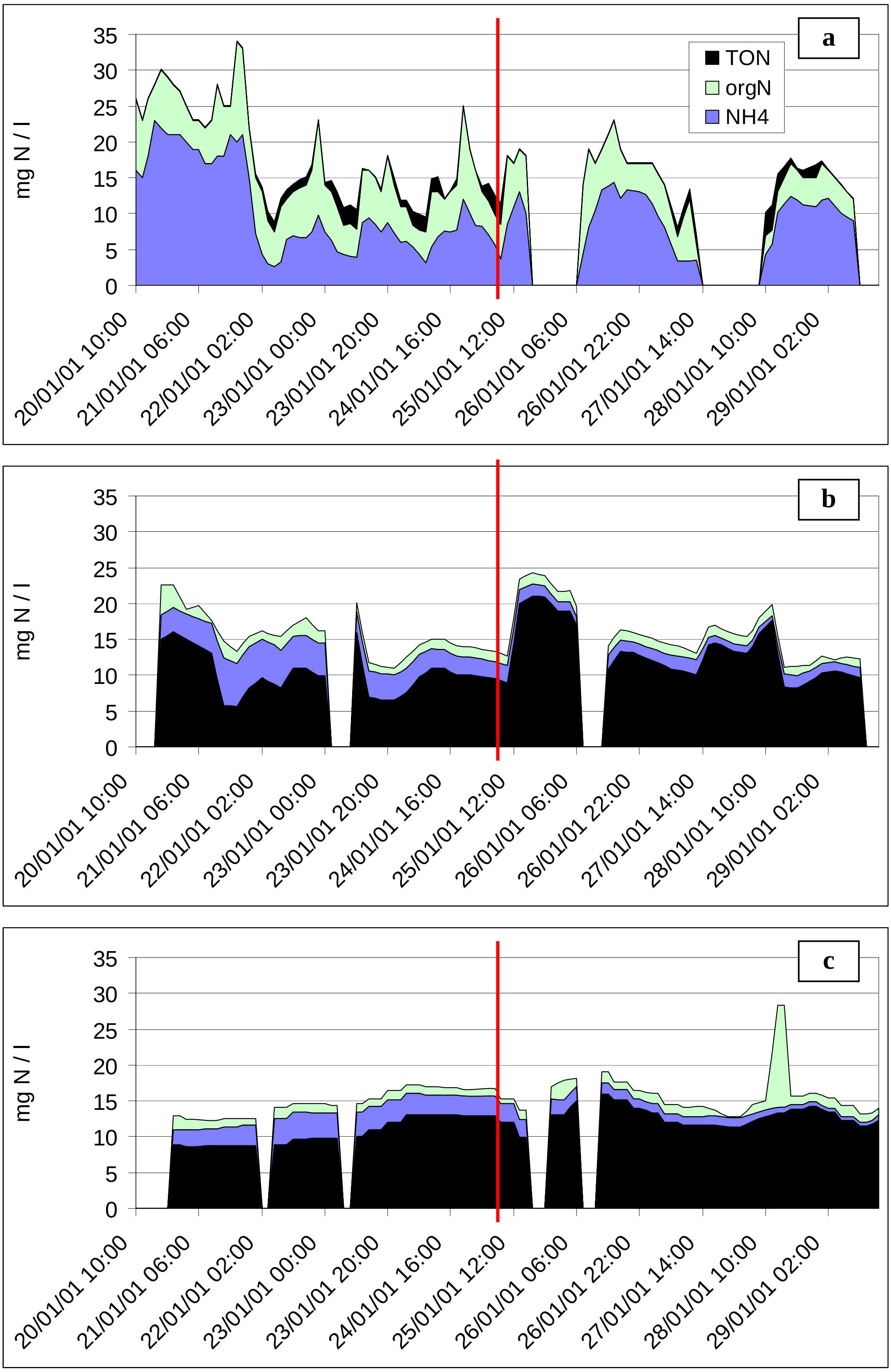

Figure 4.10. Influent and effluent nitrogen fractions in the pilot plant in Aartselaar during the winter high-frequency campaign. a=influent, b=effluent VSSF reed bed, c=effluent HSSF reed bed. Vertical lines indicate the change in loading rate.

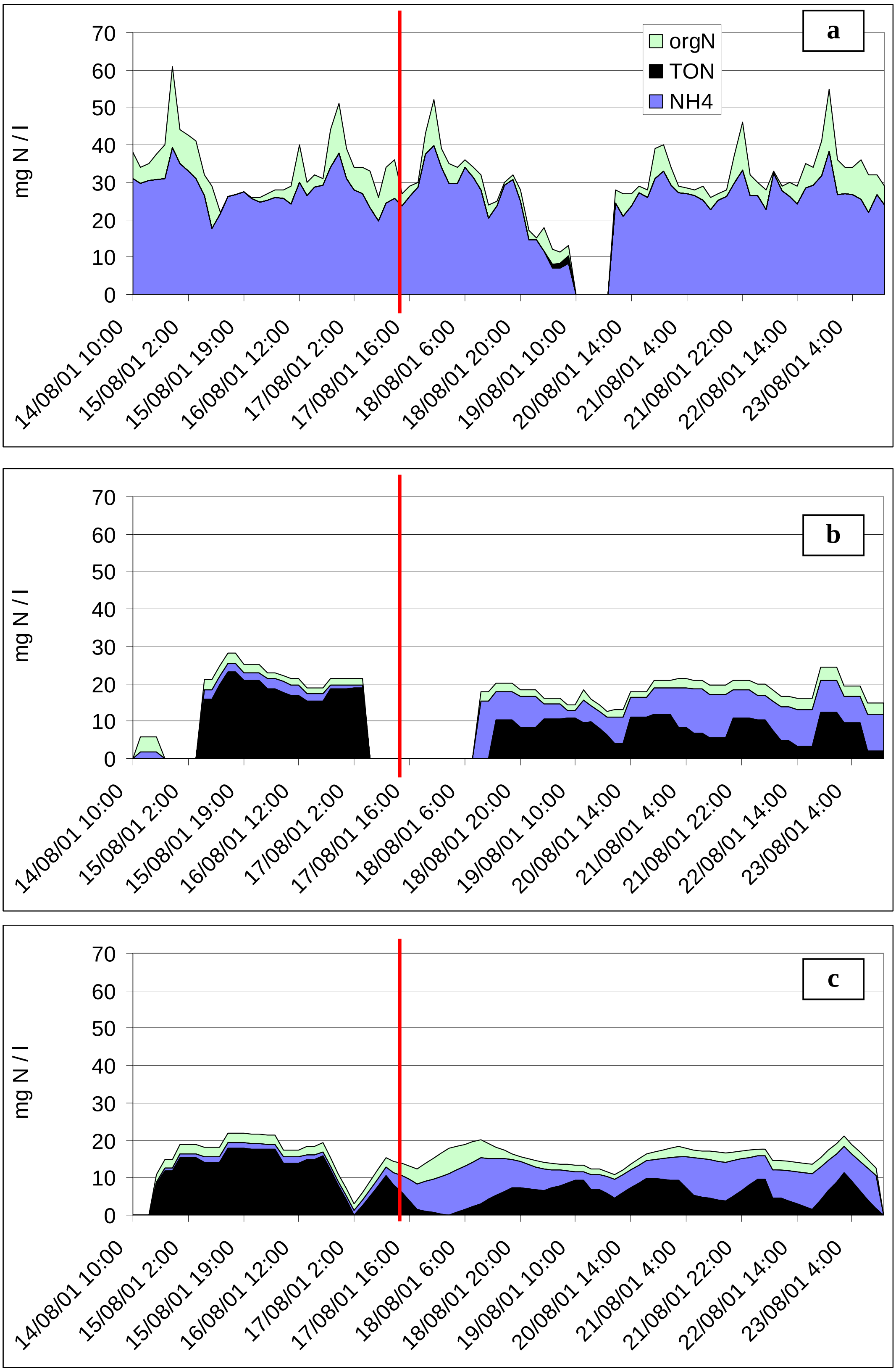

Figure 4.11. Influent and effluent nitrogen fractions in the pilot plant in Aartselaar during the summer high-frequency campaign. a=influent, b=effluent VSSF reed bed, c=effluent HSSF reed bed. Vertical lines indicate the change in loading rate.

4.4.5. Phosphate species (o-PO_4, orgP and TP)

Figures 4.12, 4.13 and 4.14 show the variations of influent and effluent TP concentrations and the different P fractions for the various measuring campaigns. Increases in flow rate are indicated by the vertical line. Seasonal and yearly variations of phosphorus concentrations and removal efficiencies are given in Table 4.6.

Ortho-phosphate removal varies considerably, from a small net production of -21% during the winter of 1998-1999 to a removal of nearly 95% during the first spring of operation. Effluent concentrations remain below 4 mg P l^{-1} for most of the time during the LT measuring campaign, below 1 mg P l^{-1} during the STjan campaign and below 2 mg P l^{-1} during the STaug one. Peaks in the influent concentrations are generally also reflected in the effluent concentrations. O-PO_4 removal seems slightly influenced by the higher hydraulic load of 3 DWF and shows higher effluent concentrations. The shift to 1.5 DWF on the contrary has no visible effect. It can furthermore be clearly observed from the data of both reed beds and of the entire system that the o-PO_4 removal capacity declines during the course of time. This is obviously due to the saturation of sorption sites or the depletion of complexation ligands. Data finally suggest a substantial effect of temperature, with better removal efficiencies during the growing season. The most probable explanations are plant uptake on the one hand and P-leaching from decaying detritus on the other hand.

Most of the organic phosphorus seems to be readily removed, except at one instance, i.e. when the flow rate was increased to 3.0 DWF during the STaug campaign. A distinct peak of organic phosphorus could then be seen in the effluent.

Total phosphorus effluent concentrations are generally in the range of 1-2 mg P l^{-1} but show relatively high sensitivity towards influent peak loadings. Assuming orgP occurs mainly in particulate form and is removed in a physical way, TP logically follows a similar pattern as o-PO_4, i.e. lower removal rates during colder periods and a distinct decline in TP removal during the course of time.

Table 4.6. Influent and effluent ortho-phosphate and total phosphorus concentrations of the VSSF and HSSF reed beds at Aartselaar and associated removal efficiencies for different seasons, years and measuring campaigns.

	o-PO_4						**TP**					
	Concentration (mg P l^{-1})			**Removal efficiency (%)**			**Concentration (mg TP l^{-1})**			**Removal efficiency (%)**		
	influent	**A**	**B**	**VSSF**	**HSSF**	**TOT**	**influent**	**A**	**B**	**VSSF**	**HSSF**	**TOT**
spring 1997	4.7	0.5	0.3	88	52	95	6.8	0.7	0.4	89	42	94
summer 1997	4.1	2.0	0.8	50	62	81	6.2	2.0	0.8	68	58	86
autumn 1997	5.2	3.3	1.5	38	52	70	6.9	3.3	1.8	52	47	74
winter 1997-1998*	2.9	2.2	1.7	24	23	41	4.6	2.4	1.8	48	25	61
spring 1998	4.6	2.1	1.3	54	37	71	6.7	3.9	1.7	43	56	75
summer 1998	5.3	2.7	1.9	49	31	64	7.6	4.9	2.1	36	56	72
autumn 1998	2.3	2.0	1.9	16	5	20	3.2	2.0	2.0	37	*-2*	36
winter 1998-1999	0.7	1.0	0.8	*-45*	17	*-21*	1.9	1.0	0.9	45	10	50
spring 1999	5.6	4.6	3.6	18	22	36	4.7	4.7	3.6	0	24	23
summer 1999	2.6	2.1	0.8	17	64	70	4.2	2.2	2.2	48	0	48
autumn 1999	3.8	4.0	2.4	*-5*	39	36	6.1	4.1	2.6	33	37	58
winter 1999-2000	1.7	1.9	1.4	*-12*	25	16	3.6	1.6	1.6	56	*-1*	56
1997*	4.5	2.0	0.9	56	55	80	6.5	2.1	1.1	68	48	83
1998	3.0	1.9	1.5	37	23	52	4.4	2.8	1.7	37	40	62
1999	4.0	3.4	2.4	13	29	38	4.9	3.4	2.7	30	22	45
springs 1997-1998-1999	4.9	2.4	1.7	51	29	65	6.1	3.1	1.9	49	38	69
summers 1997-1998-1999	4.0	2.3	1.1	42	50	71	6.0	3.0	1.7	49	43	71
autumns 1997-1998-1999	3.8	3.1	1.9	19	37	48	5.4	3.1	2.1	42	32	61
winters 1997-1998-1999*	1.2	1.4	1.1	*-17*	21	8	2.7	1.4	1.2	48	14	56
January 2001	1.7	0.2	0.1	88	42	93	2.8	0.8	0.6	72	19	78
August 2001	4.1	1.5	1.2	63	20	70	9.0	3.7	2.2	59	41	76

* 2 outliers removed, caused by malfunctioning of the primary clarifier

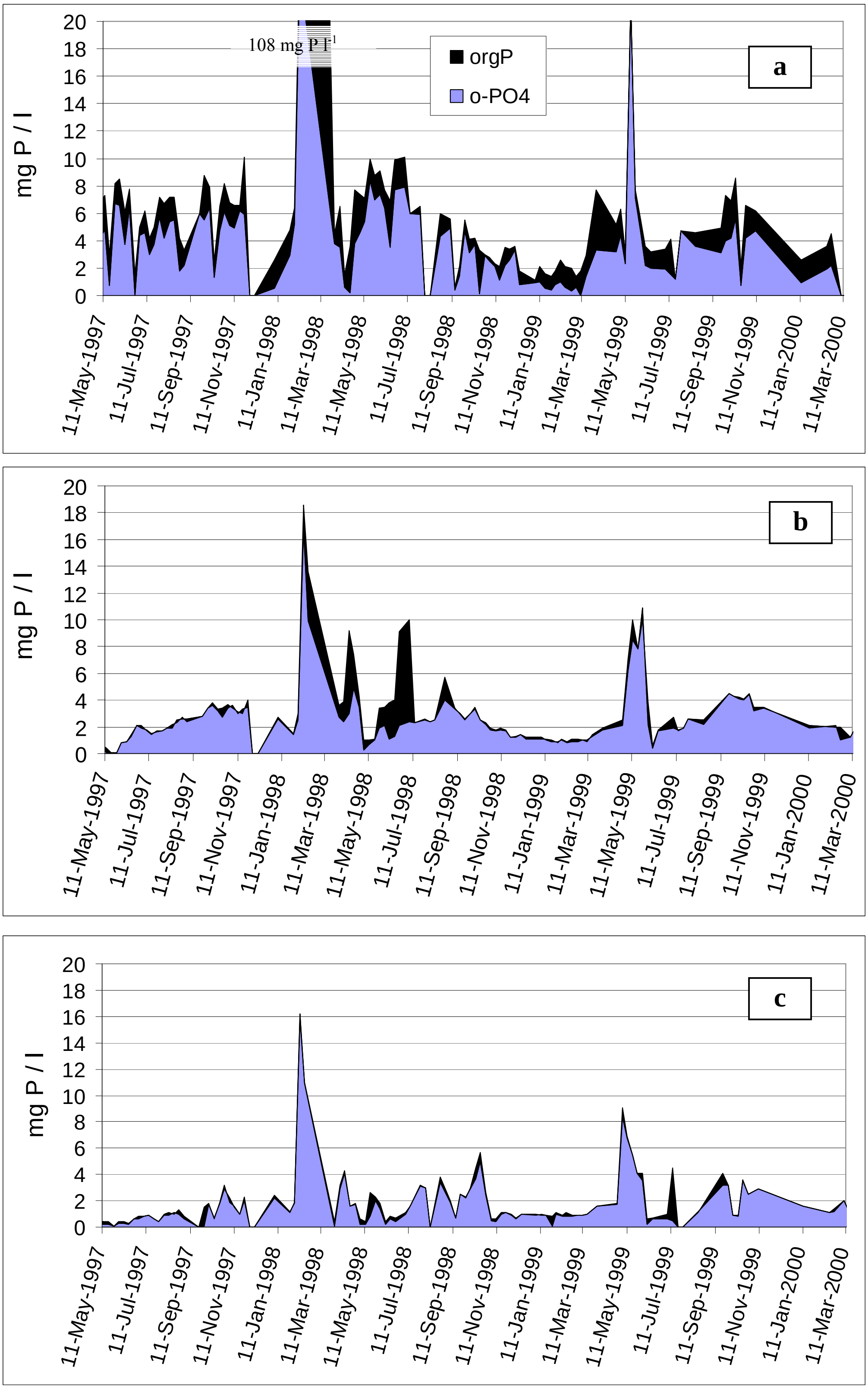

Figure 4.12. Influent and effluent phosphorus fractions in the pilot plant in Aartselaar for the long-term measuring campaign (1997-2000). a=influent, b=effluent VSSF reed bed, c=effluent HSSF reed bed.

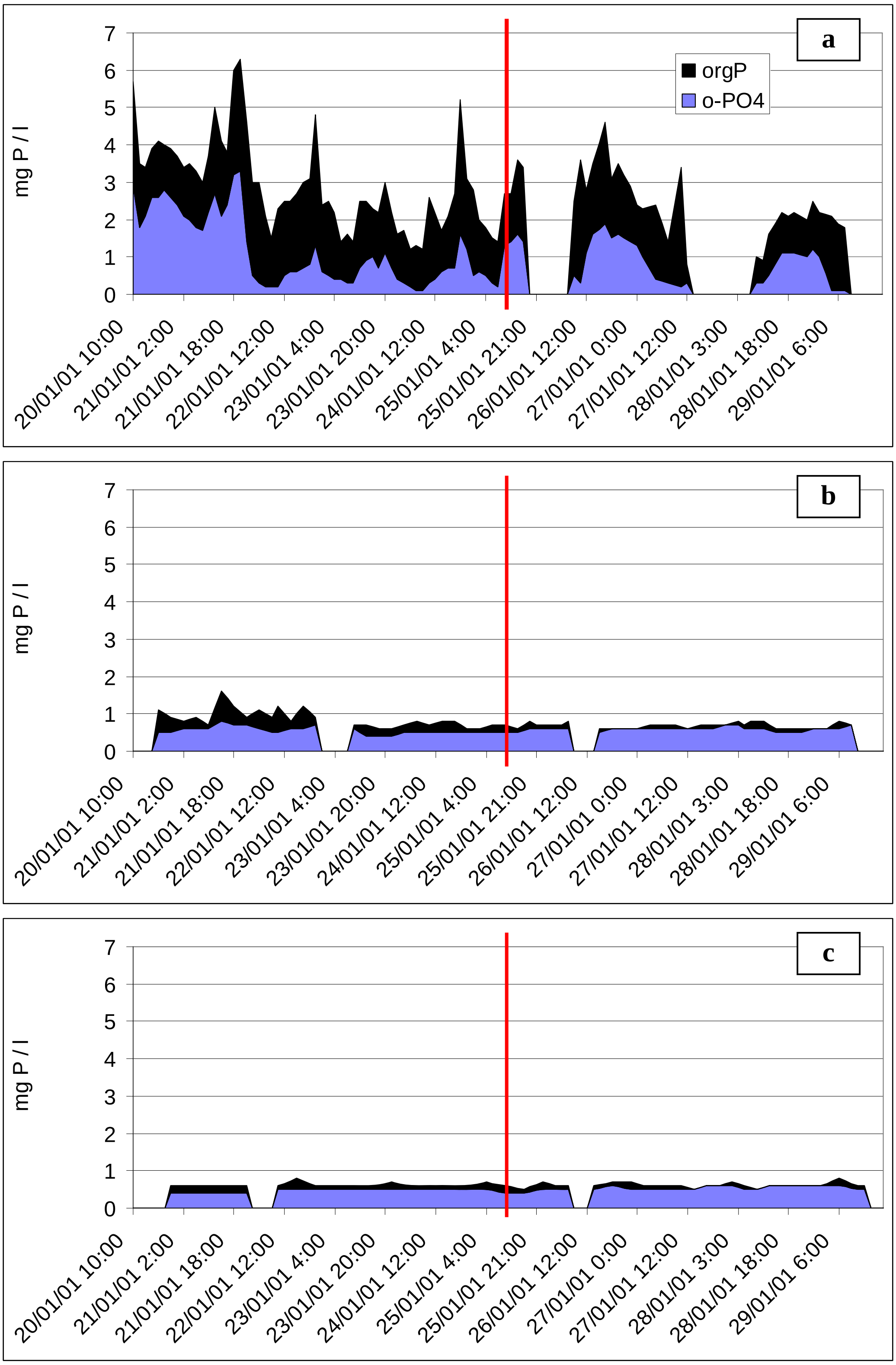

Figure 4.13. Influent and effluent phosphorus fractions in the pilot plant in Aartselaar during the winter high-frequency campaign. a=influent, b=effluent VSSF reed bed, c=effluent HSSF reed bed. Vertical lines indicate the change in loading rate.

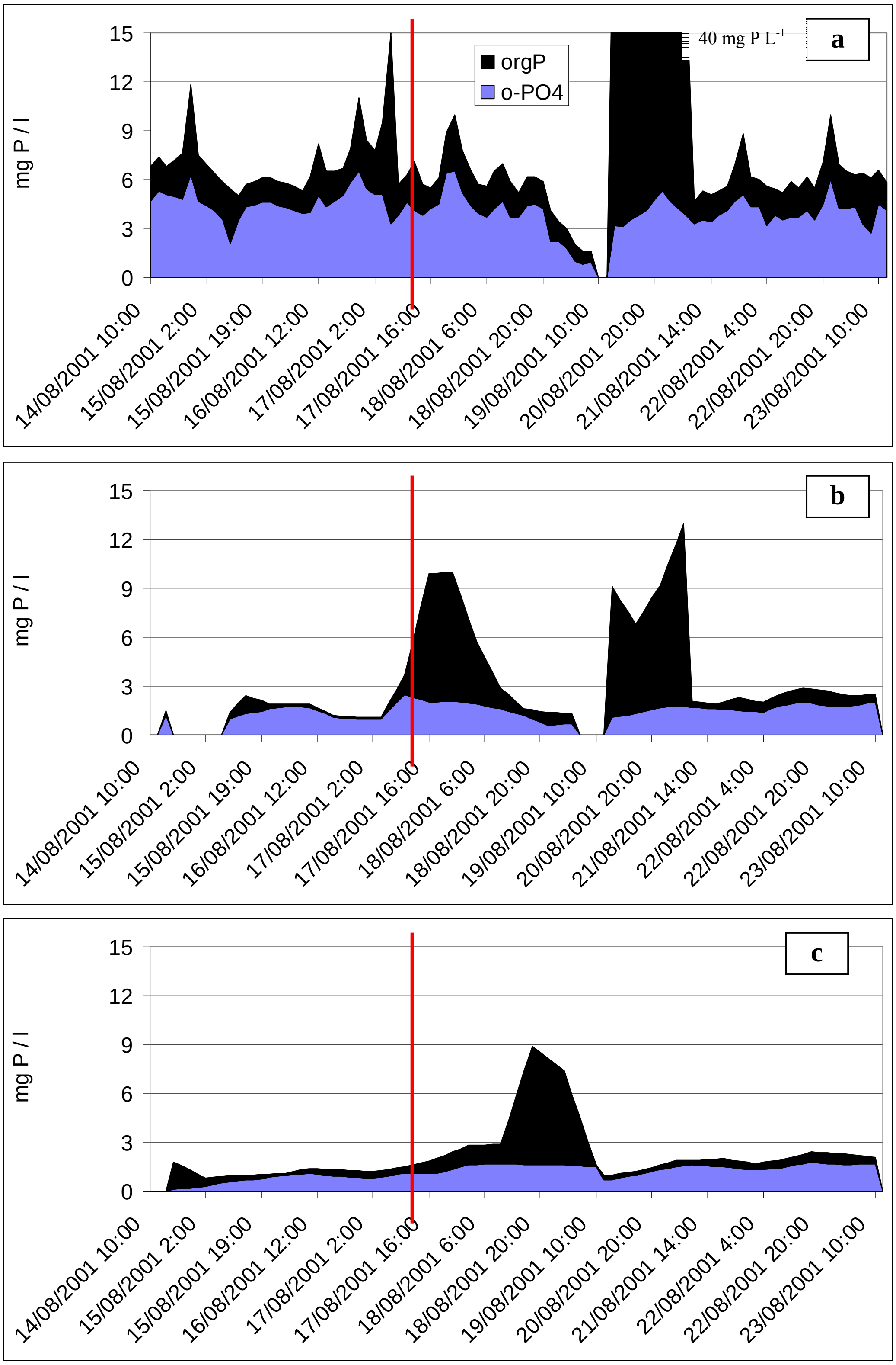

Figure 4.14. Influent and effluent phosphorus fractions in the pilot plant in Aartselaar during the summer high-frequency campaign. a=influent, b=effluent VSSF reed bed, c=effluent HSSF reed bed. Vertical lines indicate the change in loading rate.

4.5. DISCUSSION

Effluent concentrations of COD and SS in the pilot plant in Aartselaar comply with the Flemish VLAREM II standards of 250 and 60 mg l^{-1} respectively, except when the primary clarifier malfunctioned. Compared to the stringent Dutch Class IIIb standards for sensitive areas (cf. Chapter 2), this two-stage combined CW seems capable of fulfilling the 100 mg COD l^{-1} and 30 mg SS l^{-1} demands. The 30 mg TN l^{-1} standard was only exceeded during the second month of operation and when the primary clarifier malfunctioned. However, the effluent ammonium concentrations were often above the 2 mg NH_4-N l^{-1} limit. Reducing the effluent P concentrations to below 2 mg TP l^{-1} might prove more difficult with this concept as exceedances were often noted during peak loading events and in autumn conditions. Haberl *et al.* (1998) summarised the performance of 8 combined constructed wetlands consisting of VSSF and HSSF stages and found average outlet concentrations of 42 mg l^{-1} COD, 7.6 mg NH_4-N l^{-1} and 15 mg NO_3-N l^{-1}, which is in the upper range of effluent concentrations found in Aartselaar but still of comparable quality.

COD removal efficiencies in the pilot plant in Aartselaar are slightly lower than the overal 91% removal summarised in Chapter 2 for combined wetland systems. This looks somewhat surprising since the CWs in Aartselaar were operated at constant flow rates whereas the full-scale systems are regularly disturbed by peak flows. However, this seems to correspond with the findings of Ayaz and Akça (2001) that removal efficiencies are positively correlated with loading rates. This finding is in another sense backed up by the data of the HSSF wetland, i.e. the removal efficiencies of this treatment step seem inversely proportional with the effluent concentrations produced by the VSSF reed bed.

Performance data from the combined reed bed system in Oaklands Park are given by Cooper (1999). It consists of two VSSF stages in series followed by two HSSF stages in series and provides a total treatment area of 1.4 m^2 PE^{-1}. BOD_5 is reduced by 95.1% in the VSSF beds and by an additional 50% in the HSSF beds. SS is similarly reduced by 89.9% and 47.1% respectively and reaches final effluent concentrations of 9 mg SS l^{-1}.

Ammoniacal nitrogen drops by 72.3 and 20.7% resp. to 11.1 mg NH_4-N l^{-1} whereas TON increases about four-fold to 7.2 mg TON l^{-1}. Ortho-phosphates are reduced by 25.6 and 29.6% and have average effluent concentrations of 11.9 mg P l^{-1}. Data from Aartselaar do not show marked deviations from the above results, except for o-PO_4.

All data clearly show the buffering influence of the reed beds: the large influent variations (in the low-frequency as well as high-frequency data sets) cannot be retraced in the effluents of both reed beds. One should also take into account that the observed influent variations in this pilot-scale wetland probably have a smaller amplitude than would be encountered in full-scale treatment wetlands since small-scale wastewater treatment plants are notorious for their loading variations (Boller, 1997). Furthermore, during the STjan and STaug measuring campaigns, composite samples were taken which partly mask concentration variations. This indicates the need for high-frequency sampling of the influent, whilst the effluent sampling frequency may be reduced. A sudden and substantial increase of the influent loading rate can however be traced in the effluent. This was the case during the winter of 1997-1998 when the primary clarifier malfunctioned and influent concentrations therefore peaked. It also occurred when the flow rate was changed from 1 to 3 times DWF. It is hypothesised that due to the higher flow velocity, some of the settled and filtered materials were resuspended and consequently dragged out of the porous soil matrix. As soon as most of these loose materials were flushed, the effluent concentrations dropped again.

In Chapter 2, the performance of similar combined wetland systems in Flanders during different seasons was reviewed and a quite stable COD and SS removal was reported. Soto *et al*. (2000) studied a subsurface-flow wetland during two summers and two winters and found likewise only slightly higher COD removal efficiencies during summer, which were however not significantly different. Vymazal (2000) investigated the performance of 96 HSSF wetlands and concluded that temperature had little or no influence on SS removal. Data from Aartselaar allow similar conclusions, i.e. that COD and SS removal are not substantially affected by temperature.

As indicated by the data, the VSSF reed bed offers excellent nitrification conditions. Intermittent loading is indicated as an important mechanism for oxygen input by a.o. Platzer and Netter (1994) and Meuleman (1999). Additional oxygen is possibly provided by root oxygen release (Brix, 1997). pH measurements of the wastewater during the STjan and STaug campaigns (De Wilde, 2001; De Moor, 2002) confirmed that these were within the optimum interval for nitrifiers, i.e. 7.5-8.6 (Hammer and Knight, 1994). Hammer and Knight (1994) furthermore describe sharply dropping nitrification rates below 5°C. Platzer and Mauch (1997) on the contrary suggest that the impact of low temperature on nitrification is much smaller than reported in literature. Indeed, no adverse effects were found in Aartselaar during the STjan campaign and only a small decrease in ammonium removal efficiency during the LT one. Possibly, water temperatures were higher than air temperatures and the deeper layers of the filter were partly isolated from the environment (Kadlec and Knight, 1996d).

Denitrification in the HSSF wetland on the contrary seems affected by temperature and was almost completely inhibited during the STjan measuring campaign. Most authors mention 5 °C as the lower limit for denitrification, similarly as for nitrification (Hammer and Knight, 1994). Then why does nitrification continue in January 2001 while denitrification comes to a halt? Indeed, experience with activated sludge wastewater treatment indicates that nitrification is more strongly inhibited by low temperatures than denitrification (Henze *et al.,* 2000). A lack of readily available carbon seems a logical answer (van Oostrom and Russell, 1994; Ingersoll and Baker, 1998), as the influent COD concentrations were substantially higher during summer then during winter. Also, denitrification performed already substantially better after the first winter when senescent plants had degraded and released organic carbon into the HSSF bed. Platzer and Netter (1994) finally bring to the attention that high oxygen concentrations limit denitrification. Limited oxygen data of the VSSF bed effluent from the STjan campaign (De Wilde, 2001) suggest this might indeed adversely affect nitrate removal.

Average oxygen demands of the VSSF reed bed in Aartselaar for COD removal were 25.9, 15.8 and 22.0 g O_2 m^{-2} d^{-1} during respectively 1997, 1998 and 1999, when supposing that particulate COD is also aerobically converted after mineralisation. The additional oxygen demands for nitrification were 9.2, 4.1 and 6.8 g O_2 m^{-2} d^{-1} during respectively 1997, 1998 and 1999 when assuming that 4.3 g O_2 are consumed per g N converted (Cooper, 1999). Oxygen transfer capacities of VSSF reed beds were reviewed by Cooper *et al*. (1999) and were reported to be in the order of 50 to 90 g O_2 m^{-2} d^{-1}. COD removal and nitrification in Aartselaar therefore seem to consume only about 20 to 70% of the available oxygen. Platzer (1998, in Luederitz *et al*., 2001) calculated oxygen inputs via diffusion of 10 – 33 g O_2 m^{-2} d^{-1} whereas the convective input of oxygen was found to be positively correlated with the hydraulic load, i.e 6 g O_2 m^{-2} d^{-1} at 20 mm day^{-1} and 36 g O_2 m^{-2} d^{-1} at 120 mm day^{-1}. However, a higher hydraulic load will probably not lead to higher purification rates because of the reduced hydraulic residence time. Indeed, when the flow rate is raised to 3.0 DWF during August 2001, nitrification is clearly incomplete, indicating an imbalance between oxygen demand and oxygen supply. Under these conditions, the average influent oxygen demand is indeed in the order of 150 g m^{-2} d^{-1}, which is well above the oxygen transfer capability.

Laber *et al*. (2000) reviewed COD loading rates of VSSF reed beds and the relation with clogging phenomena and recommended a maximum loading rate of 80 g COD m^{-2} day^{-1}. Bavor and Schulz (1993) suggest a maximum of 40 g SS m^{-2} day^{-1} to prevent clogging. Table 4.3. shows that COD loading reaches a maximum during August 2001 with an average rate of just over 37 g COD m^{-2} day^{-1}. Since this is well below the threshold, operation at 1 DWF should prove to be sustainable on the long term. Influent SS peaked during the same period with a loading rate of nearly 12 g COD m^{-2} day^{-1}, again well below the threshold.

The observations of Fiselier (1990), Davies and Cottingham (1993) and many others that reed bed filters can get saturated with phosphorus or even become net sources of P are confirmed by the data of Aartselaar. TP removal efficiencies dropped from 83 to 45% in only three years time. During the second and third winter of operation, there was a substantial net production of ortho-phosphates in the VSSF reed bed although this observation could not be confirmed during the STjan campaign. A logical explanation would of course be that during STjan, two parallel VSSF beds were used alternately, whereas during the LT campaign only one VSSF bed was used continuously.

Cooper *et al*. (1999) elaborated on design strategies for hybrid reed bed systems and mention two possible lay-outs, i.e. HSSF – VSSF beds with recycling or VSSF – HSSF beds without recycling. The first option allows to use external carbon sources available in the wastewater and therefore likely achieves higher denitrification rates but has the disadvantage of requiring an extra pump and energy. In the second case, denitrification relies mainly on internal carbon sources which, according to the authors, should be able to support substantial denitrification due to the long hydraulic residence times of the wastewater in HSSF beds. Laber *et al*. (1997) compared the second configuration with addition of methanol as carbon source against the first one, and found slightly better TN removal efficiencies with the VSSF – HSSF – methanol system.

The overall effect of lay-out seems very limited, although loading and temperature differences render comparison quite difficult. The only variable which seems positively affected by adding a second VSSF bed to the configuration is ortho-phosphate.

As was already concluded by Vandaele *et al*. (2000), effluent standard exceedances of COD and SS only occurred when the primary clarifier was out of order. Adequate primary treatment thus seems to be of utmost importance because higher particulate influent loads cause higher effluent concentrations on a very short timescale, but also cause clogging after a couple of days, leaving the reed bed useless until the hydraulic conductivity has been restored.

One of the main conclusions for the pilot plant in Aartselaar is that it would seem to make sense to direct a small portion of the influent (10 – 15%) immediately to the HSSF beds. This would partly reduce the ‘pressure’ on the VSSF bed and therefore reduce the risk of clogging and enhance aerated conditions whilst at the same time the full capacity of the HSSF bed is used and an extra carbon source for denitrification is provided. Harvesting the plants during autumn might possibly lower N and P releases and therefore provide for lower nutrient effluent concentrations. When effluent requirements are very strict such as in sensitive areas, an additional phosphate removing filter could be added as outlined by Norvee *et al*. (2004).

4.6. MODELLING RECOMMENDATIONS

Applying the first-order k-C* model implies using averaged conditions. Averaging over at least 3 times the hydraulic residence time is recommended by Kadlec and Knight (1996c). This method therefore does not allow predicting the effluent variability and consequently the number of exceedances of a certain effluent standard. If that is of interest, one should switch to dynamic models like the ones from McBride and Tanner (2000) or Wynn and Liehr (2001).

Based on the data of the three measuring campaigns, the following conclusions and recommendations can be formulated concerning dynamic modelling of COD, SS, N and P removal processes in constructed wetlands:

- A good knowledge of the bed material is also of importance to predict the availability of sorption sites for phosphorus removal.
- An adequate influent characterisation is needed: working with daily averaged influent concentrations and flow rates is common practice but is strongly discouraged for dynamic modelling purposes. For nitrogen and phosphorus removal, knowledge about the different fractions is advisable.
- A submodel for particulate matter behaviour should be included, with the filtration efficiency being dependent on flow velocity. This model should also be able to predict clogging phenomena.
- Some production processes should be modelled as well, as was demonstrated by the extra carbon source that became available from plant debris after the first year of operation. A litter compartment is therefore recommendable.
- Measured phosphorus removal rates suggest that plant uptake is not negligible and thus should be modelled.
- Temperature dependencies should be included, especially for reactions in the nitrogen cycle.

Chapter 5

Model-based design of horizontal subsurface-flow constructed wetlands: a review

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5.1. ABSTRACT

The increasing application of constructed wetlands for wastewater treatment coupled to increasingly strict water quality standards is an ever growing incentive for the development of better process design tools. This chapter reviews design models for horizontal subsurface-flow constructed wetlands, ranging from simple rules of thumb and regression equations, to the well-known first-order k-C* models, Monod-type equations and more complex dynamic, compartmental models. Especially highlighted in this review are model constraints and parameter uncertainty.

5.2. INTRODUCTION

Treatment wetlands are either natural or constructed wetlands that are almost completely covered with emerging macrophytes and that are being managed as water quality improving systems. Some commonly used helophytes are common reed (*Phragmites australis*), cattail (*Typha* spp.) and bulrush (*Scirpus* spp.), all characterised as water-tolerant macrophytes that are rooted in the soil but emerge above the water surface (Kadlec *et al.*, 2000).

Although mainly applied for the purification of domestic wastewater, treatment wetlands are also used for purification of industrial wastewater (Panswad and Chavalparit, 1997; Mays and Edwards, 2001), agricultural wastewater (Tanner *et al.*, 1995a, 1995b; Kern and Idler, 1999; Meers *et al*., 2005) and stormwaters (Wong and Somes, 1995; Carleton *et al.*, 2000). They are furthermore applied to strip nutrients of eutrophied surface waters before these are discharged into vulnerable nature reserves (Meuleman, 1999; DeBusk *et al.*, 2000; Newman and Lynch, 2000).

It must however be stressed that treatment wetlands have several other functions. Next to water quality improvement, they can also function as a nature development area, a recreational area, a hydrological buffer or a reservoir (Bays *et al.*, 2000).

Among the treatment wetlands, HSSF CWs are a widely applied concept. Pretreated wastewater flows horizontally through the artificial filter bed, usually consisting of a matrix of sand or gravel and the helophyte roots and rhizomes. This matrix is colonised by a layer of attached microorganisms, that forms a so-called biofilm. Purification is achieved by a wide variety of physical, chemical and (micro)biological processes, like sedimentation, filtration, precipitation, sorption, plant uptake, microbial decomposition and nitrogen transformations (Kadlec and Knight, 1996 ; Wetzel, 2000).

The increasing application of CWs coupled to increasingly strict water quality standards, has been an incentive for the development of better design tools. This paper reviews some simple as well as some more elaborate design models and describes their merits as well as disadvantages with regard to the design of HSSF CWs. The focus is on the standard water quality variables Chemical Oxygen Demand (COD), Biochemical Oxygen Demand (BOD), Total Suspended Solids (TSS), Nitrogen (N) and Phosphorus (P). Special attention is also being paid to parameter uncertainty. Several models have been tested in a case study with the aim to predict the required surface area. The case study was based on an existing dataset containing influent flows and concentrations, weather conditions and effluent quality requirements.

5.3. SPECIFIC NOMENCLATURE

α	precipitation – evapotranspiration ($L\ T^{-1}$)	$k_{V,T}$	first-order volumetric rate constant at temperature T (T in °C) ($L\ T^{-1}$)
ε	void fraction of wetland bed (1)	$k_{A,20}$	first-order areal rate constant at temperature 20 °C ($L\ T^{-1}$)
τ	hydraulic retention time (T)	$k_{V,20}$	first-order volumetric rate constant at temperature 20 °C ($L\ T^{-1}$)
θ	temperature factor (1)	$k_{0,V}$	zero-order volumetric rate constant ($M\ L^{-3}\ T^{-1}$)
A	bed surface (L^2)	$k_{0,A}$	zero-order areal rate constant ($M\ L^{-2}\ T^{-1}$)
a	wetland cross-sectional area (L^2)	L_{in}	influent load ($M\ L^{-2}\ T^{-1}$)
b	time-based retardation coefficient (T^{-1})	L_{out}	effluent load ($M\ L^{-2}\ T^{-1}$)

C	concentration ($M\ L^{-3}$)	q	hydraulic loading rate HLR ($L\ T^{-1}$)
C_{in}	influent concentration ($M\ L^{-3}$)	Q	flow rate ($L^3\ T^{-1}$)
C_{out}	effluent concentration ($M\ L^{-3}$)	r	removal rate ($M\ T^{-1}$)
C^*	background concentration ($M\ L^{-3}$)	t	time (T)
d	bed depth (L)	T	influent temperature (°C)
K	half-saturation constant ($M\ L^{-3}$)	v	water velocity ($L\ T^{-1}$)
K_0	initial first-order volumetric rate constant (T^{-1})	V	wetland holding volume (L^3)
k_V	first-order volumetric rate constant (T^{-1})	W	wetland width (L)
k_A	first-order areal rate constant ($L\ T^{-1}$)	Z	wetland length (L)
$k_{A,T}$	first-order areal rate constant at temperature T (T in °C) ($L\ T^{-1}$)		

5.4. NON-MECHANISTIC MODELS REVIEW

This review of non-mechanistic models starts with simple design models like rules of thumb and regression equations. Secondly, the well-known first-order k-C* model (Kadlec and Knight, 1996c; Kadlec, 1997) and several of its extensions are treated. The overview then ends with Monod-type equations. Special attention is in each case paid to the model constraints and parameter uncertainty.

Rules of thumb. From an engineering point of view, rules of thumb are the fastest but also the roughest design methods. As an example, some of these rules for HSSF CWs described by Wood (1995) and Kadlec and Knight (1996h) are summarised in Table 5.1. However, probably the most widespread one is the use of 5 $m^2\ PE^{-1}$ (Cooper and Breen, 1998; Vymazal *et al*., 1998b). Since these rules of thumb are based on observations from a wide range of systems, climatic conditions and wastewater types, these rules of thumb show a large variation c.q. uncertainty and can thus better be used after more extensive calculations to check the design.

Table 5.1. Rule of thumb design criteria for horizontal subsurface-flow constructed wetlands.

Criterion	**Value range**	
	Wood (1995)	Kadlec & Knight (1996h)
Hydraulic retention time (days)	2 – 7	2 – 4
Max. BOD loading rate (kg BOD ha^{-1} day^{-1})	75	n.g.
Hydraulic loading rate (cm day^{-1})	0.2 – 3.0	8 – 30
Areal requirement (ha m^{-3} day)	0.001 – 0.007	n.g.

n.g. : not given

Regression equations. Considering the fact that the majority of the investigations on treatment wetlands has mainly been focusing on input-output (I/O) data rather than on internal processes data, regression equations seem to be a useful tool in interpreting and applying these I/O data. However, these black box 'models' lump a complex system like a CW into only 2 or 3 parameters, which is clearly an oversimplification. Important factors such as climate, bed material, bed design (length, width, depth) etc. are neglected, leading to a wide variety of regression equations and thus a large uncertainty of the design. A literature overview of regression equations for BOD, COD, TSS, TN and TP is presented in Table 5.2. The first two columns of Table 5.2 mention the reference and a short system description, the third column states the regression equation and the next three columns give the ranges of influent and effluent concentrations and hydraulic loading rates for which the equation is valid. The last column indicates the coefficient of determination.

As shown in Table 5.2, most of these regression equations rely on wastewater concentrations. Looking for instance at the first table entry (Brix, 1994b), this implies that for a constant BOD influent concentration, the same effluent concentration is predicted for a hydraulic loading rate (HLR) of 0.8 as well as 22 cm day^{-1}, which suggests that the HLR is a non-limiting factor within certain boundaries.

Table 5.2. Regression equations for horizontal subsurface-flow constructed wetlands.

Reference	**System**	**Equation**	**Input range**	**Output range**	**q range (cm day^{-1})**	**R²**
BOD C_{in} and C_{out} : influent and effluent concentrations (mg BOD l^{-1}) L_{in} and L_{out} : influent and effluent loads (kg BOD ha^{-1} day^{-1}) ; $L_{removed}$: load removed (kg BOD ha^{-1} day^{-1})						
Brix (1994b)	Danish and UK soil-based HSSF	$C_{out} = (0.11 * C_{in}) + 1.87$	$1 < C_{in} < 330$	$1 < C_{out} < 50$	$0.8 < q < 22$	0.74
Knight *et al*. (1993b)	US gravel beds (NADB)	$C_{out} = (0.33 * C_{in}) + 1.4$	$1 < C_{in} < 57$	$1 < C_{out} < 36$	$1.9 < q < 11.4$	0.48
Griffin *et al.* (1999)	US unplanted rock-filter	$C_{out} = 502.20 * \exp(-0.111 * T)$	$10 < T < 30$	n.g.	n.g.	0.69
Vymazal (1998c)	HSSF in Czech Republik	$C_{out} = (0.099 * C_{in}) + 3.24$	$5.8 < C_{in} < 328$	$1.3 < C_{out} < 51$	$0.6 < q < 14.2$	0.33
Reed & Brown (1995)	14 US HSSF	$L_{removed} = (0.653 * L_{in}) + 0.292$	$4 < L_{in} < 145$	$4 < L_{removed} < 88$	n.g.	0.97
Vymazal (1998b)	HSSF in Czech Republik	$L_{out} = (0.145 * L_{in}) – 0.06$	$6 < L_{in} < 76$	$0.3 < L_{out} < 11$	n.g.	0.85
Vymazal (1998c)	HSSF in Czech Republik	$L_{out} = (0.13 * L_{in}) + 0.27$	$2.6 < L_{in} < 99.6$	$0.32 < L_{out} < 21.7$	$0.6 < q < 14.2$	0.57
COD L_{in} and L_{out} : influent and effluent loads (kg COD ha^{-1} day^{-1})						
Vymazal (1998b)	HSSF in Czech Republik	$L_{out} = (0.17 * L_{in}) + 5.78$	$15 < L_{in} < 180$	$3 < L_{out} < 41$	n.g.	0.73
TSS C_{in} and C_{out} : influent and effluent concentrations (mg TSS l^{-1}) L_{in} and L_{out} : influent and effluent loads (kg TSS ha^{-1} day^{-1})						
Reed & Brown (1995)	14 US HSSF	$C_{out} = C_{in} * (0.1058 + 0.0011 * q)$	$22 < C_{in} < 118$	$3 < C_{out} < 23$	n.g.	n.g.
Knight *et al*. (1993b)	Soil-based HSSF (NADB)	$C_{out} = (0.09 * C_{in}) + 4.7$	$0 < C_{in} < 330$	$0 < C_{out} < 60$	$0.8 < q < 22$	0.67
Knight *et al*. (1993b)	HSSF (NADB)	$C_{out} = (0.063 * C_{in}) + 7.8$	$0.1 < C_{in} < 253$	$0.1 < C_{out} < 160$	$1.9 < q < 44.2$	0.09
Vymazal (1998c)	HSSF in Czech Republik	$C_{out} = (0.021 * C_{in}) + 9.17$	$13 < C_{in} < 179$	$1.7 < C_{out} < 30$	$0.6 < q < 14.2$	0.02
Kadlec *et al.* (2000c)	NADB, Severn Trent	$C_{out} = 0.76 * C_{in}^{0.706}$	$8 < C_{in} < 595$	$2 < C_{out} < 58$	n.g.	0.55
Brix (1994b)	Danish soil-based HSSF	$C_{out} = (0.09 * C_{in}) + 4.7$	$0 < C_{in} < 330$	$0 < C_{out} < 60$	n.g.	0.67
Vymazal (1998b)	HSSF in Czech Republik	$L_{out} = (0.048 * L_{in}) + 1.76$	$3 < L_{in} < 78$	$0.9 < L_{out} < 6.3$	n.g.	0.42
Vymazal (1998c)	HSSF in Czech Republik	$L_{out} = (0.083 * L_{in}) + 1.18$	$3.7 < L_{in} < 123$	$0.45 < L_{out} < 15.4$	$0.6 < q < 14.2$	0.64

Table 5.2. (contd). Regression equations for horizontal subsurface-flow constructed wetlands.

TN C_{in} and C_{out} : influent and effluent concentrations (mg TN l^{-1}) L_{in} and L_{out} : influent and effluent loads (g N m^{-2} $year^{-1}$)						
Kadlec & Knight (1996h)	NADB + others	$C_{out} = 2.6 + (0.46 * C_{in}) + (0.124 * q)$	$5.1 < C_{in} < 58.6$	$2.3 < C_{out} < 37.5$	$0.7 < q < 48.5$	0.45
Kadlec *et al.* (2000c)	Danish soil-based HSSF	$C_{out} = (0.52 * C_{in}) + 3.1$	$4 < C_{in} < 142$	$5 < C_{out} < 69$	$0.8 < q < 22$	0.63
Vymazal (1998c)	HSSF in Czech Republik	$C_{out} = (0.42 * C_{in}) + 7.68$	$16.4 < C_{in} < 93$	$10.7 < C_{out} < 49$	$1.7 < q < 14.2$	0.72
Vymazal (1998b)	HSSF in Czech Republik	$L_{out} = (0.67 * L_{in}) – 18.75$	$300 < L_{in} < 2400$	$200 < L_{out} < 1550$	n.g.	0.96
Vymazal (1998c)	HSSF in Czech Republik	$L_{out} = (0.68 * L_{in}) + 0.27$	$145 < L_{in} < 1894$	$134 < L_{out} < 1330$	$1.7 < q < 14.2$	0.96
TP C_{in} and C_{out} : influent and effluent concentrations (mg TP l^{-1}) L_{in} and L_{out} : influent and effluent loads (g P m^{-2} $year^{-1}$)						
Kadlec & Knight (1996h)	US, European, Australian HSSF	$C_{out} = 0.51 * C_{in}^{1.1}$	$0.5 < C_{in} < 20$	$0.1 < C_{out} < 15$	n.g.	0.64
Kadlec & Knight (1996h)	US HSSF	$C_{out} = 0.23 * (q^{0.6} * C_{in}^{0.76})$	$2.3 < C_{in} < 7.3$	$0.1 < C_{out} < 6$	$2.2 < q < 44$	0.60
Brix (1994b)	Danish soil-based HSSF	$C_{out} = (0.65 * C_{in}) + 0.71$	$0.5 < C_{in} < 19$	0.1 < Cout < 14	$0.8 < q < 22$	0.75
Vymazal (1998c)	HSSF in Czech Republik	$C_{out} = (0.26 * C_{in}) + 1.52$	$0.77 < C_{in} < 14.3$	$0.4 < C_{out} < 8.4$	$1.7 < q < 14.2$	0.23
Vymazal (1998b)	HSSF in Czech Republik	$L_{out} = (0.58 * L_{in}) – 4.09$	$25 < L_{in} < 320$	$20 < L_{out} < 200$	n.g.	0.61
Vymazal (1998c)	HSSF in Czech Republik	$L_{out} = (0.67 * L_{in}) – 9.03$	$28 < L_{in} < 307$	$11.4 < L_{out} < 175$	$1.7 < q < 14.2$	0.58

q expressed as cm day^{-1}

n.g. : not given

NADB : North American treatment wetlands DataBase (Knight *et al.*, 1993b)

Only a limited number of regression equations rely on both influent concentration and hydraulic loading rate as inputs to predict the effluent concentration. Consequently, only those regression equations can be used to predict the maximum allowable hydraulic loading rate based on a given influent concentration and a given effluent standard.

First-order models. The state-of-the-art in constructed wetlands' modelling consists of first-order equations (Kadlec and Knight, 1996c; Kadlec, 1997) which in case of constant conditions (e.g. influent, flow and concentrations) and an ideal plug-flow behaviour predict an exponential profile between inlet and outlet (equation 5.1):

$$\frac{dC}{dt} = -k_V C \xrightarrow{[1]} \left(\frac{C_{out} - C^*}{C_{in} - C^*}\right) = e^{(-k_V \tau)} \xrightarrow{[2],[3],[4]} \left(\frac{C_{out} - C^*}{C_{in} - C^*}\right) = e^{\left(-k_A/q\right)} \quad (5.1)$$

transformation equations: [1] $C_{in} = C(t{=}0)$ and $C_{out} = C(t{=}\tau)$, initial conditions

[2] $k_A = k_V\ \varepsilon\ d$

[3] $q = Q / A$

[4] $V = Q\ \tau = A\ d\ \varepsilon$

The background concentration C* in this model is explained by processes such as autochthonous production and/or sediment release.

Some model enhancements have been proposed to incorporate the effect of precipitation and evapotranspiration on the wetlands' performance, yielding a power law profile (equation 5.2) between inlet and outlet for steady state conditions (Kadlec, 1997):

$$\frac{C_{out} - C'}{C_{in} - C'} = (1 + \left[\alpha/q\right])^{-(1+k_A/\alpha)} \quad with \quad C' = C^* \left[\frac{k_A}{k_A + \alpha}\right] \quad (5.2)$$

The influence of temperature is commonly modelled via an Arrhenius equation (equation 5.3):

$$k_{A,T} = k_{A,20}\theta^{(T-20)} \quad and \quad k_{V,T} = k_{V,20}\theta^{(T-20)} \tag{5.3}$$

According to Kadlec and Knight (1996h), removal of BOD, TSS and TP in treatment wetlands is generally found to be independent of temperature (θ = 1.00) whereas nitrogen removal is negatively influenced by lower temperatures (θ = 1.05).

Shepherd *et al.* (2001) recently presented a time-dependent retardation model for COD removal that replaces the background concentration C* by two other parameters K_0 and b. They assumed that removal rates decrease during the course of time, because easily biodegradable substances are removed first and fast, thus leaving a solution with less biodegradable constituents and hence with slower removal kinetics. This continuous change in solution composition can be represented by a continuously varying first-order rate constant k (equation 5.4):

$$k_v = \frac{K_0}{(b\tau + 1)} \tag{5.4}$$

This retardation model was considered to be more appropriate for CW design because it allows a steady decrease in COD (or any other component) with increased treatment time rather than a constant residual COD (C*) value. Applied on data from a pilot-scale HSSF CW for winery wastewater treatment, model calibration yielded K_0 values from 9 to 12 day^{-1} and b values from 2 to 5 day^{-1}. Compared to the k-C* model, the time-dependent retardation model had more consistent parameters for COD removal data across different depths and at different loadings.

Drizo *et al.* (2000) suggested another small enhancement of the k-C* model by replacing the areal first order constant k by (k_s + k_p), representing removal by substrate and by plants respectively. Calibration was done by means of planted and non-planted pilot-scale wetlands filled with shale. N-removal (ammonium) was best fitted by a k-value of 0.065 m day^{-1} and a k_s value of 0.034 m day^{-1}. Best fits for P-removal (ortho-phosphate) were obtained with k equal to 0.084 m day^{-1} and k_s equalling 0.069 m day^{-1}, which clearly demonstrates the role of substrate sorption for phosphorus elimination.

Calibration of the parameters k, C* and θ is mostly done on the basis of input-output concentrations, and not on the basis of transect data, although the latter are to be preferred for calibration purposes (Kadlec, 2000). Because these parameters lump a large number of other characteristics representing the complex web of interactions in a CW as well as external influences like weather conditions, a large variability can be observed in reported k_A, k_V, C* and θ values. Table 5.3 presents an overview of first-order rate constants for HSSF CWs. Looking for instance at BOD removal, the reported k_A values vary between 0.06 and 1.00 m day^{-1} whereas k_v values range from 0.17 to 6.11 day^{-1}. For a given BOD influent concentration and effluent limit, the predicted maximum loading rate based on k_A values thus varies by a factor of 36. Kadlec and Knight (1996h) therefore recommend using 'global' average rate constants between these extremes.

Next to this variability, some other major drawbacks of the first-order models need to be mentioned. First of all, the equations are based on the assumptions of plug-flow and steady-state conditions. However, small-scale wastewater treatment plants under which most treatment wetlands can be ranged, are subject to large influent variations (Boller, 1997) whereas the larger ones are subject to hydrological influences (Wong and Somes, 1995; Kadlec, 1997), thus causing in both cases non steady-state conditions. Short-circuiting and dead zones are common phenomenona in CWs causing non-ideal plug flow conditions, thus jeopardising the use of the first-order model (Kadlec, 2000). Secondly, the so-called rate 'constants' do not seem to be constant at all but dependent on the influent concentrations, the hydraulic loading rate and the water depth (Kadlec, 1997, 2000). Table 5.3 also shows some influence of the void fraction, the maturity of the bed and the chosen background concentration on the rate constants.

Table 5.3. First-order rate constants for horizontal subsurface-flow constructed wetlands.

Reference	Nr. of beds	k_A (m day^{-1})	k_v (day^{-1})	Remarks
BOD				
Crites (1994)			0.8 – 1.1	0.8 = sand ; 1.1 = gravel (T °C)
Reed & Brown (1995)			1.104	k_{20} with θ = 1.06
Tanner *et al.* (1995a)	8		0.17	k_T – gravel beds
Tanner *et al.* (1995a)	8		0.22	k_{20} with θ = 1.06 – gravel beds
Wood (1995)			1.84	ε = 0.42 – medium sand (20 °C)
Wood (1995)			1.35	ε = 0.39 – coarse sand (20 °C)
Wood (1995)			0.86	ε = 0.35 – medium sand (20 °C)
Kadlec & Knight (1996h)		0.085 – 1	0.3 – 6.11	
Kadlec (1997)		0.49		C* > 3 mg l^{-1} and θ = 1.00 (20 °C)
Vymazal *et al.* (1998b)		0.19		Proposed by Kickuth
Brix (1998)		0.118 ± 0.022		Mean ± 95% limits – depends on load
Schierup *et al.* (1990a)	49	0.083		Danish systems
Cooper (1990)		0.067 – 0.1		UK systems
Brix (1994b)	70	0.16		C* = 3.0 mg l^{-1} – soil based
Brix (1994b)	70	0.068		C* = 0 mg l^{-1} – soil based
Kadlec *et al.* (2000c)		0.133		Czech republic wetlands
Kadlec *et al.* (2000c)	1	0.07 – 0.097 – 0.13 – 0.18 – 0.31 – 0.17		6 consecutive years, Czech republic wetlands
Cooper *et al.* (1996)		0.06		C* = 0 mg l^{-1} – secondary wetlands
Cooper *et al.* (1996)		0.31		C* = 0 mg l^{-1} – tertiary wetlands
Kadlec *et al.* (2000c)	14	0.17		C* = 0 mg l^{-1} – tertiary wetlands USA
Liu *et al.* (2000)			0.86	Gravel beds – soluble cBOD, 20 °C
SS				
Kadlec & Knight (1996h)		2.74		k_{20} with θ = 1 and C* > 7 mg l^{-1}
Kadlec (1997)		8.22		k_{20} with θ = 1 and C* > 7 mg l^{-1}
Kadlec *et al.* (2000c)		23.1		Laboratory colums
Kadlec *et al.* (2000c)		31.6		Large scale pilot wetland
Kadlec *et al.* (2000c)	33	0.119		Data from Czech republik
TN				
Tanner *et al.* (1995b)			0.16	k_T - gravel bed
Kadlec & Knight (1996h)		0.074		k_{20} with θ = 1.05 and C* = 1.5 mg l^{-1}
Kadlec & Knight (1996h)		0.007 – 0.1		k_T with C* = 1.5 mg l^{-1}
Wittgren & Maehlum (1997)	73		0.06	k_T - Norway
Kadlec *et al.* (2000c)		0.028		Czech systems
TP				
Tanner *et al.* (1995b)			0.14	k_T - gravel bed
Kadlec & Knight (1996h)		0.033		k_{20} with θ = 1.00 and C* = 0.02 mg l^{-1}
Wittgren & Maehlum (1997)	71		0.28	k_T - Norway

Variable-order or Monod-type models. Mitchell and McNevin (2001) identified another physical impossibility of the first-order model, namely the fact that the removal rates continue to increase with increasing loading rates (equation 5.5):

$$r = Q\ (C_{in} - C_{out}) \quad \Rightarrow \quad r = Q\ C_{in}\ (1 - \exp(-k_V \tau)) \qquad (5.5)$$

However, in most cases, a maximum allowable loading rate has been demonstrated. Therefore, Mitchell and McNevin (2001) advocate the use of a Monod-type design model, which represents first-order rate reactions for relatively low concentrations but zero-order rate reactions for high concentrations. Still with the assumption of plug flow, the model presents itself as (equation 5.6):

$$\begin{aligned} & r = k_{0,V}\ V\ \frac{C}{K + C} \quad and \quad \frac{dC}{dt} = \frac{-r}{V} \\ & \xrightarrow{[1],[2],[3],[4]} \quad \frac{dC}{dz} = -\frac{k_{0,V}\varepsilon\ a}{Q}\ \frac{C}{K + C} = -\frac{k_{0,A}}{qZ}\ \frac{C}{K + C} \end{aligned} \qquad (5.6)$$

transformation equations: [1] $k_{0,A} = k_{0,v}\ \varepsilon\ d$

[2] $q = Q / A = Q / (W * Z)$

[3] $z = v * t$

[4] $v = Q / (\varepsilon\ a)$

One other interesting feature of this model is an alternative explanation of background concentrations (C*). Indeed, if concentrations drop to near zero, the Monod equation predicts a very low reaction rate, which may prevent total decomposition of the pollutant within the given hydraulic retention time.

The authors did not try to assess parameter values, but used a graphical representation of loading and removal rates from the North-American treatment wetlands database (Knight *et al.*, 1993b) to extract some design parameters. They found a maximum allowable loading rate for HSSF CWs of 80 kg BOD ha^{-1} day^{-1} and 130 kg TSS ha^{-1} day^{-1}. Data from a.o. several Danish (Schierup *et al.*, 1990b) and UK systems (Green and Upton, 1992) show most actual loading rates well below these maximum recommended levels. Several exceptions are however mentioned where, despite significantly higher loading rates, effluent concentrations are still of acceptable quality.

Kemp and George (1997) used a comparable model to represent ammonia removal in a pilot-scale HSSF CW treating domestic wastewater. They found a $k_{0,V}$ of 7.8 mg N l^{-1} day^{-1} and a K of 5.5 mg N l^{-1}. The coefficient of determination R^2 indicated that the Monod-type model better described the variability of the data than a first-order model.

5.5. CASE STUDY OF NON-MECHANISTIC MODELS

To demonstrate the use of the above models and to illustrate the variability and uncertainty of the predictions, a case study was performed. The different design models were used to calculate the required surface area of a horizontal subsurface-flow constructed wetland, able to produce an effluent in compliance with the legal standards. Real influent data were used, collected in a pilot-scale constructed reed bed (10 P.E.) belonging to Aquafin NV and located in Aartselaar, Belgium. For a detailed description, one is referred to Vandaele *et al.* (2000) and Chapter 4. Table 5.4 gives an overview of the influent characteristics and the applied effluent standards, based on the Flemish Environmental Legislation (VLAREM II, 1995). The low influent concentrations of the CW are due to the combined effect of a mixed sewer system and an efficient primary treatment.

Table 5.4. Influent characteristics and effluent standards used in the case study.

Variable	Average influent characteristics	Effluent standards
Flow rate (m^3 day^{-1})	1.9	
BOD (mg BOD l^{-1})	48.0	25.0
COD (mg COD l^{-1})	184.7	125.0
TSS (mg TSS l^{-1})	71.0	35.0
TN (mg N l^{-1})	17.2	15.0
TP (mg P l^{-1})	2.8	2.0

Whenever possible, the minimum and maximum values of reported parameter values (Tables 5.1, 5.2 and 5.3) were applied to show the maximal variability of the areal prediction. Regression equations and area-based first-order models allow to calculate the hydraulic loading rate q, from which the required area A (A = Q / q) can be derived. Volume-based first-order models allow to calculate the hydraulic retention time τ and consequently the required volume V (V = Q * τ). An assumed water depth of 0.6 m and a pore volume of 40% was used to transform water volume into surface area (A = V / (d * ε). For the purpose of this case study, the simplest first-order model was used, i.e. without background concentrations and temperature coefficients, since many researchers do not mention values for those parameters (Table 5.3). The Monod-type model of Mitchell and McNevin (2001) could not be tested because of a lack of parameter data.

Results of the rules of thumb, the regression equations, the first-order model and the time-dependent retardation model are presented in Fig. 5.1. These different, simple design methods predict required surface areas ranging from as low as 0.1 m² up to 950 m² for the given influent data and effluent standards. Generally speaking, the rules of thumb seem to be the more conservative ones as they consistently predict larger surface areas.

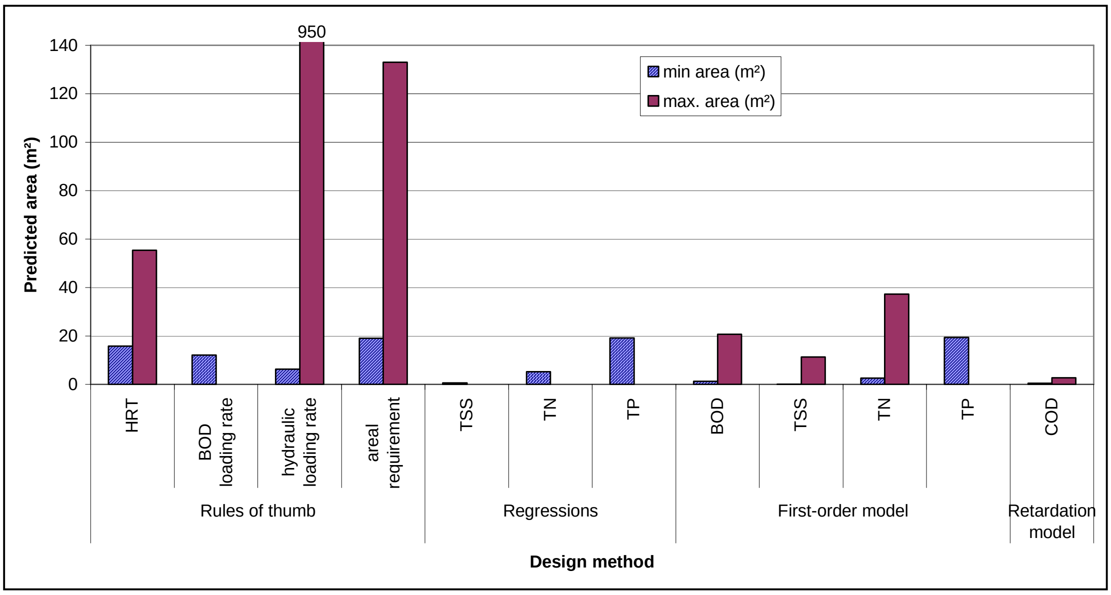

Figure 5.1. Required area predictions according to the different design methods used in the case study. Minimum and maximum areas indicate the output variability due to parameter uncertainty.

This case study clearly demonstrates that applying simple design rules implies a lot of uncertainty, both ecologically in terms of the effluent quality as economically in terms of the design size and thus investment cost. However, at this point, the k-C* is the best tool available. When following the recommendations of Kadlec and Knight (1996c) to use parameter values from a similar system (i.e. same type of wastewater, same climatic conditions, same filter material and plants etc.) and when taking into account some safety factors, an acceptable effluent quality should be guaranteed. Obviously the question arises whether or not mechanistic models which implicitly take into account pollutant transformation processes and their interactions, can lower the design uncertainty?

5.6. MECHANISTIC MODELS REVIEW

Mechanistic wetland models mathematically approach the different processes and their interactions, which aids considerably in understanding and interpreting wetland performance, or, by the words of Breen (1990), it renders the reed bed into a green box in stead of a black one. In the following sections, a short SWOT analysis (Strengths, Weaknesses, Opportunities and Threats) is presented of two such models.

5.6.1. Model of Wynn and Liehr (2001)

General model description. Only recently, a mechanistic, compartmental simulation model of an HSSF CW has been presented by Wynn and Liehr (2001), based on the PhD thesis of T.M. Gidley-Wynn (1995). The model consists of 6 interlinked submodels, representing the carbon and nitrogen cycles, the water and oxygen balances and the growth, decay and metabolism of heterotrophic and autotrophic bacteria. Phosphorus transformations are not considered since these are mainly of physical-chemical nature and the main focus was on microbial processes. One important assumption is that suspended solids removal equals 100%, so no particulate substances are leaving the bed. This was based on the fact that effluent SS levels of the studied treatment wetland were very low. Hydraulic behaviour is modelled via a tanks-in-series approach to mimic the mixing regime, and via the Darcy equation to imitate flow in a porous medium. Simulations were done with the software package STELLA II (High Performance Systems Inc.).

The required model inputs are air temperature, day length, precipitation, flow rate and the concentrations of BOD, NH_4-N, NO_3-N, organic N and dissolved oxygen. The model output consists of flow rate and the same 5 concentrations as for the input. The dynamics of the 15 state variables are modelled via 15 ordinary differential equations that contain a total of 42 parameters related to physical, microbiological and biological processes. On the one hand, this complexity of the model enables to better summarise the processes that occur within CWs as well as to demonstrate interactions between certain components. On the other hand, it requires estimation of 15 initial conditions for the state variables and knowledge about or estimation of 42 parameters, which is not a trivial task.

Strengths. One of the advantages of the STELLA code is the easily understandable syntax and the clear visual representation. Any researcher with some knowledge of mass balances should be able to interpret the code. In this sense, the limitation to (micro)biological parameters further contributes to this ease of understanding, by limiting the number of processes, interactions and parameters. Transfer of the process equations to another simulation platform therefore proves quite easy.

Another clear advantage of this model is that it makes use of routinely measured variables, i.e. BOD, NH_4, NO_3 and organic nitrogen (derived from TN). Dissolved oxygen is less often monitored, but can be easily measured by electrodes without excessive costs. Air temperature and precipitation are often available online or can be easily and cheaply measured on site.

Finally, the concept of working with aerobic and anaerobic fractions is a very handy solution to mimic the oxygen ‘mosaic’, i.e. the concurrent existence of both aerobic and anoxic zones which would otherwise not be possible in a completely mixed tank.

Weaknesses. Although using routinely measured variables such as BOD, NH_4 and NO_3 greatly enhances the possibility to find additional datasets to calibrate and/or validate the model, there is also a negative side. Fixed conversion factors are used to split total BOD and organic nitrogen into dissolved and particulate fractions, thereby largely ignoring the variability of the influent wastewater composition. In Chapter 6, this problem will be circumvented by directly measuring influent and effluent TOC and DOC instead of BOD concentrations.

Another major drawback is the lack of a particles submodel. Extreme events such as rain storms can cause a temporary presence or rise of solids in the effluent by dragging along settled solids or by surface flow, but this can not be described by this model. Solids accumulation and the effect on porosity and hydraulic conductivity is therefore also neglected. The latter process would be of minor importance for short-term simulations, but prevents adequate long-term model predictions.

A number of errors and inconsistensies were discovered in the code, although it remains unclear if they were just typing errors in the paper or were actually present in the model code. Aerobic heterotrophic growth was modelled proportionally with the Anaerobe Fraction instead of (1 – Anaerobe Fraction). An equation for ammonification seemed unused in any mass balance and an incorrect conversion factor from BOD to DOC was used.

One should also be warned that the paper is only a summary of the work done and as such leaves out many details which are nevertheless quite important when one intends to use the model for another case study. It is therefore of paramount importance to consult the original work, i.e. the PhD thesis of T.M. Gidley-Wynn (1995), and not to just copy and apply the model.

Although it is common procedure to work with carbon and nitrogen as model units in plant ecology, it is less common in wastewater treatment. Compared with the Activated Sludge Models' COD-based approach (Henze *et al.*, 2000), it yields the extra disadvantage of having to convert BOD to organic carbon. One therefore looses the possibility to consider oxygen as negative COD which allows to perform a true COD balance, and as a result one has to use different microbial yield coefficients for substrate and oxygen. Similarly, the model philosophy is partly based on previous work of Parnas (1975, in Wynn and Liehr, 2001) on C and N transformations in soils, and might therefore be less transparant and appropriate for wastewater cases.

Wynn and Liehr used data from the Mayo wetland (Maryland, USA) to calibrate their model. Flow rate, temperature and dissolved oxygen were measured daily, whereas all other wastewater variables were measured biweekly and by linear interpolation turned into daily input data. In this way, diel variations and eventual week-weekend patterns were neglected. It is also worth mentioning that the Mayo wetland has a theoretical hydraulic retention time between 2 and 4 days, so much shorter than the sampling interval.

Calibration was done graphically by visually comparing measured and simulated effluent concentrations for various parameter values. This procedure however yielded values for several microbial parameters that were one or more orders of magnitude lower than those typically mentioned in literature. Due to the complexity of the model, it is very well possible that certain parameters compensate for each other, thus causing model insensitivity to parameter changes (see e.g. Dochain and Vanrolleghem, 2001). However, it would be more reasonable to assume that certain important phenomena were not included in the model, even though they are influencing microbial reactions. As an example, diffusion limitations in the biofilm can be mentioned. Furthermore, during calibration, the wetland was only modelled as a single continuously stirred tank reactor. This, however, strongly equalises the effluent concentrations as any incoming peak is immediately diluted into the entire wetland volume.

Despite the model uncertainty and the lack of high-quality data, the calibrated model reproduced most seasonal trends of oxygen, nitrogen and carbon, clearly showed the interactions between the different cycles, but missed most of the short-term variability.

Opportunities. This ‘green box’ model of course offers many possibilities when one fully comprehends the model structure and is aware of its weaknesses and pitfalls. Scenario analyses can be applied on a range of topics, such as influence of temperature, influence of bed dimensions, different loading rates etc.

Since the code is given in the paper and the PhD thesis, it can quite easily be implemented into other simulation software and amendments can be made if required.

One adjustment made by De Wilde (2001) and Story (2003) was to include extreme events in the hydraulic submodel. Indeed, zero outflow during warm periods when evapotranspiration exceeds the influx of water, and surface flow when rain weather flows exceed the hydraulic conductivity of the reed bed are not accounted for in the original model. Of course, when one is interested in P removal, a sorption/desorption mechanism and plant uptake of phosphorus could easily be added to the code. Similarly, Story (2003) enhanced the nitrogen submodel with ammonium adsorption.

In Chapter 6, an updated version of the Wynn and Liehr (2001) model which takes into account a number of this errors, modifications and drawbacks, will be applied on a dataset of a two-stage reed bed in Saxby (UK).

Threats. As for most mechanistic models, calibration and application of this model requires lots of high-quality data with a high information content, which are unfortunately seldomly available. Also, unless end-users pick up this model and more case studies are done, data on parameter values etc. will remain scarce.

5.6.2. CW2D – The model of Langergraber (2001)

General model description. The multi-component reactive transport model CW2D (Constructed Wetland 2-Dimensional) was developed by Langergraber (2001, 2003) to model transport and reactions of the main constituents in wastewater in subsurface-flow CWs, vertical as well as horizontal flow ones. It was implemented into the source code of the simulation program HYDRUS-2D. Water flow through the variably-saturated porous media is represented by the Richard's equation. The transport model considers dispersion and diffusion, convection and also several sources and sinks such as adsorption/desorption, water uptake by plant roots etc. HYDRUS-2D furthermore allows the use of the concept of two-region, dual-porosity transport which divides the liquid phase into mobile (flowing) and immobile (stagnant) regions. Biochemical transformations in CW2D are based on the Activated Sludge Models (Henze *et* al., 2000) and are able to describe the elimination of organic matter, nitrogen and phosphorus. This includes 12 components, 9 processes and most importantly 46 parameters, excluding the parameters for the hydraulic submodel. A sensitivity analysis with 10% relative parameter changes revealed that the latter ones were the most influential, followed by the oxygen reaeration rate, yield coefficients and lysis rates for the bacteria.

Strengths. This model is fully mechanistic, with even the hydraulic submodel being a close representation of reality. Using a 2D groundwater flow model has the extra advantage that concentrations are tied to their locations, thereby creating the possibility of having both aerobic and anoxic zones in the modelled wetland. Finally, it is very recommendable to apply the microbial transformation processes of the Activated Sludge Models (Henze *et* al., 2000). The latter models are now widely accepted in wastewater treatment engineering and provide a common 'language', which makes these models more accessible. Due to their widespread application, it also has the advantage of an enormous common knowledge on stoichiometric and kinetic parameters.

Weaknesses. CW2D is currently unfit to investigate clogging phenomena since up to now only solute wastewater compounds are being considered.

Simulation results showed very good fits with data from an indoor pilot-scale constructed VSSF wetland for wastewater treatment (1m^2 surface area, 40 L day^{-1}), which the authors partly attributed to the fact that a multitude of data were available from this system for calibration purposes. Simulation results from a second indoor small-scale plot for surface water treatment (2 m^2, one downflow and one upflow chamber) also showed a good match with the measured data. However, simulation of an outdoor 40 PE two-stage (VSSF + HSSF) CW proved difficult due to hydraulic irregularities such as short-circuiting and preferential flow which could not be mimicked by the 2D model.

Opportunities. Being applicable for both vertical and horizontal subsurface-flow wetlands, a solid mechanistic structure and a good communication within the IWA Specialist Group on the Use of Macrophytes for Pollution Control seems to have convinced many researchers of the intrinsic value of CW2D. More and more case studies are emerging in which CW2D is being applied (e.g. Langergraber, 2004; Dittmer *et al*., 2004; Toscano *et al*., 2005). This results in refinements of the model structure and in increasing knowledge on parameter values, thereby reducing output uncertainty.

Threats. The most obvious drawback of the CW2D model, as for any mechanistic model, is the large amount of data that are needed to calibrate the model. Data collection of this magnitude seems for the time being limited to lab-scale or pilot-scale treatment wetlands that can be operated under strictly controlled conditions.

5.7. CONCLUSIONS AND RECOMMENDATIONS

Confronted with different models of HSSF CWs and the numerous different parameter values, the obvious question is which ones should be used and which are the most reliable ones? The case study clearly demonstrated that the predicted required surface areas are highly variable and that this variability does not only exist among the different models, but due to parameter uncertainty also within the same model category.

The rules of thumb seemed to be the more conservative design models. Since these are easily applicable, designers could be tempted to stick to those models. However, they may be guaranteeing good quality effluent, but they will likely be counteracted by economic constraints: conservative designs tend to increase the investment costs.

Mechanistic models such as the Wynn and Liehr model and the CW2D one are at this moment useful tools to gain understanding of certain processes and are capable to demonstrate several interactions within the wetland system. However, overparametrisation and a lack of experience currently limit the value of these models as design tools.

At present, the state-of-the-art k-C* model seems to be the best available design tool if the designer makes sure that all the assumptions are fulfilled and if he is aware of the pitfalls of the model. Concerning the issue of parameter uncertainty, it is advisable to implicitly take this into account during the design. If possible, parameter values should be used from constructed treatment wetlands that operate under similar conditions as the one to be constructed : climatic conditions, wastewater composition, bed material and macrophyte species.

Chapter 6

Model study of short-term dynamics of secondary treatment reed beds at Saxby (Leicestershire, UK)

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6.1. ABSTRACT

Relatively simple black-box models, such as the well-known k-C* model, are commonly applied to design horizontal subsurface-flow constructed wetlands. Important shortcomings of this model are the oversimplification of reality on the one hand, and the inability to predict short-term effluent dynamics on the other. A possible solution for these drawbacks could be the application of dynamic compartmental models. This chapter reports on the calibration requirements and the simulation results of such a dynamic model. A quantitative sensitivity analysis was used to identify the most sensitive parameters after which model predictions were optimised by adjusting those parameter values. Model fits were acceptable but missed some of the short-term dynamics observed in reality. At this point, it might therefore still be unwise to use the model as a design tool. Further model adjustments and calibration efforts are needed to enhance its reliability.

6.2. INTRODUCTION

This chapter reports on a research project that was aimed at extending and calibrating an existing dynamic model of a HSSF CW and at checking whether or not the model output would be good enough to use the model as a design tool. Firstly, the survey results of the test site are briefly summarised and important processes are indicated. Then the model structure is outlined, the calibration procedure is described and simulation results are given. Finally, during the discussion, some model flaws and calibration difficulties are identified and the applicability of the model for design purposes is assessed.

6.3. MATERIALS AND METHODS

In August 2002, a detailed data set was collected at a two-stage reed bed of Severn Trent Water Ltd. at Saxby (Leicestershire, UK), a CW designed for 47 Population Equivalents (PE) and in service since 1998. The system consists of two horizontal subsurface-flow beds connected in series, preceded by a conventional septic tank for primary treatment. Each bed has a surface area of 117 m² and an average depth of 0.6 m (Fig. 6.1). Pre-washed 5-10 mm gravel is used as filter medium. Wastewater is distributed over the entire width of the reed beds via an aboveground trough with equidistant V-shaped openings.

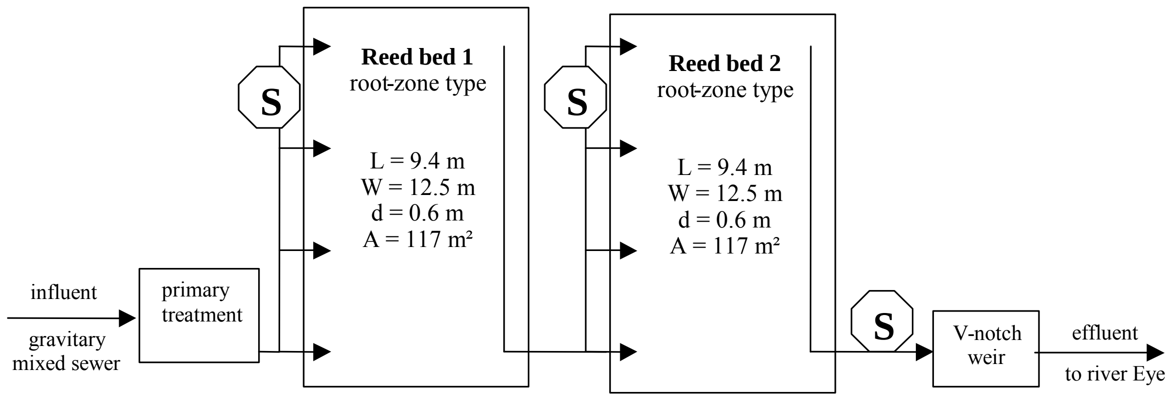

Figure 6.1. Schematic representation of the constructed treatments wetland in Saxby. S indicates location of samplers.

A two-week survey was carried out during which 8-hour composite samples were collected of the pre-settled influent, the effluent of the first bed and the effluent of the second bed (Fig. 6.1). Non-cooled automatic samplers were used. They were programmed to take one sample of 125 ml every hour and to combine 8 samples in one bottle. Composite samples are preferred because they facilitate the application of mass balances and they correspond better with the step inputs that are commonly used in simulation software. Samples were then taken to the lab on Monday, Wednesday and Friday and were thus a maximum of 2.5 days in non-refrigerated conditions.

All samples were sent to Severn Trent Laboratories and analysed for total and filtered Biochemical Oxygen Demand (BOD_t and BOD_f), Total and Filtered Organic Carbon (TOC_t and TOC_f), suspended solids (SS), ammonium (NH_4-N), total oxidised nitrogen (TON), total nitrogen (TN) and orthophosphates (o-PO_4). Occasionally, total Chemical Oxygen Demand (COD_t) analyses were carried out.

Effluent flow rates of the second reed bed were measured every 15 minutes by means of a V notch weir with an angle of 28.1° and an ISCO Model 4230 Bubbler Flow meter (Fig. 6.1), the latter device being more suitable to measure low flow rates. Simultaneously, meteorological data were collected since these have a major impact on the water balance. Precipitation was measured via an ISCO Model 674 tipping bucket rain gauge attached to the flow meter. Other meteorological data, i.e. air temperature and day length, were gathered via meteorological sites on the Internet.

6.4. SURVEY RESULTS

The daily average air temperature during the survey varied between 12 and 30 °C. Some severe rainstorms occurred on 8 and 9 August, which forced the influent flow rate from a base flow of less than 0.1 l s^{-1} to a peak flow of about 15 l s^{-1} since no Combined Sewer Overflow or bypass is provided. Corresponding hydraulic loading rates varied from as low as 5 cm day^{-1} up to about 100 cm day^{-1} during storm events. This caused temporary flooding of the beds. The treatment works nevertheless consistently produced a high quality effluent with BOD and SS concentrations lower than 10 mg l^{-1} and 30 mg l^{-1} respectively. Ammonium-nitrogen and o-PO_4 concentrations were also relatively unaffected by the fluctuating flow rates and varied between 0.9 and 7.6 mg N l^{-1} and 1.4 and 3.7 mg P l^{-1} respectively (Fig. 6.2). Remarkably, phosphorus concentrations in the effluent of the second bed are consistently higher than those of the first bed, indicating a net phosphorus production in the second bed. All in all, this CW seems to have a considerable hydraulic buffering capacity.

Average BOD, NH_4-N, TON (= NO_3 + NO_2), TN and o-PO_4 removal efficiencies (Table 6.1) can be called excellent with reference to reported literature values. SS removal on the other hand seems to be only average. When looking in terms of mass removal, this CW is capable of removing 67.9 kg SS ha^{-1} d^{-1}, 25.6 kg BOD_t ha^{-1} d^{-1} and 4.8 kg TN ha^{-1} d^{-1}. These figures clearly indicate that the beds have enough oxygenation capacity but on the other hand also provide enough anoxic regions where denitrification takes place.

Taking into account that the beds operation started more than 4 years ago and that the media consists of siliceous gravel with a low iron and calcium content, phosphorus removal also performs reasonably well. There are few signs of saturation of the sorption sites yet. The net production of phosphorus in the second bed suggests that there is some decay of organic material and/or a decrease in redox potential with subsequent P-release from Fe and Al complexes.

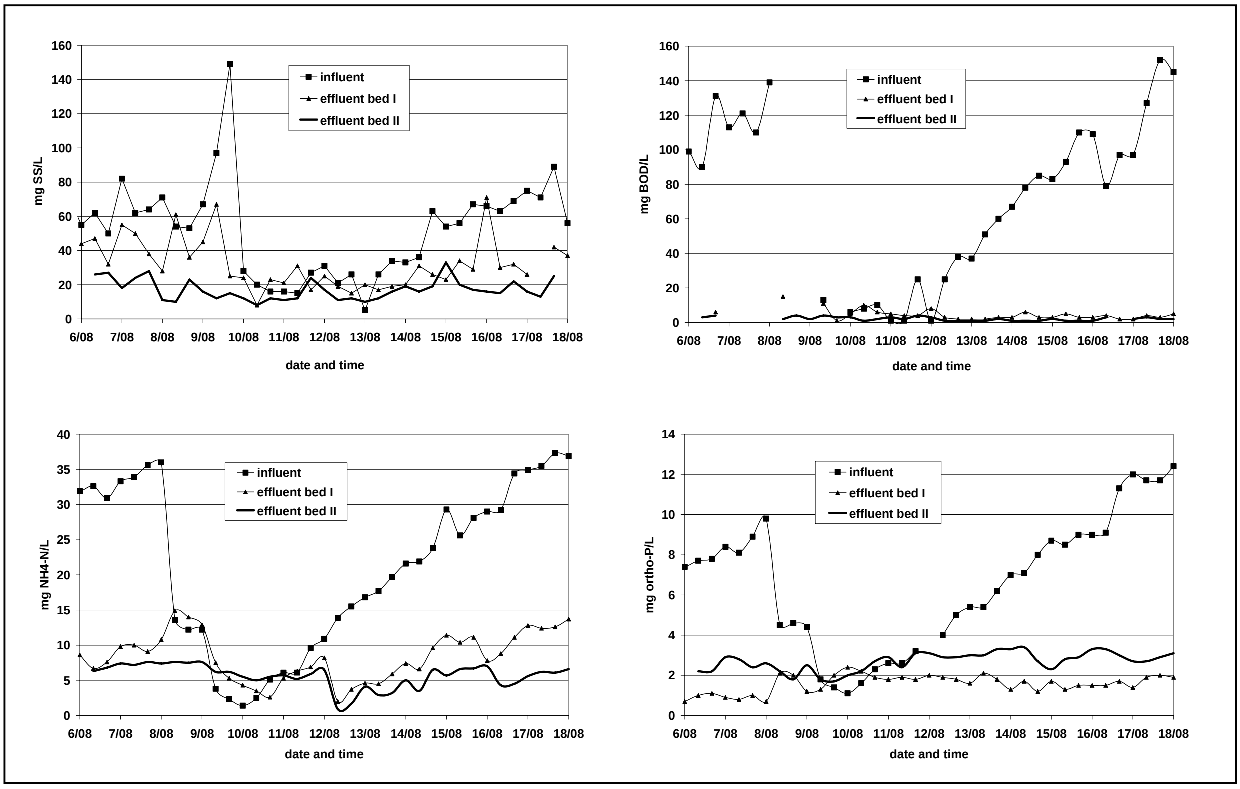

Figure 6.2. Concentration time series of suspended solids (SS), Biochemical Oxygen Demand (BOD), ammonium (NH_4-N) and ortho-phosphates (o-PO_4), measured at the constructed wetlands in Saxby from 6 till 18 August 2002. Data from pre-settled influent, effluent of the first reed bed and effluent of the second reed bed.

6.5. MODEL SETUP

For the model study of the Saxby treatment reed beds, the dynamic, compartmental model of Wynn and Liehr (2001) was used as a starting point. This model describes carbon and nitrogen transformations in a HSSF CW. Phosphorus transformations are not considered since these are mainly of physical-chemical nature and the main focus of the model is on microbial processes. This does imply that phosphorus concentrations are assumed to be non-limiting towards microbial and plant growth.

Table 6.1. Average removal efficiencies (in %) of the constructed wetlands in Saxby (based on average concentrations) and mass removal rates (based on 8-hourly samples).

	Inlet ($mg\ l^{-1}$)	Outlet Bed I ($mg\ l^{-1}$)	Outlet Bed II ($mg\ l^{-1}$)	Removal (%)	Mass removal ($kg\ ha^{-1}\ day^{-1}$)
TON	0.9 ± 1.5	2.5 ± 2.7	1.1 ± 1.4	*-25.3*	1.1
SS	52.7 ± 28.0	32.4 ± 14.8	16.6 ± 6.7	68.5	67.9
BOD_t	73.7 ± 47.2	4.6 ± 3.1	2.1 ± 1.0	97.1	25.6
BOD_f	52.2 ± 32.0	3.1 ± 1.9	1.8 ± 0.7	96.6	17.7
NH_4-N	21.7 ± 11.9	8.4 ± 3.4	5. 7 ± 1.7	73.8	3.0
KjN	22.7 ± 12.2	9.8 ± 2.6	7.7 ± 0.6	65.9	3.0
o-PO_4	6.7 ± 3.3	1.6 ± 0. 5	2.7 ± 0.5	59.6	1.0
TN	22.8 ± 11.1	12.0 ± 2.0	8.3 ± 0.7	63.8	4.8
TOC_t	31.9 ± 10.5	16. 7 ± 2.1	15.2 ± 1.1	52.4	9
TOC_f	30.0 ± 9.2	16.0 ± 1.5	14.6 ± 0.9	51.2	8.1
HLR ($cm\ day^{-1}$)	18.7 ± 29.0 (min. 4.3 – max. 101.7)				

The model requires 9 inputs, 6 regarding the influent (flow rate, BOD_t, Organic N, NH_4-N, NO_3-N and dissolved oxygen) and 3 regarding external influences (day length, air temperature and precipitation). There are 6 standard outputs that are equal to the influent inputs. One can however also keep track of certain model variables like plant growth, peat accumulation, evapotranspiration etc. if that is of interest. The dynamics of the 15 state variables are modelled via 15 ordinary differential equations that contain a total of 42 parameters related to physical, microbiological and biological processes. Microbial reactions are represented by a standard Monod equation with switching functions, which means that biofilm processes and especially diffusion limitations are neglected. To counteract this rather drastic approach, one can lower the values of the microbial kinetic parameters. For a comprehensive explanation of the model, the reader is referred to the paper of Wynn and Liehr (2001).

One important assumption of the Wynn and Liehr (2001) model is that the suspended solids removal efficiency approaches 100%, meaning that no particulate substances are leaving the reed bed. This was based on the fact that effluent SS levels of HSSF CW are generally observed to be very low. For the Saxby case, effluent SS concentrations are not really negligible: they vary between 8 and 71 mg l^{-1} in the effluent of the first reed bed, and between 8 and 33 mg l^{-1} in the effluent of the second one. However, filtered TOC and N concentrations in the effluents were observed to be nearly equal to the total concentrations, thus the assumption that only dissolved carbon and nitrogen compounds are exiting the system is still valid.

Originally, the model was written in STELLA© code (High Performance Systems Inc.). The simulations for this study were carried out in WEST© (Hemmis NV). Since WEST© works with the Model Specification Language (MSL), the model had to be recoded. During this phase, some minor model flaws were rectified (De Wilde, 2001; De Moor, 2002).

Subsurface flow is modelled by means of a classic Darcy equation. This concept was maintained although the following major adjustments were made to the water balance. Firstly, the effluent flow rate is now allowed to drop to zero if water loss by evapotranspiration exceeds the water supply as influent and precipitation. Secondly, several extra equations were added to make the model capable of dealing with flooding of the beds due to storm water peak discharges. This overland flow is modelled with a standard Manning equation to calculate flow rates dependent on bed slope, bed roughness and water height.

To represent intermediate flow behaviour, two completely mixed tanks in series were used to represent one reed bed. Unfortunately, no data from tracer tests were available to check this assumption, but the stability of the effluent concentrations (Fig. 6.2) seems to indicate a considerable degree of mixing. On the one hand, this lack of tracer test data adds to the uncertainty on the simulation results, but on the other hand, tracer test data will never be available during the design phase of a new reed bed for which purpose this model is being tested. The choice to use only 2 tanks in series was also based on the work of Wynn and Liehr (2001) who obtained reasonable results with only 1 completely mixed tank to represent a reed bed with a higher L/W ratio. Finally, one should also consider that computation time increases as the model complexity increases.

One other important adjustment concerns the carbon balance. The original model of Wynn and Liehr (2001) converts influent BOD data to Dissolved and Particulate Organic Carbon concentrations (DOC and POC) and vice versa for the effluent; the obvious advantage being that the model is able to use commonly available BOD concentrations. This conversion routine however uses several constants to translate oxygen demand into carbon concentrations, and to split total oxygen demand into dissolved and particulate fractions. In reality, these conversion values were observed to be highly variable and therefore of considerable influence on the model predictions. During this study, the model was therefore directly fed with DOC and POC data.

Based on the observed relative stability of the ammonium effluent concentrations, the model was finally extended with a Freundlich sorption isotherm equation for ammonium, as described in McBride and Tanner (2000).

Obviously, this complexity of the model, as outlined in the previous paragraphs, enables to better summarise the processes that occur within CWs as well as to demonstrate interactions between certain components. It requires however estimation of 15 initial conditions for the state variables and knowledge about or estimation of 42 parameters, which is not a straightforward task. Rousseau *et al*. (2004b) demonstrated that simply copying parameter values from another model or another case study does not guarantee reliable model predictions. Extracting parameter values from literature data can also prove to be difficult due to a large variability in reported values. For example, values of one of the parameters applied in this model, i.e. the Biomass Oxygenation Rate of *Phragmites australis* that represents root oxygen loss, are summarised by Brix (1997). Values are reported to vary between 0.02 and 12 g O_2 m^{-2} d^{-1}. Literature can thus give an indication of the possible range of parameter values, but can often not provide a crisp value. The following paragraphs therefore summarise the applied calibration routines based on the assembled input-output data and the resulting model fits.

6.6. GLOBAL SENSITIVITY ANALYSIS

Wynn and Liehr (2001) carried out a basic sensitivity analysis of this model by visual comparison of the model outputs with the measured effluent concentrations, before and after having adjusted a parameter value. They found that the model was most sensitive towards changes in parameters that affect microbial growth and substrate use directly, i.e. heterotrophic maximum growth rate, heterotrophic death rate and initial heterotrophic cell mass. Ammonium predictions where, as can be expected, significantly influenced by parameters controlling autotroph growth.

To quantify the model sensitivity and to identify the important parameters for further calibration, the method of van der Peijl and Verhoeven (1999) was used for a global sensitivity analysis. This method examines the relative change in model output (X) divided by the relative change in the value of the parameter (Param) tested:

$$S_x = \frac{\delta X / X}{\delta Param / Param}$$

To judge this change in model output (X), the Sum of Squared Errors (SSE) was used based on the deviations between the model predictions and the measured concentrations. The higher the absolute value of S_x, the more sensitive the model is towards changes of that parameter or in other words, a minor change of the parameter value causes a major change of the model predictions. S_x values were calculated for both reed beds, for DOC and NH_4 and for parameter changes of -25, -10, +10 and +25%. The results of the latter percent-wise parameter changes were fairly similar. The cut-off S_x value was arbitrarily set at 0.1.

In general, the reed bed dimensions proved to be highly sensitive parameters. This can be logically explained by the major impact of reed bed dimensions on the hydraulic residence time and thus on the water balance. Other sensitive parameters towards

DOC and NH_4 predictions are summarised in Table 6.2. Seemingly counterintuitive, the sensitivity analysis on the second reed bed revealed many more parameters with a high S_x value than the analysis on the first reed bed did. However, due to the low concentrations, other processes like for instance plant uptake become relatively more important and related parameters therefore become more sensitive.

Table 6.2. Results of the global sensitivity analysis: parameters that have a major impact on DOC and NH_4 predictions for both reed beds (S_x value ≥ 0.1).

DOC – first reed bed	**DOC – second reed bed**
• Reed bed dimensions (LxWxd) • Heterotrophic temperature factor (dimensionless) • Heterotrophic yield coefficient for NO_3 (g biomass (g NO_3-N)$^{-1}$) • Heterotrophic maximum growth rate under aerobic conditions (d^{-1}) • Root oxygen loss (g O_2 m^{-2} d^{-1}) • Heterotrophic yield coefficient for dissolved oxygen (g biomass (g O_2)$^{-1}$)	Same as bed 1 + • Hydraulic conductivity • Porosity • Autotrophic oxygen affinity constant (mg O_2 L^{-1}) • Microbial C content (g C (g biomass)$^{-1}$) • Peat C content (g C (g peat)$^{-1}$) • Heterotrophic affinity constant for organic material (mg L^{-1}) • Heterotrophic death rate (d^{-1}) • Heterotrophic oxygen affinity constant (mg O_2 l^{-1}) • Autotrophic temperature factor (dimensionless) • Autotrophic maximum growth (d^{-1}) • Autotrophic yield coefficient for oxygen (g biomass (g O_2)$^{-1}$) • Peat accumulation rate (g peat d^{-1})
NH_4 – first reed bed	**NH_4 – second reed bed**
• Reed bed dimensions (LxWxd) • Porosity • Freundlich specific NH_4 sorption rate coefficient (d^{-1}) • Freundlich exponent (dimensionless) • Freundlich solid-liquid NH_4 partition coefficient (l (kg gravel)$^{-1}$)	Same as bed 1 + • Hydraulic Conductivity • C:N ratio of reed plants (g C g N^{-1}) • C content of reed plants (g C (g biomass)$^{-1}$) • Reed growth rate (g biomass m^{-2} d^{-1})

The outcomes for DOC are generally in accordance with the findings of Wynn and Liehr (2001): microbial parameters are the more sensitive ones. However, when looking at the NH_4 transformation processes, the newly introduced Freundlich isotherm parameters prove to be the most sensitive ones.

6.7. MODEL CALIBRATION

Once the most sensitive parameters had been identified, their optimal value was determined by searching that value that results in the lowest SSE, or in other words the parameter value that results in a minimal deviation between measured and simulated concentrations. Two examples of the outcomes of this procedure are summarised in Fig. 6.3 for the parameters Biomass Oxygenation Rate and Heterotrophic Yield Coefficient for Dissolved Oxygen.

Fig. 6.3 clearly illustrates that the optimal parameter values can be different for every variable. For example, a Biomass Oxygenation Rate of 0.22 g O_2 m^{-2} d^{-1} yields a best fit (minimal SSE) for DOC but not for NH_4 where a best fit is obtained with a Biomass Oxygenation Rate value of 0.1 g O_2 m^{-2} d^{-1}. All optimal parameter values must therefore be taken into account when calibrating the model and a trade-off has to be made between the impacts on the different SSE values.

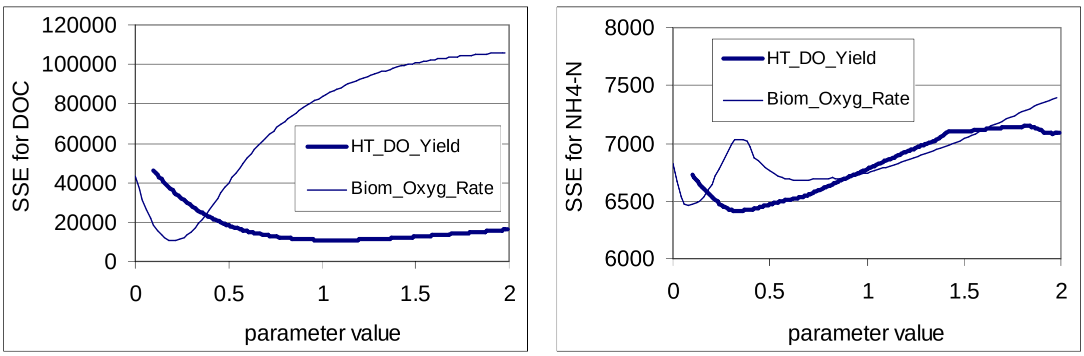

Figure 6.3. Impact of varying parameter values of the Heterotrophic Yield Coefficient for Dissolved Oxygen (thick line) and the Biomass Oxygenation Rate (thin line) on the model fits or SSEs for DOC and NH_4.

6.8. SIMULATION RESULTS

Fig. 6.4 compares simulated and measured effluent concentrations of DOC and NH_4-N of the first reed bed in Saxby. These graphs show the best possible fit, obtained by introducing the optimal parameter values into the model, as identified in the previous paragraph. One can see that the DOC effluent concentrations fit very well, except for the two small peaks at day 3 and day 5, which coincide with the storm peak flow rates. The model seems to underestimate the buffering capacity of the reed bed. Simulated NH_4-N effluent concentrations on the contrary are less dynamic than was observed in reality.

For validation purposes, the model was run again with the dataset of the second reed bed. Especially N removal was not adequately predicted. This does not immediately imply that the model is incorrect. Indeed, some parameters and initial conditions can be different for bed I and bed II. Because plants and microorganisms in the second reed bed are subjected to smaller loads, several authors have proven that e.g. growth rates are lower. Hence, new simulations with among others lower growth rates were performed and these gave somewhat better results.

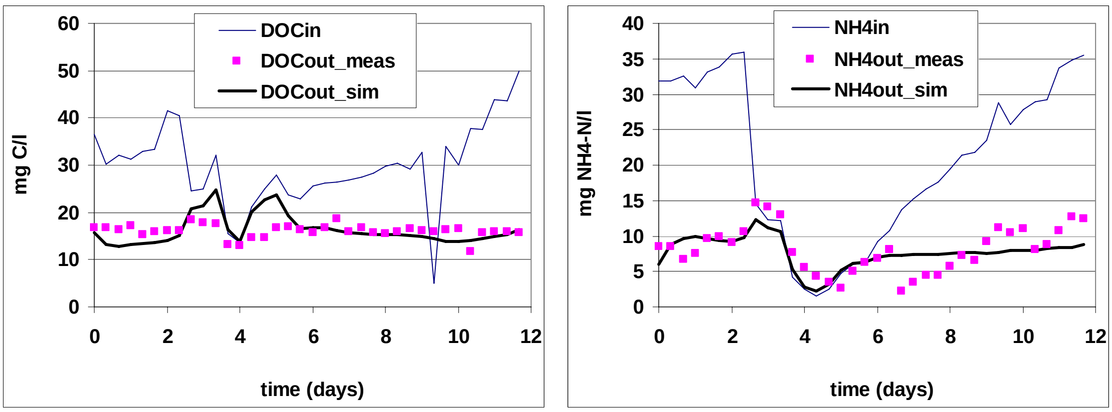

Figure 6.4. Left panel: representation of measured influent and effluent DOC concentrations at the first reed bed at Saxby and comparison with simulated effluent DOC concentrations. Right panel: representation of measured influent and effluent NH_4-N concentrations at the first reed bed at Saxby and comparison with simulated effluent NH_4-N concentrations.

Fig. 6.5 compares simulated and measured effluent concentrations of DOC and NH_4-N of the second reed bed in Saxby. These graphs show again the best possible fit, obtained by introducing the optimal parameter values into the model, as identified in the previous paragraph. Since for the second reed bed more parameters were found to be sensitive, obtaining a best fit was not obvious. Especially the model predictions of NH_4 deviate considerably from the measured concentrations.

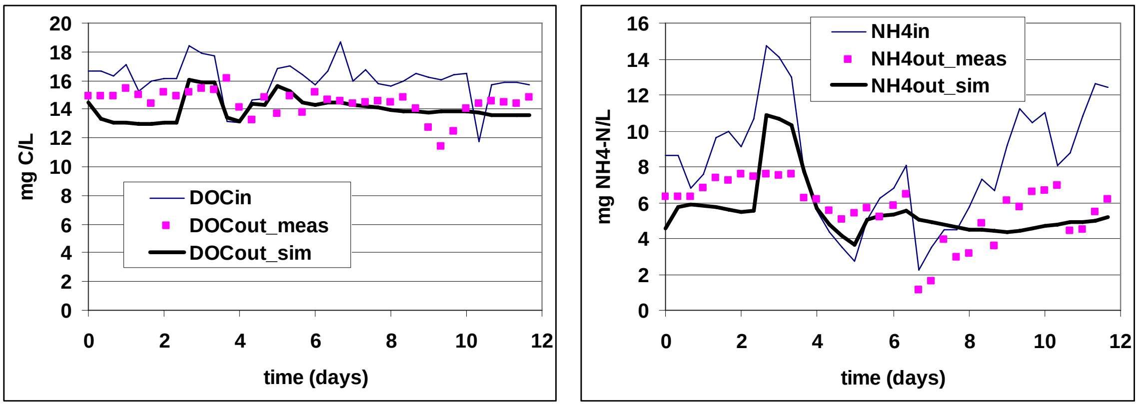

Figure 6.5. Left panel: representation of measured influent and effluent DOC concentrations at the second reed bed in Saxby and comparison with simulated effluent DOC concentrations. Right panel: representation of measured influent and effluent NH_4-N concentrations at the second reed bed in Saxby and comparison with simulated effluent NH_4-N concentrations.

6.9. DISCUSSION

Although Wynn and Liehr (2001) obtained fair results with their long-term, low-frequent dataset, the initial model results for the Saxby case were not satisfying at all. There are a number of possible causes for this discrepancy:

- *Time steps*: Wynn and Liehr (2001) used a dataset that consisted of biweekly measurements of C and N components (grab samples). They interpolated between data points to have daily inputs for the model. This is totally unlike the Saxby dataset (8-hour composite samples) and will certainly have some influence on the model performance.
- *Simulation period*: Wynn and Liehr (2001) performed a simulation over almost one year and thus covered several seasons. This was not the case for the Saxby

dataset (only summer conditions) and will again have some influence on the model output.

- *Model uncertainty*: it is quite possible that some processes occur in constructed treatment wetlands that are not included in the model. Due to external conditions, these processes might have been of minor importance in the Wynn and Liehr case, but of bigger importance in the Saxby case.
- *Measurements*: analytical uncertainties together with the use of non-cooled samplers might have caused deviations between measured and actual concentrations.

One important conclusion was derived from preliminary simulations (data not shown) and the given model predictions: knowledge of the water balance and the hydraulic behaviour or rather the degree of mixing, is of utmost importance for the model performance. Too few CSTRs in series cause every concentration peak to be flattened out whereas too many CSTRs result in false peak concentrations and, from a practical point of view, also in an increased simulation time. When gathering datasets for calibration, a simultaneous tracer test should therefore be carried out.

Because the model output does not always closely match the measured dynamics of the effluent concentrations, it might still be unwise at this point to apply the model as a design tool. Indeed, when accepting a too stable model output, a reed bed designed according to these model specifications could in reality produce an effluent that exceeds the standards from time to time. On the other hand, when accepting a too dynamic model output, the dimensions of the reed bed would probably be increased during the design phase to make sure the effluent quality will be acceptable. This will result in unnecessarily high investment costs.

6.10. CONCLUSIONS

Design of horizontal subsurface-flow constructed wetlands is usually based on the well-known state-of-the-art k-C* model (Kadlec and Knight, 1996c). One important shortcoming of this black box model is the oversimplification of reality, which results in a large uncertainty on the model predictions. Another drawback is the inability of the k-C* model to predict short-term effluent dynamics. A possible solution for these drawbacks could be the application of dynamic compartmental models.

With the dynamic model of Wynn and Liehr (2001) as a starting point, a new model was developed that reflects the conditions of the test site, a two-stage HSSF CW in Saxby (Leicestershire, UK). Several model extensions, especially the NH_4-sorption sub-model and the imitation of overland flow, significantly enhanced the model validity.

In the next phase, this new model was calibrated by means of a high-frequent dataset collected at the Saxby treatment wetlands. A quantitative sensitivity analysis revealed that reed bed dimensions had a major impact on all model predictions, which can be easily explained by the relation between the reed bed dimensions and the hydraulic behaviour. Heterotrophic kinetic parameters had most influence on the DOC predictions, whereas the parameters from the Freundlich sorption isotherm had a major impact on the NH_4-N predictions. By varying the values of these most sensitive parameters, a best fit was searched between the model outputs and the measured effluent concentrations. For optimal results, some parameters needed different values for the first and second reed bed. This can be logically explained by different governing conditions in both reed beds.

Final simulation results of both reed beds were acceptable but missed some of the dynamics observed in reality. When using this model as a design tool, this could result in a too conservative design if the model output is more dynamic than in reality, or an under dimensioned reed bed in case of a more stable model output than occurs in reality.

Further calibration and validation with other datasets is thus needed to improve the model predictions and to reduce the parameter uncertainty. Possible steps to improve the reliability of the model output are multiple. Firstly, it would be valuable to close the mass balances of carbon and nitrogen. Extra equations, and thus extra parameters, will therefore have to be added to the model, resulting on the one hand in a higher model complexity, but on the other one also in a higher model transparency. Secondly, new calibration efforts with data from other CWs should consider the following recommendations: (i) always carry out a tracer test, (ii) enhance the information content of the dataset by varying the loading rates and (iii) try to take as many direct measurements of parameters and initial conditions as possible. Finally, to be really valid for use as a design tool, the model should also be tested for seasonality.

Chapter 7

A conceptual model framework for interpreting carbon and nitrogen cycles in horizontal subsurface-flow constructed wetlands

7.1. ABSTRACT

In contrast to more conventional wastewater treatment techniques such as activated sludge plants or anaerobic reactors where major efforts have been made to develop mechanistic models as predictive tools, the state-of-the-art in constructed wetlands' modelling still consists of black-box approaches, whereby the inherent complexity of such an artificial ecosystem is entirely neglected. This chapter proposes a comprehensive model framework for horizontal subsurface-flow constructed wetlands which draws from the available modelling experience in the previously mentioned conventional treatment techniques. It focuses on microbially and plant-mediated carbon and nitrogen cycles and implicitly takes into account the competition for substrates, nutrients and electron acceptors between the different organism groups.

7.2. GENERAL DESCRIPTION

Based on the data analysis in Chapter 4, the model analysis in Chapter 5 and the experience with the Wynn and Liehr (2001) model as described in Chapter 6, a number of criteria emerged which delineated the structure of a new model of horizontal subsurface-flow CWs:

- for modelling the microbial conversions, the Activated Sludge Model (ASM, Henze *et* al., 2000) approach seems to be the most appropriate one because (i) it uses when possible closed mass balances, (ii) it facilitates communication between researchers and practitioners, (iii) model equations have proven their validity in many case studies and (iv) numerous data on parameter values are available;
- a relatively simple but accurate hydraulic submodel is needed such that it can be calibrated with limited data;
- particulate substances need to be incorporated into the model, to allow investigations of clogging and long-term assessment of hydraulic characteristics;
- meteorological influences need to be taken into account to allow for long-term simulations.

The developed model considers microbiological and plant-related processes affecting COD and nitrogen in HSSF wetlands. As for the Wynn and Liehr (2001) model, phosphorus removal is not considered and it is therefore assumed that P-concentrations are non-limiting for microbial and plant growth. The model structure allows to introduce these processes if they are of interest for the CW under investigation.

Hydraulic and hydrological submodels

With regard to mimicking the flow conditions, it was decided to stick to the continuously-stirred tanks-in-series approach, as many authors have proven its validity (cf. Kadlec and Knight, 1996c). The approach is easily comprehensible and it is easily implementable in most modelling and simulation software packages. Recently, Marsili-Libelli and Checchi (2005) presented a comparable model that is based on a network of CSTRs of unequal volume and one plug flow reactor. Being easily implementable, requiring only simple data from a tracer test and yielding excellent simulation results, this approximation of dispersed flow seems very promising.

The approach of continuously-stirred tank reactors (CSTRs) assumes a vertical uniform distribution of substrates, intermediates, products and bacteria, which may not be the case for HSSF wetlands. Studies dealing with vertical gradients in HSSF have yielded different conclusions. For example, Breen and Chick (1995) and García *et al.* (2003) have observed vertical changes for the concentration of organic matter and ammonia, whereas Headley *et al.* (2005) in contrast observed a nearly homogeneous distribution of different contaminants through the entire water depth. Vertical mixing is strongly related to hydrodynamic properties of the bulk water, and therefore linked to water velocity, hydraulic loading rate and length-to-width ratio (Headley *et al.*, 2005). For the purposes of the present study, the assumption of CSTRs should therefore be viewed as an easy and pragmatic approach. More experimental evidence on vertical gradients and the development of dynamic dispersed models will allow to improve the model presented here.

The water balance is exactly as in Wynn and Liehr (2001), i.e. based on the Darcy equation for steady flow in porous media. Effluent flow variations are related to hydraulic loading rates, evapotranspiration rates and precipitation.

Microbiological processes

Aerobic and anoxic microbial carbon and nitrogen conversion processes are mainly based on the Activated Sludge Model N° 1 (ASM1; Henze *et al*., 2000). However, several improvements of this original model developed in subsequent versions of the model (ASM 2, 2d and 3; Henze *et al*., 2000) have been taken into account in order to attain a more mechanistic approach. As an example, in the wetland model, the distinction between X_I (inert particulate COD) and X_P (inert particulate COD formed by decay of microorganisms) is not made and X_P is therefore considered as X_I. More importantly, with the assumption that HSSF CWs act as (near)perfect physical filters, preliminary mass balances showed that the pore volume would decrease much quicker than is observed in reality, due to accumulation of X_I. Indeed, X_I might be non-biodegradable within the sludge residence time of an activated sludge plant, but might be slowly degraded during the many years that it resides within the pores of the wetland. It was therefore decided to consider X_I as very slowly biodegradable, with conversion to really inert particulate COD, soluble inert COD and slowly biodegradable COD. Analogously to ADM1 (Batstone *et al*., 2002), this process will be referred to as decomposition and what was referred to as X_I in the ASM models has now been called X_C, whilst X_I is now used for the really unbiodegradable fraction.

As mentioned earlier (3.5. Dissolved oxygen balance), oxidised zones only occur close to the water surface where oxygen is provided by diffusion, and in a thin layer around the plant roots where oxygen leakage occurs. Experimental evidence from a.o. García *et al*. (2003) indeed shows average DO concentrations of 0.1 mg O_2 l^{-1} or lower and redox potentials in the order of –350 mV, suggesting the presence of anaerobic microbial pathways. Baptista (2003) analysed the bacterial diversity and activity in HSSF CWs and indeed concluded that methanogens and sulphate reducers were probably the main removers of soluble organic carbon. The experimental results of Huang *et al*. (2005) also suggest the importance of anaerobic pathways in HSSF CWs. Anaerobic microbial processes were therefore included in the model. They were drawn from the work of Kalyuzhnyi and Fedorovich (1998) on the competition between sulphate reducing and methanogenic bacteria. Their approach was preferred above the more common-place Anaerobic Digestion Model (ADM1, Batstone *et al.*, 2002) because the latter one focusses on sludge digestion rather than wastewater treatment and also because ADM1 does not take into account sulphate reduction. Using these validated models furthermore allows to apply the given parameter values with some degree of confidence, thereby possibly reducing the required calibration efforts.

To avoid sulphide accumulation in the system and therefore a strong microbial inhibition, an inverse pathway has been foreseen by adding sulphide oxidising bacteria to the model. Adding the X_{THIO} microbial community to the model was preferred above chemical sulphur oxidation as the model focuses on (micro)biological processes.

Physical processes

It is assumed, as in the Wynn and Liehr (2001) model, that suspended solids are completely removed near the inlet (at less than 1/3 of the total length) under normal operating conditions. Only at higher flow rates, wash-out of solids proportional to the flow rate has been foreseen.

Although detachment of biofilms is a commonly acknowledged process, it is assumed that sloughed parts of the biofilms are retained within the pores and are still metabolising, unless they are washed out by a peak flow.

Plant-related processes

Following the example of Wynn and Liehr (2001), the plant growth and decay model is deliberately kept simple, despite the many influencing factors that have been reported in literature. Indeed, factors such as nutrient availability, air temperature, irradiation, water level etc., all affect plant growth and/or decay to a greater or lesser extent. However, taking them all into account leads to a far more complex (ecological) model which does not fall within the scope of this work. Also, as described in Chapter 3, plant nutrient uptake is in most cases relatively insignificant compared to other nutrient removal processes. Simplifications in the plant growth model might therefore only have a small impact on nutrient removal predictions.

Plant growth is modelled by means of ‘Relative Growth Rates’ as there are many data available in literature. Plant growth is not zero-order, but depends on ammonium and nitrate concentrations. Most importantly, plant material is no longer expressed as Carbon but as COD, which is rather unusual but allows for a smooth integration with the COD-based microbial processes. Other plant-related processes include decay/senescence and physical degradation (based on Wynn and Liehr, 2001) and root oxygen loss. In practice, the contribution of plant physical degradation to the increase of organic matter in the system might be small if aerial parts are periodically harvested.

7.3. STATE VARIABLES

The model contains 26 state variables and thus 26 mass balances. Twentythree state variables are concentrations (1 to 23 in Table 7.1), two of them are areal densities (24 and 25 in Table 7.1) and one is water volume (26 in Table 7.1). The nomenclature for concentrations of dissolved components is an S, whereas particulate components are referred to with an X. When their mass is considered, the nomenclature is either MS or MX. For a more comprehensive explanation on the different wastewater fractions, one is referred to Henze *et al*. (2000) and Kalyuzhnyi and Fedorovich (1998).

7.4. INPUTS AND OUTPUTS

Inputs consist of wastewater flow rates and concentrations on the one hand, and five variables that reflect the meteorological conditions on the other hand. As outputs, effluent flow rates and concentrations are given.

The concentration vector consists of state variables 1 to 13 and 15 to 23 (Table 7.1). The climate vector contains input data on:

1. Air temperature (°C)
2. Precipitation (m day^{-1})
3. Length of day (day)
4. Season (-)
5. Water temperature (°C)

Table 7.1. State variables.

1	S_O	Dissolved oxygen (gO_2 m^{-3})
2	S_I	Inert soluble COD ($gCOD_{substrate}$ m^{-3})
3	S_F	Fermentable soluble COD ($gCOD_{substrate}$ m^{-3})
4	S_A	Acetate ($gCOD_{substrate}$ m^{-3})
5	S_{NH}	Ammonium (gN m^{-3})
6	S_{ND}	Soluble organic nitrogen (gN m^{-3})
7	S_{NO}	Nitrate (gN m^{-3})
8	S_{SO4}	Sulphate (gS m^{-3})
9#	S_{H2S}	Sulphide (gS m^{-3})
10	S_{H2}	Hydrogen (gCOD m^{-3}) (conversion factor: 16 g COD (mol H_2)$^{-1}$)
11	X_C	Very slowly biodegradable particulate COD ($gCOD_{substrate}$ m^{-3})
12	X_S	Slowly biodegradable particulate COD ($gCOD_{substrate}$ m^{-3})
13	X_{ND}	Particulate organic nitrogen (gN m^{-3})
14	X_{NH}	Sorbed ammonium (gN (kg gravel)$^{-1}$)
15	X_{BH}	Heterotrophic bacteria ($gCOD_{microbial}$ m^{-3})
16	X_{BA}	Autotrophic nitrifying bacteria ($gCOD_{microbial}$ m^{-3})
17	X_{FB}	Fermenting bacteria ($gCOD_{microbial}$ m^{-3})
18	X_{AMB}	Acetotrophic methanogenic bacteria ($gCOD_{microbial}$ m^{-3})
19	X_{ASRB}	Acetotrophic sulphate reducing bacteria ($gCOD_{microbial}$ m^{-3})
20	X_{HMB}	Hydrogenotrophic methanogenic bacteria ($gCOD_{microbial}$ m^{-3})
21	X_{HSRB}	Hydrogenotrophic sulphate reducing bacteria ($gCOD_{microbial}$ m^{-3})
22	X_{THIO}	Sulphide oxidising bacteria ($gCOD_{microbial}$ m^{-3})
23	X_I	Inert particulate COD (gCOD m^{-3})
24	X_{Pl}	Living plant biomass ($gCOD_{plant}$ m^{-2})
25	X_{Pd}	Dead standing plant biomass ($gCOD_{plant}$ m^{-2})
26	V_w	Pore water volume (m^3)

S_{H2S} represents the sum of all species whereas S_{H2S*} represents the undissociated form

7.5. MASS BALANCES

An overview of the process rates and their stoichiometry is given in Tables 7.2 and 7.3. In general, each mass balance has the following structure:

$$\frac{dMass}{dt} = \text{influx} - \text{efflux} + \text{conversion}$$

Assuming an index i for the rows and an index j for the colums in Tables 7.2 and 7.3, the total conversion rate for one component can thus be calculated as follows:

$$\text{Conversion(component j)} = \sum_{i=0}^{32} \nu_{i,j} * \rho_i$$

with ν: the stoichiometric coefficient (Table 7.2)
ρ: the process rate (Table 7.3) with units g m^{-3} d^{-1}

A general overview of the model structure can be found in Fig. 7.1. The detailed mass balances of each component are then treated in detail in the following sections. Indices of the given processes correspond with the rows and columns of Tables 7.2 and 7.3. For an explanation of the parameters, the reader is referred to Section 7.6.

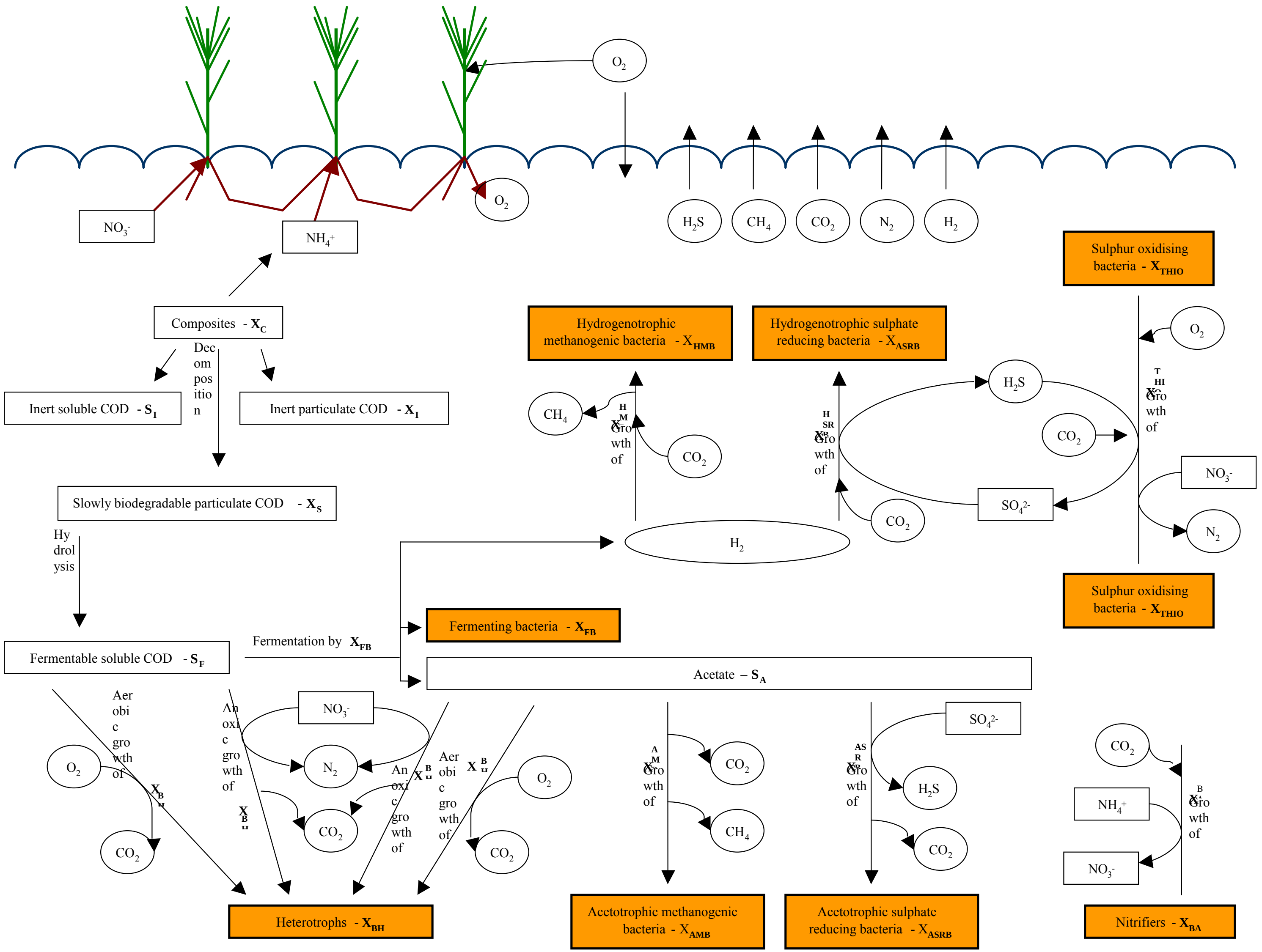

Figure 7.1. Schematic overview of the model structure.

Table 7.2. Stoichiometry of processes affecting dissolved components.

	S_O	S_I	S_F	S_A	S_{NH}	S_{ND}	S_{NO}	S_{SO4}	S_{H2S}	S_{H2}	Rate
Decomposition		f_{C_SI}									$\rho0$
Hydrolysis of organics by X_{BH} and X_{FB}			1								$\rho1$
Hydrolysis of organic nitrogen by X_{BH} and X_{FB}						1					$\rho2$
Ammonification by X_{BH} and X_{FB}					1	-1					$\rho3$
Aerobic growth of X_{BH} on S_F	$-\frac{1-Y_H}{Y_H}$		$-1/Y_H$		$-i_{XB}$						$\rho4$
Aerobic growth of X_{BH} on S_A	$-\frac{1-Y_H}{Y_H}$			$-1/Y_H$	$-i_{XB}$						$\rho5$
Anoxic growth of X_{BH} on S_F			$-1/Y_H$		$-i_{XB}$		$-\frac{1-Y_H}{2.86Y_H}$				$\rho6$
Anoxic growth of X_{BH} on S_A				$-1/Y_H$	$-i_{XB}$		$-\frac{1-Y_H}{2.86Y_H}$				$\rho7$
Aerobic growth of X_{BA}	$-\frac{4.57-Y_A}{Y_A}$				$-i_{XB}-\frac{1}{Y_A}$		$1/Y_A$				$\rho8$
Decay of X_{BH}											$\rho9$
Decay of X_{BA}											$\rho10$
Growth of X_{FB}			$-1/Y_{FB}$	$\frac{1-Y_{FB}}{1.227Y_{FB}}$	$-i_{XB}$					$\frac{1-Y_{FB}}{2.46Y_{FB}}$	$\rho11$
Growth X_{AMB}				$-1/Y_{AMB}$	$-i_{XB}$						$\rho12$
Growth of X_{ASRB}				$-1/Y_{ASRB}$	$-i_{XB}$			$-\frac{1-Y_{ASRB}}{0.55Y_{ASRB}}$	$\frac{1-Y_{ASRB}}{1.65Y_{ASRB}}$		$\rho13$
Growth X_{HMB}					$-i_{XB}$					$-1/Y_{HMB}$	$\rho14$
Growth of X_{HSRB}					$-i_{XB}$			$-\frac{1-Y_{HSRB}}{0.55Y_{HSRB}}$	$\frac{1-Y_{HSRB}}{1.65Y_{HSRB}}$	$-1/Y_{HSRB}$	$\rho15$
Aerobic growth of X_{THIO}	$-\frac{1-Y_{THIO}}{2*Y_{THIO}}$				$-i_{XB}$			$1/Y_{THIO}$	$-1/Y_{THIO}$		$\rho16$

Table 7.2. (contd). Stoichiometry of processes affecting dissolved components.

	S_O	S_I	S_F	S_A	S_{NH}	S_{ND}	S_{NO}	S_{SO4}	S_{H2S}	S_{H2}	Rate
Anoxic growth of X_{THIO}					$-i_{XB}$		$-\frac{1-Y_{THIO}}{1.59Y_{THIO}}$	$1/Y_{THIO}$	$-1/Y_{THIO}$		ρ17
Decay of X_{FB}											ρ18
Decay of X_{AMB}											ρ19
Decay of X_{ASRB}											ρ20
Decay of X_{HMB}											ρ21
Decay of X_{HSRB}											ρ22
Decay of X_{THIO}											ρ23
Plant growth on ammonium					$-i_{Xplant}$						ρ24
Plant growth on nitrate							$-i_{Xplant}$				ρ25
Plant oxygen leaching	1										ρ26
Plant decay											ρ27
Plant physical degradation											ρ28
Ammonia adsorption/desorption					1						ρ29
Physical reaeration	1										ρ30
Hydrogen gas volatilisation										-1	ρ31
H_2S volatilisation									-1		ρ32

Table 7.2. (contd). Stoichiometry of processes affecting particulate components.

	X_C	X_S	X_{ND}	X_{NH}	X_{BH}	X_{BA}	X_{FB}	X_{AMB}	X_{ASRB}	X_{HMB}	X_{HSRB}	X_{THIO}	X_{PI}	X_{Pd}	X_I	Rate
Decomposition	-1	f_{C_XS}													f_{C_XI}	ρ0
Hydrolysis of organics by X_{BH} and X_{FB}		-1														ρ1
Hydrolysis of organic nitrogen by X_{BH} and X_{FB}			-1													ρ2
Ammonification by X_{BH} and X_{FB}																ρ3
Aerobic growth of X_{BH} on S_F					1											ρ4
Aerobic growth of X_{BH} on S_A					1											ρ5
Anoxic growth of X_{BH} on S_F					1											ρ6
Anoxic growth of X_{BH} on S_A					1											ρ7
Aerobic growth of X_{BA}						1										ρ8
Decay of X_{BH}	f_P	$1- f_P$	$i_{XB} - f_P*i_{XP}$		-1											ρ9
Decay of X_{BA}	f_P	$1- f_P$	$i_{XB} - f_P*i_{XP}$			-1										ρ10
Growth of X_{FB}							1									ρ11
Growth X_{AMB}								1								ρ12
Growth of X_{ASRB}									1							ρ13
Growth X_{HMB}										1						ρ14
Growth of X_{HSRB}											1					ρ15
Aerobic growth of X_{THIO}												1				ρ16
Anoxic growth of X_{THIO}												1				ρ17
Decay of X_{FB}	f_P	$1- f_P$	$i_{XB} - f_P*i_{XP}$				-1									ρ18
Decay of X_{AMB}	f_P	$1- f_P$	$i_{XB} - f_P*i_{XP}$					-1								ρ19
Decay of X_{ASRB}	f_P	$1- f_P$	$i_{XB} - f_P*i_{XP}$						-1							ρ20
Decay of X_{HMB}	f_P	$1- f_P$	$i_{XB} - f_P*i_{XP}$							-1						ρ21
Decay of X_{HSRB}	f_P	$1- f_P$	$i_{XB} - f_P*i_{XP}$								-1					ρ22
Decay of X_{THIO}	f_P	$1- f_P$	$i_{XB} - f_P*i_{XP}$									-1				ρ23

Table 7.2. (contd). Stoichiometry of processes affecting particulate components.

	X_I	X_S	X_{ND}	X_{NH}	X_{BH}	X_{BA}	X_{FB}	X_{AMB}	X_{ASRB}	X_{HMB}	X_{HSRB}	X_{THIO}	X_{Pl}	X_{Pd}	X_{IP}	Rate
Plant growth on ammonium													1			ρ24
Plant growth on nitrate													1			ρ25
Plant oxygen leaching																ρ26
Plant decay													-1	1		ρ27
Plant physical degradation	f_{Plant}	$1 - f_{Plant}$	$i_{XPlant} - f_{Plant} * i_{XPlant}$											-1		ρ28
Ammonia adsorption/desorption				$\frac{\varepsilon}{\rho_{gravel}}$												ρ29
Physical reaeration																ρ30
Hydrogen gas volatilisation																ρ31
H_2S volatilisation																ρ32

Table 7.3. Process rates

	Rate	Expression
Decomposition	ρ0	$k_{decomp} * X_C$
Hydrolysis of organics by X_{BH} and X_{FB}	ρ1	$k_h * \left[\frac{X_S/(X_{BH}+X_{FB})}{K_X+(X_S/(X_{BH}+X_{FB}))}\right] * (X_{BH}+\eta_h * X_{FB})$
Hydrolysis of organic N by X_{BH} and X_{FB}	ρ2	$k_h * \left[\frac{X_S/(X_{BH}+X_{FB})}{K_X+(X_S/(X_{BH}+X_{FB}))}\right] * \left(\frac{X_{ND}}{X_S}\right) * (X_{BH}+\eta_h * X_{FB})$
Ammonification by X_{BH} and X_{FB}	ρ3	$k_a * S_{ND} * (X_{BH} + n_h * X_{FB})$
Aerobic growth of X_{BH} on S_F	ρ4	$\mu_H * \left(\frac{S_F}{K_{SF}+S_F}\right) * \left(\frac{S_F}{S_F+S_A}\right) * \left(\frac{S_O}{K_{OH}+S_o}\right) * \left(\frac{S_{NH}}{K_{NHH}+S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH}+S_{H2S*}}\right) * X_{BH}$
Aerobic growth of X_{BH} on S_A	ρ5	$\mu_H * \left(\frac{S_A}{K_{SA}+S_A}\right) * \left(\frac{S_A}{S_F+S_A}\right) * \left(\frac{S_O}{K_{OH}+S_o}\right) * \left(\frac{S_{NH}}{K_{NHH}+S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH}+S_{H2S*}}\right) * X_{BH}$
Anoxic growth of X_{BH} on S_F	ρ6	$n_g * \mu_H * \left(\frac{S_F}{K_{SF}+S_F}\right) * \left(\frac{S_F}{S_F+S_A}\right) * \left(\frac{K_{OH}}{K_{OH}+S_o}\right) * \left(\frac{S_{NO}}{K_{NOH}+S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHH}+S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH}+S_{H2S*}}\right) * X_{BH}$
Anoxic growth of X_{BH} on S_A	ρ7	$n_g * \mu_H * \left(\frac{S_A}{K_{SA}+S_A}\right) * \left(\frac{S_A}{S_F+S_A}\right) * \left(\frac{K_{OH}}{K_{OH}+S_o}\right) * \left(\frac{S_{NO}}{K_{NOH}+S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHH}+S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH}+S_{H2S*}}\right) * X_{BH}$
Aerobic growth of X_{BA}	ρ8	$\mu_A * \left(\frac{S_{NH}}{K_{NHA}+S_{NH}}\right) * \left(\frac{S_O}{K_{OA}+S_o}\right) * \left(\frac{K_{IA}}{K_{IA}+S_{H2S*}}\right) * X_{BA}$
Decay of X_{BH}	ρ9	$b_H * X_{BH}$
Decay of X_{BA}	ρ10	$b_A * X_{BA}$
Growth of X_{FB}	ρ11	$\mu_{FB} * \left(\frac{S_F}{K_{SFB}+S_F}\right) * \left(\frac{K_{IFB}}{K_{IFB}+S_{H2S*}}\right) * \left(\frac{K_{OFB}}{K_{OFB}+S_O}\right) * \left(\frac{K_{NOFB}}{K_{NOFB}+S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHFB}+S_{NH}}\right) * X_{FB}$
Growth of X_{AMB}	ρ12	$\mu_{AMB} * \left(\frac{S_A}{K_{SAMB}+S_A}\right) * \left(\frac{K_{IAMB}}{K_{IAMB}+S_{H2S*}}\right) * \left(\frac{K_{OAMB}}{K_{OAMB}+S_O}\right) * \left(\frac{K_{NOAMB}}{K_{NOAMB}+S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHAMB}+S_{NH}}\right) * X_{AMB}$
Growth of X_{ASRB}	ρ13	$\mu_{ASRB} * \left(\frac{S_A}{K_{SASRB}+S_A}\right) * \left(\frac{S_{SO4}}{K_{SOASRB}+S_{SO4}}\right) * \left(\frac{K_{IASRB}}{K_{IASRB}+S_{H2S*}}\right) * \left(\frac{K_{OASRB}}{K_{OASRB}+S_O}\right) * \left(\frac{K_{NOASRB}}{K_{NOASRB}+S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHASRB}+S_{NH}}\right) * X_{ASRB}$
Growth of X_{HMB}	ρ14	$\mu_{HMB} * \left(\frac{S_{H2}}{K_{H2HMB}+S_{H2}}\right) * \left(\frac{K_{IHMB}}{K_{IHMB}+S_{H2S*}}\right) * \left(\frac{K_{OHMB}}{K_{OHMB}+S_O}\right) * \left(\frac{K_{NOHMB}}{K_{NOHMB}+S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHHMB}+S_{NH}}\right) * X_{HMB}$

Table 7.3. (contd). Process rates

	Rate	Expression
Growth of X_{HSRB}	ρ15	$\mu_{HSRB} * \left(\frac{S_{H2}}{K_{H2HSRB} + S_{H2}}\right) * \left(\frac{S_{SO4}}{K_{SOHSRB} + S_{SO4}}\right) * \left(\frac{K_{IHSRB}}{K_{IHSRB} + S_{H2S*}}\right) * \left(\frac{K_{OHSRB}}{K_{OHSRB} + S_O}\right) * \left(\frac{K_{NOHSRB}}{K_{NOHSRB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHHSRB} + S_{NH}}\right) * X_{HSRB}$
Aerobic growth of X_{THIO}	ρ16	$\mu_{THIO} * \left(\frac{S_{H2S}}{K_{STHIO} + S_{H2S}}\right) * \left(\frac{S_O}{K_{OTHIO} + S_O}\right) * \left(\frac{S_{NH}}{K_{NHTHIO} + S_{NH}}\right) * X_{THIO}$
Anoxic growth of X_{THIO}	ρ17	$\mu_{THIO} * \eta_{THIO} * \left(\frac{S_{H2S}}{K_{STHIO} + S_{H2S}}\right) * \left(\frac{S_{NO}}{K_{NOTHIO} + S_{NO}}\right) * \left(\frac{K_{OTHIO}}{K_{OTHIO} + S_O}\right) * \left(\frac{S_{NH}}{K_{NHTHIO} + S_{NH}}\right) * X_{THIO}$
Decay of X_{FB}	ρ18	$b_{FB} * X_{FB}$
Decay of X_{AMB}	ρ19	$b_{AMB} * X_{AMB}$
Decay of X_{ASRB}	ρ20	$b_{ASRB} * X_{ASRB}$
Decay of X_{HMB}	ρ21	$b_{HMB} * X_{HMB}$
Decay of X_{HSRB}	ρ22	$b_{HSRB} * X_{HSRB}$
Decay of X_{THIO}	ρ23	$b_{THIO} * X_{THIO}$
Plant growth on ammonium	ρ24	$\left(\frac{1}{d_w * \varepsilon}\right) * k_{pl} * \left(\frac{S_{NH}}{K_{PNH} + S_{NH}}\right) * X_{Pl}$
Plant growth on nitrate	ρ25	$\left(\frac{1}{d_w * \varepsilon}\right) * k_{pl} * \left(\frac{S_{NO}}{K_{PNO} + S_{NO}}\right) * \left(\frac{K_{PNH}}{K_{PNH} + S_{NH}}\right) * X_{Pl}$
Plant oxygen leaching	ρ26	$[1/(d_w * \varepsilon)] * k_{ROL} * (\exp(S_{osat} - S_O) - 1)$
Plant decay	ρ27	$[1/(d_w * \varepsilon)] * b_P * X_{Pl}$
Plant physical degradation	ρ28	$[1/(d_w * \varepsilon)] * k_{degradation} * X_{Pd}$
Ammonia adsorption/desorption	ρ29	$\alpha * \left[S_{NH} - \left(\frac{X_{NH}}{PC}\right)^{1/m}\right]$
Physical reaeration	ρ30	$k_La * (S_{OSAT} - S_O)$
Hydrogen gas volatilisation	ρ31	$k_Lv * 16 * S_{H2}$
H_2S volatilisation	ρ32	$k_{Lv} * S_{H2S}$

7.5.1. Dissolved oxygen (S_O)

$$\frac{dMS_O}{dt} = (Q_{in} * S_{O\_in}) - (Q_{out} * S_O) + (pr\_1\_4 + pr\_1\_5 + pr\_1\_8 + pr\_1\_16 + pr\_1\_26 + pr\_1\_30) * V_w$$

$$pr\_1\_4 = -\left(\frac{1-Y_H}{Y_H}\right) * \mu_H * \left(\frac{S_F}{K_{SF}+S_F}\right) * \left(\frac{S_F}{S_F+S_A}\right) * \left(\frac{S_o}{K_{OH}+S_o}\right) * \left(\frac{S_{NH}}{K_{NHH}+S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH}+S_{H2S*}}\right) * X_{BH}$$

$$pr\_1\_5 = -\left(\frac{1-Y_H}{Y_H}\right) * \mu_H * \left(\frac{S_A}{K_{SA}+S_A}\right) * \left(\frac{S_A}{S_F+S_A}\right) * \left(\frac{S_o}{K_{OH}+S_o}\right) * \left(\frac{S_{NH}}{K_{NHH}+S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH}+S_{H2S*}}\right) * X_{BH}$$

$$pr\_1\_8 = -\left(\frac{4.57-Y_A}{Y_A}\right) * \mu_A * \left(\frac{S_{NH}}{K_{NHA}+S_{NH}}\right) * \left(\frac{S_O}{K_{OA}+S_o}\right) * \left(\frac{K_{IA}}{K_{IA}+S_{H2S*}}\right) * X_{BA}$$

$$pr\_1\_16 = -\left(\frac{2-Y_{THIO}}{Y_{THIO}}\right) * \mu_{THIO} * \left(\frac{S_{H2S}}{K_{STHIO}+S_{H2S}}\right) * \left(\frac{S_O}{K_{OTHIO}+S_O}\right) * \left(\frac{S_{NH}}{K_{NHTHIO}+S_{NH}}\right) * X_{THIO}$$

$$pr\_1\_26 = \frac{k_{ROL}}{d_w * \varepsilon} * (\exp(S_{o\,sat} - S_O) - 1)$$

$$pr\_1\_30 = k_L a * (S_{O\_sat} - S_O)$$

Oxygen losses occur firstly through aerobic growth of heterotrophic microorganisms. Two substrates can be used, i.e. fermentable soluble COD S_F (pr_1_4) or acetate S_A (pr_1_5). The growth rates are governed by substrate availability as well as oxygen and ammonium concentrations whilst they are inhibited by undissociated H_2S (S_{H2S*}). The ammonium term was added to prevent negative concentrations as a result of excessive nitrogen immobilisation (refer also to 7.5.5). No references for H_2S inhibition on heterotrophs were found as H_2S usually is not present in the aerobic environment where heterotrophs thrive. However, in the rootzone of wetlands, oxic, anoxic and anaerobic environments are situated close to each other and H_2S might therefore affect bacteria in aerobic wetland environments. H_2S inhibition was furthermore implemented in this model for consistency with microbial reactions described later in this chapter.

Growth of autotrophic nitrifying microorganisms is a second source of oxygen consumption (pr_1_8). Ammonium and oxygen concentrations influence growth rates whereas undissociated H_2S strongly limits growth, as was a.o. proven by Æsøy *et al.*

(1998). The reader should be aware that, for reasons of simplicity, nitrification is modelled as a one-step reaction, thus ignoring the intermediate formation of NO_2.

A similar process structure was used for aerobically growing sulphur oxidising bacteria, with *Thiobacillus denitrificans* as a typical representative (pr_1_16). Indeed, Okabe *et al.* (1999) state that *T. denitrificans* preferentially utilises oxygen over nitrate as electron acceptor in the presence of both compounds.

Besides an oxygen influx with the influent (Q_{in} * S_{o_in}), oxygen is also introduced into the system by plant root oxygen loss (pr_1_26) and atmospheric diffusion (pr_1_30). Root oxygen loss seems highly influenced by redox conditions (Stottmeister *et al.*, 2003) and oxygen demand (Wu *et al.*, 2001). It was therefore approximated as an exponential process driven by the oxygen deficit. Stein *et al.* (2003) also proved that this process was dependent on plant species, plant biomass, season etc. Since there is however little quantitative information available to underpin this hypothesis, the following equation for k_{ROL} was arbitrarily chosen to reflect these findings:

$$k_{ROL} = k_{ROLmin} + [(k_{ROLmax} - k_{ROLmin}) * (MX_{Pl} / MaxPlantBiomass)];$$

During winter, when there is no living plant biomass X_{Pl}, k_{ROL} is set to a minimum value of k_{ROLmin}. Indeed, Brix (1994a) states that there is also passive transport of air through the dead culms. During summer, the k_{ROL} is set to a maximum value of k_{ROLmax}, coinciding with the period during which plant biomass is at its maximum. During both spring and fall, there is a linear increase respectively decrease between k_{ROLmin} and k_{ROLmax}, depending on the living plant biomass.

No study seems to exist on oxygen transport from air to water moving in a gravel bed. Nevertheless, diffusion and mass transfer in the air space are several orders of magnitude larger than in water, so basically the underground water surface contacts with air containing approximately 21% oxygen (Kadlec and Knight, 1996d). As a result, physical reaeration is also approximated as a first-order process driven by the oxygen deficit as it is usually done for running waters (i.e. QUAL2 and MIKE11, Rauch *et al.*, 1998; RWQM1, Shanahan *et al.*, 2001).

For the k_La, the work of Gualtieri and Gualtieri (1999) was consulted and its value was made dependent of flow velocity (v) and depth (d) in the following way:

$$k_L a = C_{aer} * \frac{v^{a_r}}{d^b}$$

Values for C_{aer}, a_r and b are only given for rivers and are, due to the different turbulence regimes between rivers and HSSF CW, not simply transferable.

7.5.2. Inert soluble COD (S_I)

$\frac{dMS_I}{dt} = (Q_{in} * S_{I\_in}) - (Q_{out} * S_I) +$ $pr\_2\_0 * V_w$
$pr\_2\_0 = f_{C\_SI} * k_{decomp} * X_C$

Inert soluble COD flows unaltered through the reed bed. A fraction of very slowly biodegradable organics is converted to S_I during the decomposition process.

7.5.3. Fermentable, readily biodegradable soluble COD (S_F)

$$\frac{dMS_F}{dt} = (Q_{in} * S_{F\_in}) - (Q_{out} * S_F) + (pr\_3\_1 + pr\_3\_4 + pr\_3\_6 + pr\_3\_11) * V_w$$

$$pr\_3\_1 = k_h * \left[\frac{X_S/(X_{BH} + X_{FB})}{K_X + (X_S/(X_{BH} + X_{FB}))}\right] * (X_{BH} + \eta_h * X_{FB})$$

$$pr\_3\_4 = \frac{-1}{Y_H} * \mu_H * \left(\frac{S_F}{K_{SF} + S_F}\right) * \left(\frac{S_F}{S_F + S_A}\right) * \left(\frac{S_O}{K_{OH} + S_o}\right) * \left(\frac{S_{NH}}{K_{NHH} + S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH} + S_{H2S*}}\right) * X_{BH}$$

$$pr\_3\_6 = \frac{-1}{Y_H} * n_g * \mu_H * \left(\frac{S_F}{K_{SF} + S_F}\right) * \left(\frac{S_F}{S_F + S_A}\right) * \left(\frac{K_{OH}}{K_{OH} + S_o}\right) * \left(\frac{S_{NO}}{K_{NOH} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHH} + S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH} + S_{H2S*}}\right) * X_{BH}$$

$$pr\_3\_11 = \frac{-1}{Y_{FB}} * \mu_{FB} * \left(\frac{S_F}{K_{SFB} + S_F}\right) * \left(\frac{K_{IFB}}{K_{IFB} + S_{H2S*}}\right) * \left(\frac{K_{OFB}}{K_{OFB} + S_O}\right) * \left(\frac{K_{NOFB}}{K_{NOFB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHFB} + S_{NH}}\right) * X_{FB}$$

Concentrations of fermentable, readily biodegradable soluble COD S_F in the pore water increase by influx and by hydrolysis of slowly biodegradable particulate COD X_S (pr_3_1). It is assumed that hydrolysis is carried out by both heterotrophic and fermenting bacteria, the latter ones at a lower rate as can be seen from the η_h correction factor. For heterotrophs, the rate difference between aerobic and anoxic hydrolysis has been ignored, as is recommended in ASM3. By using the simplified equation pr_3_1, it is also assumed that both bacterial groups have the same K_X value.

Consumption of S_F occurs firstly by aerobic (pr_3_4) and anoxic (pr_3_6) growth of heterotrophs (the latter process is commonly known as denitrification) whereby S_F is converted to CO_2 and new cells, the ratio of which is given by the yield. Heterotrophic growth rates are dependent on substrate and ammonium availability as well as on electron acceptor concentrations (either oxygen or nitrate). Although no data were found in literature, sulphide inhibition was again added for consistency.

Growth of fermenting bacteria under anaerobic conditions fosters further removal of S_F. During their growth, S_F is converted to acetate S_A (pr_3_11). Formation of intermediate products such as butyric and propionic acid is recognised, but ignored in the present model for reasons of simplicity (refer also to 7.5.4). Their growth rate is dependent on substrate and ammonium availability and inhibited by oxygen, nitrate and sulphide.

7.5.4. Fermentation products as acetate (S_A)

$$\frac{dMS_A}{dt} = (Q_{in} * S_{A\_in}) - (Q_{out} * S_A) + (pr\_4\_5 + pr\_4\_7 + pr\_4\_11 + pr\_4\_12 + pr\_4\_13) * V_w$$

$$pr\_4\_5 = -\frac{1}{Y_H} * \mu_H * \left(\frac{S_A}{K_{SA} + S_A}\right) * \left(\frac{S_A}{S_F + S_A}\right) * \left(\frac{S_O}{K_{OH} + S_o}\right) * \left(\frac{S_{NH}}{K_{NHH} + S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH} + S_{H2S*}}\right) * X_{BH}$$

$$pr\_4\_7 = \frac{-1}{Y_H} * n_g * \mu_H * \left(\frac{S_A}{K_{SA} + S_A}\right) * \left(\frac{S_A}{S_F + S_A}\right) * \left(\frac{K_{OH}}{K_{OH} + S_o}\right) * \left(\frac{S_{NO}}{K_{NOH} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHH} + S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH} + S_{H2S*}}\right) * X_{BH}$$

$$pr\_4\_11 = \frac{1 - Y_{FB}}{1.23 Y_{FB}} * \mu_{FB} * \left(\frac{S_F}{K_{SFB} + S_F}\right) * \left(\frac{K_{IFB}}{K_{IFB} + S_{H2S*}}\right) * \left(\frac{K_{OFB}}{K_{OFB} + S_O}\right) * \left(\frac{K_{NOFB}}{K_{NOFB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHFB} + S_{NH}}\right) * X_{FB}$$

$$pr\_4\_12 = \frac{-1}{Y_{AMB}} * \mu_{AMB} * \left(\frac{S_A}{K_{SAMB} + S_A}\right) * \left(\frac{K_{IAMB}}{K_{IAMB} + S_{H2S*}}\right) * \left(\frac{K_{OAMB}}{K_{OAMB} + S_O}\right) * \left(\frac{K_{NOAMB}}{K_{NOAMB} + S_{NO}}\right)$$
$$* \left(\frac{S_{NH}}{K_{NHAMB} + S_{NH}}\right) * X_{AMB}$$

$$pr\_4\_13 = \frac{-1}{Y_{ASRB}} * \mu_{ASRB} * \left(\frac{S_A}{K_{SASRB} + S_A}\right) * \left(\frac{S_{SO4}}{K_{SOASRB} + S_{SO}}\right) * \left(\frac{K_{IASRB}}{K_{IASRB} + S_{H2S*}}\right) * \left(\frac{K_{OASRB}}{K_{OASRB} + S_O}\right) * \left(\frac{K_{NOASRB}}{K_{NOASRB} + S_{NO}}\right)$$
$$* \left(\frac{S_{NH}}{K_{NHASRB} + S_{NH}}\right) * X_{ASRB}$$

Growth of fermenting bacteria under anaerobic conditions results in the production of acetate S_A (pr_4_11). Their growth rate is dependent on substrate and ammonium availability and is being inhibited by oxygen, nitrate and undissociated H_2S. Although other organic acids such as butyric and propionic acid may be formed as intermediate products, their existence is ignored in the present model as further conversion to acetate occurs quickly. This was confirmed by Huang *et al*. (2005) who found effluent concentrations of butyric and propionic acid in a HSSF CW in the order of 1 µg l^{-1} whereas acetic acid concentrations were in the order of 10 to 20 mg l^{-1}.

Removal of S_A occurs, as for S_F, by aerobic (pr_4_5) and anoxic (pr_4_7) growth of heterotrophic microorganisms. Heterotrophic growth rates are again dependent on substrate and ammonium availability as well as on electron acceptor concentrations. Despite a lack of research data, sulphide inhibition was again added for consistency.

Further consumption of acetate occurs by anaerobically growing acetotrophic microorganisms. A first group uses organic material as electron acceptor, thereby producing carbon dioxide gas and methane (pr_4_12) whilst a second group uses sulphate as electron acceptor (pr_4_13). Both groups are inhibited by higher concentrations of oxygen, nitrate and undissociated H_2S.

7.5.5. Ammonium (S_{NH})

$$\frac{dMS_{NH}}{dt} = (Q_{in} * S_{NH\_in}) - (Q_{out} * S_{NH}) + (pr\_5\_3 + pr\_5\_4 + pr\_5\_5 + pr\_5\_6 + pr\_5\_7 + pr\_5\_8 + pr\_5\_11 + pr\_5\_12 + pr\_5\_13 + pr\_5\_14 + pr\_5\_15 + pr\_5\_16 + pr\_5\_17 + pr\_5\_24 + pr\_5\_29) * V_w$$

$$pr\_5\_3 = k_a * S_{ND} * (X_{BH} + \eta_h * X_{FB})$$

$$pr\_5\_4 = -i_{XB} * \mu_H * \left(\frac{S_F}{K_{SF} + S_F}\right) * \left(\frac{S_F}{S_F + S_A}\right) * \left(\frac{S_O}{K_{OH} + S_o}\right) * \left(\frac{S_{NH}}{K_{NHH} + S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH} + S_{H2S*}}\right) * X_{BH}$$

$$pr\_5\_5 = -i_{XB} * \mu_H * \left(\frac{S_A}{K_{SA} + S_A}\right) * \left(\frac{S_A}{S_F + S_A}\right) * \left(\frac{S_O}{K_{OH} + S_o}\right) * \left(\frac{S_{NH}}{K_{NHH} + S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH} + S_{H2S*}}\right) * X_{BH}$$

$$pr\_5\_6 = -i_{XB} * n_g * \mu_H * \left(\frac{S_F}{K_{SF} + S_F}\right) * \left(\frac{S_F}{S_F + S_A}\right) * \left(\frac{K_{OH}}{K_{OH} + S_o}\right) * \left(\frac{S_{NO}}{K_{NOH} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHH} + S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH} + S_{H2S*}}\right) * X_{BH}$$

$$pr\_5\_7 = -i_{XB} * n_g * \mu_H * \left(\frac{S_A}{K_{SA} + S_A}\right) * \left(\frac{S_A}{S_F + S_A}\right) * \left(\frac{K_{OH}}{K_{OH} + S_o}\right) * \left(\frac{S_{NO}}{K_{NOH} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHH} + S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH} + S_{H2S*}}\right) * X_{BH}$$

$$pr\_5\_8 = \left(-i_{XB} - \frac{1}{Y_A}\right) * \mu_A * \left(\frac{S_{NH}}{K_{NHA} + S_{NH}}\right) * \left(\frac{S_O}{K_{OA} + S_o}\right) * \left(\frac{K_{IA}}{K_{IA} + S_{H2S*}}\right) * X_{BA}$$

$$pr\_5\_11 = -i_{XB} * \mu_{FB} * \left(\frac{S_F}{K_{SFB} + S_F}\right) * \left(\frac{K_{IFB}}{K_{IFB} + S_{H2S*}}\right) * \left(\frac{K_{OFB}}{K_{OFB} + S_O}\right) * \left(\frac{K_{NOFB}}{K_{NOFB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHFB} + S_{NH}}\right) * X_{FB}$$

$$pr\_5\_12 = -i_{XB} * \mu_{AMB} * \left(\frac{S_A}{K_{SAMB} + S_A}\right) * \left(\frac{K_{IAMB}}{K_{IAMB} + S_{H2S*}}\right) * \left(\frac{K_{OAMB}}{K_{OAMB} + S_O}\right) * \left(\frac{K_{NOAMB}}{K_{NOAMB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHAMB} + S_{NH}}\right) * X_{AMB}$$

$$pr\_5\_13 = -i_{XB} * \mu_{ASRB} * \left(\frac{S_A}{K_{SASRB} + S_A}\right) * \left(\frac{S_{SO4}}{K_{SOASRB} + S_{SO}}\right) * \left(\frac{K_{IASRB}}{K_{IASRB} + S_{H2S*}}\right) * \left(\frac{K_{OASRB}}{K_{OASRB} + S_O}\right) * \left(\frac{K_{NOASRB}}{K_{NOASRB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHASRB} + S_{NH}}\right) * X_{ASRB}$$

$$pr\_5\_14 = -i_{XB} * \mu_{HMB} * \left(\frac{S_{H2}}{K_{H2HMB} + S_{H2}}\right) * \left(\frac{K_{IHMB}}{K_{IHMB} + S_{H2S*}}\right) * \left(\frac{K_{OHMB}}{K_{OHMB} + S_O}\right) * \left(\frac{K_{NOHMB}}{K_{NOHMB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHHMB} + S_{NH}}\right) * X_{HMB}$$

$$pr\_5\_15 = -i_{XB} * \mu_{HSRB} * \left(\frac{S_{H2}}{K_{H2HSRB} + S_{H2}}\right) * \left(\frac{S_{SO}}{K_{SOHSRB} + S_{SO}}\right) * \left(\frac{K_{IHSRB}}{K_{IHSRB} + S_{H2S*}}\right) * \left(\frac{K_{OHSRB}}{K_{OHSRB} + S_O}\right) * \left(\frac{K_{NOHSRB}}{K_{NOHSRB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHHSRB} + S_{NH}}\right) * X_{HSRB}$$

$$pr\_5\_16 = -i_{XB} * \mu_{THIO} * \left(\frac{S_{H2S}}{K_{STHIO} + S_{H2S}}\right) * \left(\frac{S_O}{K_{OTHIO} + S_O}\right) * \left(\frac{S_{NH}}{K_{NHTHIO} + S_{NH}}\right) * X_{THIO}$$

$$pr\_5\_17 = -i_{XB} * \mu_{THIO} * \eta_{THIO} * \left(\frac{S_{H2S}}{K_{STHIO} + S_{H2S}}\right) * \left(\frac{S_{NO}}{K_{NOTHIO} + S_{NO}}\right) * \left(\frac{K_{OTHIO}}{K_{OTHIO} + S_{O}}\right) * \left(\frac{S_{NH}}{K_{NHTHIO} + S_{NH}}\right) * X_{THIO}$$

$$pr\_5\_24 = (-1) * i_{XBPlant} * \left(\frac{1}{d_w * Porosity}\right) * k_{pl} * \left(\frac{S_{NH}}{K_{PNH} + S_{NH}}\right) \text{ with } k_{pl} = \text{f(season)}$$

$$pr\_5\_29 = (-1) * \alpha * \left[S_{NH} - \left(\frac{X_{NH}}{PC}\right)^{1/m} \right]$$

Ammonium is immobilised into microbial cells as part (i_{XB}) of the cell material (pr_5_4, pr_5_5, pr_5_6, pr_5_7, pr_5_8, pr_5_11, pr_5_12, pr_5_13, pr_5_14, pr_5_15, pr_5_16 and pr_5_17). In order to avoid negative ammonium concentrations due to excessive immobilisation, a Monod term was added to each microbial growth equation such that growth is limited at low ammonium concentrations. Applying the same i_{XB} value to all microbial growth reactions implicitly assumes that all bacterial cells have an equal N to COD ratio. In fact, Henze *et al*. (2000) and Kalyuzhnyi and Fedorovich (1998) use a slightly different biomass composition ($C_5H_7O_2N$ versus $C_5H_9O_3N$ respectively) but the resulting N to COD ratio hardly deviates. This different biomass composition has no effect on the other processes, as everything is converted to the same unit, i.e. COD and not dry matter.

Plant uptake also removes some ammonium from the wastewater (pr_5_24). Further losses occur through nitrification (pr_5_8) and through reversible sorption of ammonium onto the gravel (pr_5_29). The latter process was taken from the work of McBride and Tanner (2000) and is based on the reversible Freundlich sorption isotherm equation.

Concentrations of ammonium increase due to the influx and due to ammonification of soluble organic nitrogen S_{ND} (pr_5_3) which is assumed to occur by the activity of the same bacterial groups that are involved in the hydrolysis process.

7.5.6. Soluble organic nitrogen (S_{ND})

$$\frac{dMS_{ND}}{dt} = (Q_{in} * S_{ND\_in}) - (Q_{out} * S_{ND}) + (pr\_6\_2 + pr\_6\_3) * V_w$$

$$pr\_6\_2 = k_h * \left[\frac{X_S/(X_{BH}+X_{FB})}{K_X + (X_S/(X_{BH}+X_{FB}))}\right] * \left(\frac{X_{ND}}{X_S}\right) * (X_{BH} + \eta_h * X_{FB})$$

$$pr\_6\_3 = (-1) * k_a * S_{ND} * (X_{BH} + \eta_h * X_{FB})$$

Concentrations of soluble organic nitrogen S_{ND} increase by hydrolysis of particulate organic nitrogen (pr_6_2). The process rate is similar to the one of organic matter hydrolysis but it is in addition governed by the particulate N to COD ratio. It is thus assumed that both heterotrophs and fermenting bacteria drive the hydrolysis process, at unequal rates, as can be seen from the reduction coefficient η_h. S_{ND} concentrations drop by the previously described ammonification process (pr_6_3). Note that in the later Activated Sludge Model No. 3 (Henze *et al.*, 2000), ammonification is no longer considered as a separate process since it occurs at high rates. It has there been incorporated into the organic nitrogen hydrolysis process.

7.5.7. Nitrate (S_{NO})

$$\frac{dMS_{NO}}{dt} = (Q_{in} * S_{NO\_in}) - (Q_{out} * S_{NO}) + (pr\_7\_6 + pr\_7\_7 + pr\_7\_8 + pr\_7\_17 + pr\_7\_25) * V_w$$

$$pr\_7\_6 = -\frac{1-Y_H}{2.86Y_H} * n_g * \mu_H * \left(\frac{S_F}{K_{SF}+S_F}\right) * \left(\frac{S_F}{S_F+S_A}\right) * \left(\frac{K_{OH}}{K_{OH}+S_o}\right) * \left(\frac{S_{NO}}{K_{NOH}+S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHH}+S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH}+S_{H2S*}}\right) * X_{BH}$$

$$pr\_7\_7 = -\frac{1-Y_H}{2.86Y_H} * n_g * \mu_H * \left(\frac{S_A}{K_{SA}+S_A}\right) * \left(\frac{S_A}{S_F+S_A}\right) * \left(\frac{K_{OH}}{K_{OH}+S_o}\right) * \left(\frac{S_{NO}}{K_{NOH}+S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHH}+S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH}+S_{H2S*}}\right) * X_{BH}$$

$$pr\_7\_8 = \left(\frac{1}{Y_A}\right) * \mu_A * \left(\frac{S_{NH}}{K_{NHA}+S_{NH}}\right) * \left(\frac{S_O}{K_{OA}+S_o}\right) * \left(\frac{K_{IA}}{K_{IA}+S_{H2S*}}\right) * X_{BA}$$

$$pr\_7\_17 = -\frac{1-Y_{THIO}}{1.59Y_{THIO}} * \mu_{THIO} * \eta_{THIO} * \left(\frac{S_{H2S}}{K_{STHIO}+S_{H2S}}\right) * \left(\frac{S_{NO}}{K_{NOTHIO}+S_{NO}}\right) * \left(\frac{K_{OTHIO}}{K_{OTHIO}+S_{O}}\right) * \left(\frac{S_{NH}}{K_{NHTHIO}+S_{NH}}\right) * X_{THIO}$$

$$pr\_7\_25 = (-1) * i_{XBPlant} * \left(\frac{1}{d_w * Porosity}\right) * k_{pl} * \left(\frac{S_{NO}}{K_{PNO}+S_{NO}}\right) * \left(\frac{K_{PNH}}{K_{PNH}+S_{NH}}\right) \text{ with } k_{pl} = f(\text{season})$$

Nitrate losses occur microbially through heterotrophic denitrification by anoxically growing heterotrophs, thereby consuming either S_F (pr_7_6) or S_A (pr_7_7). Autotrophic denitrification by sulphur oxidising microorganisms (pr_7_17) is a second microbial process that consumes nitrate. Another N-removal process is plant uptake (pr_7_25). Aquatic plants supposedly prefer to take up ammonium instead of nitrate (Wynn and Liehr, 2001). So, when ammonium concentrations are high, nitrate uptake is low due to the addition of the nitrate switching function. Conversely, when ammonium concentrations are low, nitrate will be consumed when available.

Finally, nitrate additions occur through nitrification by aerobically growing autotrophic microorganisms (pr_7_8).

7.5.8. Sulphate (S_{SO4})

$$\frac{dMS_{SO4}}{dt} = (Q_{in} * S_{SO4\_in}) - (Q_{out} * S_{SO4}) + (pr\_8\_13 + pr\_8\_15 + pr\_8\_16 + pr\_8\_17) * V_w$$

$$pr\_8\_13 = -\frac{1-Y_{ASRB}}{0.55Y_{ASRB}} * \mu_{ASRB} * \left(\frac{S_{A}}{K_{SASRB}+S_{A}}\right) * \left(\frac{S_{SO4}}{K_{SOASRB}+S_{SO4}}\right) * \left(\frac{K_{IASRB}}{K_{IASRB}+S_{H2S*}}\right) * \left(\frac{K_{OASRB}}{K_{OASRB}+S_{O}}\right) * \left(\frac{K_{NOASRB}}{K_{NOASRB}+S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHASRB}+S_{NH}}\right) * X_{ASRB}$$

$$pr\_8\_15 = -\frac{1-Y_{HSRB}}{0.55Y_{HSRB}} * \mu_{HSRB} * \left(\frac{S_{H2}}{K_{H2HSRB}+S_{H2}}\right) * \left(\frac{S_{SO}}{K_{SOHSRB}+S_{SO}}\right) * \left(\frac{K_{IHSRB}}{K_{IHSRB}+S_{H2S*}}\right) * \left(\frac{K_{OHSRB}}{K_{OHSRB}+S_{O}}\right) * \left(\frac{K_{NOHSRB}}{K_{NOHSRB}+S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHHSRB}+S_{NH}}\right) * X_{HSRB}$$

$$pr\_8\_16 = \frac{1}{Y_{THIO}} * \mu_{THIO} * \left(\frac{S_{H2S}}{K_{STHIO}+S_{H2S}}\right) * \left(\frac{S_{O}}{K_{OTHIO}+S_{O}}\right) * \left(\frac{S_{NH}}{K_{NHTHIO}+S_{NH}}\right) * X_{THIO}$$

$$pr\_8\_17 = \frac{1}{Y_{THIO}} * \mu_{THIO} * \eta_{THIO} * \left(\frac{S_{H2S}}{K_{STHIO}+S_{H2S}}\right) * \left(\frac{S_{NO}}{K_{NOTHIO}+S_{NO}}\right) * \left(\frac{K_{OTHIO}}{K_{OTHIO}+S_{O}}\right) * \left(\frac{S_{NH}}{K_{NHTHIO}+S_{NH}}\right) * X_{THIO}$$

Sulphate reducing bacteria use sulphate as an electron acceptor for oxidation of acetate (pr_8_13) and for oxidation of hydrogen (pr_8_15). Their growth is governed by substrate availability (either S_A of S_{H2}) and as usual by ammonium concentrations. Sulphide inhibition was implemented by means of an ASM-like switching term rather than the function proposed by Kalyuzhnyi and Fedorovich (1998). Indeed, the latter model uses for example the following inhibition term:

$$\mu_{AMB} = \mu_{\max,AMB} * \frac{S_A}{K_{SA} + S_A} * \left(1 - \frac{S_{H2S*}}{K_I}\right)$$

It can be easily derived that in case of H_2S^* concentrations exceeding the K_I, growth becomes negative, which would actually lead to production of acetate and therefore to a physically meaningless model. Secondly, using the ASM-like approach renders the model code more consistent. The disadvantage is that the K_I values cannot simply be copied from Kalyuzhnyi and Fedorovich (1998), as illustrated in Fig. 7.2. For lower H_2S^* concentrations, one can see that a K_I(Monod) = 0.5 * K_I(K&F) yields a reasonable approximation. For normal domestic wastewater containing less than 200 mg SO_4^{2-} l^{-1}, inhibition at neutral pH should anyway be less than 1%.

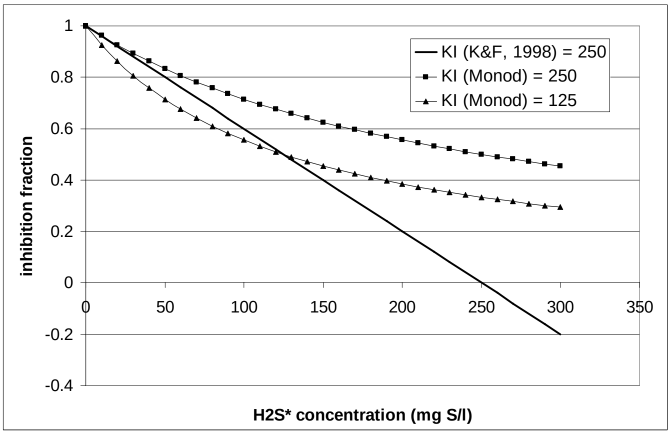

Fig. 7.2. Comparison of the Kalyuzhnyi and Fedorovich (1998) versus the ASM-Monod inhibition terms.

The opposite pathway, oxidation of H_2S to sulphate, was also included in the model for completeness. Sulphur oxidising microorganisms are capable of either using oxygen (pr_8_16) or nitrate (pr_8_17) as electron acceptors for sulphide oxidation.

7.5.9. Sulphide (S_{H2S})

$$\frac{dMS_{H2S}}{dt} = (Q_{in} * S_{H2S\_in}) - (Q_{out} * S_{H2S}) + (pr\_9\_13 + pr\_9\_15 + pr\_9\_16 + pr\_9\_17 + pr\_9\_32) * V_w$$

$$pr\_9\_13 = -\frac{1-Y_{ASRB}}{1.65Y_{ASRB}} * \mu_{ASRB} * \left(\frac{S_A}{K_{SASRB} + S_A}\right) * \left(\frac{S_{SO4}}{K_{SOASRB} + S_{SO4}}\right) * \left(\frac{K_{IASRB}}{K_{IASRB} + S_{H2S*}}\right) * \left(\frac{K_{OASRB}}{K_{OASRB} + S_O}\right) * \left(\frac{K_{NOASRB}}{K_{NOASRB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHASRB} + S_{NH}}\right) * X_{ASRB}$$

$$pr\_9\_15 = -\frac{1-Y_{HSRB}}{1.65Y_{HSRB}} * \mu_{HSRB} * \left(\frac{S_{H2}}{K_{H2HSRB} + S_{H2}}\right) * \left(\frac{S_{SO}}{K_{SOHSRB} + S_{SO}}\right) * \left(\frac{K_{IHSRB}}{K_{IHSRB} + S_{H2S*}}\right) * \left(\frac{K_{OHSRB}}{K_{OHSRB} + S_O}\right) * \left(\frac{K_{NOHSRB}}{K_{NOHSRB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHHSRB} + S_{NH}}\right) * X_{HSRB}$$

$$pr\_9\_16 = -\frac{1}{Y_{THIO}} * \mu_{THIO} * \left(\frac{S_{H2S}}{K_{STHIO} + S_{H2S}}\right) * \left(\frac{S_O}{K_{OTHIO} + S_O}\right) * \left(\frac{S_{NH}}{K_{NHTHIO} + S_{NH}}\right) * X_{THIO}$$

$$pr\_9\_17 = -\frac{1}{Y_{THIO}} * \mu_{THIO} * \eta_{THIO} * \left(\frac{S_{H2S}}{K_{STHIO} + S_{H2S}}\right) * \left(\frac{S_{NO}}{K_{NOTHIO} + S_{NO}}\right) * \left(\frac{K_{OTHIO}}{K_{OTHIO} + S_O}\right) * \left(\frac{S_{NH}}{K_{NHTHIO} + S_{NH}}\right) * X_{THIO}$$

$$pr\_9\_32 = (-1) * k_L v * S_{H2S}$$

The first four processes actually represent the same pathways as described for the sulphate reduction, but now from the point of view of product formation instead of substrate consumption. Indeed, H_2S is produced by reduction of sulphate (pr_9_13 and pr_9_15) whereas H_2S is converted to sulphate by sulphur oxidisers such as *Thiobacillus* (pr_9_16 and pr_9_17).

Dihydrogen sulphide volatilisation has also been included in the model (pr_9_32). Since H_2S concentrations in the air are very low, the driving force has been represented by the H_2S concentration.

For the majority of anaerobic bacteria, sulphide is a strong toxicant in its undissociated form which can permetate the cell membrane (Kalyuzhnyi *et al*., 1998). For this reason, sulphide inhibition is taken into account for the growth rates in the form of ASM-like switching terms. The reader is reminded that S_{H2S} represents the sum of H_2S, HS^- and S^{2-} whilst S_{H2S*} exclusively represents the undissociated form.

Although the dissociation is pH-dependent, adding an ion balance to the model for pH calculations would greatly increase its complexity and therefore the calculation efforts, although Zaher (2005) proved it to be feasible. As pH fluctuations in HSSF CW are usually small, the pH has been set as an invariable parameter for the time being, with an adjustable value between 7 and 7.5.

$$S_{H2S*} = \frac{S_{H2S}}{MM_S * \left(1 + \frac{K_{a1}}{[H^+]} + \frac{K_{a1} * K_{a2}}{[H^+]^2}\right)}$$

with: K_{a1} (H_2S) = $1e^{-7}$

K_{a2} (H_2S) = $1e^{-14}$

MM_S = 32 g mol^{-1}

7.5.10. Hydrogen gas (S_{H2})

$$\frac{dMS_{H2}}{dt} = (Q_{in} * S_{H2\_in}) - (Q_{out} * S_{H2}) + (pr\_10\_11 + pr\_10\_14 + pr\_10\_15 + pr\_10\_31) * V_w$$

$$pr\_10\_11 = \frac{1-Y_{FB}}{2.46Y_{FB}} * \mu_{FB} * \left(\frac{S_F}{K_{SFB} + S_F}\right) * \left(\frac{K_{IFB}}{K_{IFB} + S_{H2S*}}\right) * \left(\frac{K_{OFB}}{K_{OFB} + S_O}\right) * \left(\frac{K_{NOFB}}{K_{NOFB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHFB} + S_{NH}}\right) * X_{FB}$$

$$pr\_10\_14 = -\left(\frac{1}{Y_{HMB}}\right) * \mu_{HMB} * \left(\frac{S_{H2}}{K_{H2HMB} + S_{H2}}\right) * \left(\frac{K_{IHMB}}{K_{IHMB} + S_{H2S*}}\right) * \left(\frac{K_{OHMB}}{K_{OHMB} + S_O}\right) * \left(\frac{K_{NOHMB}}{K_{NOHMB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHHMB} + S_{NH}}\right) * X_{HMB}$$

$$pr\_10\_15 = -\frac{1}{Y_{HSRB}} * \mu_{HSRB} * \left(\frac{S_{H2}}{K_{H2HSRB} + S_{H2}}\right) * \left(\frac{S_{SO}}{K_{SOHSRB} + S_{SO}}\right) * \left(\frac{K_{IHSRB}}{K_{IHSRB} + S_{H2S*}}\right) * \left(\frac{K_{OHSRB}}{K_{OHSRB} + S_O}\right) * \left(\frac{K_{NOHSRB}}{K_{NOHSRB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHHSRB} + S_{NH}}\right) * X_{HSRB}$$

$$pr\_10\_31 = (-1) * 16 * k_L v * S_{H2}$$

The reader is reminder firstly that, for ease of use, hydrogen is considered as an electron donor similar to organic matter in the Kalyuzhnyi and Fedorovich (1998) model. It is therefore expressed as gCOD m^{-3} with a conversion factor of 16 gCOD (mol H_2)$^{-1}$. Of course all parameter values need to be adapted accordingly.

Hydrogen gas is produced by fermenting bacteria while converting fermentable soluble substrate S_F to acetate S_A (pr_10_11). H_2 is consumed by anaerobic hydrogenotrophic bacteria (pr_10_14 and pr_10_15). As they are all anaerobic bacteria, their growth is limited by elevated concentrations of oxygen and nitrate. Further losses occur through volatilisation (pr_10_31). As for H_2S, the driving force has been represented by the H_2 concentration since the partial pressure in the air is very small. The factor 16 is needed because H_2 is expressed as COD in this model.

7.5.11. Very slowly biodegradable particulate COD (X_C)

$$\frac{dMX_I}{dt} = (Q_{in} * X_{C\_in}) - wash\_X_C + (pr\_11\_0 + pr\_11\_9 + pr\_11\_10 + pr\_11\_18 + pr\_11\_19 + pr\_11\_20 + pr\_11\_21 + pr\_11\_22 + pr\_11\_23 + pr\_11\_28) * V_w$$

$pr\_11\_0 = (-1) * k_{decomp} * X_C$

$pr\_11\_9 = f_P * b_H * X_{BH}$

$pr\_11\_10 = f_P * b_A * X_{BA}$

$pr\_11\_18 = f_P * b_F * X_{FB}$

$pr\_11\_19 = f_P * b_F * X_{AMB}$

$pr\_11\_20 = f_P * b_F * X_{ASRB}$

$pr\_11\_21 = f_P * b_F * X_{HMB}$

$pr\_11\_22 = f_P * b_F * X_{HSRB}$

$pr\_11\_23 = f_P * b_F * X_{THIO}$

$$pr\_11\_28 = f_{plant} * \left(\frac{1}{d_w * \varepsilon}\right) * k_{degradation} * X_{Pd}$$

$wash\_X_C =$ IF (Q_{out} > threshold_flowrate)
THEN (fraction_washed_out * X_C * Q_{out})
ELSE 0

Very slowly biodegradable particulate COD obviously has an influx but, due to the assumption of 100% suspended solids removal, no efflux occurs unless the flow rate exceeds a certain threshold in which case a fraction of X_C is being washed out (wash_X_C). The latter fraction has been set proportionally to the flow rate. Settled X_C is decomposed at a very slow rate (pr_11_0).

When microorganisms die, the non-biodegradable parts of their cells (f_P) contribute to the pool of X_C (pr_11_9, pr_11_10, pr_11_18, pr_11_19, pr_11_20, pr_11_21, pr_11_22 and pr_11_23). A similar process also occurs when dead plants are physically degraded (pr_11_28): the non-biodegradable parts (f_{plant}) become X_C whereas the biodegradable parts become X_S. Using the same f_P for all bacteria implicitly assumes that they have a similar cell composition.

7.5.12. Slowly biodegradable particulate COD (X_S)

$$\frac{dMX_S}{dt} = (Q_{in} * X_{S\_in}) - wash\_X_S + (pr\_12\_0 + pr\_12\_1 + pr\_12\_9 + pr\_12\_10 + pr\_12\_18 + pr\_12\_19 + pr\_12\_20 + pr\_12\_21 + pr\_12\_22 + pr\_12\_23 + pr\_12\_28) * V_w$$

$$pr\_12\_0 = f_{C\_XS} * k_{decomp} * X_C$$

$$pr\_12\_1 = -k_h * \left[\frac{X_S/(X_{BH} + X_{FB})}{K_X + (X_S/(X_{BH} + X_{FB}))} \right] * (X_{BH} + \eta_h * X_{FB})$$

$$pr\_12\_9 = (1 - f_P) * b_H * X_{BH}$$

$$pr\_12\_10 = (1 - f_P) * b_A * X_{BA}$$

$$pr\_12\_18 = (1 - f_P) * b_F * X_{FB}$$

$$pr\_12\_19 = (1 - f_P) * b_F * X_{AMB}$$

$$pr\_12\_20 = (1 - f_P) * b_F * X_{ASRB}$$

$$pr\_12\_21 = (1 - f_P) * b_F * X_{HMB}$$

$$pr\_12\_22 = (1 - f_P) * b_F * X_{HSRB}$$

$$pr\_12\_23 = (1 - f_P) * b_F * X_{THIO}$$

$$pr\_12\_18 = (1 - f_{plant}) * \left(\frac{-1}{d_w * \varepsilon} \right) * k_{degradation} * X_{Pd}$$

$wash\_X_S =$ IF (Q_{out} > threshold_flowrate)
THEN (fraction_washed_out * X_S * Q_{out})
ELSE 0

Slowly biodegradable COD, as for all particulates, has an influx but no efflux unless a certain threshold flow rate is exceeded (wash_X_S)

When microorganisms die, the biodegradable parts of their cells ($1 - f_P$) are added to the amount of X_S in the wastewater (pr_12_9, pr_12_10, pr_12_18, pr_12_19, pr_12_20, pr_12_21, pr_12_22 and pr_12_23). It is again assumed that all microorganisms in this model have the same cell composition. A similar reaction takes place when plants are physically degraded (pr_12_28). Further additions occur through decomposition of very slowly biodegradable particulate COD (pr_12_0). X_S losses occur through hydrolysis (pr_12_1) by heterotrophic and fermenting bacteria.

7.5.13. Particulate organic nitrogen (X_{ND})

$$\frac{dMX_{ND}}{dt} = (Q_{in} * X_{ND\_in}) - wash\_X_{ND} + (pr\_13\_2 + pr\_13\_9 + pr\_13\_10 + pr\_13\_18 + pr\_13\_19 + pr\_13\_20 + pr\_13\_21 + pr\_13\_22 + pr\_13\_23 + pr\_13\_28) * V_w$$

$$pr\_13\_2 = -k_h * \left[\frac{X_S/(X_{BH} + X_{FB})}{K_X + (X_S/(X_{BH} + X_{FB}))}\right] * \left(\frac{X_{ND}}{X_S}\right) * (X_{BH} + \eta_h * X_{FB})$$

$pr\_13\_9 = (i_{XB} - (f_P * i_{XP})) * b_H * X_{BH}$

$pr\_13\_10 = (i_{XB} - (f_P * i_{XP})) * b_A * X_{BA}$

$pr\_13\_18 = (i_{XB} - (f_P * i_{XP})) * b_F * X_{FB}$

$pr\_13\_19 = (i_{XB} - (f_P * i_{XP})) * b_F * X_{AMB}$

$pr\_13\_20 = (i_{XB} - (f_P * i_{XP})) * b_F * X_{ASRB}$

$pr\_13\_21 = (i_{XB} - (f_P * i_{XP})) * b_F * X_{HMB}$

$pr\_13\_22 = (i_{XB} - (f_P * i_{XP})) * b_F * X_{HSRB}$

$pr\_13\_23 = (i_{XB} - (f_P * i_{XP})) * b_F * X_{THIO}$

$$pr\_12\_28 = (i_{XBPlant} - (f_{plant} * i_{XP})) * \left(\frac{-1}{d_w * \varepsilon}\right) * k_{degradation} * X_{Pd}$$

$wash\_X_{ND}$ = IF (Q_{out} > threshold_flowrate)
THEN (fraction_washed_out * X_{ND} * Q_{out})
ELSE 0

Particulate organic nitrogen has no efflux unless a peak flow rushes through the wetland and drags along some solids (wash_X_ND). When microorganisms die, organic nitrogen that is incorporated in the biodegradable part of their cells is released into the wastewater as X_{ND} (pr_13_9, pr_13_10, pr_13_18, pr_13_19, pr_13_20, pr_13_21, pr_13_22 and pr_13_23). All microbial groups in this model are supposed to have an equal cell nitrogen content. A similar reaction takes place when dead plants are physically degraded (pr_13_28).

X_{ND} concentrations decrease by hydrolysis into soluble organic nitrogen (pr_13_2), catalysed by heterotrophic and fermenting bacteria.

7.5.14. Sorbed ammonium (X_{NH})

$$\frac{dMX_{NH}}{dt} = \text{pr\_14\_29}$$

$$pr\_14\_29 = \frac{\varepsilon}{\rho_{gravel,bulk}} * \alpha * \left[S_{NH} - \left(\frac{X_{NH}}{PC} \right)^{1/m} \right]$$

Ammonia sorption onto the gravel surface is supposed to be reversible. It therefore has the only process rate (ρ29) which can both be positive or negative, depending on whether or not the ammonium concentration in the wastewater is higher or lower than the equilibrium concentration. One should be aware that the equation is based on the Freundlich sorption isotherm, and that the parameter values are therefore temperature-dependent.

7.5.15. Heterotrophic biomass (X_{BH})

$$\frac{dMX_{BH}}{dt} = (Q_{in} * X_{BH\_in}) - wash\_X_{BH} + (pr\_15\_4 + pr\_15\_5 + pr\_15\_6 + pr\_15\_7 + pr\_15\_9) * V_w$$

$$pr\_15\_4 = \mu_H * \left(\frac{S_F}{K_{SF} + S_F}\right) * \left(\frac{S_F}{S_F + S_A}\right) * \left(\frac{S_O}{K_{OH} + S_o}\right) * \left(\frac{S_{NH}}{K_{NHH} + S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH} + S_{H2S*}}\right) * X_{BH}$$

$$pr\_15\_5 = \mu_H * \left(\frac{S_A}{K_{SA} + S_A}\right) * \left(\frac{S_A}{S_F + S_A}\right) * \left(\frac{S_O}{K_{OH} + S_o}\right) * \left(\frac{S_{NH}}{K_{NHH} + S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH} + S_{H2S*}}\right) * X_{BH}$$

$$pr\_15\_6 = n_g * \mu_H * \left(\frac{S_F}{K_{SF} + S_F}\right) * \left(\frac{S_F}{S_F + S_A}\right) * \left(\frac{K_{OH}}{K_{OH} + S_o}\right) * \left(\frac{S_{NO}}{K_{NOH} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHH} + S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH} + S_{H2S*}}\right) * X_{BH}$$

$$pr\_15\_7 = n_g * \mu_H * \left(\frac{S_A}{K_{SA} + S_A}\right) * \left(\frac{S_A}{S_F + S_A}\right) * \left(\frac{K_{OH}}{K_{OH} + S_o}\right) * \left(\frac{S_{NO}}{K_{NOH} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHH} + S_{NH}}\right) * \left(\frac{K_{IH}}{K_{IH} + S_{H2S*}}\right) * X_{BH}$$

$pr\_15\_9 = -b_H * X_{BH}$

$wash\_X_{BH}$ = IF (Q_{out} > threshold_flowrate)
THEN (fraction_washed_out * X_{BH} * Q_{out})
ELSE 0

Growth of heterotrophs occurs on S_F and S_A both aerobically and with oxygen as electron acceptor (pr_15_4 and pr_15_5), or anoxically with nitrate as electron acceptor (pr_15_6 and pr_15_7). Growth is modelled as Monod kinetics with switching functions for substrate, ammonia and oxygen or nitrate and includes an inhibition function for undissociated H_2S. Microbial decay is modelled as a first-order process (pr_15_9).

Detachment of biofilms or so-called sloughing has been incorporated in the wash-out process.

7.5.16. Autotrophic nitrifying biomass (X_{BA})

$$\frac{dMX_{BA}}{dt} = (Q_{in} * X_{BA\_in}) - wash\_X_{BA} + (pr\_16\_8 + pr\_16\_10) * V_w$$

$$pr\_16\_8 = \mu_A * \left(\frac{S_{NH}}{K_{NHA} + S_{NH}}\right) * \left(\frac{S_O}{K_{OA} + S_o}\right) * \left(\frac{K_{IA}}{K_{IA} + S_{H2S*}}\right) * X_{BA}$$

*pr*_16_10 = - $b_A * X_{BA}$

wash_X_{BA} = IF (Q_{out} > threshold_flowrate)
THEN (fraction_washed_out * X_{BA} * Q_{out})
ELSE 0

Growth of autotrophs requires both oxygen and ammonium (pr_16_8). It is again modelled as Monod kinetics with switching functions for oxygen and ammonium and an inhibition function for undissociated H_2S. Decay (pr_16_10) is represented by a first-order process. Sloughing is taken into account via the wash-out equation.

7.5.17. Fermenting biomass (X_{FB})

$$\frac{dMX_{FB}}{dt} = (Q_{in} * X_{FB\_in}) - wash\_X_{FB} + (pr\_17\_11 + pr\_17\_18) * V_w$$

$$pr\_17\_11 = \mu_{FB} * \left(\frac{S_F}{K_{SFB} + S_F}\right) * \left(\frac{K_{IFB}}{K_{IFB} + S_{H2S*}}\right) * \left(\frac{K_{OFB}}{K_{OFB} + S_O}\right) * \left(\frac{K_{NOFB}}{K_{NOFB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHFB} + S_{NH}}\right) * X_{FB}$$

*pr*_17_18 = - $b_F * X_{FB}$

wash_X_{FB} = IF (Q_{out} > threshold_flowrate)
THEN (fraction_washed_out * X_{FB} * Q_{out})
ELSE 0

While growing, fermenting bacteria consume soluble, readily biodegradable, fermentable COD. They furthermore require ammonium for cell building and they are inhibited by elevated concentrations of oxygen, nitrate and H_2S (pr_17_11). Decay (pr_17_18) is represented by a first-order process.

One should be aware that pH inhibition has not been taken into account as Kalyuzhnyi and Fedorovich (1998) obtained acceptable results without this inhibition and, more importantly, because the pH inhibition ranges as given in ADM1 (Batstone *et al.*, 2002) are seldomly if not never encountered in constructed wetlands treating domestic wastewater.

7.5.18. Acetotrophic methanogenic biomass (X_{AMB})

$$\frac{dMX_{AMB}}{dt} = (Q_{in} * X_{AMB\_in}) - wash\_X_{AMB} + (pr\_18\_12 + pr\_18\_19) * V_w$$

$$pr\_18\_12 = \mu_{AMB} * \left(\frac{S_A}{K_{SAMB} + S_A}\right) * \left(\frac{K_{IAMB}}{K_{IAMB} + S_{H2S*}}\right) * \left(\frac{K_{OAMB}}{K_{OAMB} + S_O}\right) * \left(\frac{K_{NOAMB}}{K_{NOAMB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHAMB} + S_{NH}}\right) * X_{AMB}$$

$pr\_18\_19 = - b_{AMB} * X_{AMB}$

$wash\_X_{AMB}$ = IF (Q_{out} > threshold_flowrate)
THEN (fraction_washed_out * X_{AMB} * Q_{out})
ELSE 0

Anaerobically growing acetotrophic, methanogenic bacteria consume acetate S_A, require ammonium for cell building and are inhibited by oxygen, nitrate and H_2S (pr_18_12). Decay (pr_18_19) is represented by a first-order process.

7.5.19. Acetotrophic sulphate reducing biomass (X_{ASRB})

$$\frac{dMX_{ASRB}}{dt} = (Q_{in} * X_{ASRB\_in}) - wash\_X_{ASRB} + (pr\_19\_13 + pr\_19\_20) * V_w$$

$$pr\_19\_13 = \mu_{ASRB} * \left(\frac{S_A}{K_{SASRB} + S_A}\right) * \left(\frac{S_{SO4}}{K_{SOASRB} + S_{SO}}\right) * \left(\frac{K_{IASRB}}{K_{IASRB} + S_{H2S*}}\right) * \left(\frac{K_{OASRB}}{K_{OASRB} + S_O}\right) * \left(\frac{K_{NOASRB}}{K_{NOASRB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHASRB} + S_{NH}}\right) * X_{ASRB}$$

$pr\_19\_20 = - b_{ASRB} * X_{ASRB}$

$wash\_X_{ASRB}$ = IF (Q_{out} > threshold_flowrate)
THEN (fraction_washed_out * X_{ASRB} * Q_{out})
ELSE 0

Growth of acetotrophic, sulphate reducing bacteria consumes acetate, requires ammonium for cell building, sulphate as electron acceptor and is inhibited by oxygen, nitrate and H_2S (pr_19_13). Decay (pr_19_20) is represented by a first-order process.

7.5.20. Hydrogenotrophic methanogenic biomass (X_{HMB})

$$\frac{dMX_{HMB}}{dt} = (Q_{in} * X_{HMB\_in}) - wash\_X_{HMB} + (pr\_20\_14 + pr\_20\_21) * V_w$$

$$pr\_20\_14 = \mu_{HMB} * \left(\frac{S_{H2}}{K_{H2HMB} + S_{H2}}\right) * \left(\frac{K_{IHMB}}{K_{IHMB} + S_{H2S*}}\right) * \left(\frac{K_{OHMB}}{K_{OHMB} + S_O}\right) * \left(\frac{K_{NOHMB}}{K_{NOHMB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHHMB} + S_{NH}}\right) * X_{HMB}$$

$pr\_20\_21 = - b_{HMB} * X_{HMB}$

$wash\_X_{HMB}$ = IF (Q_{out} > threshold_flowrate)
THEN (fraction_washed_out * X_{HMB} * Q_{out})
ELSE 0

Growth of hydrogenotrophic, methanogenic bacteria consumes hydrogen, requires ammonium for cell building and, since these are anaerobic bacteria, is inhibited by oxygen, nitrate and H_2S (pr_20_14). Decay (pr_20_21) is represented by a first-order process.

7.5.21. Hydrogenotrophic sulphate reducing biomass (X_{HSRB})

$$\frac{dMX_{HSRB}}{dt} = (Q_{in} * X_{HSRB\_in}) - wash\_X_{HSRB} + (pr\_21\_15 + pr\_21\_22) * V_w$$

$$pr\_21\_15 = \mu_{HSRB} * \left(\frac{S_{H2}}{K_{H2HSRB} + S_{H2}}\right) * \left(\frac{S_{SO}}{K_{SOHSRB} + S_{SO}}\right) * \left(\frac{K_{IHSRB}}{K_{IHSRB} + S_{H2S*}}\right) * \left(\frac{K_{OHSRB}}{K_{OHSRB} + S_O}\right) * \left(\frac{K_{NOHSRB}}{K_{NOHSRB} + S_{NO}}\right) * \left(\frac{S_{NH}}{K_{NHHSRB} + S_{NH}}\right) * X_{HSRB}$$

$pr\_21\_22 = - b_{HSRB} * X_{HSRB}$

wash_X_{HSRB} = IF (Q_{out} > threshold_flowrate)
THEN (fraction_washed_out * X_{HSRB} * Q_{out})
ELSE 0

Hydrogenotrophic, sulphate reducing bacteria consume hydrogen, require ammonium for cell building, use sulphate as electron acceptor and are inhibited by oxygen, nitrate and H_2S while growing (pr_21_15). Decay (pr_21_22) is represented by a first-order process.

7.5.22. Sulphide oxidising biomass (X_{THIO})

$$\frac{dMX_{THIO}}{dt} = (Q_{in} * X_{THIO\_in}) - wash\_X_{THIO} + (pr\_22\_16 + pr\_22\_17 + pr\_22\_23) * V_w$$

$$pr\_22\_16 = \mu_{THIO} * \left(\frac{S_{H2S}}{K_{STHIO} + S_{H2S}}\right) * \left(\frac{S_O}{K_{OTHIO} + S_O}\right) * \left(\frac{S_{NH}}{K_{NHTHIO} + S_{NH}}\right) * X_{THIO}$$

$$pr\_22\_17 = \mu_{THIO} * \eta_{THIO} * \left(\frac{S_{H2S}}{K_{STHIO} + S_{H2S}}\right) * \left(\frac{S_{NO}}{K_{NOTHIO} + S_{NO}}\right) * \left(\frac{K_{OTHIO}}{K_{OTHIO} + S_O}\right) * \left(\frac{S_{NH}}{K_{NHTHIO} + S_{NH}}\right) * X_{THIO}$$

$pr\_22\_23 = - b_{THIO} * X_{THIO}$

wash_X_{THIO} = IF (Q_{out} > threshold_flowrate)
THEN (fraction_washed_out * X_{THIO} * Q_{out})
ELSE 0

Sulphur oxidising bacteria are chemoautotrophic organisms that use oxygen or nitrate to oxidise sulfide and other reduced forms of S in order to generate energy. For model completeness and to avoid possibly excessive H_2S accumulation, this group of organisms was included. Adding the X_{THIO} group to the model was preferred over chemical sulphur oxidation as the model focuses on (micro)biological processes. More importantly, Okabe *et* al. (1999) found that turnover rates of H_2S, O_2 and NO_3^- in biofilms of a rotating biological contactor were extremely short compared with possible spontaneous chemical reaction of O_2 and H_2S, indicating that aerobic and anoxic oxidation of H_2S was mediated mainly by microbial reactions. Although typically associated with acidic conditions causing concrete corrosion in e.g. sewers (Nica *et al*., 2000), certain members of this microbial group are able to thrive in near-neutral conditions (Oprime *et al.*, 2001; Oyarzun *et al.*, 2003), which are typically found in wetlands.

Growth of sulphur oxidising bacteria occurs either aerobically with oxygen as electron acceptor (pr_22_16) or anoxically with nitrate as electron acceptor (pr_22_17). Inhibition by H_2S was again added for consistency. Decay (pr_22_23) is represented by a first-order process.

7.5.23. Living plant COD (X_{BPl})

$$\frac{dMX_{Pl}}{dt} = (pr\_23\_24 + pr\_23\_25 + pr\_23\_27) * V_w$$

$$pr\_23\_24 = \left(\frac{1}{d_w * \varepsilon}\right) * k_{pl} * \left(\frac{S_{NH}}{K_{PNH} + S_{NH}}\right) * X_{Pl} \quad \text{with } k_{pl} = f_1(\text{season}) \text{ and } d_w = V_w / (L * W * \varepsilon)$$

$$pr\_23\_25 = \left(\frac{1}{d_w * \varepsilon}\right) * k_{pl} * \left(\frac{S_{NO}}{K_{PNO} + S_{NO}}\right) * \left(\frac{K_{PNH}}{K_{PNH} + S_{NH}}\right) * X_{Pl} \quad \text{with } k_{pl} = f_1(\text{season})$$

$$pr\_23\_27 = -\left(\frac{1}{d_w * \varepsilon}\right) * b_P * X_{Pl} \quad \text{with } b_P = f(\text{season})$$

Living plant biomass increases during the growth season when adequate amounts of nitrate (pr_23_25) and/or ammonium (pr_23_24) are available in the wastewater. At the onset of senescence, living biomass is converted into dead biomass following a first-order rate (pr_23_27). The first term in each equation converts area-based growth rates that are usually applied in plant growth models to volume-based ones.

7.5.24. Dead plant COD (X_{Pd})

$\frac{dMX_{Pd}}{dt} = (pr\_24\_27 + pr\_24\_28) * V_w$
$pr\_24\_27 = \left(\frac{1}{d_w * \varepsilon}\right) * b_P * X_{Pl}$ with $b_P = f_2(\text{season})$ and $d_w = V_w / (L * W * \varepsilon)$ $pr\_24\_28 = \left(\frac{1}{d_w * \varepsilon}\right) * k_{degradation} * X_{Pd}$

Dead biomass is derived from living plant biomass after the growth season ended (pr_24_27), and disappears through the process of physical degradation by for instance wind action, invertebrate consumption etc. (pr_24_28).

7.5.25. Inert particulate COD (X_I)

$\frac{dMX_I}{dt} = (Q_{in} * X_{I\_in}) - wash\_X\_I + pr\_25\_0 * V_w$
$pr\_25\_0 = f_{C\_XI} * k_{decomp} * X_C$ $wash\_X_I$ = IF (Q_{out} > threshold_flowrate)   THEN (fraction_washed_out * X_I * Q_{out})   ELSE 0

X_I represents the truly unbiodegradable particulate COD. As for the other particulate substances, X_I is assumed to remain in the pore space unless higher flow rates exert enough shear stress to drag along solids. In that case, X_I will be washed out at a rate proportional to the flow rate. Inert particulate COD is produced in the reed bed during the decomposition of very slowly biodegradable particulate COD X_C (pr_25_0).

7.5.26. Water volume

$$\frac{dV_w}{dt} = Q_{in} + \text{precipitation} - \text{evapotranspiration} - Q_{out}$$

$d_w = V_w / (L * W * \varepsilon)$

precipitation = $Precip_{in} * L * W$

evapotranspiration =
IF ($T_a > 0$)
THEN $(1.6 / 3000) * DayLength * \left(\frac{10T_a}{HeatIndex}\right)^a * L * W$
ELSE 0

outflow1 = $W * h_{outflow} * k_{hydraulic} * Bed\_Slope$

outflow2 = $\frac{V_w * k_{hydraulic}}{L^2 * \varepsilon} * \left[\frac{V_w}{L * W * \varepsilon} - h_{outflow}\right]$

outflow = IF(waterdepth < $h_{outflow}$)
THEN 0
ELSE
IF(outflow1 > outflow2)
THEN outflow1
ELSE outflow2

The Darcy equation for steady flow in porous media is applied. The hydraulic gradient is assumed to be the maximum of the bed slope or the difference in elevation between the water surface in the wetland and the outflow pipe height $h_{outflow}$.

7.6. PARAMETERS

A complete description of the reed bed requires a total of 100 stoichiometric, kinetic and other parameters. Table 7.4. summarises the applied symbols, a description of the parameter, its unit and a default value. A further discussion can be found in the following sections.

Table 7.4. Description of parameters. All values are for 20 °C.

Symbol	Description	Unit	Default value	Reference*
Microbial parameters				
Y_H	Yield for heterotrophic biomass	$gCOD_{microbial}\ (gCOD_{substrate})^{-1}$	0.67	Henze *et al*. (2000)
Y_A	Yield for autotrophic biomass	$gCOD_{microbial}\ (gN)^{-1}$	0.24	Henze *et al*. (2000)
Y_{FB}	Yield for fermenting biomass	$gCOD_{microbial}\ (gCOD_{substrate})^{-1}$	0.053	K&F (1998)
Y_{AMB}	Yield for acetotrophic methanogenic bacteria	$gCOD_{microbial}\ (gCOD_{substrate})^{-1}$	0.032	K&F (1998)
Y_{ASRB}	Yield for acetotrophic sulphate reducing bacteria	$gCOD_{microbial}\ (gCOD_{substrate})^{-1}$	0.05	K&F (1998)
Y_{HMB}	Yield for hydrogenotrophic methanogenic bacteria	$gCOD_{microbial}$ (g hydrogen)$^{-1}$	0.022	K&F (1998)
Y_{HSRB}	Yield for hydrogenotrophic sulphate reducing bacteria	$gCOD_{microbial}$ (g hydrogen)$^{-1}$	0.094	K&F (1998)
Y_{THIO}	Yield for sulphur oxidising bacteria	$gCOD_{microbial}\ (g\ S)^{-1}$	0.12	de Wit *et al*. (1995)
μ_H	Maximum specific growth rate for heterotrophic biomass	day^{-1}	6	Henze *et al*. (2000)
μ_A	Maximum specific growth rate for autotrophic biomass	day^{-1}	0.8	Henze *et al*. (2000)
μ_{FB}	Maximum specific growth rate for fermenting biomass	day^{-1}	4.1	K&F (1998)
μ_{AMB}	Maximum specific growth rate for acetotrophic methanogenic bacteria	day^{-1}	0.085	K&F (1998)
μ_{ASRB}	Maximum specific growth rate for acetotrophic sulphate reducing bacteria	day^{-1}	0.18	K&F (1998)
μ_{HMB}	Maximum specific growth rate for hydrogenotrophic methanogenic bacteria	day^{-1}	0.35	K&F (1998)
μ_{HSRB}	Maximum specific growth rate for hydrogenotrophic sulphate reducing bacteria	day^{-1}	1.8	K&F (1998)
μ_{THIO}	Maximum specific growth rate for sulphur oxidising bacteria	day^{-1}	5.28	de Wit *et al*. (1995)
f_{C_SI}	Fraction of X_C converted to S_I during decomposition	dimensionless	0.10	Batstone *et al*. (2002)
f_{C_XI}	Fraction of X_C converted to X_I during decomposition	dimensionless	0.25	Batstone *et al*. (2002)
f_{C_XS}	Fraction of X_C converted to X_S during decomposition	dimensionless	0.65	Batstone *et al*. (2002)
f_P	Fraction of microbial biomass converted to inert matter	$gCOD_{products}\ (gCOD_{microbial})^{-1}$	0.08	Henze *et al*. (2000)

Table 7.4. (contd). Description of parameters. All values are for 20 °C.

Symbol	**Description**	**Unit**	**Default value**	**Reference***
i_{XB}	Mass of nitrogen per mass of COD in microbial biomass	gN $(gCOD_{microbial})^{-1}$	0.086	Henze *et al*. (2000)
i_{XP}	Mass of nitrogen per mass of COD in products formed	gN $(gCOD_{products})^{-1}$	0.06	Henze *et al*. (2000)
K_{SF}	Half-saturation coefficient for growth of heterotrophs on fermentable substrate	$gCOD_{substrate}$ m^{-3}	4	Henze *et al*. (2000)
K_{SA}	Half-saturation coefficient for growth of heterotrophs on acetate	$gCOD_{substrate}$ m^{-3}	4	Henze *et al*. (2000)
K_{SFB}	Half-saturation coefficient for growth of fermenters on fermentable substrate	$gCOD_{substrate}$ m^{-3}	28	K&F (1998)
K_{SAMB}	Half-saturation coefficient for growth of AMB on acetate	$gCOD_{substrate}$ m^{-3}	56	K&F (1998)
K_{SASRB}	Half-saturation coefficient for growth of ASRB on acetate	$gCOD_{substrate}$ m^{-3}	24	K&F (1998)
K_{STHIO}	Sulphur half-saturation coefficient for growth of sulphur oxidising bacteria	gS m^{-3}	0.024	de Wit *et al*. (1995)
K_{SOASRB}	Sulphate half-saturation coefficient for acetotrophic sulphate reducing bacteria	gS m^{-3}	19	K&F (1998)
K_{SOHSRB}	Sulphate half-saturation coefficient for hydrogenotrophic sulphate reducing bacteria	gS m^{-3}	1	K&F (1998)
K_{H2HMB}	Hydrogen half-saturation coefficient for hydrogenotrophic methanogenic bacteria	gCOD m^{-3}	0.13	K&F (1998)
K_{H2HSRB}	Hydrogen half-saturation coefficient for hydrogenotrophic sulphate reducing bacteria	gCOD m^{-3}	0.05	K&F (1998)
K_{OH}	Oxygen half-saturation coefficient for heterotrophs	gO_2 m^{-3}	0.2	Henze *et al*. (2000)
K_{OA}	Oxygen half-saturation coefficient for autotrophs	gO_2 m^{-3}	0.4	Henze *et al*. (2000)
K_{OFB}	Oxygen inhibition constant for fermenting bacteria	gO_2 m^{-3}	0.2	Henze *et al*. (2000)
K_{OAMB}	Oxygen inhibition constant for acetotrophic methanogenic bacteria	gO_2 m^{-3}	0.0002	This work
K_{OASRB}	Oxygen inhibition constant for acetotrophic sulphate reducing bacteria	gO_2 m^{-3}	0.0002	This work
K_{OHMB}	Oxygen inhibition constant for hydrogenotrophic methanogenic bacteria	gO_2 m^{-3}	0.0002	This work

Table 7.4. (**contd**). Description of parameters. All values are for 20 °C.

Symbol	Description	Unit	Default value	Reference*
K_{OHSRB}	Oxygen inhibition constant for hydrogenotrophic sulphate reducing bacteria	$gO_2\ m^{-3}$	0.0002	This work
K_{OTHIO}	Oxygen half-saturation constant for aerobic growth of sulphur oxidising bacteria	$gO_2\ m^{-3}$	0.2	This work
K_X	Half-saturation coefficient for hydrolysis of slowly biodegradable substrate by heterotrophs and fermenters	$gCOD_{substrate}\ (gCOD_{microbial})^{-1}$	1	Henze *et al*. (2000)
K_{NOH}	Nitrate half-saturation coefficient for denitrifying heterotrophic biomass	$gNO_3\text{-}N\ m^{-3}$	0.5	Henze *et al*. (2000)
K_{NOFB}	Nitrate inhibition coefficient for fermenting bacteria	$gNO_3\text{-}N\ m^{-3}$	0.5	Henze *et al*. (2000)
K_{NOAMB}	Nitrate inhibition coefficient for acetotrophic methanogenic bacteria	$gNO_3\text{-}N\ m^{-3}$	0.0005	This work
K_{NOASRB}	Nitrate inhibition coefficient for acetotrophic sulphate reducing bacteria	$gNO_3\text{-}N\ m^{-3}$	0.0005	This work
K_{NOHMB}	Nitrate inhibition coefficient for hydrogenotrophic methanogenic bacteria	$gNO_3\text{-}N\ m^{-3}$	0.0005	This work
K_{NOHSRB}	Nitrate inhibition coefficient for hydrogenotrophic sulphate reducing bacteria	$gNO_3\text{-}N\ m^{-3}$	0.0005	This work
K_{NOTHIO}	Nitrate half-saturation coefficient for sulphur oxidising bacteria	$gNO_3\text{-}N\ m^{-3}$	0.5	This work
K_{NHH}	Ammonium half-saturation coefficient for heterotrophic biomass	$gNH_4\text{-}N\ m^{-3}$	0.01	Henze *et al*. (2000)
K_{NHA}	Ammonium half-saturation coefficient for autotrophic biomass	$gNH_4\text{-}N\ m^{-3}$	1	Henze *et al*. (2000)
K_{NHFB}	Ammonium half-saturation coefficient for fermenting bacteria	$gNH_4\text{-}N\ m^{-3}$	0.01	This work
K_{NHAMB}	Ammonium half-saturation coefficient for acetotrophic methanogenic bacteria	$gNH_4\text{-}N\ m^{-3}$	0.01	This work
K_{NHASRB}	Ammonium half-saturation coefficient for acetotrophic sulphate reducing bacteria	$gNH_4\text{-}N\ m^{-3}$	0.01	This work
K_{NHHMB}	Ammonium half-saturation coefficient for hydrogenotrophic methanogenic bacteria	$gNH_4\text{-}N\ m^{-3}$	0.01	This work

Table 7.4. (**contd**). Description of parameters. All values are for 20 °C.

Symbol	**Description**	**Unit**	**Default value**	**Reference***
K_{NHHSRB}	Ammonium half-saturation coefficient for hydrogenotrophic sulphate reducing bacteria	gNH_4-N m^{-3}	0.01	This work
K_{NHTHIO}	Ammonium half-saturation coefficient for sulphur oxidising bacteria	gNH_4-N m^{-3}	0.01	This work
K_{IH}	Sulphide inhibition constant for heterotrophs	gS m^{-3}	140	This work
K_{IA}	Sulphide inhibition constant for autotrophs	gS m^{-3}	140	This work
K_{IFB}	Sulphide inhibition constant for fermenting bacteria	gS m^{-3}	140	K&F (1998)
K_{IAMB}	Sulphide inhibition constant for acetotrophic methanogenic bacteria	gS m^{-3}	140	K&F (1998)
K_{IASRB}	Sulphide inhibition constant for acetotrophic sulphate reducing bacteria	gS m^{-3}	140	K&F (1998)
K_{IHMB}	Sulphide inhibition constant for hydrogenotrophic methanogenic bacteria	gS m^{-3}	140	K&F (1998)
K_{IHSRB}	Sulphide inhibition constant for hydrogenotrophic sulphate reducing bacteria	gS m^{-3}	140	K&F (1998)
b_H	Decay coefficient for heterotrophic biomass	day^{-1}	0.62	Henze *et al*. (2000)
b_A	Decay coefficient for autotrophic biomass	day^{-1}	0.15	Henze *et al*. (2000)
b_{FB}	Decay coefficient for fermenting bacteria	day^{-1}	0.02	K&F (1998)
b_{AMB}	Decay coefficient for acetotrophic methanogenic bacteria	day^{-1}	0.008	K&F (1998)
b_{ASRB}	Decay coefficient for acetotrophic sulphate reducing bacteria	day^{-1}	0.012	K&F (1998)
b_{HMB}	Decay coefficient for hydrogenotrophic methanogenic bacteria	day^{-1}	0.025	K&F (1998)
b_{HSRB}	Decay coefficient for hydrogenotrophic sulphate reducing bacteria	day^{-1}	0.015	K&F (1998)
b_{THIO}	Decay coefficient for sulphur oxidising bacteria	day^{-1}	0.15	This work
n_g	Correction factor for anoxic growth of heterotrophs	dimensionless	0.8	Henze *et al*. (2000)
n_{THIO}	Correction factor for anoxic growth of sulphur oxidising bacteria	dimensionless	0.8	This work
n_h	Correction factor for hydrolysis and ammonification by fermenting bacteria	dimensionless	0.1	Henze *et al*. (2000)

Table 7.4. (contd). Description of parameters. All values are for 20 °C.

Symbol	**Description**	**Unit**	**Default value**	**Reference***
k_{decomp}	First-order decomposition rate	day^{-1}	0.05	This work
k_a	Maximum specific ammonification rate	$m^3 (gCOD_{microbial}\ day)^{-1}$	0.06	Henze *et al*. (2000)
k_h	Maximum specific hydrolysis rate	day^{-1}	2	Henze *et al*. (2000)
<u>Plant parameters</u>				
k_{pl}	Plant relative growth rate, function of season	day^{-1}	0.033	Romero *e.a.* (1999)
K_{PNO}	Nitrate half-saturation coefficient for plant growth	$gNO_3\text{-}N\ m^{-3}$	0.1	K&K (1996)
K_{PNH}	Ammonium half-saturation coefficient for plant growth	$gNH_4\text{-}N\ m^{-3}$	0.3	Romero *e.a.* (1999)
b_P	Decay coefficient for living plant material, function of season	day^{-1}	0.05	This work
k_{ROL}	Root oxygen loss	$m\ day^{-1}$	0.0022 - 5	Chapter 3
k_{degrad}	First order plant physical degradation constant	day^{-1}	0.01	This work
f_{plant}	Fraction of dead plant biomass converted to inert matter	dimensionless	0.2	This work
$i_{XBPlant}$	Mass of nitrogen per mass of COD in plant biomass	$gN (gCOD_{plant})^{-1}$	0.032	Romero *e.a.* (1999)
<u>Wetland physical parameters</u>				
L	Length of the wetland	m	~wetland	
W	Width of the wetland	m	~wetland	
$h_{outflow}$	Height of elbow on drainage pipe	m	~wetland	
$k_{hydraulic}$	Hydraulic conductivity	$m\ day^{-1}$	~wetland	
Slope	Bottom slope	$m\ m^{-1}$	~wetland	
ε	Matrix material porosity as fraction	dimensionless	~wetland	
d	Depth of reed bed	m	~wetland	

Table 7.4. (contd). Description of parameters. All values are for 20 °C.

Symbol	Description	Unit	Default value	Reference*
$Q_{threshold}$	Flow rate above which solids will wash out	m^3 day^{-1}	~wetland	
f_{wash}	Fraction of particulate matter in reed bed being washed out	dimensionless	~wetland	
<u>Meteorological parameters</u>				
Heat_Index	Evapotranspiration parameter	dimensionless	36.77 (Belgium)	
a	Evapotranspiration parameter	dimensionless	1.08 (Belgium)	
<u>Other parameters</u>				
k_La	Oxygen reaeration coefficient	day^{-1}	~depth and velocity	
k_Lv	Volatilisation coefficient	day^{-1}	~depth and velocity	
S_OSAT	Oxygen saturation concentration	gO_2 m^{-3}	8.5 (~T)	
C_aer	Coefficient for calculating k_La	dimensionless	0.2	This work
a_r	Exponent in k_La equation	dimensionless	0.9	This work
b	Exponent in k_La equation	dimensionless	1.67	This work
ρ_{gravel}	Bulk gravel density	kg m^{-3}	1600	
PC	Solid/liquid partition coefficient	l (kg gravel)$^{-1}$	gravel specific	McB & T (2000)
m	Freundlich isotherm exponent	Dimensionless	gravel specific	McB & T (2000)
alpha	Specific sorption rate coefficient	day^{-1}	gravel specific	McB & T (2000)

* K&F (1998) = Kalyuzhnyi and Fedorovich (1998) ; K&K (1996) = Kadlec and Knight (1996) ; McB & T (2000) = McBride and Tanner (2000)

7.6.1. Microbial parameters

Table 7.5 gives an overview of the microbial parameter values as used in the different ASM models (Henze *et al*., 2000). Note that due to different model structures, some parameters are omitted or have another meaning in the different models and are therefore missing in the table. The third column gives an indication of the uncertainty on the ASM1-values, drawn from the work of Rousseau *et al*. (2001b).

Table 7.5. Mean values and uncertainty ranges for the microbial parameters taken from the ASM models. ~T designates that the parameters are temperature dependent with the first value for 10°C and the second one for 20°C.

	ASM1	**uncertainty**	**ASM2**	**ASM3**
k_h	1 – 3 (~T)	50 %	2 – 3 (~T)	2 – 3 (~T)
n_h	0.4 (anoxic)	20 %	0.6 (anoxic) 0.1 (anaerobic)	n.a.
k_a	0.04 – 0.08 (~T)	50 %	n.a.	n.a.
μ_H	3 – 6 (~T)	20 %	3 – 6 (~T)	n.a.
μ_A	0.3 – 0.8 (~T)	20 %	0.35 – 1 (~T)	0.35 – 1 (~T)
b_H	0.2 – 0.62 (~T)	50 %	0.2 – 0.4 (~T)	n.a.
b_A	0.05 – 0.15 (~T)	50 %	0.05 – 0.15 (~T)	n.a.
n_g	0.8	20 %	0.8	0.6
Y_H	0.67	5 %	0.63	0.63 (aerobic) 0.54 (anoxic)
Y_A	0.24	5 %	0.24	0.24
f_P	0.08	5 %	0.1	n.a.
i_{XB}	0.086	5 %	0.07	0.07
i_{XP}	0.06	5 %	0.03	0.02
K_X	0.01 – 0.03 (~T)	50 %	0.1 – 0.3 (~T)	1
K_{SF}, K_{SA}	20	50 %	4	2
K_{OH}	0.2	50 %	0.2	0.2
K_{OA}	0.4	50 %	0.5	0.5
K_{NO}	0.5	50 %	0.5	0.5
K_{NH}	1	50 %	1 (nitrification) 0.05 (immobilisaton)	1 (nitrification) 0.01(immobilisation)

n.a. not applicable or not available

Applying the Arrhenius equation 5.3 (Chapter 5) to the temperature-dependent parameters yields temperature factors θ of 1.07 for μ_H and k_a, 1.10 for μ_A and 1.12 for b_H, b_A and k_h.

No useful information was found on sulphide inhibition constants for heterotrophic and autotrophic bacteria. They were therefore put equal to the ones given in the work of Kalyuhznyi and Fedorovich (1998) on anaerobic wastewater treatment.

Parameter values for the anaerobic bacteria, as given in Table 7.4., were mostly taken from the paper of Kalyuhznyi and Fedorovich (1998). These values were determined from experiments run at 30 °C. No indication is given of their values at lower temperatures. It is therefore assumed that the same parameters are affected as in ASM1, i.e. the growth and decay rates. ADM1 (Batstone *et al*., 2002) has a short section on temperature effects. From the graph given for mesophilic bacteria, one can derive a θ factor of about 1.08. Lokshina *et al*. (2001) evaluated kinetic coefficients of integrated Monod models for low-temperature acetoclastic methanogenesis. They found a θ of 1.20 for adapted UASB biomass whereas biomass from a lake sediment had a θ of 1.11. A value of 1.11 was finally retained for the simulations described in Chapter 8. Information on oxygen and nitrate inhibition was also lacking for the anaerobic bacteria. Parameter values were therefore arbitrarily put equal to the oxygen half-saturation value of 0.0002 $gO_2\ m^{-3}$ and the nitrate half-saturation value of 0.0005 $gN\ m^{-3}$. Similarly, ammonium half-saturation constants were set equal to the ones of heterotrophic bacteria.

Some parameter values for the sulphur oxidising bacteria were obtained from the work of de Wit *et al*. (1995), i.e. half-saturation constants, the growth rate and the yield. No information on the decay rate was found nor on temperature effects. The correction factor for anoxic growth was also arbitrarily set to 0.8, based on the value for anoxically growing heterotrophs. Note that the K_{OTHIO} given in the paper of de Wit *et al*. (1995) is extremely low (1 μM) compared to the values for heterotrophs and autotrophs in the ASM models, which would make the X_{THIO} (too) dominating. It was therefore adjusted to a similar value as for heterotrophs.

7.6.2. Plant parameters

COD content. Samples of live and dead reed plants and plant litter were collected at several HSSF CWs in Flanders during the spring of 2003. Plant and litter material was then oven-dried and ground until a fine powder was obtained. Different plant parts such as stems, leaves and culms were not separated. A measured quantity was then added to a certain volume of distilled water after which a standard COD analysis was carried out. Table 7.6. gives the average values obtained.

Table 7.6. Average COD content of *Phragmites australis*. (average of 3 replicates)

	mg COD (mg DM)$^{-1}$
Alive reed plants	1.17
Dead reed plants	1.19
Reed litter	1.14

Nitrogen content. Romero *et al.* (1999) report on average 28 mg N per g plant dry weight. Taking into account the above COD contents, the nitrogen content can be calculated as 0.032 gN gCOD^{-1}. A summary of the mineral composition of several wetland plants is also given by Kadlec and Knight (1996b). They noted N-contents in *Phragmites australis* ranging from 1.6 to 4.2 % on a dry weight basis. Conversion yields 0.014 to 0.036 gN gCOD^{-1}. As living organisms tend to utilise the maximum possible amount of nutrients, the higher value might be more appropriate in the non-nitrogen-limiting environment of a HSSF CW and would correspond quite well with the one of Romero *et al.* (1999). One should however be aware that nitrogen contents may vary during the growth season and that this effect has not been incorporated in the model in an attempt to simplify certain submodels.

Plant growth rate. Romero *et al.* (1999) studied the effect of N and P concentrations on growth of *Phragmites australis* and found an average relative growth rate (RGR) of 0.033 ± 0.008 per day for the different N/P treatments. P levels seemed to have no significant effect on plant growth whereas N levels were positively correlated with plant growth. Relative growth rates of 0.026, 0.035 and 0.037 day^{-1} were measured for respectively 2.1, 7 and 14 mg NH_4-N l^{-1} in the root solution. Hartzendorf and

Rolletschek (2001) determined relative growth rates of *Phragmites australis* clones growing at lower NaCl-salinity levels (0 to 1.5 ‰) and found RGRs between 0.009 and 0.026 day^{-1}. Lissner and Schierup (1997) similarly found RGRs around 0.02 day^{-1} for lower salinity levels. Assuming that the ratio of COD to dry weigth in reed plants is relatively stable, the above-given RGRs can be used in the model without conversion. Again, under the typically non-limiting growth conditions of constructed wetlands, the higher growth rates might prevail. In this model, plant growth rate is made dependent of season, i.e. during spring a high growth rate is used, during summer a low one is used and during autumn and winter the growth is set to zero.

Nitrate half-saturation coefficient. No data were found for *Phragmites australis*. However, Kadlec and Knight (1996b) report the outcomes of a study with *Typha dominguensis* growing on different concentrations of nitrate. Growth rates were nearly maximal for nitrate concentrations in the order of 0.011 mg NO_3-N l^{-1}.

Ammonium half-saturation coefficient. Romero *et al*. (1999) applied Monod kinetics on data from their growth experiments with *Phragmites australis* and found a half-saturation coefficient of 0.3 mg NH_4-N l^{-1}.

Decay coefficient. This parameter reflects the transition rate from living to dead plant biomass or so-called litterfall. Kadlec and Knight (1996, p. 152) illustrate the overall litter production rate in a forested treatment wetland (South-Carolina, USA) where one can clearly see that litterfall occurs continuously but with marked peaks during autumn. Plant senescence depends on climatic conditions and thus on the length of the growth season. Data on standing stocks of aboveground biomass extracted from a paper on growth of *Schoenoplectus tabernaemontani* (Tanner, 2001) allowed calculation of a first-order decay coefficient with values ranging from 0.0045 to 0.0086 day^{-1}. Similary, a time series with the number of live and dead shoots of *Phragmites australis* given by Asaeda *et al*. (2003) shows a gradual decay over 10 weeks at a rate of 0.0056 day^{-1} followed by a fast decay of the remaining live shoots over a period of two weeks at a rate of 0.07 day^{-1}. Soetaert *et al*. (2004), in their model approach of reed growth in the river Scheldt estuary (Belgium), apply different first-order rates during the growth phase for leaves and stems, i.e. 0.0074 and 0.0018 day^{-1} respectively. During senescence, one overall first-order decay rate is applied for aboveground biomass of 0.1 day^{-1}.

Physical degradation constant. This parameter reflects the rate of litter decomposition. Soetaert *et al*. (2004) propose a first-order rate of 0.006 day^{-1} for dead aboveground *Phragmites* biomass. Kadlec and Knight (1996b) summarised first-order litter decomposition rates of various wetlands plant species. Reed leaves from a wetland in Austria were reported to degrade at a rate between 0.001 and 0.003 day^{-1} whilst reed stem degradation rates varied between 0.00037 and 0.00047 day^{-1}. Another study on herbaceous marsh species mentioned degradation rates up to 0.07 day^{-1}. Gessner (2000) reports exponential breakdown rates for submerged leaves between 0.0033 and 0.0051 day^{-1} whereas the culm breakdown rate was much lower, i.e. 0.0026 day^{-1}. The situation in HSSF CWs of course differs in the sense that litter falls on top of the gravel bed and is therefore not submerged. It can therefore be expected that breakdown occurs faster.

Inert fraction of dead plant material. No data could be found for this parameter. It is therefore suggested to assume a higher fraction than is used for microorganisms, since especially plant stems contain a lot of hardly biodegradable fibers. A value of 0.2 is proposed.

Root oxygen loss. As summarised earlier in Chapter 3, reported root oxygen loss rates are highly variable and range between 0.02 and 45 g O_2 m^{-2} day^{-1}. For a maximum driving force, c.q. oxygen deficit of 9 g O_2 m^{-3}, k_{ROL} can vary in the range of 0.0022 – 5 m day^{-1}.

7.6.3. Meteorological parameters

Thornthwaite's method is used to calculate daily evapotranspiration, according to the following equations:

$$\text{ETP (mm month}^{-1}\text{)} = 16 * Daylength * \left(\frac{10*T}{HeatIndex}\right)^{a} \quad \text{with T = mean monthly temperature (° C)}$$

$$\text{HeatIndex I} = \sum_{j=1}^{12} i_j \quad \text{and} \quad i_j = \left(\frac{T_j}{5}\right)^{1.514}$$

$$a = 6.75e^{-7} * I^3 - 7.71e^{-5} * I^2 + 1.792e^{-2} * I + 0.49239$$

Using the climatological data of Ukkel, home to the Royal Meteorological Institute of Belgium (http://www.meteo.be/), a HeatIndex of 36.77 and an exponent a of 1.08 are obtained.

7.7. CLOSURE

The model was finally implemented in WEST (Hemmis NV, Kortrijk, Belgium). More information on this modelling and simulation platform is available in Vanhooren *et al.* (2002). Its performance was then tested by means of two data sets from both an experimental system (0.55 m^2) and a pilot-scale system (55 m^2), the results of which are described in the next chapter.

Chapter 8

Carbon, nitrogen and sulphur cycles in horizontal subsurface-flow constructed wetlands: a model-based evaluation

8.1. ABSTRACT

Data from a 0.55 m^2 experimental and a 55m^2 pilot-scale HSSF CW were used to validate the mechanistic model as presented in Chapter 7. When taking into account the uncertainties on COD, N and S fractionations and given the low sampling frequency, it can be stated that the model was well able to describe the general trends, for different loading rates as well as for different seasons. The model seems to be able to predict porosity changes and might therefore be a useful tool to study clogging phenomena. Finally, the predicted oxygen transfer rates correlated well with the influent feeding method, i.e. a higher oxygen transfer was found for batch feeding than for continuous feeding.

8.2. INTRODUCTION

Past research on constructed wetlands often focused exclusively on pollutant removal efficiencies and tried to relate the observed concentration and load reductions to design variables such as hydraulic retention time and aspect ratio, and to system external parameters like temperature. By means of regression equations or first-order models as described in Chapter 5, simple black box models were constructed that were able to roughly reproduce the measurements but which neglected the biogeochemical cycles that led to the observations.

From the nineties on, wetland scientists gradually started to investigate the internal processes, with the aim of explaining the observed pollutant dynamics. Some examples are studies on plant uptake processes (Brix, 1994a; Romero *et al*., 1999), plant growth models (Soetaert *et al*., 2004; Asaeda & Karunaratne, 2000), phosphorus sorption dynamics (Drizo *et al*., 1997), microbial fingerprinting (Baptista, 2003), root oxygen release processes (Brix, 1994a; Stottmeister *et al*., 2003), sulphur deposits (Vymazal & Kröpfelová, 2005) etc. A more extensive overview can be found in Chapter 3 where the respective processes are described. As another example, the recent International Symposium on Wetland Pollutant Dynamics and Control (Ghent, Belgium, 4-8

September 2005) can be mentioned. This symposium was a meeting place for wetland scientists – both on constructed and natural wetlands – that were conducting leading edge research on the internal dynamics of wetlands and on pollutant cycling.

From the examples given above, one can easily understand the importance of such research, but unfortunately also notice its fragmented nature. However, mechanistic, dynamic models as described in Chapters 5, 6 and 7 offer a possibility to unify at least part of these findings and to investigate interactions between the different biogeochemical cycles.

In Chapter 7, a new model was proposed to simulate the COD, nitrogen and sulphur cycles in a horizontal subsurface-flow constructed wetland. The water balance considers the inputs influent and precipitation and the outputs effluent and evapotranspiration. Underground flow of the wastewater is approached via the Darcy equation and to mimic the dispersive characteristics, the tanks-in-series approach is proposed. Plants preferentially grow on ammonium but can also take up nitrate when there is an ammonium shortage. During fall, plants first become senescent and are then further degraded to litter. As there is an ongoing debate on the importance of plant root oxygen release, this process was included in an attempt to elucidate its impact on the transformation processes. The microbial compartment of the model consists of growth and decay of 8 microbial groups, i.e. (i) heterotrophic and autotrophic bacteria, with the equations largely based on the Activated Sludge Models (Henze et al, 2000), (ii) fermenting, acetotrophic methanogenic, acetotrophic sulphate reducing, hydrogenotrophic methanogenic and hydrogenotrophic sulphate reducing bacteria, with the equations largely based on Kalyuzhnyi and Fedorovich (1998) and finally (iii) sulphide oxidising bacteria.

This model requires 23 inputs characterising the influent (flow rate, oxygen, COD fractions, N compounds, S compounds, hydrogen and bacterial concentrations) and 5 inputs characterising the meteorological conditions (water and air temperature, daylength, precipitation and season). The 26 mass balances that make up the model contain in total 118 parameters, rendering the model extremely hard to calibrate and leaving not much hope to find a unique, identifiable parameter set (Dochain and Vanrolleghem, 2001). Calibration, if possible at all, would probably require extremely large, multivariable and high-frequent datasets, leading to excessive analysis costs.

One can however use an alternative approach, i.e. adopting default parameter values from validated models and only modify a few, highly sensitive parameters to fit the model to the sparse data. Although the reader of the Activated Sludge Models report (Henze *et al*., 2000) is indeed warned that the (*de facto* default) parameter set that is proposed in the report is only indicative, many case studies have successfully reproduced these values and they can therefore be used with some degree of confidence. The model user should be firmly aware that the model predictions – when using this approach – can by no means be considered as precise ones, i.e. 10 to 20% deviations between modelled and simulated concentrations might be rather standard than exception. However, the main advantage is that one acquires a very useful tool that can be applied to gain insight in wetland cycles, their interactions, the competition for substrates etc. Secondly, when using the model on any given data set, alternative hypotheses might surface that could help to interpret the experimental data, in this way leading to new insights. And finally, such a mechanistic model is extremely helpful to identify knowledge gaps and to point out directions for future research, as will be demonstrated later in this chapter.

8.3. DATA USED FOR ASSESSING THE MODEL VALIDITY

Data were kindly provided by Dr. Joan García from the Technical University of Catalonia (Barcelona, Spain), originating from both experimental setups and pilot-scale constructed wetlands.

8.3.1. Experimental setup (after Caselles-Osorio and García, in preparation)

The experimental HSSF CW consisted of plastic containers with a surface area of 0.55 m^2 (0.93 m length, 0.59 m width and 0.52 m height), filled with gravel extracted from a pilot-scale HSSF CW located at Les Franqueses del Vallès, Spain (Figure 8.1). The gravel layer depth was 0.4 m (porosity of 40%) and the water level was maintained at 0.05 m below the surface. Each container had a drainage pipe located on the bottom of one side to convey the effluent. In June 2004, the wetlands were planted with rhizomes of *Phragmites australis* and placed on the roof of the Hydraulics, Coastal and Environmental Engineering Department (Technical University of Catalonia, Barcelona).

Figure 8.1. Experimental HSSF CW at the Technical University of Catalonia, Barcelona.

Data from one of these HSSF CW were considered for simulation purposes. The wetland was fed daily in batch mode with fresh urban wastewater obtained from a nearby sewer which was first allowed to settle for 1 hour. Measurements were carried out during two periods in which different operational conditions were applied, from November 2004 to January 2005 and from February to March 2005 respectively (Table 8.1).

Table 8.1. Operational conditions used for the HSSF CW under study.

Period	November - January	February - March
Flow (l day^{-1})	20	30
Hydraulic retention time (day)	3	2
Surface loading rate (g COD m^{-2} day^{-1})	12.6 ± 3.2	19.7 ± 5.7

***8.3.2. Pilot-scale setup (after García et al.,* 2004a, 2004b)**

The pilot-scale HSSF CWs of Les Franqueses del Vallès near Barcelona (Spain) were created to study the effects of different depths, gravel sizes, aspect ratios and loading rates on the pollutant removal efficiencies.

A complete lay-out of the system is given in Figure 8.2. Domestic wastewater is first screened and then flows to an Imhoff tank for primary treatment. From there the water is pumped to a distribution box with a weir where the flow is split in 8 equal parts and then fed to one of eight parallel HSSF CW planted with *Phragmites australis*. All wetlands have an equal surface area of 55 m^2 and therefore operate at the same loading rate. Four different aspect ratios (L/W) were studied, i.e. A wetlands 1/1, B wetlands 1.5/1, C wetlands 2/1 and D wetlands 2.5/1. For each aspect ratio, two parallel beds were constructed, numbered 1 and 2. All number 1 beds contain gravel with a diameter of 10 mm whilst the number 2 beds contain gravel with a diameter of 3.5 mm. Beds of the types A, B and C have an average water depth of 0.5 m, beds of the D type on the contrary have an average water depth of only 0.27 m.

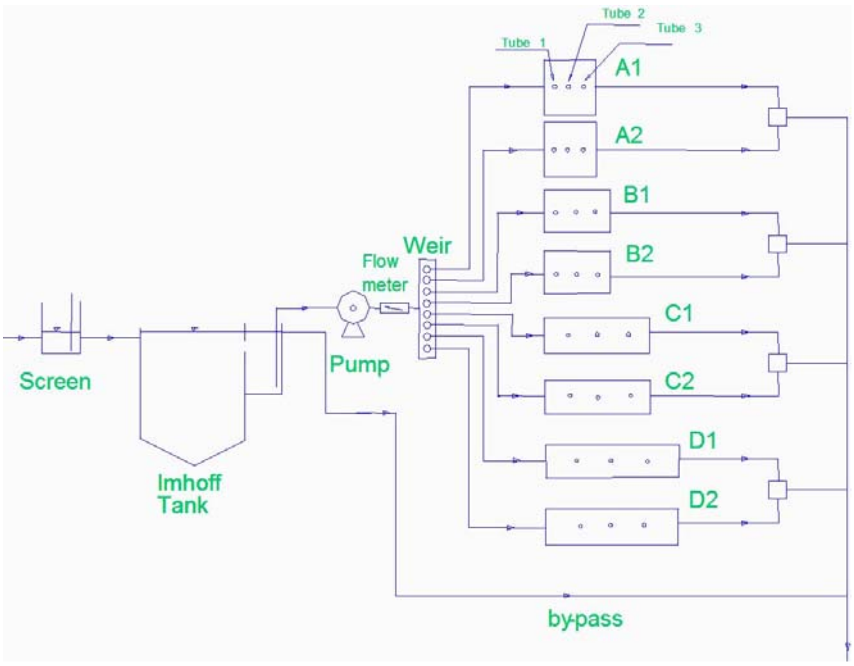

Figure 8.2. Schematic diagram of the pilot-scale HSSF CWs at Les Franqueses del Vallès (Spain). Type A beds have an aspect ratio of 1/1, B of 1.5/1, C of 2/1 and D of 2.5/1. Type 1 beds contain coarser gravel while type 2 beds contain finer gravel. Type D beds have a lower water depth.

A comprehensive data set is available from 28 May 2001 till 06 December 2003, containing information on influent and effluent concentrations of COD, BOD_5 and NH_4-N, on influent flow rates (varying between 0.9 and 3.5 m^3 day^{-1} per bed), on effluent temperatures and on meteorological variables such as precipitation and air temperature. For a more extensive explanation on the pilot-scale setup and on the treatment results, the reader is referred to García *et al.* (2003, 2004a and 2004b). Data from Bed D1 were considered for simulation purposes.

8.4. TANKS-IN-SERIES APPROACH

As recommended by many researchers, initial simulations were performed by using multiple tanks in series. Results for COD and ammonium were quite good, but effluent nitrate concentrations were far too high due to a lack of denitrification. A closer look at the mass balances learned that the different oxygen affinity constants for heterotrophs and autotrophs (K_{OH} = 0.2 mg O_2 l^{-1} and K_{OA} = 0.4 mg O_2 l^{-1}) were responsible for this. Indeed, close to the inlet zone, both bacterial groups compete for oxygen for COD removal and nitrification respectively. In a completely mixed tank, oxygen is equally spread over the tank volume whereas in reality one has a spatial heterogeneic mosaic of aerobic and anoxic microsites in the rootzone. Due to their lower oxygen affinity constant, heterotrophs will consume most of the oxygen until COD becomes limiting. Only then – and when the supply of oxygen is still adequate – will autotrophs be able to convert ammonium to nitrate in significant quantities. Depending on the respective loads, nitrification starts to occur roughly between 1/3 and 2/3 of the bed length. What remains is wastewater low in COD and ammonium, but high in nitrate. Since denitrification requires readily biodegradable COD, denitrification will be limited and nitrate effluent concentrations will therefore be high.

A solution to this problem was devised by altering the mixing model, and basically by making it consist of tanks set up in series on the one hand and adding parallel branched tanks. Figure 8.3. gives a schematic representation of this lay-out. The left I-box supplies the model with data on the influent (flow and concentrations), the upper one feeds the model with meteorological data (air and water temperature, precipitation, length of day and season). In the splitters, the water flow is divided into customisable fractions that then flow to aerobic (ae) respectively anaerobic (ana) tanks. One could imagine the aerobic tanks to represent the upper layer of the reed bed, both in contact with the air and with the plant roots whilst the so-called anaerobic tanks represent the bottom layer below the rootzone. After each set of tanks, a combiner mixes both water flows and then feeds them to the next splitter. A similar set-up was used by Benedetti et al. (2004) for the description of wastewater treatment with imperfect mixing and anaerobic zones. This allows to simulate that the water passes through aerobic, anaerobic and anoxic sites as it occurs in real systems. Initial simulations learned that the last tanks need to be anaerobic in order to prevent high effluent nitrate concentrations. This is not illogical as the wastewater always has to pass through the deeper layers in order to reach the drainage tube at the bottom.

A more practical problem of the model studies surfaced as well, i.e. the excessive simulation time. As one simulation of 140 days for the experimental system already took several hours on a Pentium 4 (for 9 aerobic and 9 anaerobic tanks), testing the influence of certain parameter variations would have consumed more time than was reasonably available. A 922 day simulation of the pilot-scale system seemed altogether unattainable. Hence, for practical reasons, the lay-out of Figure 8.3 was further used, i.e. with 8 continuously stirred tanks reactors. This reduced the 922 day simulation to about 22 minutes.

Initial simulations with this lay-out were promising, but there still seemed to be a lack of organic material near the outlet of the wetland to allow sufficient denitrification. As it would seem logical that the majority of settled substances can be found near the bottom of a HSSF CW, the splitters were reprogrammed in such a way that the fraction of solids going to the anaerobic (bottom) tanks could be adjusted to reflect sedimentation.

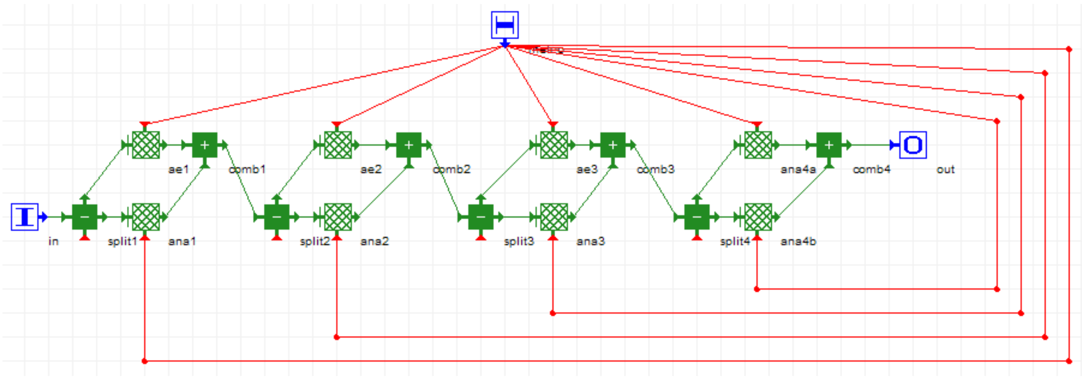

Figure 8.3. Schematic representation of the hydraulic lay-out. ae = aerobic tank, ana = anaerobic tank, split = splitter, comb = combiner, I = input, O = output.

8.5. SIMULATION RESULTS OF THE EXPERIMENTAL SYSTEM

Via trial and error, the volume ratio's between the aerobic and anaerobic tanks were determined as well as the flow split and particle split fractions. Table 8.2. lists an overview of the parameter that were either not given in Table 7.4 or changed compared to the default value given in Table 7.4.

The different wash fractions (describing at what flow rate the sediments are washed out of the CW) were set according to the observations that (i) solids usually accumulate near the inlet of a HSSF CW and (ii) measured effluent solids concentrations are usually small. The exact values were determined via trial and error. A maximum plant biomass of 300 gCOD corresponds to 1400 kg DM ha^{-1} which is well below the maximum biomass of 1.960 g DM m^{-2} $year^{-1}$ reported by Radoux and Kemp (1982). Physical reaeration constants were also set via trial and error. Different values could possibly be related to differences in turbulence near the inlet and further down in the reed bed due to the feeding mechanism.

Table 8.2. Parameter settings used to obtain the subsequent simulation results.

Parameter	Description	Units	Value
Vol_AE	Volume ratio of aerobic tanks	%	70
Vol_ANA	Volume ratio of anaerobic tanks	%	30
Flow_split	Fraction of flow to aerobic tanks	-	0.6
Particle_split	Fraction of particles to anaerobic tanks	-	0.6
kROLmax	Summer root oxygen loss	m day^{-1}	0.4
kROLmin	Winter root oxygen loss	m day^{-1}	0.1
a_r_AE	Exponent in physical reaeration equation for aerobic tanks	-	1
C_aer_AE1	Rate constant in physical reaeration equation for ae1	-	1.8
C_aer_AE2	Rate constant in physical reaeration equation for ae2	-	1.2
C_aer_AE3	Rate constant in physical reaeration equation for ae3	-	0.8
C_vol_AE	Rate constant in volatilisation equation for all aerobic tanks	-	0.1
kLa_ANA	Aeration coefficient anaerobic tanks	day^{-1}	0
kLv_ANA	Volatilisation coefficient anaerobic tanks	day^{-1}	0
WashFrac1	Fraction washed out from ae1 and ana1	-	0.04
WashFrac2	Fraction washed out from ae2 and ana2	-	0.04
WashFrac3	Fraction washed out from ae3 and ana3	-	0.004
WashFrac4	Fraction washed out from ana4a and ana4b	-	0.0001
Alpha	Specific NH_4 sorption rate coefficient	day^{-1}	0
MaxPlantBiomass	Maximum plant biomass during summer per ae tank	gCOD	300
bPWinter	Decay coefficient for living plants during winter	day^{-1}	0.0177
k_degradation	First order plant physical degradation constant	day^{-1}	0.02

Fractionation of the influent wastewater was based on standard ratios given in the ASM models (Henze *et al*, 2000), as follows: S_I = 3%, S_F = 35%, S_A = 10%, X_C = 3%, X_S = 44% and X_I = 5% of the measured influent COD. S_{ND} and X_{ND} were each given a value of 10% of the measured NH_4-N. S_O in the influent was estimated to be as low as 0.2 mg O_2 l^{-1}. S_{NO}, S_{H2S}, S_{H2} and all bacterial groups were assumed not to be present in the influent. S_{SO4} was derived from measured sulphate concentrations.

Before showing the simulation results, the reader should be aware of one important difference between the experimental and the simulated setup. The experimental system was in fact fed in batch mode (20 or 30 liter of wastewater applied every morning over a period of about 20 minutes, with doubled portions on Mondays and Fridays to cover the weekend). However, feeding the model with such batch data led to considerable numerical instabilities because of zero outflows and it was therefore decided to change the simulation such that the 20 or 30 liter influent feeding was equally spread over the day.

A similar warning needs to be given for the effluent in the sense that measured effluent concentrations were obtained from the displaced volume of wastewater during feeding, whereas simulated data can be considered as daily grab samples from the effluent.

8.5.1. COD removal

Figure 8.4. compares measured and simulated effluent concentrations of COD in the experimental system. Until day 90, 20 l day^{-1} of wastewater was applied whereas from day 90 onwards 30 l day^{-1} was applied. Unfortunately, between day 50 and 90 no data were collected. For the simulations this period was bridged by using a constant influent (data not shown).

Measured and simulated effluent concentrations seem to be in good agreement, with some exceptions. The higher predicted concentrations between day 25 and day 38 seem to be caused by an increased washout of solids to the subsequent tanks because of some violent storms. At day 96, the peak indicates that the model is underestimating the capability of the wetland to deal with loading variations. However, in general, both the measured and simulated effluent data show that the higher hydraulic loading rate has a very low influence on the removal efficiency.

The reader is reminded again that due to the sampling method (one sample of the mixed, displaced effluent volume), variations of measured concentrations might be less pronounced than those of the simulated effluent concentrations.

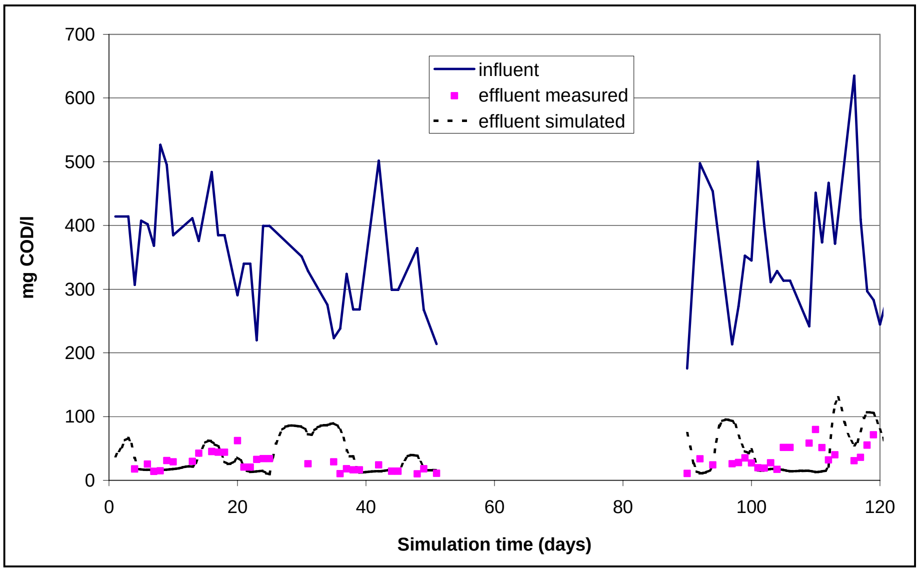

Figure 8.4. Simulated effluent COD concentrations (dotted line) compared with measured influent (thick line) and effluent (squares) concentrations.

8.5.2. Nitrogen removal

Figure 8.5. shows the measured ammonium influent concentrations and compares the simulated with the measured NH_4-N effluent concentrations.

Before day 50, a HRT of 3 days was applied whereas after day 90 the HRT was reduced to 2 days. This change clearly has an effect on ammonium removal, as is evident from both measured and simulated effluent data. During the first period, the model seems to underestimate ammonium removal by some 3 to 4 mg N l^{-1}. Increasing the aerobic volume and/or the oxygen transfer rates only helped partially and resulted in very high effluent nitrate concentrations, which were not observed in reality (see below).

After changing the flow rate at day 90, the simulation shows a marked effluent peak which was not observed in reality. This seems to indicate that the wetland has a quicker adaptation capacity than what the model predicts. Indeed, the predicted concentrations of nitrifying bacteria increase significantly but too slowly (see 8.5.4). Note that a similar (but smaller) peak could also be noticed in the simulated COD effluent concentrations.

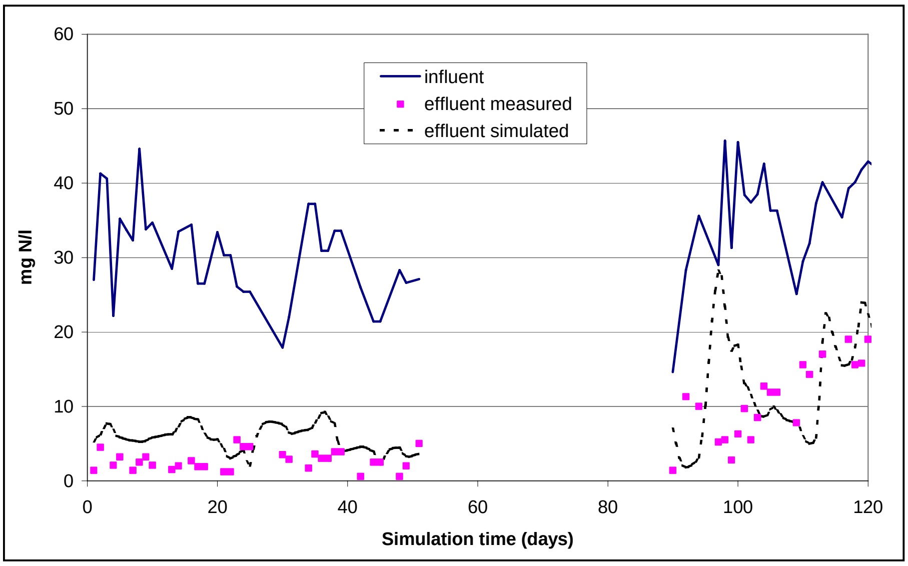

Figure 8.5. Simulated effluent NH_4-N concentrations (dotted line) compared with measured influent (thick line) and effluent (squares) concentrations.

Some sporadic measurements of the nitrate effluent concentration were available at the TU Catalunia and all showed low concentrations (usually below 2 mg N l^{-1}), as commonly observed in other HSSF CW (cf. Chapters 4 and 6). Figure 8.6. presents the simulated NO_3-N effluent time series.

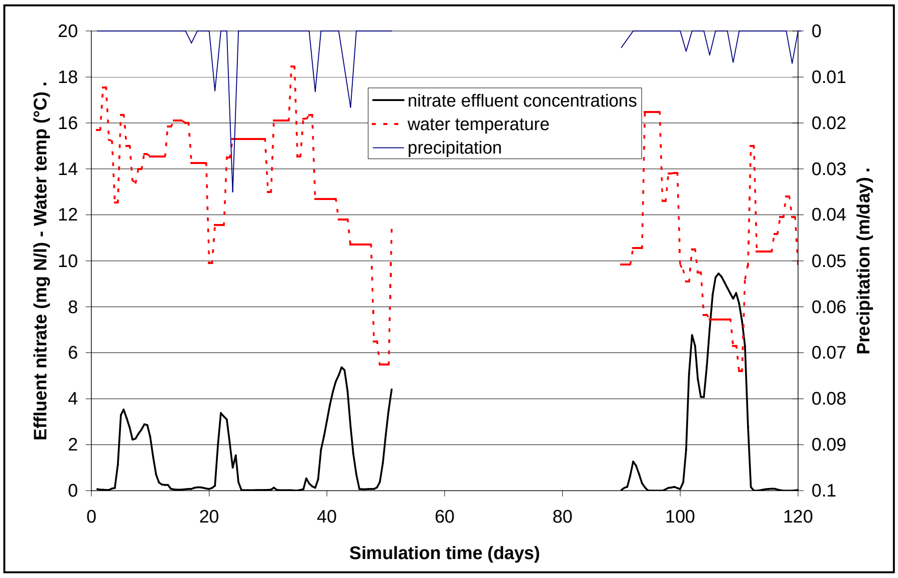

Figure 8.6. Simulated effluent NO_3-N concentrations (thick line) and their coincidence with rainstorms (thin line) and/or low temperatures (dotted line).

It can be easily observed that nitrate effluent peaks coincide with either rain events (as was also observed in Aartselaar, Chapter 4 and Saxby, Chapter 6) and/or with colder periods. Indeed, the highest peak (from day 100 to day 112) is caused by slow adaptation of firstly autotrophs and then heterotrophs to the low water temperatures.

8.5.3. Sulphur conversions

Sulphate removal data are summarised in Figure 8.7. Simulated effluent concentrations are considerably more variable than the measured effluent SO_4 concentrations and the model clearly overpredicts sulphate removal. The variations are triggered by changes in temperature and organic loading which in term influence the oxygen concentrations. Zero effluent concentrations seem to coincide with higher water temperatures (cf. Figure 8.6). Again, one is reminded that due to the sampling and feeding approach, it is not unlogical that measured concentrations show less variations than the simulated ones.

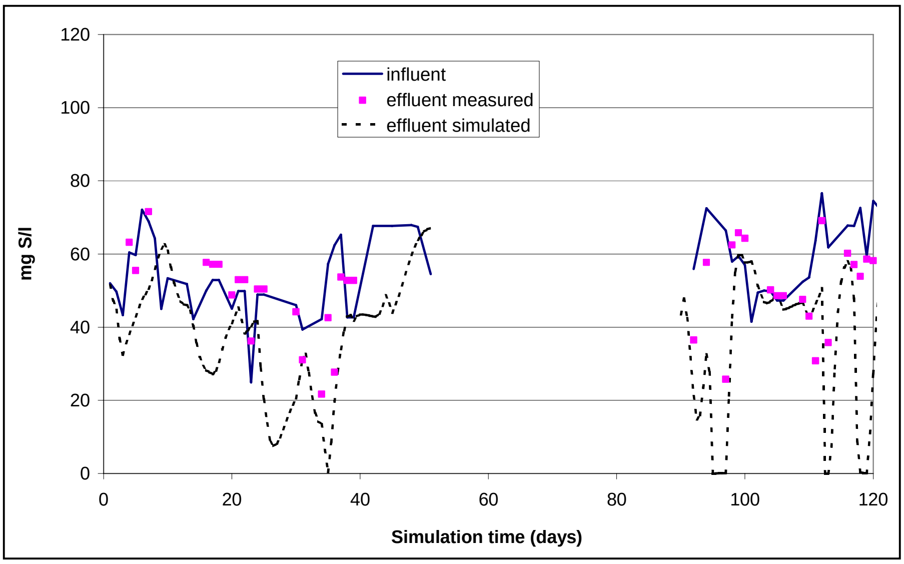

Figure 8.7. Simulated effluent SO_4-S concentrations (dotted line) compared with measured influent (thick line) and effluent (squares) concentrations.

Due to the completely mixed tank approach, the oxygen and nitrate inhibition constants for sulphate reducing bacteria had to be set very low, i.e. 0.0002 mg O_2 l^{-1} and 0.0005 mg NO_3-N l^{-1} respectively (arbitrarily set to 1/1000 of the oxygen saturation constant of heterotrophic bacteria). Apparently, these inhibition constants should be even lower in order to reflect the measured effluent S concentrations.

Simulated undissociated hydrogen sulphide concentrations in the tanks nevertheless remain low and reach maximum concentrations of 0.56 mg S l^{-1} (for the set pH 7), which are far below the inhibitory level.

8.5.4. Spatial and temporal variations of bacterial concentrations

Heterotrophic and fermenting bacteria become the most abundant organisms in the simulated wetland. Heterotrophs logically have the highest densities in the aerobic tanks and near the inlet, where substrate is still abundant, whereas the fermenting bacteria are more abundant in the anaerobic tanks near the outlet (Figure 8.8).

Bacteria concentrations may be converted from COD units to DM units by using the conversion factor of 1.222 gCOD (g biomass)$^{-1}$ given by Kalyuzhnyi and Fedorovich (1998).

Higher loading rates clearly result in increasing concentrations of both bacterial groups. Heterotrophs present in the anaerobic tanks remain active and can grow aerobically or anoxically by using the oxygen and nitrate that is passed on from the previous aerobic tank. Since the influent is considered to be as good as free of oxygen and nitrate, no heterotrophs are present in ana1.

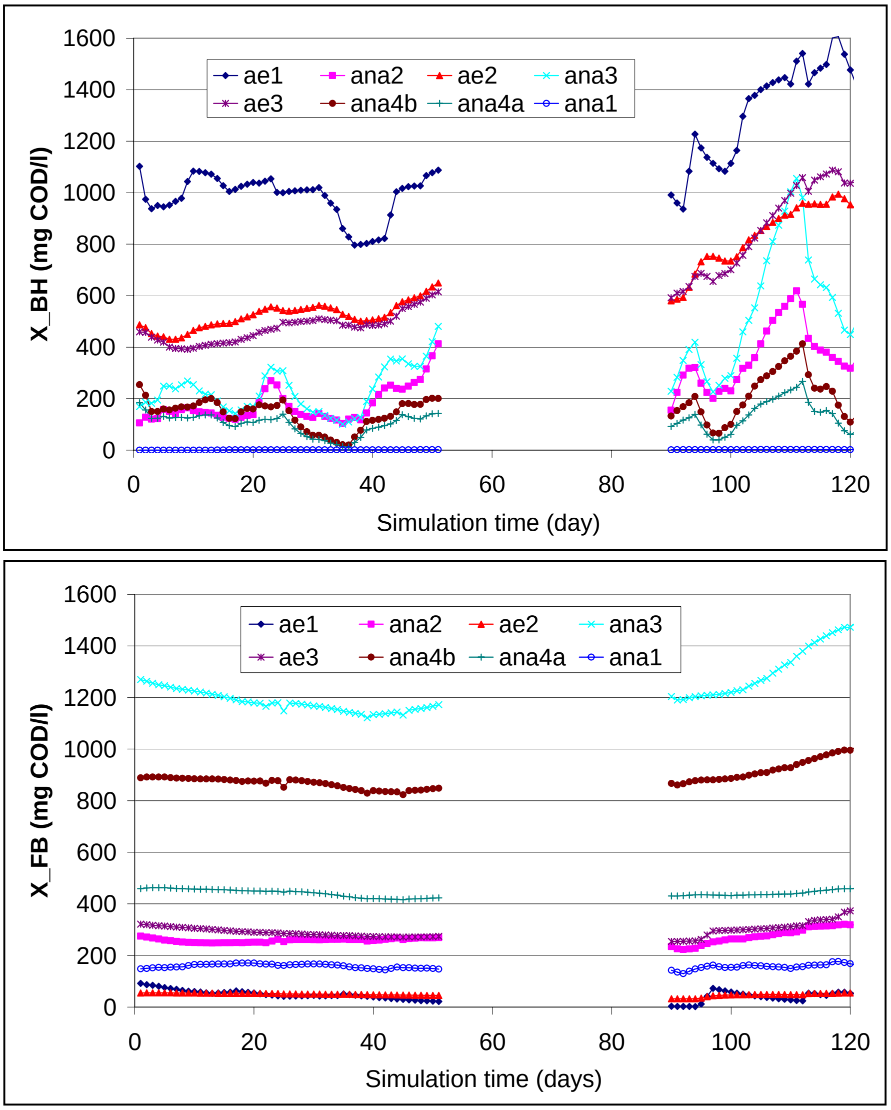

Figure 8.8. Spatio-temporal distribution of the two most abundant bacterial groups, i.e. heterotrophs (upper panel) and fermenting bacteria (lower panel).

Autotrophic nitrifying bacteria are mostly found in the aerobic tanks with concentrations varying between 30 and 50 mg COD l^{-1} during the first period (3 day HRT) and reaching a maximum concentration of 80 mg COD l^{-1} in ae3 during the second period. At first sight surprising, many nitrifiers can also be found in ana3. They can possibly thrive in this so-called anaerobic tank because of the oxygen surplus in ae2 which ends up in ae3 and ana3 (Figure 8.9). As for the heterotrophs, nitrifiers are absent in ana1 because of a lack of oxygen.

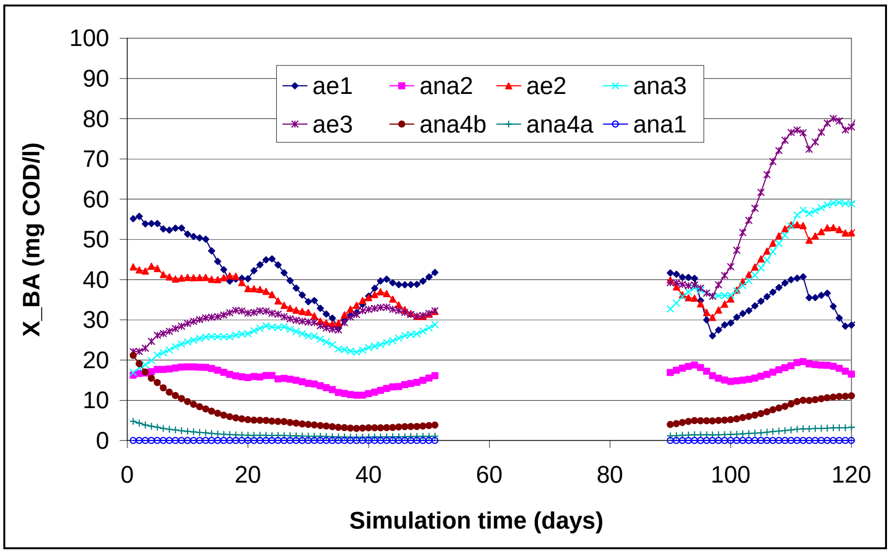

Figure 8.9. Spatio-temporal distribution of nitrifying bacteria.

Acetotrophic methanogenic bacteria are predicted to be mainly present in the anaerobic tank 4b (140 to 160 mg COD l^{-1}), ana3 (40 to 80 mg COD l^{-1}, with the lowest concentrations during period 2) and ana4a (35 to 40 mg COD l^{-1}). Aerobic tanks ae1, ae2 and ae3 and anaerobic tanks ana1 and ana2 all have AMB concentrations below 20 mg COD l^{-1}. Hydrogenotrophic methanogenic bacteria on the contrary only occur in minor quantities (less than 20 mg COD l^{-1} during period 1, below 10 mg COD l^{-1} during period 2).

Acetotrophic sulphate reducing bacteria (ASRB) seem to play a minor role under these conditions. Concentrations of ASRB are below 10 mg COD l^{-1} in ana4b and even lower than 5 mg COD l^{-1} in all other tanks. Hydrogenotrophic sulphate reducing bacteria on the other hand are more abundant with quantities around 200 – 250 mg COD l^{-1} in ana4b, concentrations around 100 mg COD l^{-1} in ana3 and ana4a and concentrations below 50 mg COD l^{-1} for all other tanks.

The Thiobacillus group is nearly non-existing, with concentrations in all tanks below 10 mg COD l^{-1}. It might either be outcompeted by heterotrophs and nitrifiers for oxygen and nitrate, and/or it could indicate that hydrogen sulphide volatilisation is rather high, thus provoking substrate limitation.

8.5.5. Estimated oxygen transfer rates

Because of a lack of literature data c.q. valid equations for HSSF CW, root oxygen release and physical reaeration are considered as one lumped process. Estimated oxygen transfer rates (pr_1_26 and pr_1_30 in Chapter 7) vary between 15 - 17 g O_2 m^{-2} day^{-1} during period 1, and between 17 - 20 g O_2 m^{-2} day^{-1} during period 2, so there is a good agreement between both periods. Oxygen concentrations in the effluent are consistently below 1 mg l^{-1}.

8.5.6. Estimated porosity evolution

The spatio-temporal evolution of porosity is shown in Figure 8.10. Lowest porosities are found after 120 days in anaerobic tanks 1 and 3. This correlates well with the fact that more than 50% of the solids are routed to the bottom layers. However, in reality one mostly observes the sharpest decline near the inlet, and not between 50% and 75% of the bed length. This indicates a problem with the wash-out settings. However, adjusting these settings to achieve a more realistic porosity profile resulted in higher nitrate effluent concentrations due to the reduction of available COD for denitrification in the final anaerobic zones.

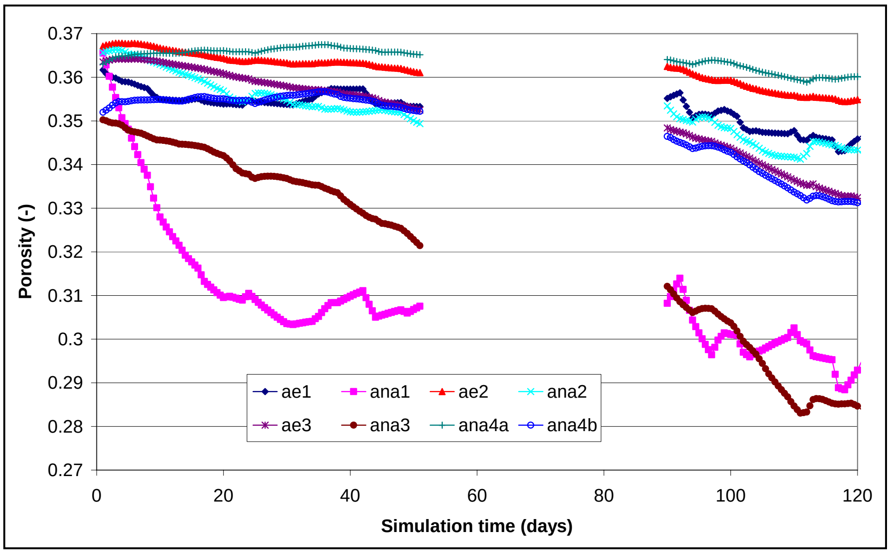

Figure 8.10. Spatio-temporal distribution of porosity.

8.6. SIMULATION RESULTS OF THE PILOT-SCALE SYSTEM

The appropriate volume ratio's between the aerobic and anaerobic tanks as well as the flow split and particle split fractions were again determined via trial and error. Table 8.3. gives an overview of the adapted parameters compared to Table 7.4.

Table 8.3. Parameter settings used to obtain the subsequent simulation results.

Parameter	Symbol	Units	Value
Vol_AE	Volume ratio of aerobic tanks	%	50
Vol_ANA	Volume ratio of anaerobic tanks	%	50
Flow_split	Fraction of flow to aerobic tanks	-	0.5
Particle_split	Fraction of particles to anaerobic tanks	-	0.6
kROLmax	Summer root oxygen loss	m day^{-1}	0.1
kROLmin	Winter root oxygen loss	m day^{-1}	0.02
a_r_AE	Exponent in physical reaeration equation for aerobic tanks	-	0.9
C_aer_AE1	Rate constant in physical reaeration equation for ae1	-	0.1
C_aer_AE2	Rate constant in physical reaeration equation for ae2	-	0.1
C_aer_AE3	Rate constant in physical reaeration equation for ae3	-	0.1
C_vol_AE	Rate constant in volatilisation equation for all aerobic tanks	-	0.1
kLa_ANA	Aeration coefficient anaerobic tanks	day^{-1}	0
kLv_ANA	Volatilisation coefficient anaerobic tanks	day^{-1}	0
WashFrac1	Fraction washed out from ae1 and ana1	-	0.005
WashFrac2	Fraction washed out from ae2 and ana2	-	0.003
WashFrac3	Fraction washed out from ae3 and ana3	-	0.003
WashFrac4	Fraction washed out from ana4a and ana4b	-	0.001
MaxPlantBiomass	Maximum plant biomass during summer per ae tank	gCOD	30000
bPWinter	Decay coefficient for living plants during winter	day^{-1}	0.0177
k_degradation	First order plant physical degradation constant	day^{-1}	0.02
Alpha	Specific NH_4 sorption rate coefficient	day^{-1}	0

As before, the different wash fractions were determined via trial and error. A maximum plant biomass of 30000 gCOD per tank corresponds to 1400 kg DS ha^{-1} (Radoux and Kemp, 1982). Physical reaeration constants were also set via trial and error. As there is very little information available about both physical and biological reaeration, both processes were for the moment considered as one. This explains the different parameter values obtained for the experimental and the pilot-scale system. Indeed, a change in parameter values of the physical reaeration equation is possibly compensated by a change in parameter values of the plant reaearation equation.

The influent pollutant fractionation was based on standard ratios given in the ASM models (Henze *et al*, 2000), as follows: S_I = 5%, S_F = 40%, S_A = 10%, X_C = 20%, X_S = 23% and X_I = 2% of measured COD. S_{ND} and X_{ND} each 10% of measured NH_4-N. S_O was estimated to be as low as 0.2 mg O_2 l^{-1}. S_{NO}, S_{H2S}, S_{H2} and all bacterial groups were assumed to be absent in the influent. S_{SO4} was fixed at 25 mg S/l based on a small number of analyses available.

The reader should be pointed to the fact that this time both the real system as well as the simulated system are continuously fed and both systems are 'grab sampled'. One should also be aware that influent samples of the pilot-scale system were only taken weekly or biweekly per week. For the simulation, the influent concentration was therefore kept constant for several days until a new measurement was available. In reality, influent variations will probably have been much higher (cf. Chapters 4 and 6).

8.6.1. COD removal

Figure 8.11 compares measured and simulated COD effluent data over a period of 922 days. COD removal tends to be slightly overestimated by the model, especially from day 600 onwards. Initially this seems to be caused by an overestimated oxygen transfer in winter. Indeed, due to the low water temperatures between day 600 and day 650, the oxygen saturation concentration increases considerably. As a consequence, the driving force ($S_{OSAT} - S_O$) rises and with it the oxygen transfer. After day 700, there seems to be no obvious reason for the overestimated COD removal associated with the model structure. However, field observations from that period report a reduced plant growth after the feeding was interrupted for a certain period, which could have had an impact on the plant root oxygen leakage.

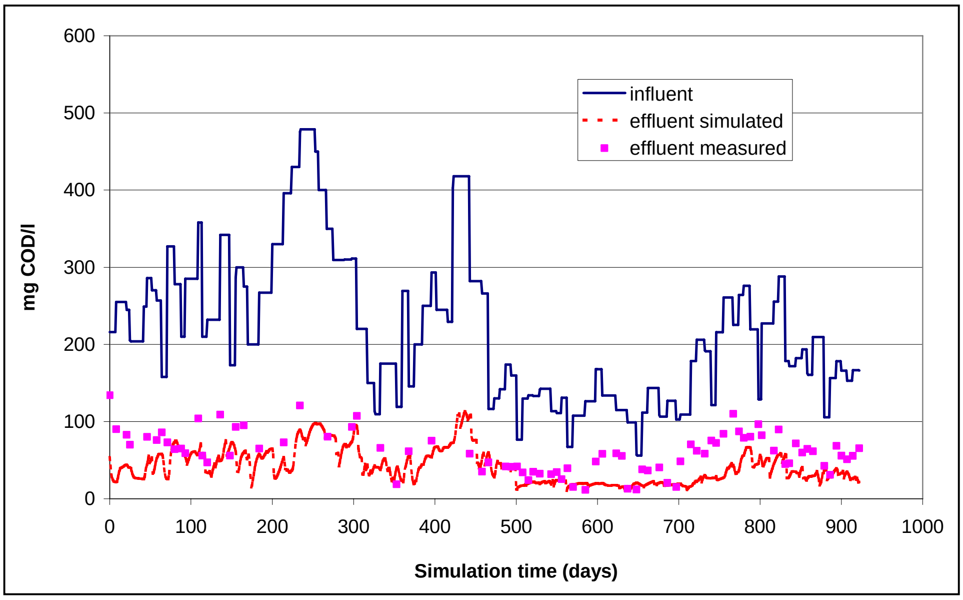

Figure 8.11. 922 days time series of influent COD (thick line), simulated effluent COD (dotted line) and measured effluent COD (squares) with day 0 = 28 May 2001.

8.6.2. Nitrogen removal

In Figure 8.12, a 922 day time series of measured effluent ammonium concentrations is compared with simulated data. In general, the model seems capable of reproducing the trends, but shows some peak effluent concentrations which in reality were not noticed.

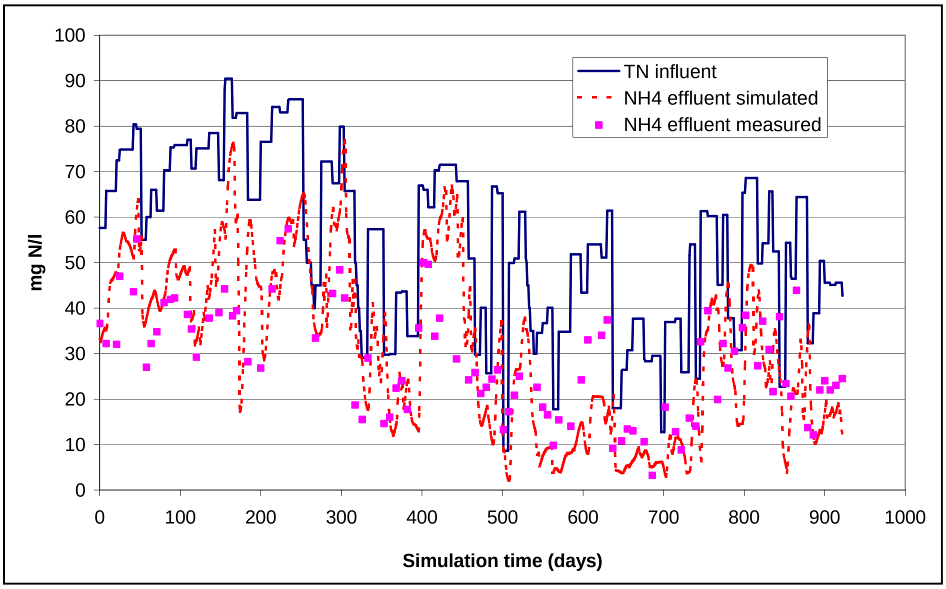

Figure 8.12. 922 days time series of influent TN (thick line), simulated effluent NH_4 (dotted line) and measured effluent NH_4 (squares) with day 0 = 28 May 2001.

Except for the second winter where concentrations rise up to 20 mg N l^{-1} (Figure 8.13), simulated effluent nitrate concentrations are generally low, a fact commonly noticed in HSSF CW (Chapters 4 and 6). This behaviour corresponds again with the higher oxygen transfer rates obtained at low water temperatures. Indeed, oxygen concentrations in the tanks reach such concentrations that anoxic growth of heterotrophic bacteria is inhibited.

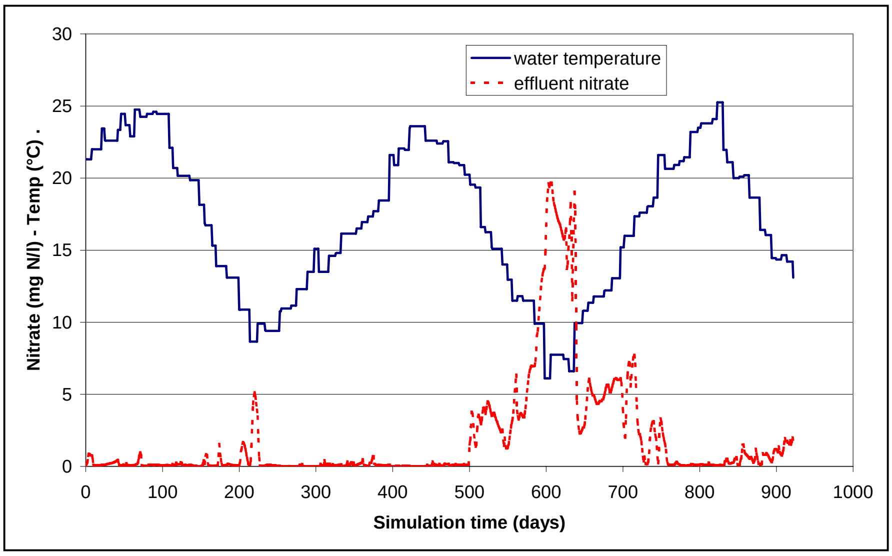

Figure 8.13. 922 days time series of simulated effluent nitrate concentrations (dotted line) compared to measured water temperatures (thick line) with day 0 = 28 May 2001.

8.6.3. Sulphur transformations

Only few measurements are available on sulphate removal at Les Franqueses del Vallès (García *et al.*, in press) but they indicate highly variable removal efficiencies. Figure 8.14. presents the simulated effluent sulphate variations for a fixed influent concentration of 25 mg S l^{-1}. As was also evident from the experimental system, sulphate removal seems to form an alternative pathway which can quickly be switched on or off. Effluent concentrations surpassing the influent concentrations are caused by the concentrating effect of evapotranspiration. H_2S concentrations in all tanks remained again well below inhibitory levels (for the set pH 7).

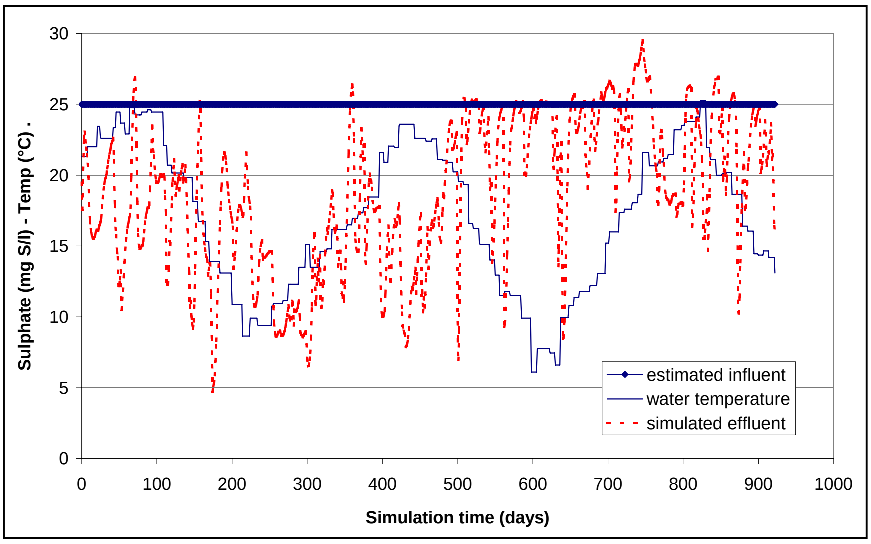

Figure 8.14. 922 days time series of simulated effluent sulphate (dotted line) compared to measured water temperatures (thin line) and estimated influent concentration (thick line) with day 0 = 28 May 2001.

8.6.4. Spatial and temporal variations of bacterial concentrations

As for the experimental system, and as could be logically expected, heterotrophs are mainly present in the aerobic tanks (Figure 8.15). Counterintuitively, their abundance is higher during winter than during summer, which seems to indicate that they are not temperature sensitive. However, an similar pattern is often observed in activated sludge wastewater treatment plants, due to the lower decay rates at lower temperatures. Concentrations of heterotrophic bacteria in the anaerobic tanks are roughly one order of magnitude lower than in the aerobic ones. Heterotrophs survive in these anaerobic tanks because they receive oxygen and nitrate from the previous aerobic tanks.

Fermenting bacteria occur mostly near the inlet of the system, both in the aerobic and anaerobic tanks because they are less inhibited by oxygen than the methanogenic and sulphate reducing bacteria (Fig. 8.15).

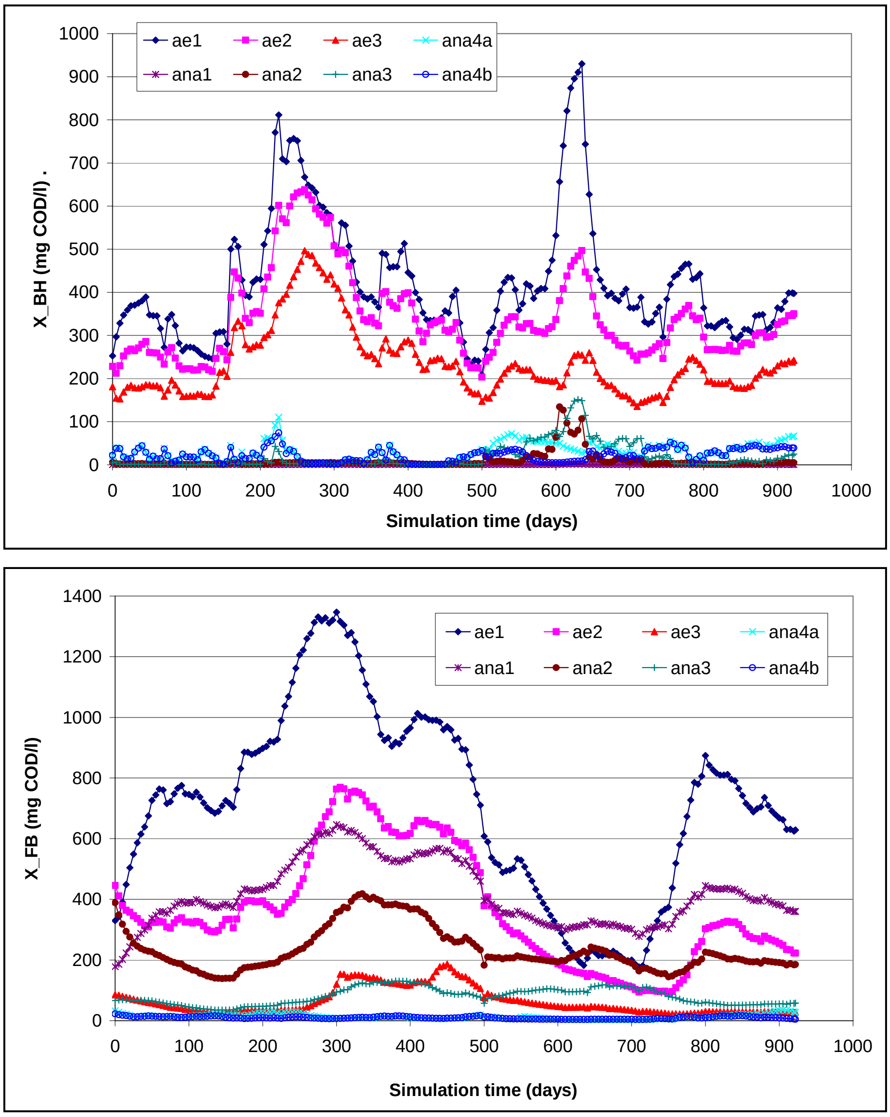

Figure 8.15. Spatial and temporal variations of heterotrophic bacteria (upper panel) and fermenting bacteria (lower panel) with day 0 = 28 May 2001.

Autotrophic, nitrifying bacteria on the contrary do not occur near the inlet, but are mainly found in the aerobic tanks ae2 and ae3. As for the heterotrophsthey increase in abundance, especially during the second winter, instead of being reduced in numbers due to colder temperatures. As was stated before, this coincides with higher oxygen concentrations in the water because of the higher solubility at low water temperatures.

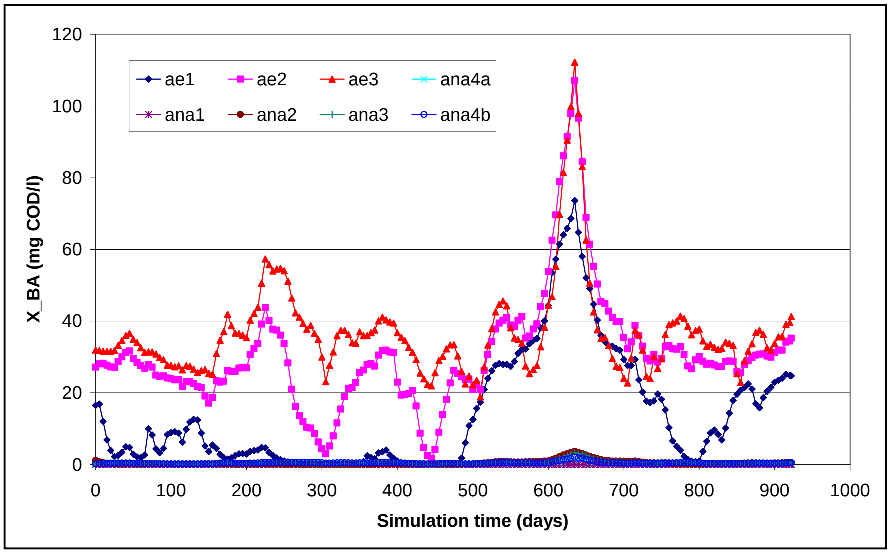

Figure 8.16. Spatial and temporal variations of nitrifying bacteria with day 0 = 28 May 2001.

Both the acetotrophic methanogenic bacteria and the acetotrophic sulphate reducing bacteria seem to be unable to grow under the specific conditions of this wetland. From initial concentrations of 1 mg COD l^{-1} or lower, they decline to concentrations near zero for all tanks.

Hydrogenotrophic methanogenic bacteria are present in insignificant numbers as well, except in anaerobic tank 2 where one peak of 27 mg COD l^{-1} can be noticed around day 450 (summer 2002). Hydrogenotrophic sulphate reducing bacteria are only present in significant quantities in anaerobic tanks ana2 (fluctuates between 20 - 100 mg COD l^{-1}) and ana3 (fluctuates between 10 and 40 mg COD l^{-1}).

The Thiobacillus group does not seem to thrive under these conditions and reaches maximum quantities of only 5 mg COD l^{-1}.

8.6.5. Estimated oxygen transfer rates

As for the experimental system, root oxygen release and physical reaeration are considered as one lumped process. The estimated oxygen transfer rates are much lower than those of the experimental system and vary between 4 and 9 g O_2 m^{-2} day^{-1}. Oxygen concentrations in the effluent are consistently below 1 mg l^{-1}.

8.7.6. Estimated porosity variations

Figure 8.17 summarises the porosity variations based on estimated densities and water contents of the joint COD fractions. As is often observed in reality, solids accumulate mainly near the inlet.

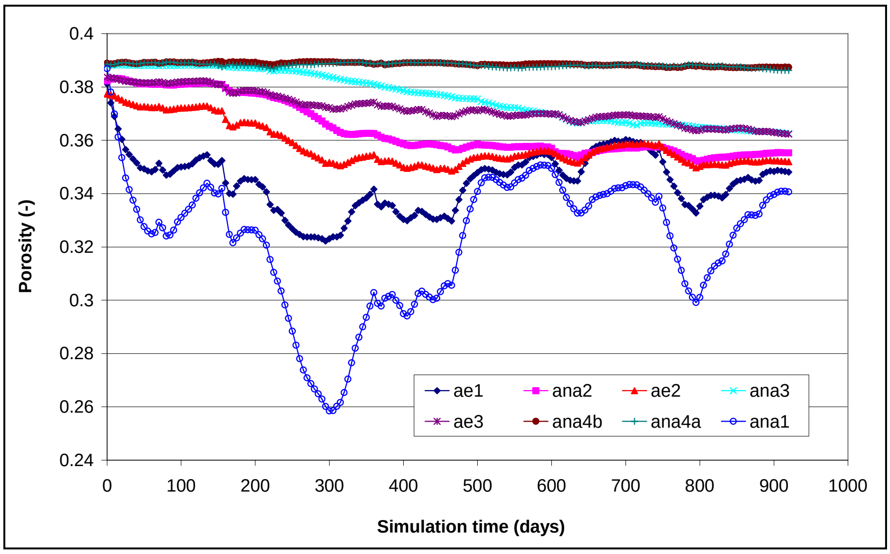

Figure 8.17. Spatial and temporal variations of porosity with day 0 = 28 May 2001.

8.7. DISCUSSION

8.7.1. Parallel tank approach

The parallel tank approach, with both aerobic and anoxic tanks was found to be the only possible solution to recreate the typical mosaic of redox conditions that can be found in the rootzone of a HSSF CW. Establishing the correct volume ratio between both was done via trial and error in this study. However, from the experimental data of Les Franqueses del Vallès (García *et al.*, 2004b) it became apparent that the reed beds with similar aspect ratios, the same surface area but different gravel layer and water depths exhibited equal removal efficiencies, despite the unequal hydraulic residence times. Excavations showed that the roots of *Phragmites australis* only penetrated up to a maximum of 15-20 cm, which is for unknown reasons less than the usual 50-60 cm. Knowing that aerobic processes are far more efficient than anaerobic ones, one might assume that the contribution of the lower (supposedly anaerobic) layers towards pollutant removal are therefore low. It it thus hypothesised that the volume ratio between aerobic and anaerobic tanks might be correlated with the rooting depth of the plants.

Other authors circumvented the 'mosaic' problem by using spatially explicit models. Langergraber (2001) for instance used a 2D model, thus allowing oxygen concentrations to form gradients, eventually leading to anaerobic zones. For a more detailed explanation, see also Chapter 5. In earlier attempts to model surface-flow wetlands (Rousseau, 1999) and HSSF CW (De Wilde, 2000), the biofilm model of Rauch *et al*. (1998) was applied. The latter model allows to define several layers inside the biofilm according to the availability of oxygen and nitrate. These two alternative approaches were however not used in the current model. The 2D model was not used because of calibration difficulties of the hydraulic submodel and because of the absence of particulate processes in that model. The biofilm model was not used because it only contains aerobic and anoxic processes and most importantly because it requires even more parameters than the currently developed model and it requires reliable data on biofilm thicknesses.

Wynn and Liehr (2001) encountered similar difficulties with their model, and solved the 'mosaic' problem by defining a time-variable aerobic and anoxic fraction of heterotrophs depending on the bulk oxygen concentration. The latter relation was also based on an empirical approach rather than on a scientific basis.

8.7.2. COD, N and S transformations

COD predictions were generally in good agreement with the measured data, except for some peaks in the experimental system and a moderate underestimation in the pilot-scale system. Some peak predictions were also noticed in the Saxby model study (Chapter 6), which seems to indicate that HSSF CW have a higher buffer capacity than is mathematically modelled here. Especially at higher hydraulic loading rates, it is not inconceivable that the mixing conditions change due to higher turbulence, which implicates that a lower number of tanks-in-series is needed during such events. Note that a similar observation was also made for the predicted NH_4-N effluent concentrations.

Data from the experimental system indicated that a lower HRT had no substantial influence on the COD removal efficiencies: a decrease from 92.7 to 89.5% was noted when the HRT decreased from 3 to 2 days. Model results lead to similar observations: efficiencies dropped from 89.1 to 86.5%. The mechanism behind this is a slightly higher oxygen transfer rate, partly physically due to the higher turbulence and partly biologically due to the higher oxygen demand, which allows the heterotrophs to grow in larger numbers. Indeed, the 33% increase in COD loading rate combined with the higher oxygen availability results in a 50 to 60% increase of heterotrophs in the aerobic tanks.

A lower HRT on the contrary did have a significant impact on TN removal: observed TN removal efficiencies dropped from 92.6 to 74.8%, the simulated ones from 84.2 to 70.5%. Clearly, the extra oxygen input is most beneficial to the heterotrophs, and not to the nitrifiers. One can see that the nitrifying bacteria in the first aerated tank are outcompeted by the heterotrophs. Only in the number 3 tanks they show a significant increase in concentration.

The overall predicted removal efficiency of 80.8% for the pilot-scale system at Les Franquèses del Valles slightly exceeds the measured one which amounted to 72.3% due to the lower predicted than observed effluent concentrations during the last 200 days.

As mentioned before, plant growth was suboptimal during this last growth season and this might have affected COD removal.

Concerning sulphate removal, the results from both systems seem to indicate that the related processes can be quickly switched on and off, depending on loading conditions and temperatures. Acetotrophic sulphate reducers are outcompeted in both systems, so sulphate removal mainly occurs by mediation of hydrogenotrophic sulphate reducing bacteria. The reverse pathway was foreseen in the model (sulphide oxidation by the *Thiobacillus* group), but seems to be of minor importance. There is however some uncertainty about the H_2S volatilisation process and this might affect the growth rates of *Thiobacillus* since H_2S is their main substrate.

The main lesson learned from the sulphur cycle predictions is that sulphate can play an important role in HSSF CW and it is thus being recommended that this variable should be more routinely monitored. It is further worth mentioning that metals like Fe and Mn play a role in the sulphur cycle (e.g. Vymazal *et al*., 1998b). As this would however further complicate the model and data on Fe and Mn concentrations are rarely available, these processes were ignored in the current model.

8.7.3. Variations of bacterial concentrations

Bacterial concentrations in both systems were more or less as expected, i.e. dominance of heterotrophic bacteria and fermenting bacteria and strictly aerobic bacteria (such as nitrifiers) mostly present in the aerobic tanks. Sulphur oxidising bacteria, acetotrophic sulphate reducing bacteria and hydrogenotrophic methanogenic bacteria on the contrary were only present in very low quantities. As explained before, for *Thiobacillus* some uncertainty exists with respect to the volatilisation of its main substrate H_2S. For the acetotrophic sulphate reducing bacteria and hydrogenotrophic methanogenic bacteria, some assumptions had to made on their inhibition by oxygen, nitrate and temperature which could have an important effect on their simulated growth rates.

The fact that higher concentrations of heterotrophs occurred during the colder winterperiods might first strike as strange, but then many authors have proven that BOD c.q. COD removal is quite temperature insensitive. One way to explain this might be the lower decay rates at lower temperatures. There is some experimental evidence from natural reed stands that heterotrophic bacteria do not follow the annual temperature cycle. Indeed, Roos and Trueba (1977, in Samson, 1994) found 33% of all bacteria to be heterotrophic during fall, in contrast to only 3.4% during spring and 12% during summer.

8.7.4. Oxygen transfer rates

The experimental system clearly shows higher oxygen transfer rates than the pilot-scale one, i.e. 15-20 versus 4-9 g O_2 m^{-2} day^{-1} respectively.

Literature data on root oxygen losses were summarised in Chapter 3 and ranged between 0.02 and 12 g g O_2 m^{-2} day^{-1}. Taking into account that the calculated oxygen transfer rates from both HSSF CW lump root oxygen loss and physical reaeration, and when assuming similar root oxygen leakage rates, it becomes apparent that the physical reaeration is much higher in the experimental than in the pilot-scale system. This would not seem to be illogical, given the different loading modes, i.e. batch-wise versus continuous-flow. Prior manipulation of the 20 or 30 liter of wastewater used as influent for the experimental system might already introduce some oxygen in the wastewater. Secondly, pouring the wastewater into the experimental HSSF CW in a brief period of time certainly creates a lot more turbulence than the continuous introduction of wastewater in the pilot-scale HSSF CW.

The higher turbulence in the experimental system might also have a side effect, i.e. more washout of solids from the first to the subsequent tanks could occur, thereby increasing the oxygen demand throughout the wetland. This could stimulate root oxygen release.

Although not evident from the effluent oxygen concentrations, O_2 concentrations in the individual aerobic tanks fluctuate considerably and seem to be quite sensitive to the water temperature because of its influence on the oxygen solubility. Especially during the second winter with water temperatures going down to as low as 6 °C, oxygen transfer rates are predicted to be too high. The higher oxygen concentrations in turn cause inhibition of anoxic heterotrophic growth and therefore higher effluent nitrate concentrations.

With the current knowledge and the current model structure, it was unfortunately not possible to distinguish between physical reaeration and plant root oxygen losses. Many studies have tried to quantify root oxygen loss, but it was found to be too dependent on factors such as oxygen demand, redox conditions, plant species, plant age etc. It is therefore proposed to focus experimental work on unravelling the relations between flow rates, water depth, gravel size on the one hand, and physical reaeration on the other hand. Methodologies using tracer gases such as propane have a.o. been described by Boumansour and Vasel (1998). Once the physical reaeration can be adequately described, the model can be applied to estimated root oxygen loss.

8.7.5. Estimated porosity variations

Estimating the solid volumes c.q. porosities requires some assumptions about the density of organic material and its water content. However, since these are constants, the absolute values of the porosity are dependent on them, but the trends are not. The variations in the pilot-scale system compare well with field observations, i.e. the highest solids accumulation is found near the inlet and near the bottom. For the experimental system however, there is considerable accumulation in the third tank. Simulations showed that this could be prevented by adjusting the fractions of solids washed out from each tank, but then the predicted nitrate effluent concentrations deteriorated.

These observations again seem to confirm the importance of internal particle transport and the need for additional research into these processes.

8.8. CONCLUSIONS

Taking into account the uncertainties on wastewater fractionation, considering the measurement frequencies that are low in comparison to the dynamics of the load variations often observed in reality, and taking into account that mostly default biokinetic parameter values were used, the model does a fairly good job in predicting effluent concentrations and removal efficiencies.

Of course this model cannot be used (yet) for design purposes, but it might provide a framework and a ‘language’ for discussion of experimental results between wetland scientists. It is a useful tool to obtain insights in the different interactions in a HSSF CW and in the competition by several microbial groups for substrates.

One of the main lessons learnt from this experience is that sulphate should be taken up in the suite of routinely monitored variables and that more attention should be paid to adequate wastewater characterisation c.q. fractionation.

Some important knowledge gaps were identified which might point out directions for future research. It would first of all be very helpful to obtain more information on the physical reaeration process, in relation to parameters such as water velocity, porosity, water depth, water temperature etc. One this process is better characterised, it might be more easy to define the importance of plant root oxygen release. A second research need concerns the behaviour of particulate substances inside the gravel matrix. Filtration and settling processes as well as resuspension and transport processes need to be more adequately quantified if one wants a better understanding of HSSF CW behaviour.

Chapter 9

Operation and maintenance of constructed wetlands

9.1. ABSTRACT

Although a good performance of any system starts with a correct design and construction, maintaining optimal performance during several decades requires regular checks, routine maintenance and servicing, and an appropriate response to any failures that might occur. This chapter starts by reviewing standard Operation and Maintenance tasks for different types of CWs and the frequency with which they should be carried out. Special attention is then devoted to record keeping, as this deviates quite a lot from more technical wastewater treatment plants. Finally, some common operational problems are described and a summary of troubleshooting possibilities is given.

9.2. STANDARD OPERATION AND MAINTENANCE

Maintenance and operation of constructed wetlands (CWs) is fairly easy due to the virtual absence of mechanical and/or electrical parts (Vymazal, 1998b). It is nevertheless being recommended to check larger systems (> 500 PE) on a daily basis. During this routine maintenance, attention should be focused on pre-treatment units as well as inlet and outlet structures of the reed beds. In practice however, insufficient maintenance is often observed, resulting in uneven flow distribution and consequently local overloading (see e.g. Chapter 2). Initially, treatment efficiency seems to be unaffected, but progressive deterioration of the system can irreversibly reduce the performance in the long term.

Kadlec and Knight (1996e) more or less concur and indicate that monitoring and adjustment of flows, water levels, water quality and biological parameters are the only day-to-day activities required to achieve successful performance in CWs. Other operation and maintenance activities such as repair of pumps, dikes and control structures, vegetation management, and removal of accumulated mineral solids must be carried out at much lower frequencies. Kadlec *et al.* (2000) also recommend to include cover estimates and observations concerning plant health as a routine part of operational monitoring. Because plants grow slowly and are important for maintaining the performance of wetland treatment systems, problems must be anticipated or prevented before they cause irreversible damage.

One of the continuing debates in CWs management, is whether or not the plants should be harvested. Main advantages of harvesting are: (i) export of nutrients and (ii) prevention of thick layers of dead material with stagnant water in FWS CW which are ideal pest breeding places (Greenway *et al*., 2002). Leaving the plants however also has certain advantages: (i) provision of a detritus layer that can adsorb trace metals in FWS CW, (ii) provision of a carbon source for denitrification in FWS CW and (iii) creation of an isolating layer of dead plant material on top of SSF CW during winter. The latter item means that harvesting ideally occurs just before the new growing season. Kadlec and Knight (1996j) nevertheless advise against harvesting as it may alter the ecological functioning of wetlands.

A list of needed maintenance operations on VSSFF CWs is given by Liénard *et al*. (2004), Table 9.1.

Table 9.1. Maintenance tasks for a 1000 PE VSSF constructed wetland (after Liénard *et al*., 2004).

Task	**Frequency**	**Duration**	**Total (h year^{-1})**
Gate operation, control of siphons	2x / week	5 min	9
Preliminary treatment: bar screen	1x / week	10 min	9
General inspection of filters and weed control	1x / week	15 min	13
On-going operational records	1x / week	20 min	18
Vegetation cutting on dikes and surroundings	6x / year	8 hours	48
Check-up and cleaning of the distribution system	2x / year	3 hours	6
Cleaning of the manholes	2x / year	1.5 hours	3
Cutting and disposal of reeds	1x / year	80 hours	80
Extraction of sludge	1x / 10 years	60 hours	6
TOTAL			192

The reported time consumption of 192 hours per year is valid for a 1000 PE wetland. One can see that the major time consumption is made up by vegetation management. For comparison: a 1000 PE waste stabilization pond only requires 100 hours of maintenance per year, mainly because reed cutting is unnecessary; a 400 PE VSSF reed bed system requires 103 maintenance hours per year. Especially when topography allows gravity feeding and there are no electromechanical parts, all these tasks can be carried out by unskilled operators. Similar tasks and frequencies may be expected for HSSF CWs.

Table 9.2 shows an operation and maintenance schedule for free-water-surface CWs as exemplified by Merz (2000), indicating the different tasks and frequencies at which they should be carried out. The same author also gives a similar schedule for associated wetlands' facilities, such as roads, surrounding grass land, a flow recording station etc.

Table 9.2. O&M schedule for free-water-surface constructed wetlands (after Merz, 2000).

W = weekly M = monthly E = after event B = bi-annually	O&M activity	Inlet structure	Inlet zone	Rock work	Macrophyte zone	Outlet structure	Wetland embankments
Solids accumulation	Solids removed over 75 mm depth. Determined by regular inspections and depth measurement. Do not damage margin vegetation during removal. Dispose solids in approved location.		E,M				
Debris	Remove from inlet zone, macrophyte zone and ensure outlet weirs are clear. Dispose debris in approved location.		E,M	E,M	E,M	E	
Scour damage	Undertake inspections following events, report and undertake remedial work to structures, earthforms and vegetated areas. Repair any bank erosion. Minimise disturbance to vegetation.	E,M	E,M	E,B		E,M	E,M
Noxious plants	Identify invasive and noxious weeds and remove preferably by physical means. Apply chemical eradication methods using approved methods and chemicals. Wetland level may be temporarily lowered to help identify nuisance weeds. Dispose weeds in an approved location.		M		M		
Harvesting need	Floating plants should be drawn off if very dense. Emergent plants can be cut or control burned after lowering the water level. Burning should be restricted to early spring and be of low intensity. Dispose of surplus plants in an approved location.				M		
Structure check	Inspect, report, undertake repairs	E,M		E,M		E,M	E,M
Mosquito checks	Regular inspections to identify problems. Report complaints. Regular changes in water level, native fish stocking, check on vegetation densities, avoid stagnant zones, and/or seek specialist advise from Department of Health.		M		W		
Replanting need	Replace dead wetland plants with approved species. Control water depth during replanting establishment period. Check areas tending to channelise and short-circuit and replant accordingly. Ensure minimal disturbance to existing plants during replanting.				M		
Water level adjustment	Take particular care during plant establishment phase. Make adjustments at the outlet weir structure. Assess wetland ability to cope with variations of inflow. Lower water levels prior to a forecasted wet event.				W	E,W	

9.3. RECORD KEEPING

A list of typical minimum monitoring requirements for successful operation of CWs is given by Kadlec and Knight (1996j). They stress that monitoring is one of the most important aspects of treatment wetlands operation and provides a “system-level barometer” of wetland health and performance. It is furthermore recommended - for ease of handling, for operators’ safety and to avoid creating preferential flow paths - to add boardwalks to the design to facilitate access to monitoring stations. A list of minimum monitoring requirements, compiled by Kadlec and Knight (1996) is given in Table 9.3.

Table 9.3. Minimum monitoring requirements (after Kadlec and Knight, 1996).

Recommended parameters	**Recommended sample locations**	**Minimum sample frequency**
Inflow and outflow water quality	Inflow(s) and outflow(s)	
a. temperature, DO, pH, EC, BOD_5, TSS, Cl^- and SO_4 (every system)		a. Weekly
b. NOx, NH_4, TKN, TP (as required by permits)		b. Monthly
c. metals, organics, toxicity (as required by permits)		c. Quarterly
Flow	Inflow(s) and outflow(s)	Daily
Rainfall	Adjacent to wetland	Daily
Water stage	Within wetland	Daily
Plant cover for dominant species	Near inflow, near wetland centre, near outflow	Annualy

The state of Florida has an extensive and quite strict legislation concerning monitoring of both constructed and natural treatment wetlands (Kadlec and Knight, 1996f). Pretreatment should be at least secondary for CWs, whilst nitrification and P removal are obliged for natural ones. Monitoring efforts are separated in baseline monitoring (only for natural wetlands, one year) and operational monitoring, and include water quality, sediment and biological parameters, as given in Table 9.4.

Table 9.4. Wetland monitoring requirements in Florida (after Kadlec and Knight, 1996f).

		Baseline monitoring	Operational monitoring	
	Parameter	Natural Wetland	Natural Wetland	CW
Water quality	Temperature	Monthly (diel)	Monthly (diel)	Quarterly (diel)
	DO	Monthly (diel)	Monthly (diel)	Quarterly (diel)
	pH	Monthly	Monthly	Quarterly
	EC	Monthly	Monthly	Quarterly
	Colour	Monthly	Monthly	
	$cBOD_5$	Monthly	Monthly	Quarterly
	TSS	Monthly	Monthly	Quarterly
	TP	Monthly	Monthly	Quarterly
	$o\text{-}PO_4$	Monthly	Monthly	
	TKN	Monthly	Monthly	Quarterly
	TAN	Monthly	Monthly	Quarterly
	NO_3	Monthly	Monthly	Quarterly
	SO_4	Quarterly	Quarterly	Quarterly
	FC	Monthly	Monthly	Quarterly
	Chl. a	Quarterly	Quarterly	Quarterly
	Priority pollutants (non-metallic)	Once	Annually	
	Metals (Hg, Pb, …)	Once	Semesterly	
	Water stage	Continuously	Continuously	Continuously
Sediment	pH	Once	Annually	
	TP	Once	Annually	
	TKN	Once	Annually	
	TAN	Once	Annually	
	NO_3	Once	Annually	
	S	Once	Annually	
	Metals (Hg, Pb, …)	Once	Semesterly	Annually
Biological	Benthic macroinvertebrates	Quarterly	Quarterly	
	Woody vegetation		Annually	Annually
	Herbaceous vegetation		Quarterly	Quarterly
	Fish		Quarterly	Once
	Mosquitoes		Monthly*	
	Threatened/endangered species		Annually	Annually
	Woody plant tissues (metals, TKN, TP)		After 5 year	
	Leafy and woody plant tissues (TP, TKN, Fe, Zn)		Annually	

* only during growing season (April to November)

9.4. PARTICULAR O&M PROBLEMS AND TROUBLESHOOTING

Vymazal (1998b) separates operational problems into two categories: those resulting from poor maintenance and those associated with parts of the system that were not properly designed or built. Billeter *et al*. (1998) add that problems can also result from faulty instructions by the owners, their forgetfulness or the erroneous view that low technology wastewater treatment plants do not need maintenance. Indeed natural wastewater treatment systems are frequently considered to be a ‘build-and-forget’ solution not needing any attention at all. When denied the minimal amount of maintenance even natural systems need, failing treatment systems are often reported (e.g. Chapter 2 and Chapter 10).

Severn Trent Water Ltd., one of the larger water utilities in the UK, operates more than 300 CWs for tertiary treatment. They are most often HSSF CWs preceded by a rotating biological contactor, a trickling filter, a small activated sludge plant or a submerged aerated filter. Cooper *et al*. (2004) surveyed more than 120 of these tertiary treatment wetlands and noted in many cases problems with sludge deposition, inlet flow distributor problems, outlet collector problems, weed infestation, tree growth and above-ground flow. Despite these problems, all effluents were still compliant with the regulatory consents. The authors therefore call CWs “very forgiving and abuse tolerant”. As a conclusion, they suggest that reed beds should be inspected at least once per month and more frequent if there are known problems. Weeds should be removed at 6-month intervals, as is the case with saplings in order to avoid tree roots puncturing through the plastic liner. All in all, tertiary treatment CWs of this scale (< 2000 PE) require only a few days maintenance per year.

In a similar way, Chapter 10 presents the results of a survey of 12 HSSF storm water treatment CWs of Severn Trent Water Ltd. and concludes that quick on-site surveys with a number of very simple methods provide valuable information on a range of factors that can influence the design life of reed beds. Measuring sludge layer thicknesses allows to assess solids accumulation and can act as an early warning sign for clogging. Plant heights and weed proliferation are a good visual sign of otherwise hidden water level problems. Weed control, a thorough maintenance of the inlet distribution system and a correct setting of the outlet level were identified as crucial factors contributing to the performance and the longevity of the beds. The reader is referred to Chapter 10 for more details about this survey.

Constructed wetlands are possible breeding spots for mosquitoes. Greenway *et al.* (2002) state that a wetland with a high biodiversity (no monospecific stands) and an extensive food web will cause low mosquito nuisance. Knight *et al.* (2000b) elaborate on a number of design strategies and management procedures to counteract mosquito proliferation and potentially related diseases. These strategies are especially important when the wetlands are constructed close to human settlements or in arid areas where formerly no stagnant water was present. As for many environmental problems, source reduction is the first and most important measure. Mosquitoes preferably deposit their eggs in stagnant waters containing relatively high amounts of organic matter and nutrients. It is clear that source reduction is therefore contradictory to many design principles of above-ground flow CWs and leaves little or no options at all. Whilst total prevention is not possible, adequate pretreatment and the resulting lower organic loading rate can substantially lower the numbers of mosquitoes, which is a positive argument for tertiary treatment wetlands. Subsurface-flow CWs are obviously less favourable breeding grounds unless surface flow occurs due to clogging. When the design incorporates multiple basins and associated flow paths, it is possible to periodically bypass one or more basins and empty them. Present mosquito larvae will be largely eradicated. Lower water depths will result in higher flow velocities and therefore less suitable conditions for mosquito breeding. Intermediate open water areas support the growth of predatory invertebrates and fish and therefore reduce the number of mosquito larvae. Finally, a careful bottom grading during the construction phase is absolutely necessary to prevent the occurrence of stagnant zones. Next to the above-mentioned design principles, a number of operational control measures are available to reduce mosquito nuisance. The most radical solution is chemical treatment with insecticides. This is not only an unsustainable solution, it is also a very expensive one: spraying a moderately big wetland of 10 ha by helicopter can cost up to US$ 4000.

Some biological control agents are also available: microbial ones (bacteria, viruses, protozoans and fungi) and multicellular ones (nematodes, cyclopoid copepods, predaceous aquatic insects and larvivorous fish). *Bacillus thuringiensis israeliensis* and *B. sphaericus* have been successfully used on a number of wetlands systems (Reed *et al.*, 1988). Greenway *et al.* (2002) studied four tertiary FWS CWs and hypothesized that macroinvertebrates probably are crucial for control of mosquito larvae. Macroinvertebrate densities were found to be dependent on plant species, water quality, water depth and diel dissolved oxygen concentrations. The use of *Gambusia* sp., a larvivorous fish, which is proposed by many authors, is strongly discouraged by Greenway (2002) since mosquito larvae only form a minor part of its diet and it thus feeds on other useful macroinvertebrates.

Another major threat to the wetland as a whole and the vegetation in particular are muskrats. Especially systems planted with *Scirpus* spp. or *Typha* spp. are vulnerable since the animals use the plants both as a food source and as nesting material (Reed *et al.*, 1988). *Phragmites* spp. does not seem to serve as a food source and is therefore less vulnerable. In extreme cases, trapping the muskrats might be necessary.

Kadlec and Knight (1996i) provide a list of the most common physical, chemical and biological factors that can lead to poor plant growth and propose some corrective measures.

Odour nuisance can occur in some cases, certainly in water hyacinth and duckweed ponds where the thick floating mat of plants limits oxygen input into the system. Anaerobic conditions are therefore quite likely to occur and could produce objectionable hydrogen sulphide odours when the wastewater contains high amounts of sulphates. Reed *et al.* (1988) suggest the following measures for floating plant systems: (i) provide supplemental aeration if necessary, (ii) harvest at most 20% of the plants at each time to keep the lagoon fully covered and (iii) locate the ponds at least 0.4 km from any habitation. Kadlec and Knight (1996j) reach similar conclusions for wetland treatment systems and suggest: (i) to reduce the loading rates of BOD and ammonium if needed and (ii) to create aerobic environments by means of shallow basins or by implementation of cascading outfall structures.

Table 9.5. Potential factors resulting in wetland vegetation maintenance problems (Kadlec & Knight, 1996i).

Problem	Corrective measures
Water stress (levels too low)	Raise outlet weirs, add more water, or provide supplemental irrigation to maintain adequate soil moisture
Flood stress (levels too high)	Lower outlet weirs or reduce flow to lower water levels
Macronutrient stress (N, P, K)	Fertilize as required to promote healthy plant growth
Micronutrient stress (Fe, Mg, …)	Add micronutrients as required to promote healthy plant growth
Dissolved oxygen stress a. organic loading b. ammonia loading c. smothering (sludge or solids) d. tight soils	Reduce the input of oxygen demanding substances (BOD and NH_4); lower water levels; reduce the input of solids; design with loamy topsoil to provide a suitable rooting medium
Pathogens/herbivory a. insects b. plant diseases c. mammals	Tolerate without chemical controls as much as possible. Burn during winter months to reduce insect and pathogen resting stages; trap and remove mammals as necessary
Weather/physical a. frost b. heat c. wind d. excessive ETP	Maintain flooded conditions to regulate favourable root temperatures; use suitable topsoil to provide plant stability

Clogging of subsurface-flow CWs is a tangible risk and is mainly influenced by loading rates of BOD and/or SS (and thus the level of pretreatment), the hydraulic loading rate and the particle size and distribution of the matrix material as well as the wastewater particles (Winter and Goetz, 2003). Blazejewski and Murat-Blazejewska (1997) and Kadlec and Watson (1993) also identified the following processes as important factors: (i) biofilm development, especially at higher ambient temperatures, (ii) development of an inorganic gel of Ca compounds and (iii) peptisation of soil colloids and collapsing macropores between aggregates. Clogging can be counteracted by lowering loading rates, by changing the pumping frequency (e.g. longer resting intervals in between loading) or by leaving one or more beds to rest. During this resting period, organic material that blocks the pores can be composted and the hydraulic conductivity thus restored. When most pores are filled with inorganic material and the hydraulic conductivity is too low, the only solution is to excavate the bed and either refill it with new matrix material or refill it with the same matrix material after rinsing (Cooper *et al*., 2004).

Chapter 10

Impact of operational maintenance on the asset life of storm reed beds

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10.1. ABSTRACT

This chapter reviews the operation of storm reed beds to determine whether the current system of operational maintenance is contributing to premature process failures and if not, to identify other factors of importance. Twelve storm reed beds of the horizontal subsurface-flow type, at seven locations in the South Warwickshire area of the United Kingdom, were surveyed. Each survey consisted of a site visit, an interview with the operators in charge and an assessment of the treatment performance based on routine monitoring data. Although some sites suffered from varying degrees of sludge accumulation, surface blinding and/or weed growth, all effluent concentrations remained far below the consent levels. Thorough operational maintenance on a reed bed is proven to be important for the asset life. However, there are other factors or features of a reed bed that play a more pivotal role in premature process failure such as the lack of pre-treatment and a premature operation of the storm overflow.

10.2. INTRODUCTION

Combined Sewer Overflows (CSOs) are becoming increasingly undesirable for river water quality considerations (Mulliss *et al.*, 1997) and multiple approaches have been adopted to reduce their impact (Zabel *et al.*, 2001). Storm water detention tanks are a common preventive measure but at small-scale wastewater treatment plants they are unpopular with the water companies because they require additional site visits and attendance time. As a consequence, operating costs can increase considerably. Another potential drawback of detention tanks is the virtual absence of pollutant removal processes because of very short hydraulic residence times. This concept is therefore increasingly being abandoned in favour of storm water treatment facilities (Griffin and Pamplin, 1998). Whilst CSO treatment options are multiple (Geiger, 1998), this paper focuses on CWs as they present an eco-friendly and cost-effective solution in rural areas to minimize CSO effects on the receiving water course (Scholes *et al.*, 1999; Carleton *et al.*, 2001).

Severn Trent Water is the world's fourth largest privately-owned water company - serving over 8 million customers across the heart of the UK, stretching from the Bristol Channel to the Humber river, and from mid-Wales to the East Midlands. The company has more than 700 facilities serving populations of less than 2000. About 200 of these facilities rely on rotating biological contactors (RBCs) for wastewater treatment. A policy decision has been taken to provide capacity in the RBCs for 6 times dry weather flow (DWF). Higher flows are firstly routed through a Copasac™ chamber fitted with bags made of woven polypropylene with a 2 to 10 mm mesh that are most effective in capturing plastic and other floatables. Further treatment occurs through storm reed beds of the horizontal subsurface-flow type where a surface of about 0.5 m^2 PE^{-1} is provided (Green and Martin, 1996). This process flow sheet is visualized in Figure 10.1.

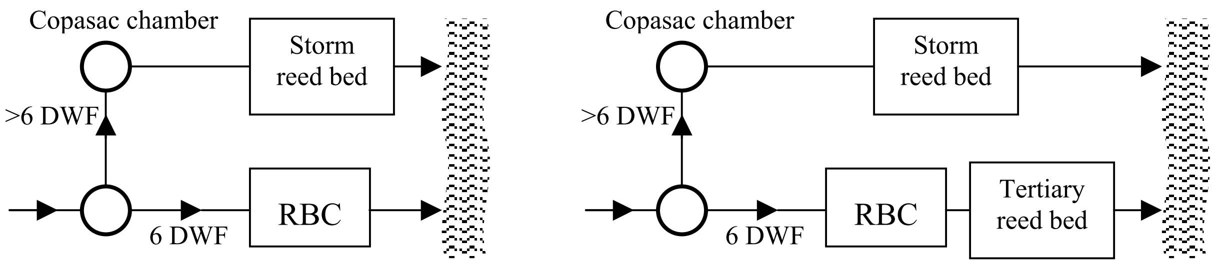

Figure 10.1. Process flow sheets indicating deployment of storm reed beds (Griffin and Pamplin, 1998).

Whilst design and performance of storm reed bed systems have formerly been positively evaluated (Green and Martin, 1996), little is known about their optimal management and most importantly about their design life expectancy. Operational problems and premature failure are therefore not uncommon.

The life expectancy of CWs is defined by Bavor *et al.* (1995) as the period of time over which sustained pollutant removal can be achieved at the mean loading rate. For HSSF CWs it seems to be mainly limited by accumulation of mineral solids in the pore space, mainly near the bottom of the gravel bed. Hydraulic conductivity is therefore less impacted than in the case of uniform pore blockage (Kadlec *et al.*, 2000).

10.3. AIM OF THE STUDY

The storm reed beds that were surveyed during this study date from the early 1990s and there are already some indications that they will not last their expected asset life of 20 years. Consequently there is a need to investigate the factors influencing the design life of storm reed beds and especially the rate of solids accumulation and degradation. The working life of the reed bed should match the physical life of the assets, otherwise there is a danger of early write-off of these CWs.

10.4. MATERIAL AND METHODS

Twelve storm reed beds at 7 locations in the South Warwickshire area of the United Kingdom were surveyed. All reed beds are of the horizontal subsurface-flow type. They have been filled with pre-washed 5-10 mm gravel and planted with *Phragmites australis*. The inlet distribution system consists of a number of equidistant vertical riser pipes. Other basic design features are summarized in Table 10.1. All reed beds are operated by Severn Trent Water Ltd.

Each survey consisted of a site visit, an interview with the operators in charge and an assessment of the treatment performance through time using routine monitoring data.

Table 10.1. Basic design features and consent levels of the investigated storm reed beds. Consents are expressed in the following order: mg BOD l^{-1} / mg SS l^{-1} / mg NH_4-N l^{-1}.

Location	Design size (PE)	Number of reed beds	Total reed bed area (m^2)	Year of construction	Summer effluent consents
Napton	947	1	595	1992	15 / 25 / 10
Snitterfield	1,172	2	2,368	1994	15 / 25 / 5
Lighthorne Heath	1,154	2	700	1992	10 / 20 / 5
Fenny Compton	599	1	500	1993	10 / 15 / 5
Ettington	822	2	750	1993	15 / 25 / 5
Ilmington	701	2	780	1992	15 / 25 / 5
Bearly	709	2	1,408	1993	25 / 45 / 10

Site surveys

For each site surveyed a data collection form (DCF) was devised in order to gather data from the field. The parameters investigated were:

- *General data*: data concerning age, dimensions, capacity (as PE) and type were collected from the Severn Trent Water reed bed data spreadsheet and checked on site.
- *Reed growth*: reed heights were roughly estimated at 15 different spots in each reed bed according to the following grid: 0, 50 and 100% of the bed width and 0, 25, 50, 75 and 100% of the bed length. The outlet of the bed corresponds to 100% width, 100% length.
- *Reed density*: reed density was assessed as low, medium or high, based on the surveyor's experience and *inter*-site comparison.
- *Reed condition*: reed condition was subjectively assessed as poor, good or excellent, based on the two previous indicators as well as on signs of chlorosis, and *inter*-site comparison.
- *Sludge depth*: the sludge layer thickness on top of the gravel bed was measured by dipping a rule into the ground until it hit the gravel surface. This depth of sludge and leaf litter was then recorded. Measurements were carried out at 15 different spots in each reed bed according to the above-described grid.
- *Weed growth*: In order to measure the percentage weed cover, general observations were made by walking around and through the reed bed taking note of the position(s) of the weeds in pictorial form and estimating how much of the total reed bed was actually covered by weeds.
- *Site-specific issues*: issues like high infiltration of groundwater into the sewerage, flow split problems, rag/solids problems or remediation were assessed on site or obtained from the operators.
- *Depth of water*: in normal conditions, the wastewater level should be some 6 cm under the gravel surface. This level might be raised from time to time for weed control purposes. The water depth was measured by means of a rule in case of surface water or by digging a small pit and measuring the depth of the water table. This was again carried out at 15 different locations according to the above-described

grid. If no water was encountered at 6 cm below the gravel surface, no further digging was carried out and the water depth was noted as > 6 cm.

- *Flow distribution*: the type and number of inlet structures was indicated on the DCF schematically: vertical riser pipes, horizontal slotted pipes or troughs/channels. Finally, the flow distribution was assessed based on factors such as clogged pipes, unequal flows out of different pipes and a visual inspection of moist patches in the inlet zone.
- *Primary treatment*: presence of screening and pre-settlement units was indicated on the DCF.

Interview with the operators

The operators were asked closed questions with a limited number of answering options for ease of evaluation and analysis. The questions asked were: How often are the reed beds inspected? How often is the flow distribution inspected? Have the reeds ever been cut down or removed? How often is the inlet and outlet cleaned and how? Is there any weed control and, if so, what type and when was sludge last removed from the bed?

Data treatment

BOD, SS and NH_4-N effluent concentrations collected over the last couple of years, obtained from the Severn Trent Water performance database, were checked against the consents and were also graphically interpreted using MS Excel™ to determine whether or not there were any clearly visible trends. Other data were graphically interpreted using MS Excel™. The Spearman's Rank Correlation Coefficient Test was used to identify correlations between the averages of two variables.

10.5. RESULTS AND DISCUSSION

Surface area. Surface areas range from 0.6 m^2 PE^{-1} at Lighthorne Heath and Napton to 1.99 m^2 PE^{-1} at Bearley. Therefore all storm beds surveyed have a larger surface area than the optimum of 0.5 m^2 PE^{-1} recommended by Green and Martin (1996). The advantage of these storm beds having a larger than recommended surface area is that they provide increased retention time. Thus in theory, they may produce better effluent quality. They also have increased treatment capacity which may be useful in the future if further development occurs in the catchment area.

Pre-settlement. Most studies advise pre-settlement of wastewater before it enters the reed bed system in order to reduce the sewage strength which, if too high, may cause problems with plant growth and also to reduce solids which may dramatically shorten the system life by clogging the pores. None of the studied storm reed bed influents is however subjected to pre-settlement, although they would greatly benefit from it. If settlement tanks were to be constructed they would require considerably more maintenance than the reed beds. The storm tanks would need to be emptied and cleaned out in order to prevent septic conditions which could lead to odour problems on site. This type of maintenance is very labour intensive and time consuming which would negate the benefits offered by reed beds (Griffin and Pamplin, 1998).

Plant height (Figure 10.2). Reeds are strongly inhibited at Napton and Ilmington I and II, with an estimated 30 to 60% of the bed surface now covered by weeds. Lighthorne Heath I also shows significant reed growth inhibition but weed coverage is still low (approximately 5% of the bed surface) which suggests that the decline of the reed stand only started recently.

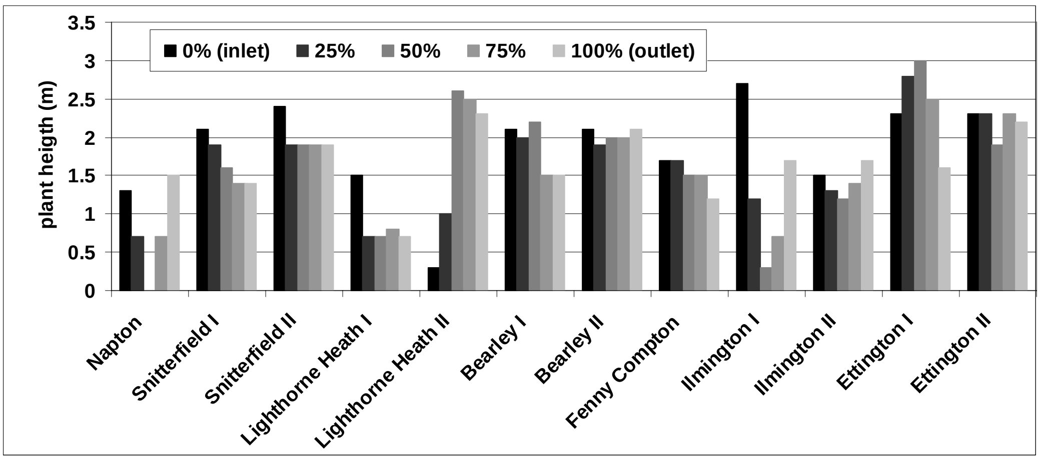

Figure 10.2. Reed heights at different locations between inlet and outlet of twelve UK storm reed beds. Bars represent averages of 3 reed height measurements at 0%, 50% and 100% of the bed width.

Small patches of weeds near the inlet zone of Lighthorne Heath II seem to have outcompeted reed plants. The most abundant plant growth was observed at Ettington I. Another important observation is that reed plants tend to be shorter near the outlet side. Kadlec and Knight (1996) indeed suggest that macronutrient limitations might occur in the downstream areas of a wetland.

Sludge accumulation (Figure 10.3). A mixture of sludge and leaf litter has accumulated on all parts of the Ettington I and II reed beds and is of particular concern since it has penetrated into the outlet zone. This implies a chance of sludge washout during storm events and a possible breaching of the effluent consents. Ilmington I and II, in contrast, only have considerable sludge accumulation in the inlet zone and sludge washout is thus not likely to occur in the near future. It nevertheless hinders a good influent distribution over the entire bed width. There does not seem to be a design-related explanation for both cases since the provided area is more than sufficient (0.91 m² PE^{-1} at Ettington and 1.11 m² PE^{-1} at Ilmington) and their length/width ratio also corresponds to commonly accepted design guidelines (0.4 for both systems). Influent loads might thus be higher than expected and/or the storm overflow operates prematurely. All other reed beds seem to be relatively unaffected by sludge accumulation.

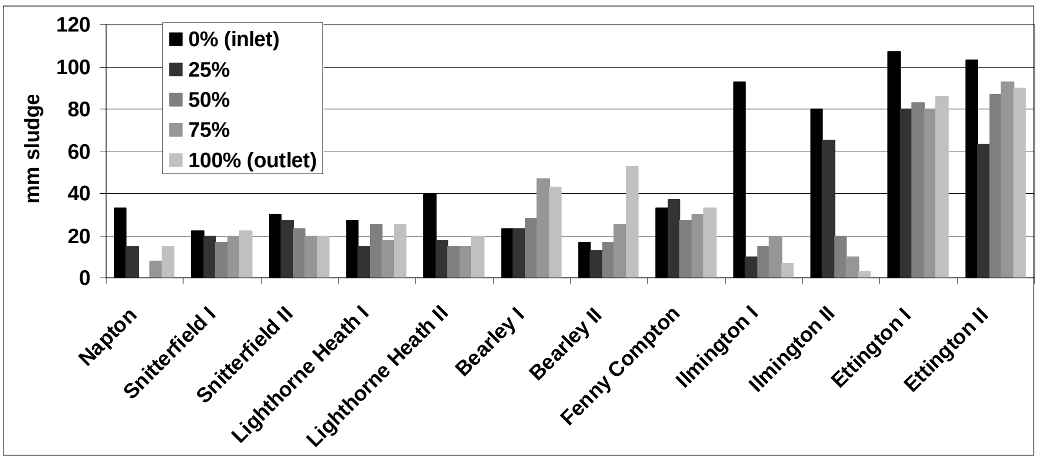

Figure 10.3. Sludge depths at different locations between inlet and outlet of twelve UK storm reed beds. Bars represent averages of 3 sludge measurements at 0%, 50% and 100% of the bed width.

Water level (Figure 10.4). At Napton, Snitterfield I & II, Lighthorne Heath I & II, Bearly II and Fenny Compton, the water level remains at least 60 mm below the gravel bed surface. At Bearly I, water levels are closer to the gravel surface but remain underground. Surface water occurs at Ilmington I & II, but only in the inlet zone, due to sludge accumulation. Only Ettington I & II are struck by serious surface blinding.

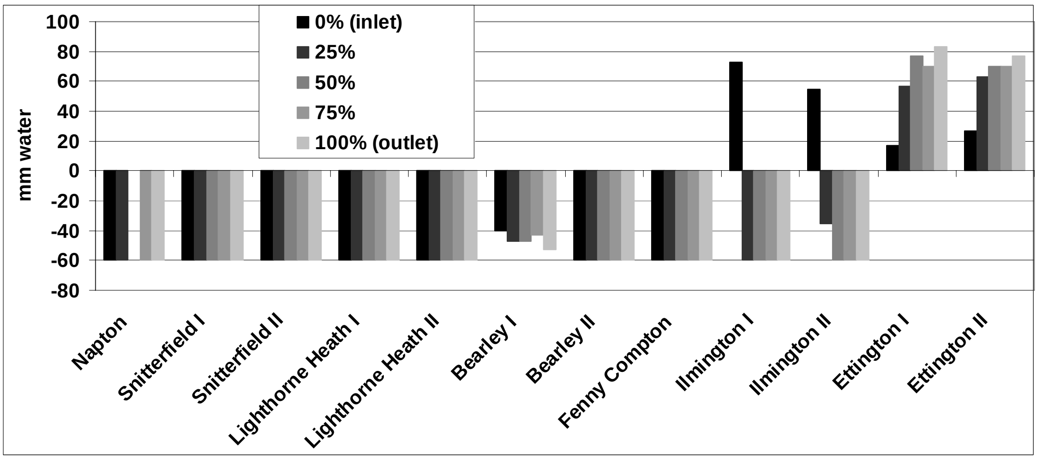

Figure 10.4. Water depths or water heights at different locations between inlet and outlet of twelve UK storm reed beds. Bars represent averages of 3 measurements at 0%, 50% and 100% of the bed width. Water levels lower than 60 mm under the gravel surface are represented as – 60 mm.

Correlations. Averaged variables were compared using the Spearman's Rank Correlation Coefficient. Since all reed beds were put in operation shortly after each other, age was proven to be no factor in this study for reed growth, sludge accumulation nor water level. No significant correlations were furthermore found between reed growth and sludge accumulation or water level. Only the water level proved to be highly correlated with the sludge accumulation ($P < 0.01$). Indeed, water surfaces in the inlet zones of Ilmington I & II, which coincides with significant sludge accumulation in these zones. Surface blinding at Ettington I & II correlates with pore blockages due to excessive sludge quantities.

Operation, maintenance and management (Figure 10.5). Most storm reed beds are inspected biweekly or monthly. This frequency is lower than the one recommended by

Vymazal (1998b) but is probably adequate since storm reed beds operate discontinuously. Confusingly, at 6 out of the 12 reed beds, inspection of the flow distribution is claimed to be carried out only occasionally, whereas at 11 out of the 12 reed beds, cleaning of the inlet is claimed to be done at least once per month. This can however be explained by a different perception of the concept 'cleaning' between operators and surveyors. Some surveys indeed revealed that nearly half of the vertical riser pipes in the inlet zone were blocked by plant debris and sludge, which was clearly not the result of one-month's accumulation. Reed cutting and removal as well as sludge removal are not a standard policy of Severn Trent Water Ltd. and have therefore never been done until now. However, considerable sludge accumulation at the storm reed beds of Ilmington and Ettington will probably need to be counteracted by desludging and consequent replanting of the beds.

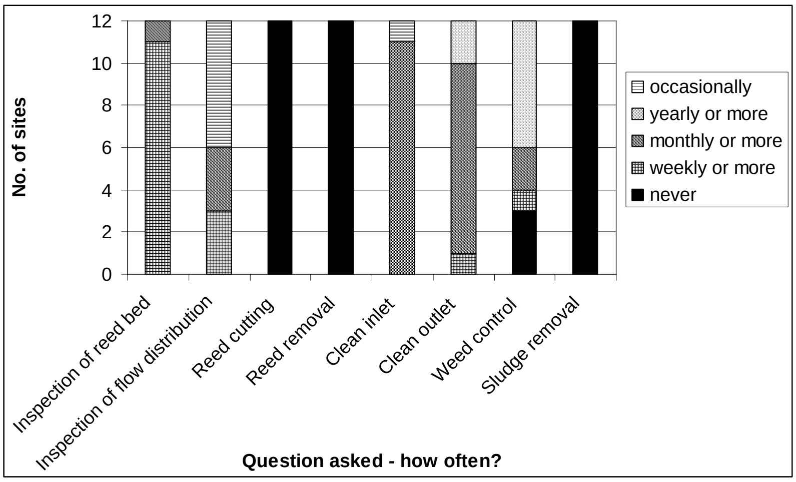

Figure 10.5. Frequency of inspection and maintenance of 12 storm reed beds.

Treatment performance. All storm reed beds were proven to perform exceptionally well. Data gathered from 2000 till 2002 (at least 30 effluent samples per location) clearly demonstrate that all effluent concentrations are far below the consent levels (cf. Table 10.1). Varying degrees of sludge accumulation, weed growth, surface blinding and unequal flow distribution therefore seemed to have only minor effects on the treatment performance of the selected storm reed beds.

10.6. CONCLUSIONS AND RECOMMENDATIONS

Quick surveys with simple methods, as in this study, have been proven to provide valuable information on a range of factors that can influence the design life of storm reed beds. Measuring sludge layer thicknesses provides an assessment of solids accumulation and can act as an early warning sign for clogging. Plant heights and weed proliferation are a good visual sign of otherwise hidden water level problems.

Operational maintenance is an important factor in ensuring longevity of a reed bed. However, observations from the on-site surveys indicate that it is not the frequency with which the maintenance activities are being undertaken that is having an effect on the performance of reed beds, but the thoroughness with which these tasks are being carried out. This concurs with the conclusions of Cooper *et al*. (1996), Billeter *et al*. (1998) and others, that natural treatment systems are frequently but wrongly considered to be a ‘build-and-forget’ solution and thus do not need any attention.

All of the sites surveyed would no doubt benefit from pre-settlement, especially those sites that suffer from very high sludge accumulations in the inlet zone of the bed. However, if settlement tanks were to be constructed they would require considerably more maintenance than the reed beds. This type of maintenance is very labour-intensive and time-consuming which would negate the benefits offered by reed beds.

Other factors or features of a reed bed also play a role in premature process failure and are thus important to the asset life. It is apparent that at some sites the storm overflow operates prematurely. This not only causes strong sewage to be applied to the bed, deteriorating the effluent quality but the life of the bed may be dramatically shortened due to excessive sludge accumulation.

Weed control, sufficient screening of the influent, a thorough maintenance of the inlet distribution system and a correct setting of the outlet level were identified as crucial factors contributing to the performance and the longevity of the beds.

Chapter 11

General discussion, conclusions and perspectives

11.1. INTRODUCTION

Constructed wetland technology – commonly known as reed bed technology - emerged during the 1950s in Germany and has for many years been considered a marginal technology with limited applicability. Gradually however, experience with full-scale systems and innovative experimental set-ups led to sometimes radical changes in design and operation and an ever-increasing application of this technology (Vymazal, 1998a). A non-exhaustive list of uses was given in Chapter 1 and encompassed domestic, agricultural and industrial wastewaters, often containing mixtures of organic and inorganic substances in varying concentrations. Removal of all these pollutants can only be accomplished by a vast array of biological, physical and chemical processes, as was explained in detail in Chapter 3.

Despite the massive number of papers on natural systems for wastewater treatment, many knowledge-gaps still exist. Indeed, until recently, field-scale research concentrated rather on pollutant removal efficiencies and mainly tried to relate the observed performances to influencing variables such as HRT, temperature etc. without much speculation on the basic processes behind the observations. Recent investigations do focus more and more on pollutant dynamics but this research still tends to be very fragmented and is often carried out on a lab-scale, making it difficult to extrapolate the outcomes to a larger scale. As a result, many quantitative data have been assembled without the necessary theoretical foundations. A structured approach is thus absolutely needed to optimise the design and efficiency of these natural wastewater treatment systems.

In view of this, the three major contributions of this thesis are:

1. Higher-than-usual sampling frequencies have been applied in an attempt to attain more insights in the dynamics of CWs. To this purpose, both a pilot-scale 10 PE two-stage combined constructed wetland (VSSFF + HSSF CW) and a 47 PE two-stage constructed wetland (HSSF + HSSF CW) were monitored.

2. A new conceptual model framework for interpreting carbon, nitrogen and sulphur cycles in a HSSF CW has been developed and proved to be a valuable tool for interpreting experimental data and for identifying knowledge gaps.

3. Strong arguments are given to apply a minimum maintenance effort as even well-designed constructed wetlands can fail when denied adequate maintenance. A literature review on minimum monitoring efforts and O&M tasks was used to deduce a number of guidelines for CW operation.

11.2. PERFORMANCE

11.2.1. Survey results

The 10 PE pilot-scale combined CW (Aartselaar, Belgium) as well as the 47 PE two-stage HSSF CW (Saxby, UK) showed some remarkable similarities. Both systems firstly showed a very high buffering capacity as effluent concentrations only vaguely reflected drastic changes in influent hydraulic and/or organic loading rates. Secondly, for all pollutants except nitrogen, the contribution of the second stage wetland to removal was only minor and they mostly functioned as a sort of backup system in case the first stage became overloaded. However, for the combined VSSF and HSSF system, nitrification and denitrification clearly took place in the separate stages 1 and 2 respectively, thereby rendering the HSSF CW indispensable for a good TN removal. Another important observation was that denitrification only reached high rates after the first winter of operation and it was therefore speculated that decaying plants and litter are a major carbon source for denitrification as most COD present in the wastewater was already removed in the first stage.

11.2.2. Design and operation recommendations

Although, due to frequent clogging problems, the concept of combined constructed wetlands (VSSF + HSSF CWs) has been abandoned in Flanders in favour of HSSF CW, results from the pilot-scale wetland and from the wetland survey in Flanders (Chapter 2) indicate that such combined systems yield one of the highest possible removal efficiencies. This corresponds with the statement of Vymazal *et* al. (1998b) that the effluent quality appears to improve with the complexity of the facility, a statement also backed up by the most recent literature (Belmont *et al*., 2005). It is however also the most technically complex system that results in higher investment costs. Some recommendations are nevertheless given on the design and maintenance of these combined systems which could help to prevent clogging and to optimise treatment performance:

- fine gravel should be used as matrix material for the VSSFF bed instead of coarse sand;
- a larger influent distribution network on the VSSFF CW is needed to ensure adequate spreading of the wastewater over the entire surface area;
- harvesting the plants from the VSSF beds after each growth season seems necessary to prevent the resulting litter from clogging the pores in the upper layers. As the dead plants create an isolating layer, harvesting is preferably done after the winter;
- diverting a 10 to 20% portion of the influent to the second stage HSSF CW not only reduces the loading rate of the VSSF CW and thus reduces the risk of clogging, but it also ensures that there is a carbon source available for denitrification in the second stage. Field observations often showed reduced plant growth in the second stage reed beds and diverting some primary treated wastewater with higher nutrient contents to the HSSF CW might solve this problem;
- mixing straw or another carbon source with the gravel in the HSSF CW will ensure that there is a carbon source for denitrification during the start-up phase. One should however take care not to reduce the hydraulic conductivity too much;
- sustained phosphorus removal seems only possible by addition of an extra treatment step such as a small filter bed containing matrix material with a high P-sorption capacity (Norvee *et al*., 2004; Seo *et al*., 2005).

11.2.3. Monitoring recommendations

As both studied wetland systems proved to have a very high peak-shaving efficiency, rather low effluent sampling frequencies in the order of several days are still acceptable, except when extreme loading events occur. In such case, a flow meter with a certain threshold value could be used to trigger a higher sampling frequency. Influent variations on the contrary were quite extreme, as also mentioned by Boller (1997) and sampling frequencies here should be rather in the order of several hours. For mass balance purposes one could however also opt to collect flow-based composite samples. In any case, flow data are very important when a reliable estimate of the CW performance is needed.

When collecting data for the purpose of model calibration, one should make sure to measure, if reasonably possible, all required model inputs. This mostly includes fractionation into particulate and dissolved COD/BOD, readily and slowly biodegradable and inert COD, reduced and oxidised nitrogen species, organic and inorganic phosphorus species and according to the recommendations of Chapter 8 also sulphate. Sulphate was not measured in the Aartselaar and Saxby systems, mainly because of the timeline of the research in this thesis. However, because the pilot-scale system included a VSSF CW which typically introduces a lot of oxygen in the system, it is hypothesised that redox potentials throughout the bed were not low enough to trigger sulphate reduction. For the Saxby system it might have been more interesting to follow sulphate concentrations although the organic loading rate was quite low and the redox potentials inside the bed therefore might also have been higher than the sulphate-reducing range.

A valuable data set for model calibration is usually also one with a high information content. As it was proven that wetlands are quite insensitive to small load variations, it is recommended to incorporate some extreme events in the monitoring campaign. For field-scale campaigns, one might await a storm event in order to evaluate the systems' behaviour under higher hydraulic loading rates. As for COD, N and P loading rates, it might still be practicable for CWs up to several hundred PEs to artificially spike the influent with a cheap COD, N and/or P component and to evaluate its impact on the effluent concentrations.

11.3. MODEL-BASED EVALUATION

As demonstrated before in the 'Performance' section and many other literature sources, wetlands do a good job in treating wastewater, but the underlying mechanisms are still rather speculative. Many researchers have attempted to capture CW behaviour in simple models like regression equations and the k-C* model, and although they often perform quite well, they remain black-box models, unable to explain the internal mechanisms. This became very clear in Chapter 5 when examining the parameter values proposed in different literature sources. The variability was very high because many influencing factors are not accounted for in these models. As a result, when attempting to apply these models for design purposes, the predicted required surface areas vary within a range of magnitude of 10^4.

Mechanistic models are thought to be very useful to render the black box white, and after a SWOT analysis of several of these models, the one of Wynn and Liehr (2001) was adopted and adapted to simulate the measured performance of the Saxby CW, already discussed before. The one major advantage of this model is that it uses routinely measured variables like BOD and NH_4 as inputs. However, such approach required several assumptions on and simplifications of the wetland processes. BOD and nitrogen mass balances were therefore not closed and this renders interpretation of the model outputs quite difficult. Also, processes affecting particle concentrations in the wastewater were completely ignored, thereby making the model unfit for predicting clogging effects. Finally, recent papers (Baptista, 2003; García *et al.*, in press) stress that anaerobic processes play an important role in HSSF CWs and these were lacking in the model of Wynn and Liehr (2001).

It was therefore decided to develop a new conceptual model framework to interprete carbon, nitrogen and sulphur cycles in HSSF CWs. Given the widespread application of the Activated Sludge Models (Henze *et al.,* 2000) and given the many 'offspring models' like the Anaerobic Digestion Model N° 1 (Batstone *et al.*, 2002) and the River Water Quality Model N° 1 (Shanahan *et al.*, 2001) which make use of the same philosophy, it was decided to adopt a similar process structure. The model incorporates physical processes such as physical reaeration and wash-out of solids, biological processes such as plant uptake of nutrients and microbiological processes representing the competition between aerobic, anoxic and anaerobic bacteria for substrates and electron acceptors.

As such a complex model is typically overparameterised, default parameter values from the different validated submodels were used when available and it was then tested on two datasets, one from an experimental 0.55 m^2 setup and one from a pilot-scale 55 m^2 HSSF CW. Although in these initial attempts the model was not always very precise in predicting effluent concentrations, it did a fair job in responding to load changes and seasonal variations and it was able to tie the different removal efficiencies to the loading method, i.e. batch-wise and continuous respectively. Most importantly, it proved to be a very valuable tool to interpret the experimental data, a very useful framework to foster discussion and an important instrument to identify knowledge gaps.

One should anyhow be aware that the model incorporates only the major (expected) processes. However, these aquatic ecosystems – artificial as they may be – are so complex that probably dozens of processes have not been covered. Investigations on the biota of SSF CW for instance revealed the presence of significant quantities of macro-invertebrates such as oligochaetes, springtails, beetles etc. (Pauwels, 2004; Verheire, 2003) which are thought to play an important role in the foodweb by ingesting larger organic particles, grazing the biofilm etc.

Speculating about the future of the model, it would seem logical to have a similar evolvement as the activated sludge models did:

1. Reducing the parameter uncertainty by calibration with data of different systems, loading rates, climates etc;

2. Reducing the model uncertainty by addition, elimination or transformation of process equations;
3. Extending the model with other relevant processes such as phosphorus removal, heavy metal removal etc;
4. Coupling the CW model to river water quality models and sewer models, also called ‘integrated urban wastewater systems’ (IUWS, cf. Meirlaen, 2002);
5. Coupling of chemical water quality models with ecological water quality models (Adriaenssens, 2004; Goethals, 2005; Dedecker, 2005).

Within suggestions 1, 2 and 3, multidisciplinary research should certainly be stimulated. As many of the incorporated processes are also observed in natural wetlands, buffer strips (Leeds-Harrison *et al.*, 1999; Dhondt *et al*., 2004), controlled flooding areas (Du Laing *et al*., 2003) etc., these research areas could also benefit from the model and vice versa. The last two suggestions would fit within the tasks resulting from the implementation of the European Water Framework Directive (2000) which, by imposing a good ecological quality for every water body, focuses more on the immission-based approach rather than only on the emission-based one. One is referred also to section 11.4 for a further discussion on this topic.

11.4. OPERATION AND MAINTENANCE

During many field visits, in Belgium as well as elsewhere, it was often noticed that CWs were not well maintained. Cooper *et al*. (1996) also made this observation and called it the ‘build-and-forget’ mentality. Indeed, because they are ‘natural’ systems and because they are promoted as wastewater treatment systems with low maintenance requirements, owners and operators tend to misinterpret this as ‘no maintenance needed’. Twelve stormwater treatment reed beds in the UK were surveyed and it was concluded that operational maintenance was an important factor in ensuring the longevity of a CW. A detailed economic analysis was not made, but it is clear that the costs of more frequent and more thorough maintenance are relatively insignificant compared to the benefit of being able to operate for several more years before a CW has to be replaced. To this end, a literature review was made to compile a list of monitoring requirements and minimal maintenance efforts.

11.5. CONSTRUCTED TREATMENT WETLANDS IN A BROADER PERSPECTIVE

Designing and operating CWs for optimal treatment performance is the rather narrow-minded 'engineering approach' where the system boundaries are clearly defined by the CW itself. However, from an economical and ecological point of view, the objective should rather be to have a good ecological quality in the receiving water course, and this at a minimum cost.

CWs can often be found in rural or remote areas where no sewer system is present and where people are thus – legally – obliged to treat their own wastewater. In many cases, the agro- and natural ecosystems in those rural and remote areas are intersected with many small brooks and watercourses to which the many anthropogenic discharges are a potential ecological threat. Depending on the use of these surface waters (e.g. recreation, potable water production, fishing), different quality standards apply which in turn can be translated into different effluent standards (cf. European Water Framework Directive, 2000).

Rousseau *et al*. (2003) investigated the impact of CWs on a small rural catchment area by comparing the river water quality before and after the start-up of a reed bed, upstream and downstream the discharge point of the reed bed and by comparing the effluent load of the constructed wetland with the other pollutant loads that enter the watercourse. Data from two different reed bed systems in Belgium and the UK were used. Both CWs removed a great deal of pollutants and had a strong peak shaving capacity, thus avoiding peak loads to be discharged into the receiving water courses. The impact on the water quality of the brooks was however less clear, for a number of reasons. First of all, several CSO events per year regularly disturb the aquatic ecosystem. Secondly, it is not illogical to assume that the gathering of wastewater – although treated – at one single discharge location, shows other pollution patterns than the same amount of untreated wastewater being discharged at a number of different locations. And the last, probably most important reason, is that both wastewater treatment systems were situated in a rural area with intensive farming activities. Manure, drainage water and point sources from non-sewered houses and farms most likely deliver a considerable fraction of the pollution load to the watercourses. These case studies have therefore convincingly demonstrated the **need for an integrated approach**. CWs or small-scale wastewater treatment systems in general are quite useless if the watercourse receives several other untreated discharges. One small-scale wastewater treatment plant might be a drop in the ocean, but a multitude of them works can significantly contribute to the river water quality and avoid exceeding the self-purification capacity.

Another way to surpass the strict engineering approach has been briefly touched in Chapter 1, i.e. to incorporate water reuse possibilities in CW projects and to make use of the so-called ancillary benefits like recreation, selling economically valuable plants etc. Especially in developing countries, this subject already received major attention, but given the increasing water demand and scarcity, it may well become a crucial issue in developed countries as well. Van Minh and De Pauw (2005) for instance give an interesting overview of the different types of wastewater-based aquaculture in the south of Vietnam. Constructed wetlands – being a low-cost, easily maintainable and highly efficient alternative to conventional wastewater treatment plants – have a strong potential for application in developing countries since their warm tropical and subtropical climates stimulate biological treatment and productivity. However, these systems have not yet found widespread use, due to lack of awareness and local expertise to develop these technologies on a local scale.

With these last words, it is the authors' sincere hope that this thesis may have contributed to spreading out the message that natural systems for wastewater treatment, in particular the constructed wetlands, will increasingly continue to play their role in the broad context of the need for sustainable development.

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LIST OF ABBREVIATIONS

ADM	Anaerobic Digestion Model
AMINAL	Administratie Milieu, Natuur en Landinrichting (Environment, Nature, Land and Water Administration)
ASM	Activated Sludge Model
BOD	Biochemical Oxygen Demand
COD	Chemical Oxygen Demand
CSO	Combined Sewer Overflow
CSTR	Continuously Stirred Tank Reactor
CW	Constructed Wetland
DO	Dissolved Oxygen
DOC	Dissolved Organic Carbon
DWF	Dry Weather Flow
ETP	EvapoTransPiration
FC	Faecal Coliforms
FWS	Free-Water-Surface
HDPE	High Density PolyEthylene
HLR	Hydraulic Loading Rate
HRT	Hydraulic Retention Time
HSSF	Horizontal SubSurface-Flow
KJN	Kjeldahl Nitrogen
LT	Long Term
MSL	Model and Simulation Language
NADB	North American treatment wetlands DataBase
O&M	Operation and Maintenance
o-PO_4	Orthophosphate
orgN	Organic Nitrogen
orgP	Organic phosphorus
PE	Population Equivalent
POC	Particulate Organic Carbon
RBC	Rotating Biological Contactor
RGR	Relative Growth Rate
SME	Small Medium Enterprises
SSF	SubSurface-Flow
ST	Short Term
TN	Total Nitrogen
TON	Total Oxidised Nitrogen
TP	Total Phosphorus
TSS	Total Suspended Solids
VLM	Vlaamse Landmaatschappij (Flemish Land Agency)
VMM	Vlaamse Milieumaatschappij (Environment Agency of Flanders)
VSSF	Vertical SubSurface-Flow
WWTP	WasteWater Treatment Plant

SUMMARY

Under certain circumstances, wastewater treatment by activated sludge units and clarifiers appears to be an unfit technology. Mainly in developing countries where know-how, funding and assets are limited, one often recurs to inexpensive, low-technological but nevertheless efficient methods such as waste stabilisation ponds and/or constructed wetlands (CWs). Even in economically stronger countries, sustainable alternatives are often needed for these discharge points that cannot be connected to a conventional wastewater treatment plant due to technical, economical or ecological constraints. *In situ* treatment by means of CWs offers a potential alternative in certain cases.

How this green technology evolved and which types currently are in operation around the world, is being described in Chapter 1. Purification processes are then summarised and the role of some important internal and external influencing parameters such as pH and temperature is discussed. A brief economical analysis of costs and benefits concludes this introductory chapter.

For Chapter 2, a database on 107 CWs in Flanders (Belgium) has been assembled and analysed to summarise the available experience. For each type of CW, an overview is given of treatment performance and its seasonal variations. Free-water-surface CWs exhibited the lowest treatment performance whereas vertical subsurface-flow CWs seemed most efficient, with the exception of nitrogen removal. Indeed, adding a horizontal subsurface-flow CW as polishing step was clearly beneficial because of enhanced denitrification. Season c.q. temperature mainly influenced nutrient removal with lower removal efficiencies during cold periods. Investment costs proved to be highly variable and strongly dependent on the type of CW and on the design capacity. Finally, from practical experience, it appears that the specific legislation on CWs and certainly its enforcement fail and that many owners/operators have a wrong perception of the required maintenance of such a treatment system. Non-stringent effluent standards, the lack of compliance monitoring and the often-noted misconception that natural systems are able to manage themselves, cause neglection of many CWs.

Since treatment efficiency of both horizontal and vertical subsurface-flow CWs was positively evaluated in Chapter 2, Chapters 3 and 4 further exploit these technologies. Firstly, detailed mass balances for water, solids, organic material, nitrogen and phosphorus demonstrate that purification in CWs is accomplished by a complex array of interacting physical, chemical and biological processes. Influencing factors such as temperature, pH, C/N ratio etc. are also being discussed in detail. This theoretical framework is then applied on three data sets from a pilot-scale two-stage reed bed (Aquafin Ltd, Aartselaar, Belgium). Short and long-term dynamics are being compared and the influence of influent load and temperature on treatment performance is assessed. Higher loads mainly caused a transient effect on the effluent concentrations shortly after the load increment, but the concentrations then quickly leveled off at the earlier level. Ammonium was the only exception as the oxygen demand at higher loads exceeded the oxygen transfer capacity of the vertical subsurface-flow CW. Seasonal performance variations were not detected for COD and suspended solids but were obvious for nitrogen removal as denitrification seemed inhibited by cold temperatures. Phosphorus removal also fluctuated substantially and seemed to be correlated to the plant growth and decay processes.

Having demonstrated the obvious qualities of CWs, the following chapters of the thesis are devoted to two crucial topics, i.e. design and maintenance of CWs. Only horizontal subsurface-flow CWs (HSSF CWs) are further discussed, as these are the most widespread type of CW within a European context.

Chapter 5 elaborates on model-based design of HSSF CWs, starting with simple rules of thumb, continuing with the state-of-the-art k-C* model and ending with dynamic, mechanistic models. A simple case study has been used to prove that the performance of black box models is not satisfactory. Indeed, different models and within-model parameter variations caused the predicted required surface area for a 10 PE case to vary between 0.1 and 950 m^2. Dynamic models are still in a premature stage but offer interesting perspectives.

Chapter 6 therefore presents a model study with such a mechanistic model, applied on data of a two-stage HSSF CW at Saxby (UK). As a starting point, the model of Wynn and Liehr (2001) was chosen as it gives a quantitative description of carbon and nitrogen transformations. After a number of changes to the model structure and after parameter estimation, this model seemed able to predict general trends in effluent quality, but missed some of the short-term dynamics. Due to a number of non-closed mass balances, the lack of an adequate description of particulate processes and the absence of anaerobic processes, it was decided to develop a new conceptual model that was able to describe and explain the interactions between the C, N and S cycli.

Chapter 7 presents this new mechanistic model of a HSSF CW in which 8 different microbial communities, together with the reed plants and a number of physical processes, interact and clean up the wastewater. The model equations are among others based on the widely spread and commonly accepted ‘Activated Sludge Models’. One advantage of this approach is that it enhances communication between wetland scientists as it introduces a sort of ‘common language’. Another advantage is that literature provides lots of parameter values as these models already have been applied in many case studies.

Calibration of such a complex model proves to be a very difficult task and would require many more data then are available up till now. It was therefore decided to use the default parameter values from each validated submodel. The model was then used to simulate an experimental HSSF CW of 0.55 m^2 and a pilot-scale HSSF CW of 55 m^2. Despite the many uncertainties, the model did a good job in predicting the effluent quality and most importantly it allowed to better explain the data.

Although a sound design forms the basis to a good performance, adequate operation and maintenance throughout the lifespan of a CW are of equal importance. Chapters 9 and 10 attempt to refute the widespread ‘reed beds are a build-and-forget solution’ mentality. Firstly, Chapter 9 reviews maintenance tasks, monitoring requirements and the frequency with which they should be carried out. Frequently occuring operational problems are described and troubleshooting guidelines are supplied. These rather theoretical recommendations are then being tested in a case study in Chapter 10. Twelve stormwater treatment CWs were examined by means of a site visit, an interview with the operators and by reviewing available effluent data. These investigations revealed that several CWs suffered from sludge accumulation, surface blinding and weed growth, but not to such an extent that the effluent quality was unsatisfactory. It has nevertheless been proved that adequate maintenance positively contributes to a longer lifespan of CWs.

Chapter 11 finaly summarises the most important findings of each chapter and lists some suggestions for future research.

SAMENVATTING

Afvalwaterzuivering onder de klassiek gekende vorm van beluchtingsbekkens en bezinkers blijkt in bepaalde omstandigheden geen haalbare kaart te zijn. Vooral in ontwikkelingslanden, waar kennis en middelen vaak ontbreken, dient men zijn toevlucht te nemen tot laag-technologische, goedkopere maar evengoed efficiënte methoden zoals stabilisatievijvers en/of artificiële moerassystemen, verder 'constructed wetlands (CWs)' genoemd. Ook in economisch meer welvarende landen zoekt men vaak naar duurzame alternatieven, daar waar aansluiting op een conventionele afvalwaterzuiveringsinstallatie omwille van technische, economische of ecologische redenen onmogelijk blijkt. *In situ* zuivering met CWs kan eventueel een pasklaar antwoord bieden.

Het ontstaan van deze groene technologie wordt kort beschreven in hoofdstuk 1 waarna dieper ingegaan wordt op de verschillende types van CWs die op heden toegepast worden. Vervolgens passeren de verschillende zuiveringsprocessen de revue, waarbij aandacht besteed wordt aan een aantal interne en externe invloedsfactoren zoals pH, temperatuur enz. Om af te sluiten worden heel summier een aantal kosten en baten op een rijtje gezet.

Hoofdstuk 2 beschrijft meer specifiek de ervaringen met CWs in Vlaanderen op basis van gegevens over 107 rietvelden. Per type wordt vooreerst een overzicht gegeven van hun respectievelijke zuiveringsresultaten tijdens de verschillende seizoenen. Hieruit bleek dat vloeirietvelden voor alle variabelen het minst efficiënt waren, percolatierietvelden daarentegen het meest efficiënt, uitgezonderd voor stikstof waar door het toevoegen van een wortelzonerietveld als tweede trap de TN verwijderingsefficiëntie nog gevoelig steeg door een verhoogde denitrificatie. Verder kan algemeen gesteld worden dat seizoen c.q. temperatuur vooral een duidelijke invloed heeft op nutriëntenverwijdering. Een analyse van de investeringskosten toonde aan dat deze zeer variabel waren, en sterk afhankelijk van het type CW en de ontwerpcapaciteit. Uit de praktijkvoorbeelden bleek tenslotte dat de wetgeving ter zake en de handhaving ervan een aantal lacunes vertoont en vooral ook dat vele eigenaars een verkeerde

perceptie hebben van het benodigde onderhoud van een dergelijk systeem. Te lakse effluentnormen en het gebrek aan controle ervan, gecombineerd met de veel voorkomende opvatting dat natuurlijke systemen zichzelf onderhouden, zorgt ervoor dat vele CWs er verwaarloosd bijliggen en daardoor hun doel totaal voorbijschieten.

Wegens de goede resultaten die genoteerd werden in hoofdstuk 2 voor zowel percolatie- als wortelzonerietvelden, gaan hoofdstukken 3 en 4 nader in op beide technologieën. Er wordt vooreerst aangetoond dat afvalwaterzuivering bewerkstelligd wordt door een complex web van interagerende fysische, chemische en biologische processen, en dit aan de hand van de massabalansen voor water, zwevende stoffen, organisch materiaal, stikstof en fosfor. Telkens worden ook de verschillende invloedsfactoren gedetailleerd besproken, zoals temperatuur, pH, C/N verhoudingen etc. Dit theoretisch kader wordt dan in hoofdstuk 4 toegepast op drie data sets die verzameld werden in een experimenteel tweetrapsrietveld van Aquafin NV in Aartselaar. Korte en lange termijn processen worden met elkaar vergeleken en de invloed van influentbelasting en seizoen op de zuiveringsresultaten wordt nagegaan. Hogere belastingen bleken hoofdzakelijk een effect te hebben op het moment van de omschakeling, nadien stabilizeerden de effluentconcentraties zich. Enkel ammonium vormde daarop een uitzondering want bij hogere belastingen overschreed de zuurstofvraag duidelijk de zuurstoftransfercapaciteit van het percolatierietveld. Een eventuele seizoensinvloed was niet merkbaar bij CZV en ZS, maar des te duidelijker bij TN door een inhibitie van de denitrificatie gedurende de koudere periodes. TP verwijdering varieerde ook doorheen de seizoenen en leek vooral gecorreleerd met de cyclus van plantengroei en –afsterving.

Nu de kwaliteiten van CWs duidelijk aangetoond werden, worden de verdere hoofdstukken gewijd aan twee cruciale topics, met name de ontwerpfase en het onderhoud van CWs eens ze in bedrijf werden genomen. Hierbij wordt gefocust op wortelzonerietvelden (WZRV) aangezien deze op Europese schaal het vaakst gebruikt worden.

Hoofdstuk 5 geeft een overzicht van modelgebaseerd ontwerp van WZRV, gaande van eenvoudige vuistregels, over het ‘state-of-the-art’ k-C* model tot dynamische, mechanistische modellen. Een gevalstudie toont duidelijk aan dat de eenvoudige ‘black box’ modellen niet voldoen. Model- en parametervariaties zorgen ervoor dat de berekende oppervlakte voor 10 inwonerequivalenten varieert van 0,1 tot 950 m^2. Dynamische modellen staan nog in de kinderschoenen maar openen interessante perspectieven.

Vandaar dat in hoofdstuk 6 een dergelijk mechanistisch model toegepast werd op data van een tweetraps WZRV in Saxby (VK). Als uitgangspunt werd het model van Wynn en Liehr (2001) gebruikt dat koolstof en stikstof transformaties beschrijft. Mits een aantal wijzigingen aan de model structuur en na parameterschatting bleek dit model in staat om de algemene trends in effluentkwaliteit weer te geven, maar een deel van de korte-termijn dynamiek ging verloren. Omwille van een aantal niet-gesloten massabalansen, de afwezigheid van particulaire processen en het ontbreken van anaërobe processen, werd besloten een nieuw conceptueel model te ontwikkelen dat in staat was om de verschillende interacties tussen C, N en S cycli te verklaren.

In hoofdstuk 7 wordt, op basis van de verzamelde kennis, een nieuw mechanistisch model van WZRV voorgesteld waarin 8 verschillende microbiële gemeenschappen, samen met de rietplanten en een aantal fysische processen, interageren en voor zuivering van het afvalwater zorgen. De vergelijkingen zijn onder andere gebaseerd op de bekende en wijd verspreide ‘Activated Sludge Models’. Dit heeft niet alleen als voordeel dat het de communicatie bevordert door het invoeren van een gemeenschappelijke ‘taal’, maar vooral dat in de literatuur voldoende informatie kan worden teruggevonden over parameterwaarden.

Kalibratie van een dergelijk complex model is een zeer moeilijke taak en zou veel meer gegevens vereisen dan tot nu toe beschikbaar zijn. Daarom werden van alle gevalideerde submodellen telkens de standaard parameter waarden gebruikt waarna het model werd losgelaten op 2 data sets van respectievelijk een experimenteel WZRV van 0.55 m^2 en een pilootschaal WZRV van 55 m^2. De resultaten hiervan worden beschreven in hoofdstuk 8. Met inachtneming van alle onzekerheden bleek het model toch behoorlijk goed in staat om de effluentkwaliteit te voorspellen en liet het vooral toe om de experimentele resultaten beter te verklaren.

Alhoewel logischerwijze een goed ontwerp aan de basis ligt van een goede performantie, zijn de daaropvolgende bedrijfsvoering en het onderhoud eveneens van cruciaal belang. Hoofdstukken 9 en 10 proberen de ingeburgerde 'reed beds are a build-and-forget solution' mentaliteit te doorbreken. Hiertoe wordt in hoofdstuk 8 aangevangen met een beknopte literatuurstudie over onderhoudstaken, onderhoudsfrequentie en de vereiste monitoring. Verder worden een aantal courante operationele problemen beschreven en mogelijke oplossingen aangereikt. Deze theoretische aanbevelingen worden in het daaropvolgende hoofdstuk getoetst door middel van een gevalstudie. Twaalf 'stormwater' CWs werden onderzocht aan de hand van enkele *in situ* metingen, een interview met de operatoren en beschikbare effluent gegevens. Hieruit bleek dat bij een aantal CWs slibopstapeling, oppervlaktestroming en onkruidgroei voorkwamen, maar dit nergens tot een onvoldoende effluentkwaliteit leidde. Er werd niettemin aangetoond dat doeltreffend onderhoud bijdraagt tot een langere levensduur van CWs.

Hoofdstuk 11 tenslotte vat nog eens de belangrijkste conclusies van de verschillende hoofdstukken samen en reikt nog een aantal perspectieven aan voor verder onderzoek.

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MEERS E., **D.P.L. ROUSSEAU**, N. BLOMME, E. LESAGE, G. DU LAING, F.M.G. TACK and M.G. VERLOO. Tertiary treatment of the liquid fraction of pig manure with *Phragmites australis*. *Water, Air and Soil Pollution*, 160, 15-26.

ROUSSEAU D.P.L., P. GRIFFIN, P.A. VANROLLEGHEM and N. DE PAUW (2005). Model study of secondary treatment reed beds at Saxby (Leicestershire, UK). *Journal of Environmental Science and Health part A*, 40(6-7), 1479 - 1492.

ROUSSEAU D.P.L., D. HORTON, P. GRIFFIN, P.A. VANROLLEGHEM and N. DE PAUW (2005). Impact of operational maintenance on the asset life of storm reed beds. *Water Science and Technology,* 51(9), 243-250.

MEERS E., **D.P.L. ROUSSEAU**, E. LESAGE, E. DEMEERSSEMAN and F.M.G. TACK (in press). Physico-chemical P removal from the Liquid Fraction of Pig Manure as an Intermediary Step in Manure Processing. *Water, Air and Soil Pollution.*

LESAGE E., **D.P.L. ROUSSEAU**, F.M.G. TACK and N. DE PAUW (submitted). Accumulation of metals in a horizontal subsurface flow constructed wetland treating domestic wastewater in Flanders, Belgium. *The Science of the Total Environment.*

Other journal publications

ROUSSEAU D., P. GOETHALS, J. MEIRLAEN, J. VERBOVEN, P. VANROLLEGHEM and N. DE PAUW (1999). Monitoring and modelling of free-water-surface constructed wetlands. *Communications in Applied Biological Sciences*, 64(5a), 191-196.

ROUSSEAU D., F. VERDONCK, R. CARRETTE, C. THOEYE, J. MEIRLAEN and P.A. VANROLLEGHEM (2000). Risk assessment tool for the design of wastewater treatment plants. *Communications in Applied Biological Sciences*, 65(4), 219-223.

GOETHALS P.L.M., U. WIEME, J. BOLS, J. MEIRLAEN, **D. ROUSSEAU**, A. VAN GRIENSVEN, V. VANDENBERGHE, L. VAN VOOREN, W. BAUWENS, N. DE PAUW and P. VANROLLEGHEM (2000). Quality assurance of automated measurement stations for the ‘on-line’ quality monitoring of surface waters. @*WEL*, 6, 1-9 (in Dutch).

ROUSSEAU D., W. DE WILDE, C. JONKERS, D. GEENENS, P.A. VANROLLEGHEM and N. DE PAUW (2001). Short-term behaviour of an experimental constructed wetland at low temperatures. *Communications in Applied Biological Sciences*, 66(4), 233-237.

ROUSSEAU D.P.L., P.A. VANROLLEGHEM and N. DE PAUW (2004). Constructed wetlands for wastewater treatment in Flanders: design, operation and performance. *Afvalwaterwetenschap*, 3(1), 13-33 (in Dutch).

DIADOVSKI, I., RAYKOVSKA, Y., DE PAUW, N. and ROUSSEAU, D. (2005). Ecotoxicological assessment of water and sediment pollution of the Iskar River below Samokov. *Bioautomation*, 2, 70-75.

Publications in international proceedings with peer-review

ROUSSEAU D., F. VERDONCK, O. MOERMAN, R. CARRETTE, C. THOEYE, J. MEIRLAEN and P.A. VANROLLEGHEM (2000). Development of a risk assessment based technique for design/retrofitting of WWTPs. In: Proceedings WATERMATEX 2000, 5th International Symposium on Systems Analysis and Computing in Water Quality Management, 18-20 september 2000, Gent, Belgium, 5/17 – 5/24.

BIXIO D., G. PARMENTIER, **D. ROUSSEAU**, F. VERDONCK, J. MEIRLAEN, P.A. VANROLLEGHEM and C. THOEYE (2001). Integrating risk analysis in the design/simulation of activated sludge systems. In: Proceedings WEFTEC 2001 (on CDROM).

BIXIO D., R. CARRETTE, **D. ROUSSEAU**, F. VERDONCK, C. THOEYE, J. MEIRLAEN and P.A. VANROLLEGHEM (2001). A quantitative risk analysis tool for design/simulation of wastewater treatment plants. In: Proceedings IWA 2nd World Water Congress, Berlin, Germany, 15-18 October 2001 (on CDROM).

ROUSSEAU D., D. GEENENS, P.A. VANROLLEGHEM and N. DE PAUW (2002). Short-term behaviour of constructed reed beds: pilot plant experiments under different temperature conditions. In: Proceedings IWA 8th International Conference on Wetland Systems for Water Pollution Control, Arusha, Tanzania, 16-19 September 2002, 128-139.

ROUSSEAU D.P.L., N. DE PAUW, D. GEENENS, P. GRIFFIN and P.A. VANROLLEGHEM (2003). Do constructed wetlands contribute to a better river water quality? In: Proceedings Second World Wide Workshop for Young Environmental Scientists, 14-17 May 2003, Vitry-sur-Seine, 61-71.

VERDONCK F.A.M., **D. ROUSSEAU**, D. BIXIO, C. THOEYE and P.A. VANROLLEGHEM (2003). Added value of concentration-duration-frequency curves of wastewater treatment effluent quality. In: Proceedings MODSIM 2003: Integrative Modelling of Biophysical, Social and Economic Systems for Resource Management Solutions, 14-17 July 2003, Townsville, Australia (on CDROM)

ROUSSEAU D.P.L., P. GRIFFIN, P.A. VANROLLEGHEM and N. DE PAUW (2003). Model study of secondary treatment reed beds. Publicationes Instituti Geographici Universitatis Tartuensis, 94, 179-183.

ROUSSEAU D.P.L., D. HORTON, P. GRIFFIN, P.A. VANROLLEGHEM and N. DE PAUW (2004). Impact of operational maintenance on the asset life of storm reed beds. In: Proceedings 9th IWA International Conference on Wetland Systems for Water Pollution Control, Volume 2, 26-30 September 2004, Avignon, France, 567-575.

ROUSSEAU D.P.L., G. DU LAING, M. VERLOO and N. DE PAUW. Hydroponic treatment of landfill leachate and wastewater of a glass recuperation site (2004). In: Proceedings 9th IWA International Conference on Wetland Systems for Water Pollution Control, 26-30 September 2004, Avignon, France (on CDROM).

LESAGE E., A. VAN DE MOORTEL, **D. ROUSSEAU**, F.M.G. TACK, N. DE PAUW and M. VERLOO (2004). Effect of aquatic macrophytes on the removal efficiency of heavy metals in horizontal subsurface flow constructed treatment wetlands. In: Proceedings 9th IWA International Conference on Wetland Systems for Water Pollution Control, 26-30 September 2004, Avignon, France (on CDROM).

DU LAING G., **D. ROUSSEAU**, E. LESAGE, F. TACK and M. VERLOO (2004). Removal of Zn from water by two floating macrophytes, *Lemna minor* and *Azolla pinnata*. In: Proceedings 9th IWA International Conference on Wetland Systems for Water Pollution Control, 26-30 September 2004, Avignon, France (on CDROM).

<u>Publications in international proceedings without peer-review</u>

ROUSSEAU D. (1999). Modeling of helophyte filters for treatment of domestic wastewater. In: Proceedings NecoV workshop on ‘Natural systems for wastewater treatment in The Netherlands and Flanders’, 21 October 1999, Antwerp, Belgium (in Dutch).

NEDELTCHEVA T. and **D. ROUSSEAU** (2002). Existing indices and criterion for physico-chemical, structural-morphological and biological assessment of water quality in Belgium and other EU countries (in Bulgarian). In: Proceedings International Symposium and Young Scientist School Bioprocess Systems BioPS’02, 28 October 2002, Bulgarian Academy of Sciences, Sofia, Bulgaria, II.40-II.46.

NEDELTCHEVA T. and **D. ROUSSEAU** (2002). Integrated ecological assessment of the water quality used in EU countries (in Bulgarian). In: Proceedings International Symposium and Young Scientist School Bioprocess Systems BioPS’02, 28 October 2002, Bulgarian Academy of Sciences, Sofia, Bulgaria, II.47-II.52.

DIADOVSKI I., L. BRANKOVA and **D. ROUSSEAU** (2003). Ecotoxicological Impact Assessment of Some Heavy Metals from Point Sources of Pollution on the River Flow. Proceedings 12th International Symposium "Ecology 2003", 3-5 June, Burgas, Bulgaria, 125-134.

DIADOVSKI I., I. IVANOV, **D. ROUSSEAU** and L. BRANKOVA (2003). Biological Removal of Nitrogen, Phosphorus and Harmful Substances from Point Source of Pollution in Natural Conditions. Proceedings 12th International Symposium "Ecology 2003", 3-5 June, Burgas, 135-142.

Abstracts

ROUSSEAU D., W. DE WILDE, C. JONKERS, D. GEENENS, P.A. VANROLLEGHEM and N. DE PAUW (2001). Short-term behaviour of an experimental constructed wetland at low temperatures. NecoV Wintermeeting, 12-13 December 2001, Antwerpen, 34-35.

LESAGE E., S. ROGGEMAN, N. DE PAUW, G. DU LAING, E. MEERS, **D. ROUSSEAU**, F. TACK and M. VERLOO (2003). Copper tolerance of some aquatic macrophytes commonly used for wastewater treatment. 1st International Seminar on the Use of Aquatic Macrophytes for Wastewater Treatment in Constructed Wetlands, 8–10 mei 2003, Lissabon, Portugal, 637.

ROUSSEAU D.P.L., P.A. VANROLLEGHEM and N. DE PAUW (2003). Constructed wetlands in Flanders: 17 years of experience. Eight International Symposium on Biogeochemistry of Wetlands, 14-17 september 2003, Gent, 37.

LESAGE E., A. VANDEMOORTEL, F. TACK, **D. ROUSSEAU**., N. DE PAUW and M. VERLOO (2004). Influence of helophytes on the removal of heavy metals in rootzone constructed wetlands. Wintermeeting NecoV, 14-15 January 2004, Gent, 40 (in Dutch).

ROUSSEAU D.P.L., P.A. VANROLLEGHEM and N. DE PAUW (2004). Natural systems for wastewater treatment in Flanders: fiction and facts. Wintermeeting NecoV, 14-15 January 2004, Gent, 21 (in Dutch).

STORY A., **D.P.L. ROUSSEAU**, P.A. VANROLLEGHEM and N. DE PAUW (2004). Integrated modelling of rootzone reedbeds and the receiving water body. NecoV Wintermeeting, 14-15 January 2004, Gent, 51 (in Dutch).

LESAGE E., **D.P.L. ROUSSEAU**, K. DE BLOCK, D. VANTHUYNE, F.M.G. TACK, N. DE PAUW and M.G. VERLOO (2005). Assessing Cu and Zn accumulation in the gravel filter medium of a constructed wetland receiving domestic wastewater. 8th ICOBTE Conference, April 2005, Adelaide, Australia.

LESAGE E., **D.P.L. ROUSSEAU**, F.M.G. TACK, M.G. VERLOO and N. DE PAUW (2005). Assessing trace metal accumulation in a constructed wetland receiving domestic wastewater. 9th International Symposium on Biogeochemistry of Wetlands, March 20-23, Baton Rouge, Louisiana, USA.

LESAGE E., **D.P.L. ROUSSEAU**, A. STORY, N. DE PAUW, P.A. VANROLLEGHEM, F.M.G. TACK and M.G. VERLOO (2005). Pilot-scale horizontal subsurface flow treatment wetlands for model calibration purposes. 9th International Symposium on Biogeochemistry of Wetlands, March 20-23, Baton Rouge, Louisiana, USA.

NGUYEN L.T.H., **D.P.L. ROUSSEAU**, S. HEYLEN and N. DE PAUW (2005). Re-colonisation of the pelagial of a shallow lake by benthic macroinvertebrates after removal of polluted sediment. Shallow Lakes Conference, 5-9 June, Dalfsen, The Netherlands.

ROUSSEAU, D.P.L., J. GARCIA, A. STORY, E. LESAGE, N. DE PAUW and P.A. VANROLLEGHEM (2005). Carbon and nitrogen cycles in horizontal subsurface-flow constructed wetlands: model-based evaluation. International Symposium on Wetland Pollutant Dynamics and Control, September 4-8, Ghent, Belgium.

MEERS E., **D. ROUSSEAU**, E. LESAGE and F. TACK (2005). Tertiary treatment of the liquid fraction of pig manure with constructed wetlands. International Symposium on Wetland Pollutant Dynamics and Control, September 4-8, Ghent, Belgium.

LESAGE E., **D. ROUSSEAU**, F. TACK and N. DE PAUW (2005). Accumulation of trace metals in constructed treatment wetlands in Flanders, Belgium. International Symposium on Wetland Pollutant Dynamics and Control, September 4-8, Ghent, Belgium.

MEERS E., E. LESAGE, **D. ROUSSEAU** and F. TACK (2005). Cu and Zn distribution in a constructed wetland for treatment of agricultural effluents. International Symposium on Wetland Pollutant Dynamics and Control, September 4-8, Ghent, Belgium.

Research reports

ROUSSEAU D. (1999). Monitoring and modelling of free-water-surface constructed wetlands. MSc thesis Faculty of Agricultural and Applied Biological Sciences, Ghent University, 124 p. + annexes (in Dutch).

GOETHALS P.L.M., **D. ROUSSEAU** and N. DE PAUW (2000). Project Management Unit 415 - Canal Tan Hoa – Lo Gom sanitation and urban upgrading project. Report Mission 2: Design of a natural system for urban wastewater treatment and assessment of industrial wastewater discharges in the Bing Hung Hoa Canal in HCMC'. 55 p. + annexes.

VASEL J.-L., N. DE PAUW H. JUPSIN, **D. ROUSSEAU** and P. GOETHALS, (2000). Project Management Unit 415 - Canal Tan Hoa – Lo Gom sanitation and urban upgrading project. Pre-feasibility study of a waste stabilisation pond system for the treatment of polluted canal water, 18 p.

ROUSSEAU D., G. DU LAING, M. VERLOO and N. DE PAUW (2002). Feasibility study: Applicability of constructed wetland technology for treatment of different wastewater flows from a landfill. Ghent University, Laboratory of Environmental Toxicology and Aquatic Ecology and Laboratory of Analytical Chemistry and Applied Ecochemistry, 55 p. + annexes.

ROUSSEAU D. (2002). Survey and model study of the secondary treatment reed beds at Saxby (Leicestershire). Ghent University, Laboratory of Environmental Toxicology and Aquatic Ecology, 41 p.

ROUSSEAU D. (2002). Assessment of operational problems at five Severn Trent Water reed bed sites: Butlers Marston, Claverdon, Combrook, Knightcote and Stretton-on-Fosse. Ghent University, Laboratory of Environmental Toxicology and Aquatic Ecology, 25 p.

GOETHALS P.L.M., **D. ROUSSEAU**, J.-L. VASEL and N. DE PAUW (2003). Audit PMU415 lagoon project: appreciation report for phase 2 of the detailed design study: comments on the final design and research possibilities. Ghent University, Gent, Belgium, 96 p.

REVIEWS FOR INTERNATIONAL JOURNALS

- Water Research (4)
- Aquatic Ecology (2)
- Journal of Environmental Science and Health part A (2)
- Ecological Engineering (1)
- Aquaculture Research (2)
- Journal of Environmental Engineering and Science (1)
- Environmental Pollution (1)

ATTENDED SYMPOSIA, CONFERENCES AND WORKSHOPS

Symposium ’25 years Aquatic Ecology’, 13 November 1998, Nijmegem, The Netherlands.

IAWQ 4th Specialised Conference on Small Wastewater Treatment Plants, 18-21 April 1999, Stratford-upon-Avon, UK.

Workshop ‘Applications of membrane technology’, Technological Institute, 19 May 1999, Leuven, Belgium.

13th Forum for Applied Biotechnology (FAB), 22 – 23 September 1999, Ghent, Belgium (poster presentation).

NecoV workshop ‘Natural systems for (waste)water treatment in The Netherlands and Flanders’, 21 October 1999, Antwerp, Belgium (platform presentation).

NecoV Wintersymposium, 8-9 December 1999, Antwerp, Belgium (poster presentation).

Workshop ‘Small-scale wastewater treatment’, Technological Institute, 18 May 2000, Antwerp, Belgium.

IWA 7th International Conference on Wetland Systems for Water Pollution Control, 11-16 November 2000, Lake Buena Vista, Florida, USA.

NecoV Wintersymposium ‘Towards a sustainable restoration of ecosystems’, 13-14 December 2000, Wageningen, The Netherlands.

Agriflora 2001. Workshop ‘Rainwater and wastewater : treatment and use possibilities’, 12 January 2001, Flanders Expo, Ghent, Belgium.

Workshop ‘Water for Tomorrow’, WWF and VMM, 2 May 2001, Ghent, Belgium.

Forum for Water Research in Flanders, VWN, AMINAL-Water and VMM, 17 May 2001, Brussels, Belgium.

First Plenary Assembly of ETNET 21, European Network for Education and Training Environment-Water. International Colloquium ‘Knowledge transfer for environment water’, 18-19 June 2001, Delft, The Netherlands.

Workshop ‘River bank management’, AMINAL-Water, 20 June 2001, Bruges, Belgium.

ESF Summer School ‘Buffer zones for water pollution control’, 29 August – 07 September 2001, Ghent, Belgium.

7th PhD Symposium of the Faculty of Agricultural and Applied Biological Sciences, 10 October 2001, Gent, Belgium (poster presentation).

Workshop ‘Natural river banks and buffer strips, theory and practice of a multifunctional water management’, Instituut of Nature Conservation and AMINAL–Water, 12 October 2001, Brussels, Belgium.

NecoV Wintersymposium ‘New Developments in Ecology’, 12-13 December 2001, Antwerp, Belgium (poster presentation – poster award).

Upscaling and downscaling in landscape ecology, 13 June 2002, Utrecht, The Netherlands.

IWA 8th International Conference on Wetland Systems for Water Pollution Control, 16-19 September 2002, Arusha, Tanzania (platform presentation).

NecoV Wintersymposium ‘Ecology at major construction sites’, 15 January 2003, Leiden, The Netherlands.

Second World Wide Workshop for Young Environmental Scientists, 14-17 May 2003, Domaine de Chérioux, Vitry-sur-Seine, France (platform presentation).

Eight International Symposium on Biogeochemistry of Wetlands, 14-17 September 2003, Ghent, Belgium (platform presentation).

International Conference on Constructed and Riverine Wetlands for Optimal Control of Wastewater at Catchment Scale, 29 September – 2 October 2003, Tartu, Estonia (platform presentation).

Workshop ‘Water pollution and small-scale wastewater treatment’, 28 November 2003, KAHO Sint-Lieven Hogeschool, Ghent, Belgium (platform presentation).

NecoV Wintersymposium ‘Ecotechnics and Nature Development’, 14-15 January 2004, Ghent, Belgium (platform presentation, co-organiser).

Royal Society of Natural Sciences DODONAEA, ‘Natural systems for wastewater treatment: facts and fiction, 11 May 2004, Gent (platform presentation).

9th IWA International Conference on Wetland Systems for Water Pollution Control, 26-30 September 2004, Avignon, France (1 platform presentation – 1 poster presentation – 2 poster presentations as co-author).

NecoV Wintersymposium ‘Changing contacts: ecological aspects of connection and isolation’, 26 January 2005, ‘s Hertogenbosch, The Netherlands.

International Symposium on Wetland Pollutant Dynamics and Control, 4-8 September 2005, Ghent, Belgium (1 platform presentation – 1 platform presentation as co-author – 2 poster presentations as co-author, co-organiser).

ORGANISATION OF SCIENTIFIC MEETINGS AND CONFERENCES

Co-organiser ‘NecoV Wintersymposium – Ecotechnics and Nature Development’, Ghent, Belgium, 14-15 January 2004.

Co-organiser ‘International Symposium on Wetland Pollutant Dynamics and Control’, Ghent, Belgium, 4-8 September 2005.

STAYS ABROAD AND TRAVEL GRANTS

February – June 1998:	ERASMUS student exchange with: Ecole Nationale Supérieure Agronomique de Rennes 65, Rue Saint Brieuc, 35042 Rennes cedex, France
July – August 2002:	FWO travel grant (V 4.060.02N) for research project at: Severn Trent Water Ltd - Technology and Development Avon House, St. Martin's Road, Coventry CV3 6PR, UK
10-17 June 2003:	FWO travel grant (V 4.109.03N) Central Laboratory of General Ecology (Sofia, Bulgaria) Bulgarian Academy of Science
29/9 – 2/10 2003:	FWO conference grant (C 17/5 – CVW. D 5) International Conference on Constructed and Riverine Wetlands for Optimal Control of Wastewater at Catchment Scale, Tartu, Estonia
31/8 – 7/9 2004:	FWO travel grant (V 4.103.04N) Central Laboratory of General Ecology (Sofia, Bulgaria) Bulgarian Academy of Science
26 – 30 September 2004:	FWO conference grant (C 17/5 – KL. D 5) 9th IWA International Conference on Wetland Systems for Water Pollution Control, Avignon, France

ACTIVE PARTICIPATION IN PROJECTS BESIDES PhD RESEARCH

Development of supporting techniques for a hydro-informatics system for water quality management of rivers. Cooperation between Ghent University (Department of Applied Ecology and Environmental Biology, Department of Applied Mathematics, Biometrics and Process Control) and Free University of Brussels (Department of Hydrology). Funded by Fund for Scientific Research Flanders (FWO G. 0102.97N)

Tan Hoa – Lo Gom Canal sanitation and urban upgrading project in Ho Chi Minh City, Vietnam. Funded by Belgian Technical Cooperation.

Preservation, protection and restoration of the Kraenepoel and surroundings (Aalter, Belgium): development of macro-invertebrates after restoration. Funded by EU-Life (LIFE 98 NAT/B/005172), Environment, Nature, Land and Water Administration (AMINAL division Nature) and the Community of Aalter.

Ecotoxicological assessment of point and diffuse pollution impact on the aquatic ecosystems: application for the Iskar river catchment. Bilateral coöperation with the 'Central Laboratory of General Ecology' (Sofia, Bulgaria). Funded by Fund for Scientific Research Flanders (FWO V 4.044.02N).

TEACHING ACTIVITIES

Practical exercises at Ghent University

2000–2005: **Aquatic Ecology** for Bio-engineers, MSc. Environmental Sanitation, MSc. Aquaculture and MSc. Nematology (15h each year). Main topic: plankton analysis.

2000–2004: **Ecological Water Quality Assessment** for Bio-engineers, MSc. Environmental Science and Technology and MSc. Environmental Sanitation (30h each year). Main topic: biological water quality assessment by means of macro-invertebrates.

2000–2004: **Algae Culture** for MSc. Aquaculture (15h each year). Main topics: cultivation techniques and growth monitoring.

2000–2004: **Conservation Biology** for MSc. Environmental Sanitation (15h each year). Topic: excursions to nature reserve areas.

2001–2004: **Natural Systems** for (Waste)water Treatment for MSc. Environmental Sanitation and MSc. Aquaculture (15h each year). Topics: design calculations of constructed wetlands and waste stabilisation ponds; excursions.

2001–2004: **Ecohydraulics and Nature Development** for Bio-engineers (8h each year). Topics: design calculations of constructed wetlands and waste stabilisation ponds; excursions.

International Training Courses

15-21 May 2004: COMENIUS/GRUNDTVIG In Service Training BV-2004-010 'Water and Life', Paphos, Cyprus. Practical exercises on ecological water quality assessment via the BISEL method.

Supervision of script students and trainees

Wouter De Wilde, Bio-engineer land- and forestry management (2000-2001). Monitoring and modelling of subsurface-flow constructed wetlands.

Saskia Lammens, MSc. Environmental Science and Technology (2000-2001). Monitoring and optimisation of the Kasteelvijvers ('Castle ponds') of Brewery Palm.

Jeroen Moernaut, MSc. Environmental Science and Technology (2000-2001). Nutrient cycles in subsurface-flow wetlands.

Bernard Timmers, MSc. Environmental Science and Technology (2000-2001). Carbon cycle and pathogen removal in subsurface-flow constructed wetlands.

Dierik De Middeleer, MSc. Environmental Science and Technology (2000-2001) Development of a sampling scheme for the monitoring of vertical subsurface-flow constructed wetlands, treating dairy wastewater.

Annemie Boone, BSc. Chemistry – option Environment (2000-2001). Macro-invertebrates in aquatic ecosystems.

An De Moor, Bio-engineer environmental technology (2001-2002). Monitoring and modelling of horizontal subsurface-flow constructed wetlands.

Jan Dick, MSc. Environmental Science and Technology (2001-2002). Applicability of reed bed technology for the treatment of landfill leachate.

Natalie Verheire, Technical Engineer (2002-2003). Clogging phenomena in vertical subsurface-flow constructed wetlands: possibilities of bioremediation.

Joachim Remue, BSc. Chemistry – option environmental sanitation (2002-2003). Clogging phenomena in reed beds.

Anke Story, Bio-engineer environmental technology (2002-2003). Impact of horizontal subsurface-flow constructed wetlands on the receiving surface waters: a model study.

Bram Van Renterghem, Bio-engineer land- and forestry management (2002-2003). Treatment of combined sewer overflow by means of floating macrophyte mats: a model study.

Kevin Pauwels, BSc Chemistry – option environmental sanitation (2003-2004). Integrated assessment of the operational problems of reed beds and implications for their life span.

Abigail Marie L. Torres, MSc. Environmental Sanitation (2004-2005). Design and start-up of pilot-scale horizontal subsurface-flow constructed wetlands.

Isabelle Van Thournout, MSc. Environmental Science and Technology (2004-2005). Ecology of treatment wetlands: assessment by means of avifauna and macroinvertebrates.

Melanie Franck – MSc. Environmental Science and Technology (2004-2005). Ecology of treatment wetlands: assessment by means of plankton and macrophytes.

Jury member of script students at Ghent University

Pieter De Grauwe, Bio-engineer environmental technology (2002-2003). Biogeochemical behaviour of Pb, Ni and Zn in river marginal reed beds of the Scheldt river. Promoters: Prof. Dr. ir. M. Verloo and ir. G. Du Laing.

Wouter Moors, Bio-engineer environmental technology (2002-2003). Biogeochemical behaviour of Cd, Cu and Cr in river marginal reed beds of the Scheldt river. Promoters: Prof. Dr. ir. M. Verloo and ir. G. Du Laing.

Nathalie Blomme, Bio-engineer environmental technology (2002-2003). Feasibility of helophyte filters for tertiary treatment of liquid manure. Promoters: Prof. Dr. ir. F. Tack and ir. E. Meers.

Sarah Roggeman, MSc. Environmental Science and Technology (2002-2003). Tolerance of helophyte applied for rhizofiltration of heavy metal laden wastewater. Promoters: Prof. Dr. ir. F. Tack and ir. E. Meers.

Annelies Van de Moortel, Bio-engineer environmental technology (2003-2004). Distribution and removal of heavy metals in helophyte filters. Promoter: Prof. Dr. ir. F. Tack.

Ellen Demeersseman, Bio-engineer environmental technology (2003-2004). Polishing of liquid manure via phytoremediation. Promoters: Prof. Dr. ir. F. Tack and ir. E. Meers.

Benjamin De Meyer, Bio-engineer environmental technology (2003-2004). Seasonal fluctuations in metal mobility in intertidal zones of the Scheldt river. Promoters: Prof. Dr. ir. M. Verloo and ir. G. Du Laing.

Jury member of script students outside Ghent University

Patrik Cromphout, Technical Engineer at Hogeschool Gent (2004-2005). Assessment of treatment methods for combined sewer overflows. Promoters: Prof. ir. C. Vlerick and ir. Ludwig Buts.

MEMBERSHIPS

1999 – present: Dutch-Flemish Ecological Society (NecoV)
2000 – present: International Water Association (IWA)