CIWEM

GOOD PRACTICE IN WATER AND ENVIRONMENTAL MANAGEMENT

NATURAL WASTEWATER TREATMENT

D.D.MARA

SCHOOL OF CIVIL ENGINEERING
UNIVERSITY OF LEEDS

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ABBREVIATIONS

BOD	Biochemical oxygen demand (5-day, 20°C)
CAPEX	Capital expenditure
CVF	Compact vertical-flow (CW)
CW	Constructed wetland(s)
DEM	Deutschmark(s)
DWF	Dry weather flow
FC	Faecal (i.e., thermotolerant) coliforms
FWS	Free-water-surface (CW)
HLR	Hydraulic loading rate(s)
ICE	Institution of Civil Engineers
IWA	International Water Association
NWT	Natural wastewater treatment
OPEX	Operational expenditure
OTR	Oxygen transfer rate(s)
O&M	Operation and maintenance
p.e.	Population equivalent
RBC	Rotating biological contactor(s)
RF	Rock filter(s)
RW	Raw wastewater
SS	Suspended solids (= total suspended solids, TSS)
SSHF	Subsurface horizontal-flow (CW)
SUDS	Sustainable drainage systems
TOC	Total organic carbon
UASB	Upflow anaerobic sludge blanket (reactor)
USD	United States dollar(s)
UWWTD	Urban Waste Water Treatment Directive
VF	Vertical-flow (CW)
WSP	Waste stabilization pond(s)
WHO	World Health Organization

NOTATION

A	Area, m^2 or ha
C	Concentration, mg/l
D	Depth, m
e	Base of Naperian logarithms
e	Evaporation or evapotranspiration, mm/d
k_1	First-order rate constant for BOD removal, d^{-1}
k_A	First-order area-based rate constant for BOD removal, m/d
k_B	First-order rate constant for FC removal, d^{-1}
k_N	First-order area-based rate constant for ammonia-N removal, m/d
L	BOD concentration, mg/l
N	Number of FC per 100 ml
n	Number of maturation ponds
P	Population (or population equivalents) served
Q	Flow, m^3/d
ε	Porosity
θ	Retention time, d
λ_S	Surface BOD loading rate, kg/ha d
φ	Arrhenius constant

Subscripts:

e	effluent
F	facultative (pond)
i	influent
M	maturation (pond)
M1	first maturation (pond)
P	polishing (pond)

CONTACTS

NWT ASSOCIATIONS

UK-based

Constructed Wetland Association
www.constructedwetland.org
Secretary: Paul Cooper
The Ladder House, Cheap St, Chedworth, Cheltenham GL54 4AB

Water Association
Contact: Simon Charter
Ebb and Flow Ltd (see below).

International

International Water Association
Alliance House, 12 Caxton St, London SW1H 0QS
Specialist Groups on:
(a) the use of macrophytes in water pollution control (i.e., constructed wetlands),

(b) waste stabilization ponds, and

(c) small water and wastewater systems.

www.iwahq.org.uk – click on 'Groups'.

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[1]These are small specialist firms. Many of the well-known larger UK consultants also advise on NWT.

1

INTRODUCTION

1.1 NATURAL WASTEWATER TREATMENT

Natural wastewater treatment systems (NWT) are biological treatment systems that require no or very little electrical energy. This is in contradistinction to conventional treatment systems such as activated sludge (Table 1.1).[1] Instead they rely on entirely natural processes, principally biochemical and in particular photosynthetic reactions, to provide the energy required for wastewater treatment. NWT systems are anaerobic or aerobic, and some have both aerobic and anaerobic zones. Because they are not energy-intensive processes, they require a greater volume or area to enable wastewater treatment to proceed to the level required. There is thus a trade-off: either more money is spent on land (either by purchase or by leasing) for NWT, or more money is spent on electromechanical equipment and electrical energy for conventional treatment processes such as activated sludge. Land, of course, is an appreciating asset – a good example of this is from the city of Concord in California, where the city bought land in 1955 for waste stabilization ponds at a cost of USD 50 000 per ha, and in 1975 the land was worth USD 370 000 per ha; inflation in this 20-year period was ~100 percent, so the profit in real terms was USD 270 000 per ha (Oswald, 1976).[2]

In the United Kingdom large areas of land for wastewater treatment are not generally available, and this limits the application of NWT to small populations of a few hundreds where sufficient land is normally available. This last statement is often disputed ("land is just not available"), but in 2003 there were ~680 000 ha of 'set-aside' land (land which the farmers are paid not to farm) in the UK (Department for Environment, Food and Rural Affairs, 2004).[3] The population of the UK was ~60 million in 2003 (Office for National Statistics, 2005), so there were then ~110 m^2 of set-aside land per person - i.e., theoretically more than enough for natural wastewater treatment for in fact the whole of the UK population, and certainly in practice for small villages. So land *is* available. Furthermore it should be available at reasonable cost: in 2003 the annual payment to farmers in England for set-aside land was £240 per ha. Farmers often have an emotional attachment to their land and they may not always be willing to sell it (current farmland prices are approaching £8000 per ha; RICS, 2005), but they might well be prepared to lease it (as, for example, in the case of the land for the waste stabilization ponds at Scrayingham in North Yorkshire – Chapter 4). If the overall land area requirement for NWT were taken as 20 m^2 per person (which is a very high value), a

[1]The UK water industry is a major consumer of electrical energy. Figures from Water UK (2003, 2004) show that energy consumption for wastewater treatment increased substantially from 437 kWh per Ml of wastewater treated in 1998/99 to 814 kWh per Ml in 2002/03; it decreased to 645 kWh per Ml in 2003/04 (Water UK, 2005). For a wastewater flow of 200 litres per person per day, the energy consumption for wastewater treatment is therefore ~48 kWh per person per year. The CO_2 emission from electricity generation from natural gas is 0.1 million tonnes of C (i.e., 0.37 million tonnes of CO_2) per TWh (Department of Trade and Industry, 1998), so the CO_2 emission due to wastewater treatment is ~17 kg per person per year.

[2]The profit is quoted in 1975 dollars. Expressed in 2005 dollars, it is close to USD 1 million per ha (Sahr, 2005).

[3]Set-aside payments have been replaced by the Single Payment Scheme, but set-aside itself continues (Rural Payments Agency and Department for Environment, Food and Rural Affairs, 2004).

Table 1.1 Typical electrical energy requirements of various treatment processed treating a wastewater flow of 1 million US gallons per day (3780 m^3/d)

Treatment process	Electrical energy usage (kWh per year)
Activated sludge	1 000 000
Aerated lagoons	800 000
Rotating biological contactors	120 000
Waste stabilization ponds	Nil

Source: Middlebrooks *et al.* (1982).

village of 500 people would require 1 ha of land for NWT; and even if the farmer were paid as much as £2000 per year for the use of 1 ha of land for NWT (i.e., ~8 times the 2003 set-aside payment and ~25 percent of its sale value), the land rental cost would be only £4 per person per year.

For small rural communities in the UK where NWT is appropriate the wastewater can be expected to be wholly domestic and with essentially no persistent pollutants that would contaminate the land and require removal or remediation prior to the land being put to alternative uses when at some stage in the future it is no longer needed for NWT. If the land is being returned to farmland, then no special precautions are necessary; if for some other purpose (housing, for example), then it may be prudent to let the subsoil rest for a few months prior to being covered with new topsoil.

1.2 MAIN TYPES OF NWT IN THE UK

In the UK the types of NWT systems in current use are:

1. Septic tanks,
2. Constructed wetlands (CW) (Figure 1.1), and
3. Waste stabilization ponds (WSP) (Figure 1.2).

Septic tanks are briefly considered in Chapter 2. CW and WSP are described in some detail in Chapter 3 and 4, respectively. Although they are not much used in the UK, rock filters (Chapter 5) are a proven low-cost tertiary treatment process used to 'polish' WSP effluents which are generally high in algal suspended solids. Guidelines for NWT technology selection, based on land area requirements, performance and costs, are presented in Chapter 6.

Advice on small NWT systems for populations up to ~50 is given in *Sewage Solutions* by Grant *et al*. (2005). The New Zealand manual *Sustainable Wastewater Management: A Handbook for Smaller Communities* (Ministry of the Environment, 2003) is also very useful.

1.3 ADVANTAGES AND DISADVANTAGES OF NWT FOR UK VILLAGES

The advantages of NWT systems for UK villages are low costs (both CAPEX and OPEX), simple maintenance requirements and, assuming good design and good operation and maintenance, high performance and robustness. Their principal perceived disadvantage is the large land area that they require; they are also often (but erroneously) thought to present odour problems and to be unable to produce effluents of an acceptable quality. Specific advantages and disadvantages are detailed in Chapters 3 and 4 for CW and WSP, respectively.

1.4 FLOWS AND LOADS

The most recent UK Code of Practice (British Water, 2005) recommends the following values for small wastewater treatment systems, with 'small' being defined as up to 1000 population equivalents (p.e.):

Wastewater flow:	200 litres per p.e. per day
BOD:	60 g per p.e. per day
Ammonia (NH_3 + NH_4^+):	8 g N per p.e. per day

This Code of Practice is for package plants, but these values are equally applicable to NWT systems. Designers may consider a BOD contribution of 60 g per p.e. per day to be on the high side. This value is the formal definition of 1 p.e. in the Urban Waste Water Treatment Directive (Council of the European Communities, 1991), but in rural areas it is much lower - in rural France, for example, it is 35-40 g per p.e. per day (Pujol and Liénard, 1990). In this Manual a value of 50 g per p.e. per day is used.

All types of wastewater treatment plants, including NWT plants, serving small rural communities can be adversely affected by the discharge of toxic materials by householders (e.g., water-soluble paints, paint-strippers, wood preservatives, biocides of all types, disinfectants) as the available dilution with wastewater from other households is small. To avoid this the community needs advice on the safe disposal of toxic materials. Similarly, to protect NWT serving motorway service areas and those receiving wastewater from restaurants and/or fast food outlets, grease traps must be installed prior to discharge to sewer; the grease traps require regular inspection to ensure that they are being used properly (again, advice on grease disposal must be provided).

Wastewater effluent flows <50 m^3 DWF per day (equivalent to a population of up to 250, assuming a dry-weather wastewater flow of 200 litres per person per day) in England and Wales are not required to be self-monitored under the Environment Agency's monitoring certification scheme ('MCERTS'); self-monitoring is also not required for effluents with only descriptive consents (Environment Agency, 2005).

1.5 PRELIMINARY TREATMENT

Preliminary treatment (screening, grit removal, flow measurement) is fully described in a separate CIWEM Manual of Practice (forthcoming) (see also Marais and van Haandel, 1996, and US Environmental Protection Agency, 2003). In general, preliminary treatment ahead of NWT processes is very simple: there may be only coarse (e.g., 50-mm) screening and no grit removal. If the wastewater is pumped to the treatment site, even screening is often omitted.

1.6 HEALTH AND SAFETY

The Health and Safety aspects of NWT plants are not arduous: the site should be enclosed by a chain-link fence; access should be via a gate which is normally kept locked. There should be suitable notices fixed to the fence at several locations advising what the works are and what danger exists (for example, "Sewage Treatment Ponds. Danger: Deep Water. Keep Out").

Maintenance workers should be provided with (and required to wear) protective clothing (including high-visibility jackets, gloves and boots), and their vehicle should contain a lifebuoy (in the case of waste stabilization ponds), a first-aid kit and a mobile telephone.

2

SEPTIC TANKS

2.1 DESCRIPTION

Septic tanks are the most familiar and common NWT system in the UK and elsewhere. They have been widely used in the UK since Cameron and Cummins patented the system in 1895 (Cooper, 2001a). Developments of the septic tank include Imhoff tanks in 1906 (Seeger, 1999) and, more recently, upflow anaerobic sludge blanket (UASB) reactors in 1977 (van Haandel and Lettinga, 1994).

Septic tanks are anaerobic treatment systems, although in UK winters they act essentially as solids settlement and storage units as anaerobic microbial activity is close to zero at low winter temperatures. They are fully described in a separate CIWEM Manual of Practice (forthcoming).[1] Large septic tanks are briefly described here as these are suitable for the primary treatment of wastewater from small villages in the UK (see also US Environmental Protection Agency, 2000).

2.2 SEPTIC TANKS FOR VILLAGES IN THE UK

Septic tank design in the UK is formally set out in British Standard BS6297:1983 *Code of Practice for Design and Installation of Small Sewage Treatment Works and Cesspools*.[2] The relationship between tank size (C, litres) and population served (P) is given as:

$$C = 180P + 2000 \qquad (2.1)$$

subject to a minimum size of 2720 (i.e., to serve up to four persons).

This sizing is confirmed for England and Wales in 'Approved Document H' of the Building Regulations 2000 (Office of the Deputy Prime Minister, 2002), and for Scotland in the Building (Scotland) Regulations 2004 (Scottish Building Standards Agency, 2004).

Large prefabricated septic tanks (2.6 m diameter) are available in the UK up to a size of 72 000 litres (Figure 2.1).[3] A 72 000-litre tank followed in series by on of 36 000 litres would, according to equation 2.1, serve a population of:

$$P = \frac{(72\,000 + 36\,000) - 2000}{180} = 580$$

However, in accordance with the recommendations of British Water (2005) (see Chapter 1), the wastewater flow should now be taken as 200 litres per person per day, so that the population served by these two septic tanks in series is:

$$P = \frac{(72\,000 + 36\,000) - 2000}{200} = 530 \text{ (say, 500)}$$

There are many villages in the UK with populations below this level where large septic tanks could be advantageously used for primary treatment before constructed wetlands

[1]This Manual of Practice should be consulted for detailed information on septic tanks design, operation (e.g. emptying frequency, sludge disposal) and maintenance.

[2]This standard is currently under revision. See also Payne and Butler (1993) (who estimated that there were then around 819 000 septic tanks in England and Wales).

[3]Titan Pollution Control, West Portway, Andover, Hampshire SP10 3LF (www.titanpc.co.uk).

(Chapter 3) or waste stabilization ponds (Chapter 4). Costs are low: for example, the cost (including delivery and installation) of the two tanks (76 000 and 36 000 litres) for a village of ~500 people is around £75 per person.

For populations of 500 - 2000[4] Imhoff tanks can be advantageously used (they are sometimes used, for example, in France prior to treatment in waste stabilization ponds). They may be regarded as 'old-fashioned' but this is a mistaken view, at least on the grounds of cost and performance.[5] However, UASBs (van Haandel and Lettinga, 1994) are much too complex for small treatment works where their operational benefits (particularly biogas separation and recovery) are not really relevant.

[4]2000 is the limit of 'small' in the Urban Waste Water Treatment Directive (Council of the European Communities, 1991).

[5]Prefabricated concrete Imhoff tanks for up to 1000 p.e. are manufactured by, for example, Gazebo s.p.a., Gatteo (FC), Italy (www.gazebo.it/doc_ing/fosse_imhoff.htm).

3

CONSTRUCTED WETLANDS

3.1 TYPES OF CONSTRUCTED WETLANDS

There are five main types of constructed wetlands (CW) (also often called reed beds after the plant most commonly grown in them: *Phragmites australis*, the common reed):

(a) free-water-surface CW,

(b) subsurface horizontal-flow CW,

(c) vertical-flow CW,

(d) raw-wastewater vertical-flow CW, and

(e) sludge-drying CW.

The first three types provide secondary or tertiary treatment and thus the wastewater is pretreated in a septic tank (Chapter 2) (or other simple solids/liquid separator) or, for example (and as routinely used by Severn Trent Water; Griffin, 2003), a rotating biological contactor (RBC). The fourth is a recent development in France; at present there is only one small system serving 2 p.e. in the UK. The last type is not considered in this Manual as it is described in the CIWEM Manual of Practice on Sludge Treatment and Disposal (forthcoming).

Detailed information on CW is given by IWA Specialist Group (2000) and Berland and Cooper (2001), and in the issues of *Water Science and Technology* which contain the proceedings of the IWA international conferences on CW.[1] Cooper (2001b, 2003), Cooper et al. (1996), Green and Upton (1995), Griffin (2003), Griffin and Pamplin (1998), Nuttal et al. (1997) and Upton et al. (1995) detail CW practice and design in the UK (a less positive view is given by Hiley, 1995). There is also a CIWEM Factsheet on Reed Bed Wastewater Treatment[2] and the UK-based Constructed Wetland Association.[3]

3.2 FREE-WATER-SURFACE CW

These CW are the most common type of CW used in the United States for domestic wastewater treatment (US Environmental Protection Agency, 1993), but they are not used for this purpose to any extent in the UK. In the US they are used for the tertiary treatment of high-quality secondary effluents from large populations (up to 500 000 p.e.) and hence require very large areas of land (which would not be available for this purpose in the UK).

Another disadvantage of the system is that the open water, which is shaded by the plants, encourages mosquito breeding, often to the extent of major nuisance. [In France, when waste stabilization ponds (Chapter 4) were introduced in the 1970s, it was then common practice to plant the downstream half of the second maturation pond with reeds (in an attempt to shade out the pond algae and so reduce effluent BOD and suspended solids), but mosquito breeding was such a problem that the practice was discontinued.]

3.2.1 Mine Drainage Waters

FWS-CW planted with *Phragmites* have been successfully used in the UK to treat mine

[1] Issues available on-line at www.iwaponline.com/wst/toc.htm are: vol. 32, no. 3 (1995); vol. 35, no. 5 (1997); vol. 40, no. 3 (1999); vol. 44, no 11-12 (2001); vol. 48, no. 5 (2003); and vol. 51, no. 9 (2005).
[2] www.ciwem.org.uk/policy/factsheets/fs3.asp
[3] www.constructedwetland.org

drainage waters (Jarvis and Younger, 1999; Batty and Younger, 2002; Coal Authority, 2005). Currently the largest example is at Morlais in the county of Swansea (Figure 3.1), which was designed for a maximum flow of 300 l/s and commissioned in 2003. The water is first settled in two sedimentation lagoons (total area of 1 ha) and then in the FWS-CW (3 ha). The iron concentration is reduced from ~30 mg Fe/l to <1 mg Fe/l (and usually to <0.2 mg Fe/l).[4]

3.3 SUBSURFACE HORIZONTAL-FLOW CW

3.3.1 Description

Subsurface horizontal-flow CW (SSHF-CW) are the type of CW most commonly used for domestic wastewater treatment in the UK, to where they were introduced from Germany in 1985 (Cooper *et al.*, 1989, 1990). Secondary SSHF-CW receive settled wastewater (e.g., septic tank effluent) and tertiary SSHF-CW receive secondary effluents (RBC effluents). The current design used by Severn Trent Water, which has ~400 SSHF- CW, is given in Figure 3.2 (Griffin, 2003). A typical SSHF-CW is shown in Figure 3.3. The bed medium (soil in the original German reed beds) is now most commonly 5 - 10-mm gravel, although coarse sands are sometimes used. The bed depth is usually 0.6 m (this is the maximum depth to which *Phragmites* roots grow) and the medium extends above the water surface by ~0.1 m. The bed is usually lined with a 0.5 - 0.6-mm thick impermeable LDPE or butyl rubber geomembrane.

Although reeds are the plants by far most commonly grown in CW, a variety of other aquatic macrophytes are used - for example, rushes: *Typha latifolia* (bulrush), *Schoenoplectus lacustris* (club rush), and *Juncus effusus* (soft rush). Ornamental flowers can also be grown in CW: for example, yellow flag iris (*Iris pseudacorus*), canna lilies (*Canna* spp.), and arum or calla lilies (*Zantedeschia* spp.) (Belmont *et al.*, 2005). In principle, any plant that can be grown hydroponically can be grown in SSHF-CW (this includes non-root food crops, although clearly UK water authorities and companies are unlikely to become food producers; however, owners of private CW may see this as an advantage. The health risks from consuming foods produced in this way are minimal; WHO, 2006).

3.3.2 Design

SSHF-CW are designed for BOD removal as plug-flow reactors, as follows:

$$L_e = L_i e^{-k_1\theta} \qquad (3.1)$$

where L_e and L_i are the mean effluent and influent BOD (mg/l), respectively; k_1 is the first-order rate constant for BOD removal (day^{-1}); θ is the mean hydraulic retention time (days); and e is the base of Naperian logarithms. The value of k_1 depends on temperature (T, °C):

$$k_{1(T)} = k_{1(20)}\varphi^{T-20} \qquad (3.2)$$

where φ is an Arrhenius constant.

The retention time is volume/flow, and the flow is the mean of the inflow and outflow; therefore:

$$\theta = \frac{\varepsilon AD}{0.5(Q_i + Q_e)} \qquad (3.3)$$

where ε is the porosity of the bed medium (~0.35 - 0.4 for pea gravel); A, the CW area (m^2); D, the CW depth (m); and Q_i and Q_e are the inflow and outflow, respectively (m^3/d).

The outflow is the inflow less the loss due to evapotranspiration:

where e is the net evapotranspiration (i.e.,

$$Q_e = Q_i - 0.001eA \qquad (3.4)$$

evapotranspiration - rainfall) (mm/d). Thus:

$$\theta = \frac{2\varepsilon AD}{2Q_i - 0.001eA} \qquad (3.5)$$

A can now be calculated from known (and/or assumed) values of the parameters in the above equations.

A simpler approach, but still one based on equation 3.1, is typically used in the UK:

[4] Information kindly provided by Parsons Brinckerhoff Ltd, Bristol. See also: www.coal.gov.uk/resources/environment/morlaisminewatertreatmentscheme.cfm

evapotranspiration is ignored[5] temperature and depth are not considered explicitly, and, rather than using k_1 as defined above, an area-based k value (k_A, m/d) is used, as follows (Cooper, 2001):

$$A = \frac{Q_i (\ln L_i - \ln L_e)}{k_A} \quad (3.6)$$

For secondary SSHF-CW the design value of k_A is 0.06 m/d and A is typically 5 m^2 per person; effluent BOD and SS are both ~20 mg/l, but ammonia removal is negligible. For tertiary SSHF-CW k_A is taken as 0.31 m/d and A is typically 0.5 -0.7 m^2 per person; ammonia is removed by nitrification (Cooper, 2001). Severn Trent Water uses a tertiary SSHF-CW area of 0.7 m^2 per person (Griffin, 2003).

The values of L_e used in the above equations are mean values as the equations were developed before it became the practice to specify effluent quality parameters as 95-percentile values. In very general terms a mean value (used in these equations) is about half the required 95-percentile value.

3.3.3 Ammonia Removal

Ammonia is nitrified in the micro-aerobic zones around the roots of the plants and then some of resultant nitrate (~35-40 percent, although there is considerable variation from site to site) is denitrified in the bulk anoxic zone of the gravel bed (Tanner, 2001).[6] Ammonia removal is modelled by equation 3.1 rewritten as follows (Huang *et al.*, 2000):

$$C_e = C_i e^{-k_N \theta} \quad (3.7)$$

where C_e and C_i are the mean ammonia concentrations in the CW effluent and influent respectively (mg N/l); and k_N is the first-order rate constant for ammonia removal at T°C (m/d); θ is given by equation 3.5. For secondary SSHF-CW the variation of k_N with temperature in the range 6-20°C is given by:

$$k_{N(T)} = 0.126(1.008)^{T-20} \quad (3.8)$$

According to Griffin (2005), tertiary SSHF-CW remove ~1-3 mg N/l, although there is some evidence that this increases as the bed matures. Severn Trent Water therefore designs the secondary treatment process for nitrification and does not rely on the tertiary SSHF-CW for any additional removal.

3.3.4 Phosphorus Removal

Phosphorus is removed principally by two mechanisms: adsorption on to the bed medium, and precipitation (mainly as apatite [$Ca_5(PO_4)_3$(F, Cl, OH)]) followed by crystallization (Brix *et al.*, 2001; Molle *et al.*, 2003). Use of media with high P-adsorptivities (e.g., calcite, crushed marble, blast furnace slag) in the bed of a SSHF-CW improves P removal; however, removal is limited and in some cases the P is desorbed after a few weeks or months and appears in the effluent as a high-P pulse. In general vertical-flow CW are better at removing P than SSHF-CW (see section 3.4.1).

3.3.5 Role of Plants

A review of the CW literature (Mara, 2004b) revealed consistent evidence that the plants in SSHF-CW play no role in the removals of BOD, SS, P and faecal bacteria - i.e., there are no significant differences in the percentage removals achieved in planted CW and unplanted controls (as found, for example, by Gersberg et al., 1985; Hiley, 1995; Wood, 1995; Mæhlum and Stålnacke, 1999; Ayaz and Akça, 2001; Coleman *et al.*, 2001; Tanner, 2001; Baptista *et al.*, 2003; and Regmi *et al.*, 2003).

As noted above, the plants have a crucial role in nitrogen removal by providing aerobic conditions adjacent to their roots for nitrification to occur; some of the nitrate so formed is then denitrified in the bulk anoxic zone of the bed. This suggests that the plants are only needed for treatment (as opposed to, for example, aesthetics) when the environmental regulator has specified a discharge consent for ammonia-nitrogen; otherwise it may be better to leave the bed unplanted and to increase the size of the bed medium - i.e., to have a rock filter (Chapter 5). However, the plants are not

[5] This is of course an acceptable assumption in winter; however, in summer a significant proportion of the influent water may be lost through evapotranspiration. Widdas (2005) reports rates of up to 25 mm/day in summer and ~1400 mm/year in Europe. This has consequences for effluent quality when expressed in concentration terms (see the design example in Section 3.7).

[6] In the absence of nitrate, sulphate is used as a source of oxygen and this can lead to odour from H_2S, especially in summer.

active in winter (they transport only enough oxygen to their roots to prevent them from rotting) and the removal of ammonia is lower in winter than in summer (Figure 3.4); indeed it may often be close to zero [Andersson *et al*. (2005) reported a variation in total N removal over a 4-year period in a free-water-surface CW in southern Sweden from ~63 percent in July to ~1 percent in December; see also IWA Specialist Group (2000)].

Despite not contributing to performance (other than ammonia removal in summer), the plants do nevertheless have an important role in SSHF-CW: they prevent the bed from clogging (the bed medium is 5-10-mm gravel, as opposed to the 40-60-mm-rock used in rock filters; see Chapter 5). The major function of the plants is associated with their roots and rhizomes which provide hydraulic pathways through the bed and maintain its hydraulic conductivity at higher rates than those occurring in unplanted beds (the roots and rhizomes expand the bed surface by several cm when the root zone is fully developed, so demonstrating the power of the growing roots). Another important factor is 'wind rock': when the wind blows, the plants sway and this creates small gaps between the base of the stems and the surface of the bed; this punctures the surface and so helps to maintain the bed conductivity.

3.3.6 Storm Sewage Overflow

At some of its small treatment works Severn Trent Water treats 6×DWF in an RBC and the RBC effluent, together with any storm flow >6×DWF, is treated in a combined tertiary/stormwater SSHF-CW sized at 1 m^2 per person. The company has agreed a framework for the relaxation of consent conditions during storm events with the Environment Agency (Table 3.1).

3.3.7 Surface Water Run-off

Constructed wetlands, mainly FWS-CW and SSHF-CW, are used in the UK for surface water run-off from some urban areas, highways and airports; they are 'sustainable drainage systems' (SUDS) which provide a storage and treatment function (Figure 3.5). Shutes *et al.* (2005) give a comprehensive review which should be consulted for further details.

3.4 VERTICAL-FLOW CW

The original concept of vertical-flow constructed wetlands (VF-CW), which are downward-flow systems usually planted with *Phragmites,* was that they were used for tertiary treatment, principally for the removal of ammonia-nitrogen, in a cycle over a few days of load and rest; their action was that of a very simple, discontinuous form of nitrifying trickling filter. However, their role has been reappraised over the last 15 years and now, as reviewed by Cooper (2003, 2005a) (on which much of the following text is based), they continuously receive settled wastewater[7] and are sometimes followed by a tertiary SSHF-CW to reduce effluent SS, so forming a 'hybrid' system. Their design has become much more sophisticated and these recent (or 'second generation') VF-CW are often now referred to as 'compact' vertical-flow constructed wetlands (CVF-CW,

Table 3.1. Consent conditions in dry weather and during storm events.

Consent in dry weather (BOD/SS/Ammonia-N, mg/l)	Consent during storm events[a] (BOD/SS/Ammonia-N, mg/l)
25/45/15	40/60/15
20/30/10	30/50/15
15/25/5	25/45/10

a When the storm overflow is in operation (>6×DWF).
Source: Griffin (2003).

Figure 3.6) (Weedon, 2003). The most usual sizing of CVF-CW is 2 m^2 per person.

The hydraulic loading rate (HLR, litres of settled wastewater per m^2 of filter surface area per day, equivalent to mm/day), the oxygen transfer rate (OTR, g O_2 per m^2 of filter surface area per day), and the size grading of the bed medium are the three critical parameters which control CVF-CW performance (effluent quality, no surface flooding). Surface flooding of the filter does not occur at HLR of <800 mm/day. The minimum value found for OTR is ~28 g O_2/m^2 day. Bed depths are 0.5-1 m; the bed medium grading is typically as follows for a bed depth of 0.7 m:

(a) top 50 mm: 1-mm sand,

(b) next 350 mm: 5-10-mm gravel,

(c) bottom 300 mm: 30-60-mm rounded stones.

Sand alone has been used in 1-m deep CVF-CW (Weedon, 2003; Brix and Arias, 2005); the sand grading is important: it should have a d_{10} between 0.25 and 1.2 mm, a d_{60} between 1 and 4 mm, with a coefficient of uniformity (= d_{60}/d_{10}) of <3.5 (the clay and silt fraction should be <0.5 percent). A recent innovation is the use of crushed waste glass (Figure 3.7).

The oxygen supply is used for both BOD removal and nitrification. Thus OTR is given by Equation 3.9, where 4.3 is the O_2 demand of nitrification (g O_2 per g ammonia-N nitrified).

This equation can be expressed in terms of C_e as shown in Equation 3.10.

OTR is likely to be a function of temperature, but no relationship has been established. Kayser *et al.* (2002) reported the following variation of nitrification performance (i.e., percentage of influent ammonia nitrified) with temperature in a tertiary VF-CW treating the effluent from a facultative waste stabilization pond in northern Germany:

T < 5°C	~50 percent
5°C < T < 10°C	~70 percent
T > 10°C	~90 percent

3.4.1 Phosphorus Removal

Currently few small natural wastewater treatment plants in the UK have a discharge consent for phosphorus (one example is the Tigh Mor Trossachs waste stabilization ponds in Perthshire shown in Figure 4.1; the effluent, which discharges into the pristine Loch Achray, is required to have ≤3 mg P/l). Most of the research and development work on P removal in CW has been done by investigators in Europe and the United States (IWA Specialist Group, 2000).

The main mechanisms of P removal in VF-CW are precipitation, adsorption on to the bed medium and subsequent crystallization (Brix *et al.*, 2001; Molle *et al.*, 2003). There has been considerable effort made in identifying and evaluating suitable P-adsorbing media. Calcite, crushed marble, crushed waste concrete, sea-shell sand and blast furnace slag have all been investigated (Arias *et al.*, 2003; Brix *et al.*, 2001; Arias and Brix, 2005; Korkusuz *et al.*, 2005; Kostura *et al.*, 2005; Molle *et al.,* 2003; Søvik and Kløve, 2005). Rather than adding these P-adsorbents to a VF-CW, it is better from an engineering perspective (for ease of replacing the medium when it is P-saturated) to have a separate filter for P removal. For example, Arias *et al.* (2003) used three upflow calcite filters in series between two VF-CW; P removal was ~2.3 kg P per m^3 of calcite filter. Blast furnace slag appears to be a particularly good P-adsorbent (Korkusuz *et al.*, 2005; Kostura *et al.*, 2005), although it may introduce high metal concentrations in the final effluent. However, more work is needed to develop design guidelines (e.g., upflow vs SSHF filters, number

$$\mathrm{OTR} = \frac{Q[(L_\mathrm{i} - L_\mathrm{e}) + 4.3(C_\mathrm{i} - C_\mathrm{e})]}{A} \qquad (3.9)$$

$$C_\mathrm{e} = C_\mathrm{i} - \frac{(\mathrm{OTR})(A/Q) - (L_\mathrm{i} - L_\mathrm{e})}{4.3} \qquad (3.10)$$

[7]Including wastewater separated in an Aquatron (www.aquatron.se), as used by Weedon (2003).

of filters, optimal medium selection, how best to replace the medium when exhausted, and so on). Alternatively, P removal could be achieved by chemical dosing of the CW influent with removal of the precipitates in the CW bed, although for small works this may not be wholly practical.

3.4.2 Effluent Polishing

The tertiary SSHF-CW for SS removal which sometimes follows a CVF-CW is generally sized at ~0.5 m^2 per person. Alternatively, an unaerated rock filter may be used (Chapter 5).

3.5 Physical Design

Both SSHF-CW and VF-CW are lined with an impermeable plastic liner (at least 0.5 mm thick), unless the soil has an in-situ coefficient of permeability of $\leq 10^{-7}$ m/s, in order to maintain the bed water level and avoid any groundwater pollution.

3.5.1 SSHF-CW

The bed is generally at a longitudinal slope of 1 in 100 (from inlet to outlet) and the outlet is often adjustable to provide the required wastewater depth in the bed (Figure 3.2). The length-to-breadth ratio is in the range 2-10 to 1, with a preference for higher values as this makes the influent distribution easier and more uniform across the width. García *et al*. (2004) found that a depth of 0.27 m in a SSHF-CW planted with *Phragmites* yielded better process efficiency than a depth of 0.5 m; they also confirmed the importance of the areal hydraulic loading rate (i.e., Q_i/A; cf. equation 3.6), but found the length-to-breadth ratio and the bed medium size to be less important (at least within their experimental ranges of 1-2.5 to 1 and 3.5-10-mm gravel, respectively).

The wastewater depth in SSHF-CW is a compromise: if shallow beds are used, the surface area has to be large enough to ensure the required hydraulic throughput and retention time can be achieved; if it is too deep, and the head requirement may be excessive and pumping becomes necessary.

3.5.2 VF-CW, including CVF-CW

Uniform distribution over the wetland surface is crucial. This is closely achieved by dosing the bed at approximately hourly intervals through a network of perforated half-pipes (e.g., gutters) on the bed surface; the objectives are to flood the surface so that oxygen is trapped in the bed voids for use by the bacteria on the bed medium surfaces, and to allow the wastewater to trickle down through the bed before the next dose arrives (hence the critical nature of the HLR). The gravity-operating dosing chamber shown in Figure 4.10 may be used; alternatively the wastewater can be pumped intermittently from a wet well after the primary treatment stage.

The design challenge for VF-CW is to ensure that the influent wastewater does not drain through the bed medium so fast that the bed is unable to flood, but it must pass through the bed at a sufficient rate that the bed has drained by the start of the next doing cycle. Design is complicated by the fact that the bed drainage time changes with time as solids accumulate in the bed.

3.5.3 Planting

Spring and early summer is the optimal time for planting; planting later does not allow for sufficient time for the plants to establish good root growth and they are therefore likely to be either killed or have their growth retarded by frosts. A planting density of four plants per m^2 provides good cover at reasonable cost; commercially grown seedlings offer the simplest and most effective method of establishment. The bed should be flooded after planting to prevent rabbits damaging the immature plants. Provided that regular weeding is undertaken in the first year, and low water levels in the bed are avoided, a dense stand of reeds will develop which requires little attention. However, there may be evidence of plant yellowing and poor growth towards the downstream end of the bed in the first two seasons.

3.6 OPERATION AND MAINTENANCE

Operation and maintenance for both SSHF-CW and CVF-CW is very simple. During the first year of operation the beds need to be weeded to remove invading plants; thereafter this is not normally necessary. The whole works (preliminary and primary treatment units and the CW) should be checked regularly, preferably at least twice per month for SSHF-CW, and several times a week for VF-CW

(including CVF-CW), particularly to ensure that wastewater distribution over the surface is adequate. The water level should be checked at each visit to ensure it is just below the bed surface. In spring the water level may be raised to flood the bed and discourage the growth of invasive weeds which may outcompete the wetland plants if they are allowed to become established. Inlet structures, especially siphonic inlets, should be water-jetted once every 2-3 months.

In late autumn or early winter the reeds in CVF-CW are cut down to a height of ~250 mm. This is not generally done with SSHF-CW, but it may be necessary if 'lodging' occurs - i.e., when a thick layer of wind-flattened reed stems forms a dense thatch over part of the bed surface which prevents plant regrowth in the spring.

3.7 CVF-CW TREATING RAW WASTEWATER

Compact vertical-flow CW systems treating raw wastewater (RWVF-CW) have gradually been developed in France over the past 20 years to treat the wastewater from villages of up to ~1500 people (most serve ~200-700 people) (Groupe Macrophytes et Traitement des Eaux, 2005; Molle *et al.*, 2005; Paing and Voisin, 2005). There were over 400 plants in operation by the end of 2004, with more than 100 commissioned in that year alone. They comprise two stages:

(a) three RWVF-CW in parallel, which discharge into:

(b) two secondary VF-CW in parallel.

Each of these five units is sized at 0.4m^2 per person, giving a total of 2m^2 per person, for separate sewerage systems; for combined systems each unit is sized at 0.5m^2 per person - i.e., a total of 2.5 m^2 per person. However, this sizing is likely to be too small to give the level of nitrification needed to achieve a 95-percentile effluent quality of $\leq$5 mg N/l in the UK.

Only one of the first-stage RWVF-CW is used at any one time: it receives screened wastewater in batches from a self-priming siphon tank at an effective hydraulic loading rate of 0.37 m^3/m^2 day for 3-4 days and is then rested for 6-8 days, during which time the other two units are used sequentially (these design figures are based on 120 g COD (i.e., ~60 g BOD) per person per day, 60 g SS per person per day, 10-12 g TKN per person per day and a wastewater flow of 150 litres per person per day). The second-stage units are alternately loaded, with each being operated for 6-8 days. An operator (usually an employee of the village who also looks after village green spaces and the local cemetery) visits the plant for two hours twice a week to change the units and to do any required simple maintenance.

In some schemes there are three secondary units so that each series of primary and secondary units is operated for one week and then rested for two weeks. In this case the operator visits the plant only once a week.

Clogging of the primary unit may be a problem initially, and also at the end of winter or early spring, before the plants are established or start regrowing. However, the 1-week rest period normally ensures that this is not a major problem.

3.8 CW DESIGN EXAMPLES

A CW system is to be designed for a village with a population of 250. Design parameter values are:

Flow = 200 litres per person per day

BOD = 50 grams per person per day

Ammonia = 8 g N per person per day

Design temperature (winter) = 7°C

Summer temperature = 15°

3.8.1 Solutions

3.8.1.1 Secondary subsurface horizontal-flow CW

The flow is 50 m^3/day and the BOD and ammonia concentrations are 250 mg/l and 40 mg N/l, respectively. Assume that primary treatment in a septic tank achieves 40 percent BOD removal (i.e., the tank effluent BOD = (0.6 × 250) = 150 mg/l), but increases the ammonia concentration (due to partial ammonification of the organic N in the raw wastewater) to 50 mg/l.

The secondary SSHF-CW is designed according to equation 3.6 to produce a mean effluent BOD of 20 mg/l:

$$A = \frac{Q_i(\ln L_i - \ln L_e)}{k_A} = \frac{50(\ln 150 - \ln 20)}{0.06}$$
$$= 1680 \text{ m}^2$$

(i.e., 6.7m^2 per person).

The effluent ammonia concentration in winter is given by equations 3.5, 3.7 and 3.8:

$$k_{N(T)} = 0.126(1.008)^{T-20} = 0.126(1.008)^{7-20}$$
$$= 0.114 \text{ d}^{-1}$$

$$C_e = C_i e^{-k_N(\varepsilon AD/Q)}$$
$$= 50 e^{-[0.114\ (0.4 \times 1680 \times 0.6/50)]}$$
$$= 20 \text{ mg N/l}$$

i.e., an ammonia removal of 60 percent. In summer:

$$k_{N(T)} = 0.126(1.008)^{15-20}$$
$$= 0.121 \text{ d}^{-1}$$

$$C_e = 50 e^{-[0.121\ (0.4 \times 1680 \times 0.6/50)]}$$
$$= 19 \text{ mg N/l}$$

i.e., an ammonia removal of 62 percent.

Evapotranspiration Widdas (2005) quotes an evapotranspiration rate of up to 25 mm/day in Europe. Taking a value of 15 mm/day as a typical maximum in a UK summer, the effluent flow is given by equation 3.4 as:

$$Q_e = Q_i - 0.001eA$$
$$= 50 - (0.001 \times 15 \times 1680)$$
$$= 25 \text{ m}^3/\text{d}$$

(i.e., a wastewater loss due to evapotranspiration of 50 percent).

3.8.1.2 Compact vertical-flow CW

The area per person is 2 m^2, so for 250 people the area is 500 m^2. The mean effluent ammonia concentration is given by equation 3.10, assuming an OTR of 28 g O_2/m^2 day, as:

$$C_e = C_i - \frac{(\text{OTR})(A/Q) - (L_i - L_e)}{4.3}$$
$$= 50 - \frac{(28)(500/50) - (150 - 20)}{4.3}$$
$$= 15 \text{ mg N/l}$$

(i.e., an ammonia removal of 70 percent).

FIGURES

Figure 1.1 Constructed wetland (within the black lines) at Airton wastewater treatment works, North Yorkshire

Figure 1.2 One of the four secondary facultative ponds at Hawkwood College, near Stroud, shortly after commissioning and planting of the marginal plants in August 2005 (see also Figure 4.8)

Figure 2.1 18 000-litre prefabricated cylindrical septic tank
Photograph courtesy of Titan Pollution Control

Figure 3.1 Free-water-surface constructed wetlands treating acid mine drainage water at Morlais, Swansea
Photograph courtesy of Parsons Brinckerhoff Ltd

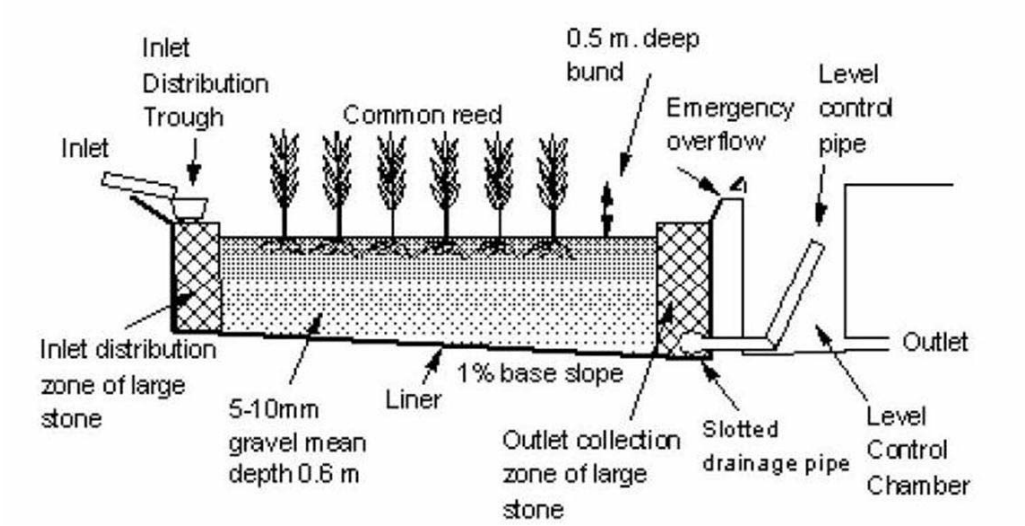

Figure 3.2 Longitudinal section of a subsurface horizontal-flow constructed wetland.
Figure courtesy of Severn Trent Water (Griffin, 2003)

Figure 3.3 Subsurface horizontal-flow constructed wetland at Airton, North Yorkshire, in winter (Figure 1.1 shows this CW in summer)

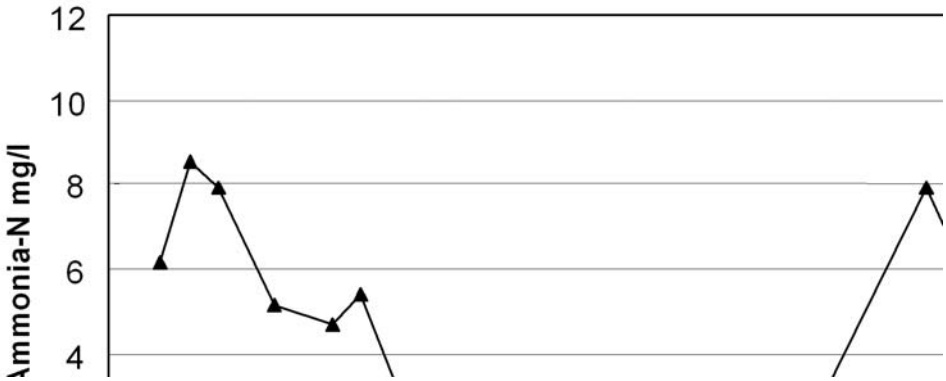

Figure 3.4 Ammonia concentrations in the effluent of a tertiary SSHF-CW planted with Typha at Esholt, Bradford, during April 2004 - April 2005
Figure courtesy of Ms Michelle Johnson, School of Civil Engineering, University of Leeds

Figure 3.5 Free-water-surface constructed SUDS wetland at Appleton Court, Wakefield, West Yorkshire
Photograph courtesy of Dr Nigel Horan, School of Civil Engineering, University of Leeds.

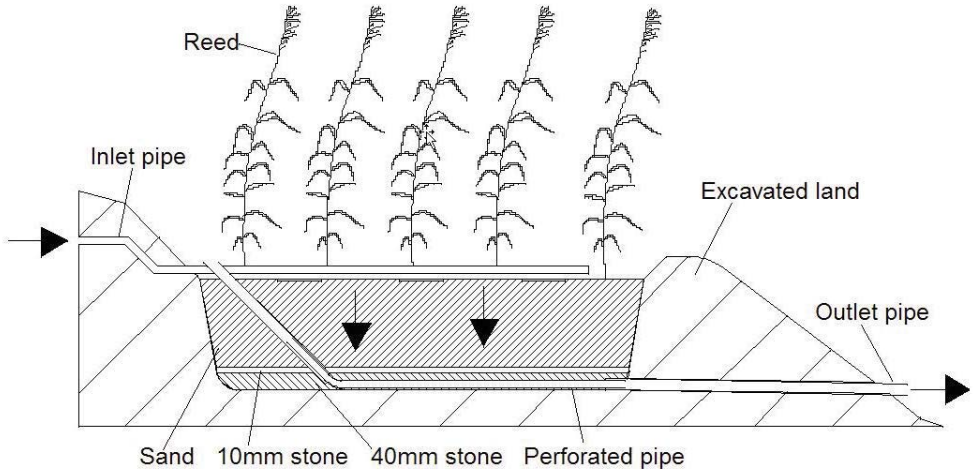

Figure 3.6 Longitudinal section of a compact vertical-flow constructed wetland
Source: Weedon (2003)

Figure 3.7 Bed medium of recycled glass in the VF-CW at Bernard Matthew Ltd, Great Witchingham, Norfolk
Photograph courtesy of Dr Nigel Horan, School of Civil Engineering, University of Leeds

Figure 4.1 The primary facultative pond at Tigh Mor Trossachs, Perthshire, serving a holiday home complex (top). The facultative pond is followed by two maturation ponds (bottom). The final effluent discharges into Loch Achray (beyond the second maturation pond)

Pond design by Iris Water and Design, Castleton, North Yorkshire

Figure 4.2 Primary facultative pond at Scrayingham, North Yorkshire
Pond design by Iris Water and Design, Castleton, North Yorkshire
The Scrayingham WSP won Yorkshire Water the 2005 ICE Yorkshire Award and the 2005 BCIA Environmental Award (Kitching, 2005; BCIA, 2005)

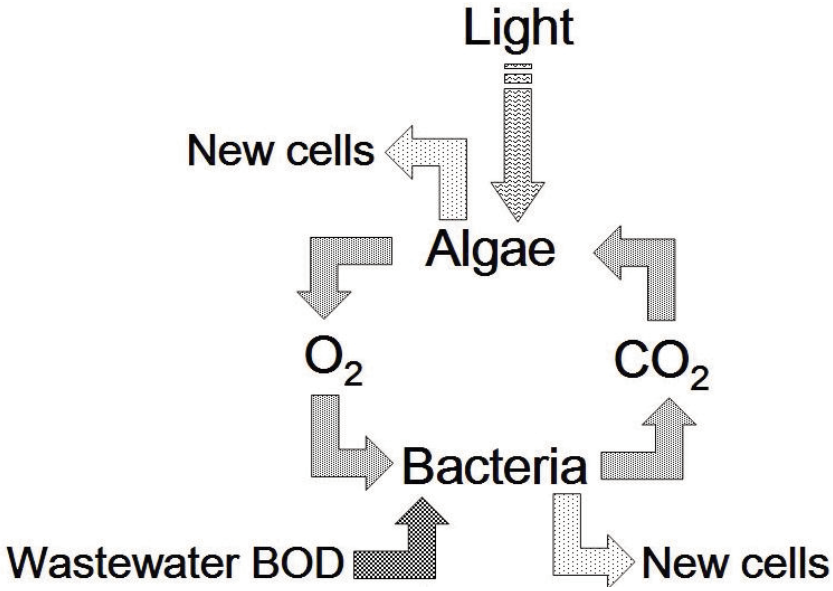

Figure 4.3 The mutualistic relationship between algae and bacteria in facultative and maturation ponds

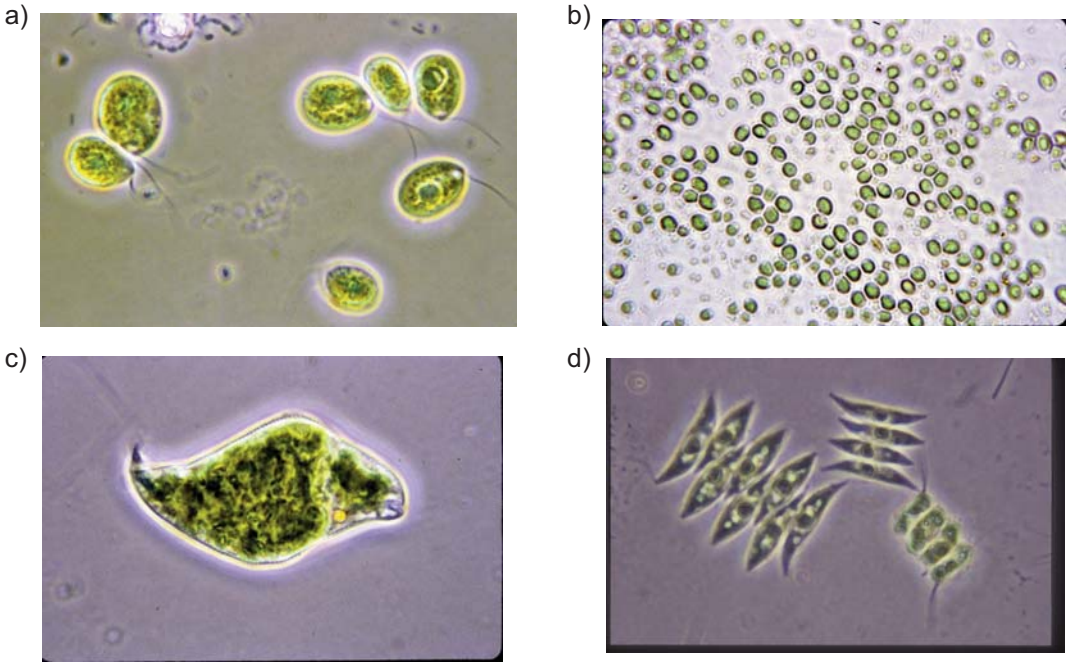

Figure 4.4 Algae typically found in facultative ponds: (a) *Chlamydomonas*; (b) *Chlorella*; (c) *Euglena;* (d) *Scenedesmus*
Photomicrographs courtesy of Professor Francisco Torrella, University of Murcia

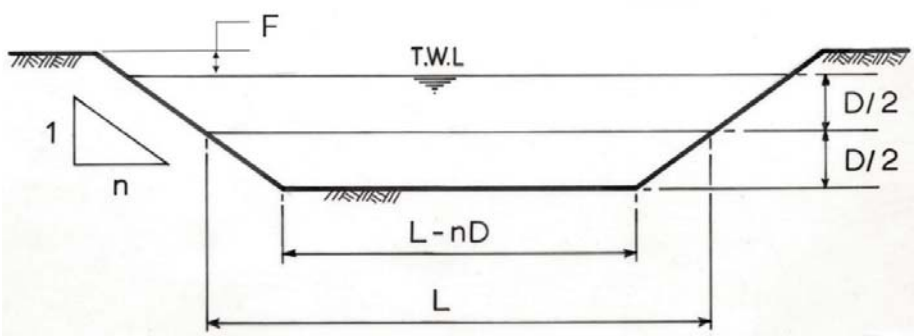

Figure 4.5 Determination of base dimensions of a pond from its mid-depth dimensions (shown here for its mid-depth length, L). F is the freeboard (at least 0.5 m to prevent wind-induced waves overtopping the embankment)

Figure 4.7 The primary Aero-fac® lagoon at Errol, by Dundee, showing the wind-powered aerators
Photograph courtesy of Ms Michelle Johnson, School of Civil Engineering, University of Leeds

Figure 4.8 One of the four secondary facultative ponds at Hawkwood College, near Stroud (see also Figure 1.2). The pond contents are internally circulated by a 200-W submersible pump (housed at P) which pumps the contents at a rate of ~100 l/min to the top of the 'flowform' cascade (bottom left). This induces a gentle circular motion in the pond. (A similar cascade can be seen in the primary facultative pond shown in Figure 4.1.) Each of the four ponds has a different design of cascade, each of which induces a slightly different circulation pattern in the pond; this will permit the 'best' system, in terms of performance and biodiversity, to be established. Apart from treating wastewater, the idea of this WSP system was to produce a very aesthetic, tranquil locality for contemplation and meditation (a bench will be located on the small gravelled area shown in the top right corner)
Pond design by Ebb & Flow Ltd, Nailsworth, Gloucestershire

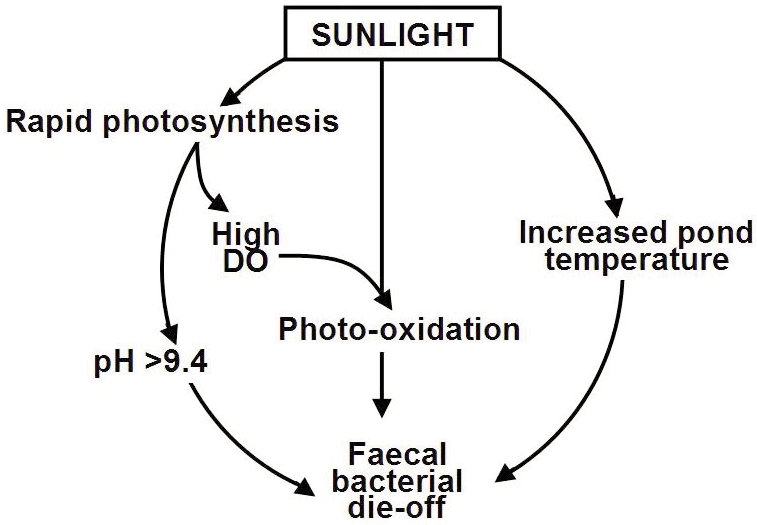

Figure 4.9 Principal mechanisms of faecal bacterial die-off in facultative and maturation ponds

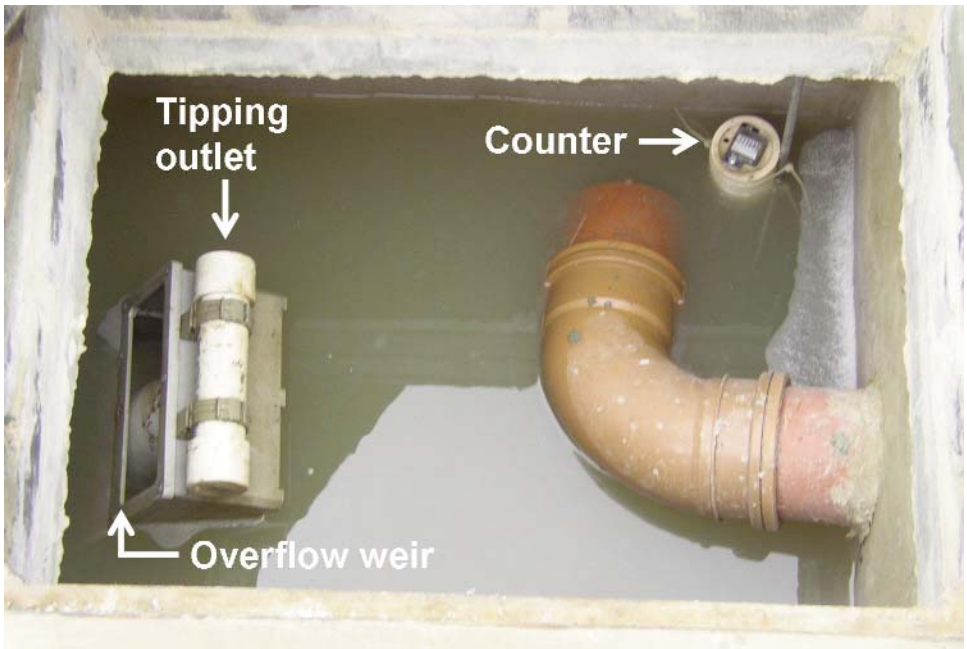

Figure 4.10 Dosing chamber feeding septic tank effluent to the secondary facultative ponds at Hawkwood College, near Stroud. A, inlet from septic tank. When the wastewater rises to the level of the effluent weir, it overflows into the outlet which then quickly tips over and the chamber contents are discharged into the receiving pond. The counter has an electrode set at the height of the overflow weir, so enabling the daily (or weekly) flow to be determined [= (chamber volume, m^3) × (difference in counter readings over a 24-hour (or 7-day) period)]

Chamber design by Mark Moodie (formerly of Elemental Solutions, Orcop, Hereford)

Figure 4.11 Scum baffle at the inlet of a primary facultative pond in France

Figure 4.12 Primary facultative pond at Botton Village, near Castleton, North Yorkshire, showing marginal planting
Pond design by Iris Water and Design, Castleton, North Yorkshire

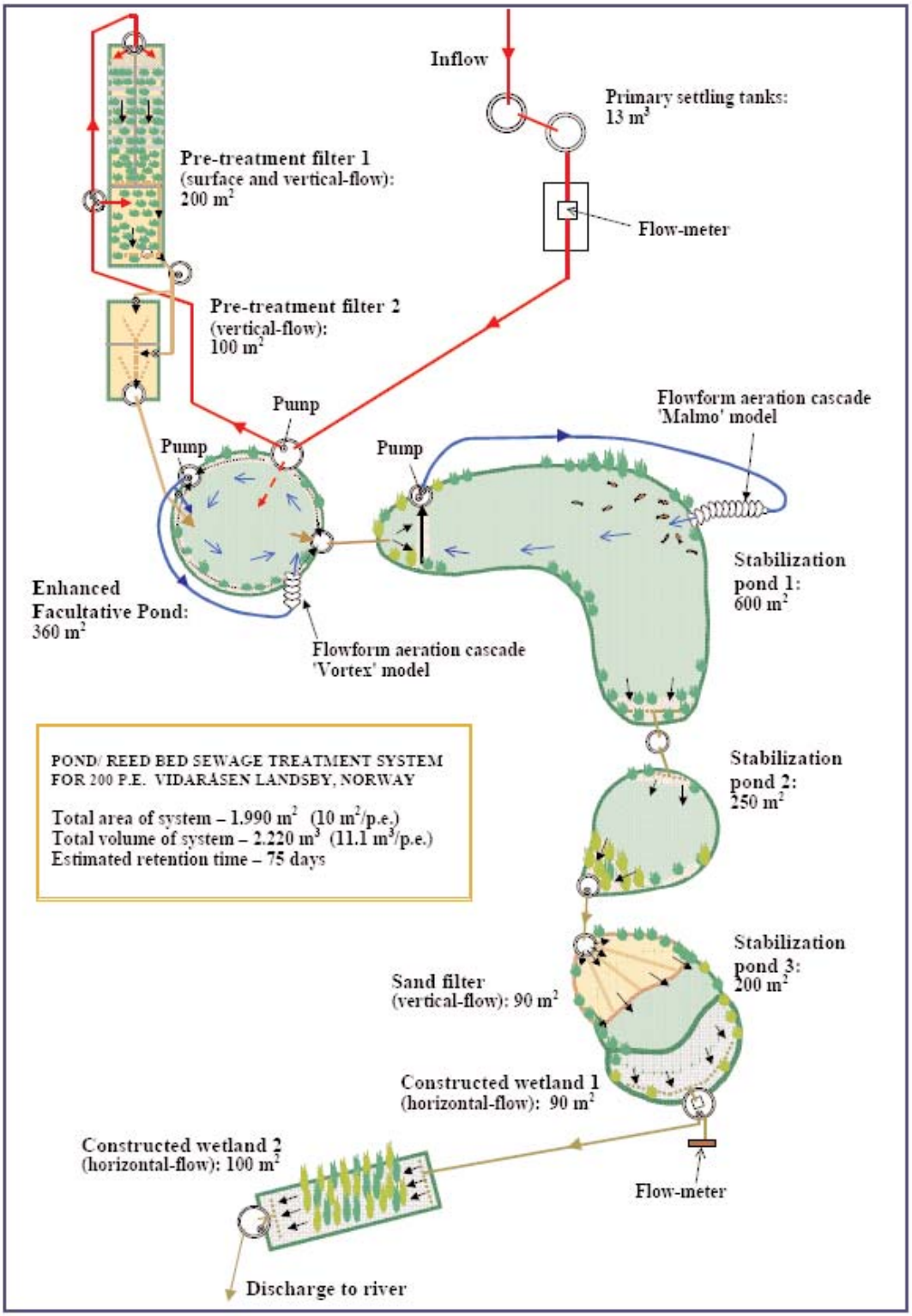

Figure 4.13 Combined CW-WSP treatment system at Vidaråsen, Norway
Source: Browne and Jenssen (2005)

Figure 5.1 Rock filter around the outlet of the facultative pond at Scrayingham, North Yorkshire
Pond design by Iris Water and Design, Castleton, North Yorkshire

Figure 5.2 Shallow partially planted pond receiving the effluent from the facultative pond at Scrayingham, North Yorkshire, showing the cross-pond gravel filters (top, February 2005; left, August 2005)

Pond design by Iris Water and Design, Castleton, North Yorkshire

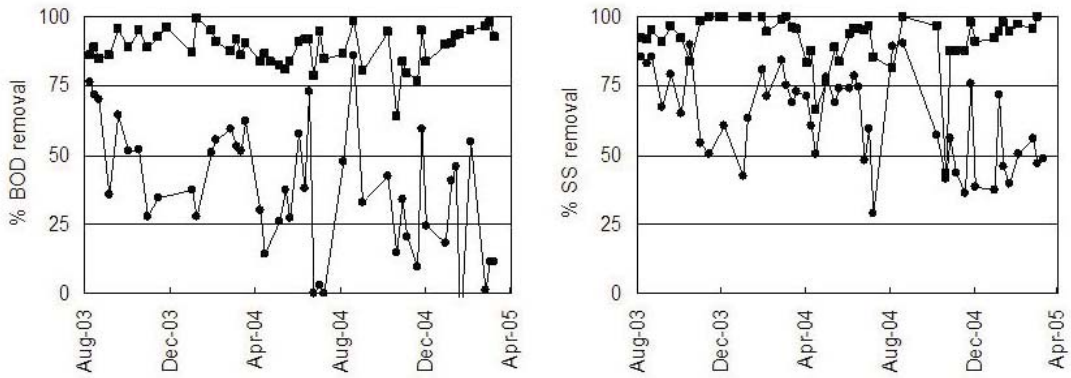

Figure 5.3 BOD and SS removals in aerated (■) and unaerated (•) rock filters at Esholt, Bradford
Source: Mara and Johnson (2005)

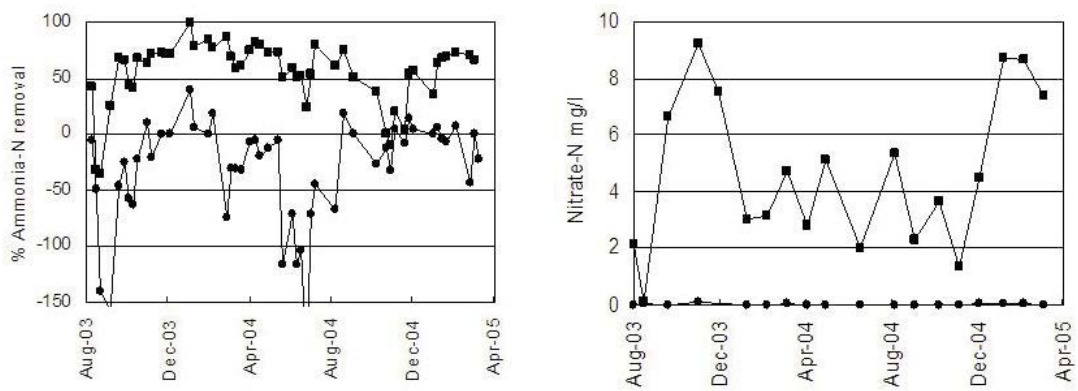

Figure 5.4 Ammonia removal and nitrate production in aerated (■) and unaerated (•) rock filters at Esholt, Bradford. [The negative ammonia removals in the unaerated filter are due to the partial ammonification of the algal organic nitrogen in the influent from the facultative pond.]
Source: Mara and Johnson (2005)

Figure 5.5 The pilot-scale rock filters (bottom, aerated RF; middle, unaerated RF) and subsurface horizontal-flow constructed wetland (top) at Esholt, Bradford. Each unit is 4 × 0.5 × 0.5 m. Above, early summer; below, winter

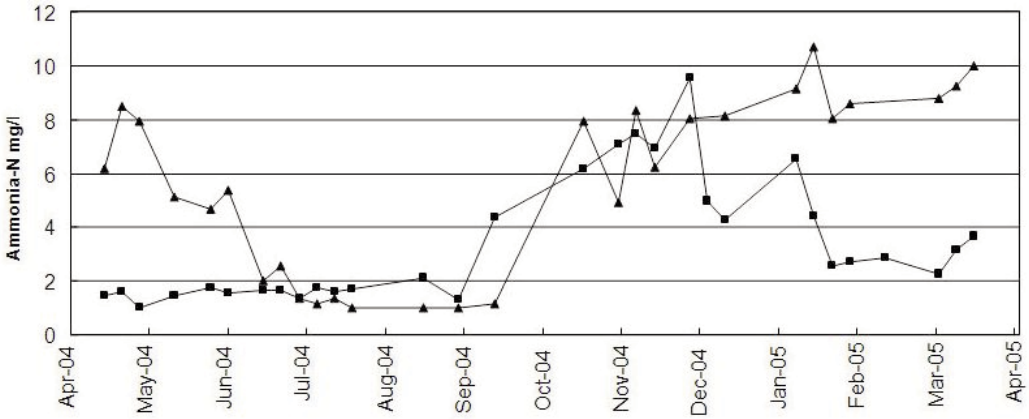

Figure 5.6 Ammonia concentrations in the effluents of the pilot-scale aerated rock filter (■) and subsurface horizontal-flow constructed wetland (▲) at Esholt, Bradford
Figure courtesy of Michelle Johnson, School of Civil Engineering, University of Leeds

4

WASTE STABILIZATION PONDS

4.1 INTRODUCTION

Waste stabilization ponds (WSP) have not been as popular in the UK as constructed wetlands (Chapter 3). There are only ~50 systems and all but two are privately owned (the two exceptions are Yorkshire Water's WSP at Scrayingham in North Yorkshire and Scottish Water's Aero-fac®[1] lagoons at Errol, by Dundee). These privately owned WSP serve small populations (2-1000 people) in individual homes, holiday apartment complexes (Figure 4.1), rural 'self-sufficient' communities (for example, those operated by the Camphill Trust[2]), privately owned Estate villages, and a motorway service area (Abis, 2002). However, performance data have been only been reported for one full-scale UK WSP system (Mara *et al*., 1998); more data are available from the University of Leeds' pilot-scale WSP located at Yorkshire Water's wastewater treatment works at Esholt, Bradford; Mara *et al*., 2002). In contrast to the UK, there are close to 3000 WSP systems in France (Cemagref and Agences de l'Eau, 1997; Racault and Boutin, 2005). Bucksteeg (1987) reported ~1100 systems in Germany; this number has now grown to ~2500, including ~1500 in Bavaria alone (Schleypen, 2003).

An introduction to WSP for non-specialists is given by Peña Varón and Mara (2004). More detailed information is given in Mara and Pearson (1998), Mara (2004) and Shilton (2006), as well as in the issues of *Water Science and Technology* which contain the proceedings of the IWA international and regional conferences on WSP.[3]

Properly designed and constructed WSP systems are robust, simple to operate and maintain, produce excess sludge very infrequently, and do not smell. They require a greater land area than conventional electromechanical treatment plants, but this is not a serious disadvantage for small rural communities (this point is discussed further in Chapters 1 and 6).

WSP systems in the UK comprise a facultative pond and one or two maturation ponds. Anaerobic ponds are not used, doubtless because of a fear of odour release (but they are commonly used in southern Germany and odour release is not experienced).

4.2 FACULTATIVE PONDS

4.2.1 Description

Facultative ponds are either 'primary' facultative ponds, which receive untreated wastewater (i.e., after only preliminary treatment) (Figure 4.2), or 'secondary' facultative ponds, which receive the effluent from septic tanks[4] (or anaerobic ponds). In both cases the pond working depth is 1–2 m, with 1.5 m being most commonly used. Wastewater treatment is achieved by the mutualistic activities of bacteria and algae

[1]Aero-fac is a registered trademark of LAS International (Europe) Ltd, King's Lynn PE34 3ES.

[2]www.camphilll.org.uk

[3]Issues available on-line at www.iwaponline.com/wst/toc.htm: vol. 31, no. 12 (1995); vol. 33, no. 7 (1996); vol. 42, no.10-11 (2000); vol. 45, no. 1 (2002); vol. 48, no.2 (2003); and vol. 51, no. 12 (2005).

[4]See the design examples in Section 4.8 which show the advantage, in terms of reduced land area requirements, of pretreatment in a septic tank.

(Figure 4.2): the usual genera of heterotrophic bacteria found in biological wastewater treatment plants oxidize the BOD and, and this is the microbiological feature unique to facultative and maturation ponds, several genera of mainly green micro-algae (Figure 4.4 and Table 4.1) photosynthetically produce the oxygen needed by the bacteria; and the bacteria produce the CO_2 fixed into cell carbon by the algae as they photosynthesize.[5] The general equation for algal photosynthesis is (Oswald, 1988):

$$106CO_2 + 236H_2\mathbf{O} + 16NH_4^+ + HPO_4^{2-} \xrightarrow{\text{light}} C_{106}H_{181}O_{45}N_{16}P + 118\mathbf{O_2} + 171H_2O + 14H^+$$

This shows that oxygen is produced as a by-product from water and that 1 g of algae produces ~1.55 g of oxygen (sufficient to satisfy the oxygen demand of 1.55 g of ultimate BOD or ~1 g of BOD_5). The algae most commonly found in the fairly turbid waters of facultative ponds are motile genera as these can optimize their position in the water column in relation to environmental factors, particularly the incident light intensity. The algae also have an important role in removing faecal bacteria (see Section 4.3).

The effluent from both primary and secondary facultative ponds (and also maturation ponds) contain high numbers of algae which contribute to effluent SS and BOD. The BOD in facultative

Table 4.1: Algal species commonly found in facultative and maturation ponds

Alga	Facultative ponds	Maturation ponds
Euglenophyta		
Euglena[*, E]	+	+
Phacus[*, E]	+	+
Chlorophyta		
Chlamydomonas[*, E]	+	+
*Chlorogonium**	+	+
Eudorina	+	+
*Pandorina**	+	+
*Pyrobotrys**	+	+
Ankistrodesmus	–	+
Chlorella[E]	+	+
Micratinium	–	+
Scenedesmus[E]	–	+
Selenastrum	–	+
*Carteria**	+	+
Coelastrum	–	+
Dictyosphaerium	–	+
Oocystis	–	+
*Volvox**	+	–

Notes:

A. *, motile; E, alga found by Abis (2002) in primary facultative ponds at Esholt, Bradford; +, present; –, absent.
B. An identification key to pond algae is given in Mara and Pearson (1998).

[5]Some O_2 and some CO_2 enters the pond from the atmoshere, but most is produced by the pond algae and bacteria

pond effluents (also in maturation pond effluents) is thus expressed as either 'unfiltered BOD', which includes the BOD due to the algae, or 'filtered BOD' which excludes it (filtered BOD is measured in the filtrate from standard filtration procedures for measuring SS). Unfiltered BOD removal in facultative ponds in the UK is 70–90 percent, filtered BOD removal >95 percent, and SS removal >90 percent (Abis, 2002; Abis and Mara, 2003, 2004, 2005a).

The Urban Waste Water Treatment Directive (Council of the European Communities, 1991) requires WSP effluents to contain ≤25 mg filtered BOD/l and ≤150 mg SS/l. This recognises the difference between algal and non-algal BOD and SS. In the receiving watercourse the algae produce more O_2 during daylight hours than they consume by respiration at night, so they make a positive contribution to the DO balance in the receiving watercourse. Furthermore WSP algae are consumed by protozoa and rotifers in the stream.

4.2.2 Process design

Facultative ponds are designed on the basis of a permissible BOD surface loading (λ_S, expressed in units of kg BOD per hectare per day):

$$\lambda_S = \frac{10 L_i Q}{A_F} \qquad (4.1)$$

where L_i is the influent BOD (mg/l); Q the inflow (m^3/d); and A_F the facultative pond area (m^2).

The permissible loading varies with mean monthly temperatures (T, °C) as follows (Mara, 1987):

$$\lambda_S = 350(1.107 - 0.002T)^{T-25} \qquad (4.2)$$

subject to λ_S = 80 kg/ha d at temperatures ≤8°C. Since winter temperatures in the UK are <8°C, the design loading adopted is 80 kg/ha d (Abis, 2002; Abis and Mara, 2003, 2004). This design loading is used in New Zealand (Ministry of Works and Development, 1974) and is close to the value used in France (83 kg/ha d; Cemagref and Agences de l'Eau, 1997).

Thus, for this design loading, the facultative pond area is given by:

$$A_F = \frac{L_i Q}{8} \qquad (4.3)$$

In fact this area is the mid-depth area of the pond, from which the surface and base areas and hence dimensions (using a length-to breadth ratio of 2–3 to 1) are determined, as shown in Figure 4.5.

4.2.2.1 Retention time

The mean hydraulic retention time (θ_F, days) is volume/flow. For facultative ponds the flow is the mean of the inflow and outflow:

$$\theta_F = \frac{A_F D_F}{0.5(Q_i + Q_e)} \qquad (4.4)$$

where D_F is the facultative pond depth (1.5 m); and Q_i and Q_e are the inflow and outflow, respectively (m^3/d).

The outflow is the inflow less losses due to evaporation and seepage. Assuming seepage is negligible (see Section 4.5), then:

$$Q_e = Q_i - 0.001 e A_F \qquad (4.5)$$

where e is the net evaporation (i.e., evaporation – rainfall) (mm/d). Thus:

$$\theta_F = \frac{2 A_F D_F}{2Q_i - 0.001 e A_F} \qquad (4.6)$$

Effluent BOD

The unfiltered BOD in the facultative pond effluent (L_e, mg/l) is calculated from the first-order equation:

$$L_e = \frac{L_i}{1 + k_{1(T)}\theta_F} \qquad (4.7)$$

where $k_{1(T)}$ is the value of the first-order rate constant for unfiltered BOD removal at T°C (day^{-1}), given by:

$$k_{1(T)} = 0.3(1.05)^{T-20} \qquad (4.8)$$

This design value of k_1 at 20°C (0.3 day^{-1}) is for

primary facultative ponds; for secondary ponds it is 0.1 day^{-1}.

The filtered BOD is ~0.3L_e. This assumes that 70 percent of the effluent BOD is due to the algae (in practice the range is 70–90 percent) (Abis, 2002; Abis and Mara, 2003).

Facultative ponds in the UK loaded at 80 kg BOD/ha d produce an effluent complying with the UWWTD requirements for WSP effluents (Abis, 2002; Abis and Mara, 2003). However, there is currently only one pond system in the UK (the Aero-fac® lagoon system at Errol, by Dundee; see Section 4.2.4) which has had the UWWTD pond effluent quality applied to it.

The design procedure is illustrated in the design example given in Section 4.8.

4.2.3 Odour

WSP that are not overloaded do not smell. Field observations in summer 2002 on two full-scale WSP systems in North Yorkshire and the pilot-scale WSP at Esholt, Bradford, using three human noses and an electronic nose (described in Figueiredo, 2002) found no odour (less, in fact, than at conventional wastewater treatment works). Early work in the United States found no odour from WSP when the sulphate concentration in the raw wastewater was <500 mg SO_4^{2-}/l (higher concentrations would lead to correspondingly higher in-pond sulphide concentrations with consequently greater risks of H_2S release) (Gloyna and Espino, 1969).[6]

4.2.4 Mixed facultative ponds

Abis (2002) found that the algae in primary facultative ponds 'struggled' to survive in winter at temperatures <5°C and light intensities of ~20 W/m^2. Gentle mixing (really, gentle stirring or circulation) of the ponds is beneficial, and this can be achieved by floating electric mixer/circulator pumps[7] or by wind-powered aerator/mixers.[8] Stirred ponds are usually 2–3 m deep (vs 1.5 m for unstirred ponds). Electric mixer/circulator pumps (Figure 4.6) are inexpensive: a 250-watt unit for a facultative pond serving a population of up to ~500 costs around USD 4600 (f.o.b.)[7]; energy input is minimal: ~0.1 W/m^3 (vs ~5 W/m^3 in a completely mixed aerated lagoon, for example).

Wind-powered aerator/mixers are used at Scottish Water's Aero-fac® lagoon system at Errol, by Dundee (Figure 4.7). These lagoons are also provided with supplementary diffused aeration which switches on automatically when the dissolved oxygen concentration in the lagoon falls below 4 mg/l. The final effluent quality is much better than required: ~8 mg unfiltered BOD/l and ~6 mg filtered BOD/l (the consent is ≤30 mg filtered BOD/l) (LAS International, 2005; see also Salih, 2004 and Horan *et al*., 2005).[9] The cost of the Errol lagoons was £840 000 for a design population of 2000 (the whole scheme, including interceptor sewers, rising main, inlet works and effluent outfall to the River Tay, cost £1.6 million, or £800 per person, in 2001).

A good alternative for small facultative ponds (serving up to around ~500 people) is to pretreat the wastewater in a septic tank (Chapter 2; see also the design example in Appendix I to this Chapter) and/or internally circulate the facultative pond contents by means of a pump and a cascade (e.g., a series of 'Flowforms'[10]) (Figure 4.8).

4.3 MATURATION PONDS

4.3.1 Description

The principal function of maturation ponds is threefold: (a) to reduce the BOD and SS in the facultative pond effluent; (b) to remove faecal bacteria; and (c) to reduce the concentration of ammonia-nitrogen. The decision whether to have maturation ponds or rock filters (Chapter 5), or constructed wetlands (Chapter 2), should be taken carefully as maturation ponds have a large land area requirement (for example, in France a facultative pond designed with 6 m^2 per person is followed by two maturation

[6]The maximum permissible sulphate concentration in drinking water is 250 mg/l; sulphate concentrations in wastewater are higher than in drinking water as detergents contain up to 40 percent $NaSO_4$ (w/w).
[7]For example, the model Enviro 700 floating circulator pump manufactured by Sunset Solar Systems Ltd, Assiniboia, SK S0H 0B0, Canada (www.pondmill.com).
[8]For example, the Mark 3 wind-powered aerator/mixer manufactured by LAS International (Europe) Ltd, King's Lynn PE34 3ES (www.lasinternational.com).
[9]The population currently served is ~1200 (*vs* the design population of 2000).
[10]For example, www.flowforms.com. For a more philosophical (indeed 'anthroposophical') account see Moodie (1997).

ponds, each with an area of 2.5 m^2 per person; Cemagref and Agences de l'Eau, 1997).

4.3.1.1. Faecal bacterial removal

In facultative and maturation ponds the following mechanisms are mainly responsible for the die-off of faecal bacteria (Figure 4.9):

(a) high sunlight intensity increases the in-pond temperature and faecal bacteria die more quickly with increasing temperature;

(b) algal demand for CO_2 during periods of rapid photosynthesis (which generally occur in the late morning and early afternoon) is greater than its supply from the in-pond bacteria (Figure 4.2); as a result carbonate and bicarbonate ions dissociate to provide more CO_2:

$$2HCO_3^- \rightarrow CO_3^{2-} + H_2O + CO_2$$

$$CO_3^{2-} + H_2O \rightarrow 2OH^- + CO_2$$

The OH- ions accumulate and can cause the in-pond pH to rise above 9.4, which is the critical threshold for faecal bacterial die-off (Parhad and Rao, 1974; Pearson et al., 1987); even in the UK in winter in-pond pH on a very sunny afternoon can rise to >10 in a primary maturation pond with faecal coliform numbers <1000 per 100 ml in the pond effluent.11

(c) The combination of high visible light intensity and high dissolved oxygen concentrations (>15 mg/l) leads to very rapid photo-oxidative death of faecal bacteria; this effect is enhanced at high in-pond pH values (Curtis *et al*., 1992).

4.3.1.2 Ammonia removal

In facultative and maturation ponds ammonia is removed mainly by the following mechanism:

algal uptake → sedimentation of organic nitrogen in dead algal cells → accumulation in pond sludge (with partial ammonification of the organic nitrogen and feedback to the bulk pond liquid phase).

Some ammonia may be lost by volatilization at high pH, but in fact the loss observed is very small (Epworth, 2004; Camargo Valero and Mara, 2005).

4.3.2 Process design

Maturation pond depths are usually 1-1.5 m (with a preference for 1 m). The first maturation pond is designed subject to three constraints:

(a) its retention time should not be greater than that of the preceding facultative pond,

(b) its retention time should not, in temperate climates, be less than 5 days, and

(c) the surface BOD loading on it should not be more than that on the facultative pond (and preferably no more than 70 percent of the facultative pond loading).

Considering constraint (c) first, and writing equation 4.1 for the first maturation pond, with $Q/A = D/\theta$ and $L_i = L_{e(Fac)}$ (as determined from equation 4.7):

$$\lambda_{S(M1)} = \frac{10 L_{e(Fac)} D_{M1}}{\theta_{M1}} \qquad (4.9)$$

where the subscript M1 refers to the first maturation pond. Rearranging and writing $\lambda_{S(M1)}$ as 0.7 $\lambda_{S(Fac)}$:

$$\theta_{M1} = \frac{10 L_{e(Fac)} D_{M1}}{0.7 \lambda_{S(Fac)}} \qquad (4.10)$$

The maturation ponds can now be designed either for faecal bacterial removal or ammonia-N removal (or both).

4.3.2.1 Faecal bacterial removal

The bacteria of interest are faecal (or thermotolerant) coliforms or (and preferably) *Escherichia coli.* The design equations of Marais (1974) are used, as follows:

$$N_e = \frac{N_i}{(1 + k_{B(T)}\theta_F)(1 + k_{B(T)}\theta_{M1})(1 + k_{B(T)}\theta_M)^n} \qquad (4.11)$$

[11]Personal observation, Michelle Johnson (School of Civil Engineering, University of Leeds).

where N_e and N_i are the number of faecal bacteria per 100 ml of final effluent and untreated wastewater, respectively; $k_{B(T)}$ the first-order rate constant for faecal bacterial removal (day^{-1}); θ_M the retention time in each maturation pond subsequent to the first maturation pond (days); n the number of maturation ponds subsequent to the first (which, at this stage of the design, are assumed to be of the same size and shape). The value of $k_{B(T)}$ is strongly temperature-dependent:

$$k_{B(T)} = 2.6(1.19)^{T-20} \qquad (4.12)$$

Equation 4.12 was derived from field data in the temperature range 2-21°C.

Equation 4.11 is rearranged as follows:

$$\theta_M = \frac{\left(\dfrac{N_i}{N_e(1 + k_{B(T)}\theta_F)(1 + k_{B(T)}\theta_{M1})}\right)^{1/n} - 1}{k_{B(T)}} \qquad (4.13)$$

This equation is then solved for n = 1, then for n = 2, and so on, until the calculated value of θ_M is less than 5 days (the minimum permissible retention time to avoid massive hydraulic short-circuiting and algal wash-out). The designer then selects the most appropriate combination of n and θ_M (usually the one requiring least land). The procedure is illustrated in the design example given in Section 4.8.

The area of each maturation pond (including the first) is determined as follows:

$$A_M = \frac{2Q_i}{2D_M + 0.001e\theta_M} \qquad (4.14)$$

where Q_i is the inflow (i.e., the outflow from the previous pond, determined from equation 4.5). The outflow from the pond whose area is being calculated is then determined; it is used as the inflow to the next maturation pond.

4.3.2.2 Ammonia-N removal

The equation of Pano and Middlebrooks (1982) for temperatures below 20°C (developed in the United States, but found to give reasonable results for ponds in the UK – Abis, 2002) is used:

$$C_e = \frac{C_i}{1 + (A/Q)(0.0038 + 0.000134T)e^x} \qquad (4.15)$$

where C_e and C_i are the effluent and influent ammonia concentrations (mg N/l), respectively; and e is the base of Naperian logarithms; and x = (1.041 + 0.044T)(pH - 6.6). The equation is applied first to the facultative pond, and then in turn to each maturation pond, in order to determine the ammonia concentration in the final effluent.

4.4 POLISHING PONDS

Polishing ponds are short-retention-time ponds occasionally used as a final treatment stage at conventional treatment works. Their main function is to 'smooth out' fluctuations in effluent BOD and SS so that the effluent complies with its consent requirements. Their retention time is ~1 day (longer retention times would encourage algal growth, especially in summer, with a consequent increase in effluent BOD and SS). Most polishing ponds are not akin to maturation ponds (which, as described above, have entirely different functions), although some have been used specifically for bacterial removal (Toms *et al*., 1975). When designed for faecal bacterial removal, the following version of equation 4.13 should be used:

$$\theta_P = \frac{(N_i / N_e)^{1/n} - 1}{k_{B(T)}} \qquad (4.16)$$

where θ_P is the retention time in each of n polishing ponds (days); and N_i and N_e are the *E. coli* numbers per 100 ml of the influent to the first polishing pond and the effluent from the last, respectively.

4.5 PHYSICAL DESIGN

The physical design of WSP is at least as important as process design: a study of malfunctioning WSP in France found that half were malfunctioning because of problems (mainly geotechnical problems) which were not adequately addressed during the design stage (Drakides and Trotouin, 1991).

Particular attention should be paid to the WSP location. The site should be at least 200 m

from the nearest houses, and it should slope gently to allow inter-pond flow by gravity. The soil should have an in-situ coefficient of permeability of $<10^{-7}$ m/s, otherwise the ponds should be lined. Embankment slopes are commonly 1 in 3 internally and 1 in 2-2.5 externally;[12] the embankments are planted with grass to minimize erosion. The length-to-breadth ratio of primary facultative ponds is typically 2-3 to 1; for secondary facultative and maturation ponds it can be much higher (up to 10 to 1). Liquid depths are generally 1.5 m in facultative ponds and 1 m in maturation ponds. In order to prevent embankment erosion by wind-induced waves, the embankment should be protected with precast concrete paving slabs set at top water level (stone rip-rap and lean in-situ concrete may also be used).

Conventional preliminary treatment (screening and grit removal) is not normally required at small WSP installations. In France a coarse (50-mm) screen is often used to remove large objects (Drakides and Trotouin, 1991). If necessary, simple grit removal channels can be used (Marais and van Haandel, 1996). Figure 4.10 shows an inlet dosing chamber which also serves as a flow recorder.

Simple inlets and outlets should be located in diagonally opposite corners of the pond. A scum baffle around the inlet reduces material floating on the pond surface (Figure 4.11). Inlet pipes should discharge close to the side of the pond and below the pond surface to minimize floating materials. Outlet pipes should be protected by a scum guard to prevent blockage due to floating material which might enter the pipe.

Many WSP in the UK have been designed with marginal plants (Figures 1.2, 4.8 and 4.12). This improves site aesthetics and aquatic biodiversity, but it is not known if there is any resultant measurable effect on performance. There is, however, some evidence that marginal planting decreases the likelihood of duckweed infestation and blooms of *Daphnia.*

WSP hydraulics is an area now better understood (Shilton and Harrison, 2003a,b). As well as retaining scum and other floating material, the inlet scum baffle shown in Figure 4.11, provided it extends well down into the pond (preferably to ~1.2 m), reduces the momentum of the influent and so minimizes hydraulic short-circuiting.

Full details of WSP physical design are given by Environment Protection Agency (2004) and in Mara and Pearson (1998).

4.6 SAMPLING AND PERFORMANCE EVALUATION

A low-cost protocol for sampling WSP effluents and for the minimum evaluation of pond performance is given by Pearson *et al.* (1986); this publication should be consulted for further details.

4.7 OPERATION AND MAINTENANCE

WSP O&M is very simple and comprises the following routine tasks:

(a) removal of screenings and grit from the inlet works;

(b) cutting the grass on the embankments and removing it so that it does not fall into the pond;

(c) removal of floating scum and floating macrophytes, (e.g., duckweed) from the surface of facultative and maturation ponds (this is required to maximize photosynthesis and surface re-aeration and prevent fly and mosquito breeding);

(d) removal of any accumulated solids in the inlets and outlets;

(e) repair of any damage to the embankments caused by rodents, rabbits or other animals; and

(f) repair of any damage to external fences and gates.

Routine O&M in France is done by a two-person mobile crew which visits each WSP system for half a day every fortnight (Cemagref and Agences de l'Eau, 1997). This is feasible as there are several WSP systems in any one area. In the UK routine O&M of the privately owned WSP systems is done only occasionally (perhaps once every 4-8 weeks).

[12]Small ponds are often simply excavated and, where necessary, protected against storm run-off by French drains.

Mosquito breeding in WSP is not usually a problem, provided the ponds are properly operated and maintained. Abis (2002) found mosquito breeding in primary facultative ponds loaded at 60 kg BOD/ha d, but not in ponds loaded at ≥80 kg BOD/ha d. Stringham (2002) gives advice on mosquito control in WSP, including recommendations for suitable mosquito larvicide selection and application.

Sludge accumulates in primary facultative ponds at a rate of 0.08-0.16 m^3 per person per year (Abis and Mara, 2005b). In France the average rate is 0.11 m^3 per person per year (Racault and Boutin, 2005). Sludge removal is required after ~10 years when the pond is up to one-third full of sludge. Proprietary sludge removal systems (e.g., pontoon-mounted sludge pumps) are available.[13]

4.8 WSP DESIGN EXAMPLE

A WSP system is to be designed for a village with a population of 250. Design parameter values are:

Flow = 200 litres per person per

BOD = 50 grams per person per day

Ammonia concentration = 30 mg N per litre

Design temperature (winter) = 5°C

- What is the effluent BOD from the facultative pond?
- How many maturation ponds are required to produce an effluent with 10 mg ammonia-N/l?
- How many maturation ponds would be required in summer (15°C) to reduce the *E. coli* count from 5 × 10^7 per 100 ml to ≤10^5 per 100 ml? (This would allow the effluent to be used for restricted irrigation – i.e., for the irrigation of all crops except those eaten uncooked; see WHO, 2006).

4.8.1 Solutions

4.8.1.1 Primary facultative pond

The flow is 50 m^3/day and the BOD concentration is 250 mg/l.

The design temperature is <8°C, so the design BOD loading is 80 kg/ha day. Thus, from equation 4.1:

$$A_F = \frac{10L_i Q}{\lambda_S} = \frac{10 \times 250 \times 50}{80} = 1563 \text{ m}^2$$

From equation 4.6 with e = 0 (i.e., negligible evaporation in winter) and with D_F = 1.5 m:

$$\theta_F = \frac{A_F D_F}{Q} = \frac{1563 \times 1.5}{50} = 47 \text{ days}$$

At 5°C the value of $k_{1(T)}$ is given by equation 4.8 as:

$$k_{1(T)} = 0.3(1.05)^{5-20} = 0.14 \text{ day}^{-1}$$

The unfiltered effluent BOD is given by equation 4.7 as:

$$L_e = \frac{L_i}{1 + k_{1(T)}\theta_F} = \frac{250}{1 + (0.14 \times 47)} = 33 \text{ mg/l}$$

Therefore the unfiltered effluent BOD is ~0.3 × 33 ≈ 10 mg/l.

4.8.1.2 Maturation ponds – ammonia removal

A series of maturation ponds is designed for ammonia removal. First the ammonia-N concentration in the facultative pond effluent is calculated from equation 4.15 with an assumed pH value of 7.5:

$$\begin{aligned} x &= (1.041 + 0.044T)(\text{pH} - 6.6) \\ &= (1.041 + 0.044\times5)(7.5 - 6.6) \\ &= 1.135 \end{aligned}$$

$$\begin{aligned} C_e &= \frac{C_i}{1 + (A/Q)(0.0038 + 0.000134T)e^x} \\ &= \frac{30}{1 + (1563/50)(0.0038 + 0.000134\times5)e^{1.135}} \\ &= 21 \text{ mg N/l} \end{aligned}$$

The retention time in the first maturation pond (depth = 1 m) is given by equation 4.10:

$$\theta_{M1} = \frac{10L_{e(Fac)}D_{M1}}{0.7\lambda_{S(Fac)}} = \frac{10 \times 33 \times 1}{0.7 \times 80} = 6 \text{ days}$$

[13]Brain Associates, Carmarthen SA33 6JB.

Its area is:

$$A_{M1} = \frac{Q\theta_{M1}}{D_{M1}} = \frac{50 \times 6}{1} = 300 \text{ m}^2$$

The ammonia-N concentration in the first maturation pond effluent is calculated with an assumed pH value of 7.5:

$$C_e = \frac{21}{1 + (300/50)(0.0038 + 0.000134 \times 5)e^{1.135}}$$

$$= 19 \text{ mg N/l}$$

This is a removal of ~10 percent. Thus a total of eight maturation ponds, each with a retention time of 6 days, would be required to produce an effluent with ~9 mg ammonia-N/l.

4.8.1.3 Maturation ponds – E. coli removal

The value of the first-order rate constant for *E. coli* removal at 15°C is given by equation 4.12:

Equation 4.11 is used to determine first the number of *E. coli* in the facultative pond effluent:

Try one maturation pond with the minimum retention time (calculated above) of 6 days:

$$k_{B(T)} = 2.6(1.19)^{T-20} = 2.6(1.19)^{15-0} = 1.1 \text{ day}^{-1}$$

which is not satisfactory. Increase the retention time to 10 days:

$$N_{e(Fac)} = \frac{N_i}{1 + k_{B(T)}\theta_F}$$

$$= \frac{5 \times 10^7}{1 + (1.1 \times 47)} \approx 10^6 \text{ per 100 ml}$$

$$N_{e(M1)} = \frac{10^6}{1 + (1.1 \times 6)} \approx 1.3 \times 10^5 \text{ per 100 ml}$$

which is satisfactory.

$$N_{e(M1)} = \frac{10^6}{1 + (1.1 \times 10)} \approx 8 \times 10^4 \text{ per 100 ml}$$

4.8.2 Alternative Solutions

4.8.2.1 Septic tank and secondary facultative pond

For a population of 250 equation 2.1 gives the septic tank volume as:

$$C = 200P + 2000 = (200 \times 250) + 2000$$

$$= 52\,000 \text{ litres}$$

This capacity can be provided by two septic tanks in series, the first with 36 000 litres and the second with 18 000 litres. BOD removal can be estimated as 40 percent, so the tank effluent BOD is (0.6 × 250) = 150 mg/l.

The secondary facultative pond has an area of:

$$A_F = \frac{10L_iQ}{\lambda_S} = \frac{10 \times 150 \times 50}{80} = 938 \text{ m}^2$$

This area is 40 percent less than that of the primary facultative pond calculated above (3.75 m^2 per person *vs* 6.25 m^2 per person).

From equation 4.6 with e = 0 (i.e., negligible evaporation in winter) and with D_F = 1.5 m:

$$\theta_F = \frac{A_F D_F}{Q} = \frac{938 \times 1.5}{50} = 28 \text{ days}$$

The value of $k_{1(T)}$ in secondary facultative ponds at 5°C is:

$$k_{1(T)} = 0.1(1.05)^{5-20} = 0.05 \text{ day}^{-1}$$

The unfiltered effluent BOD is:

$$L_e = \frac{L_i}{1 + k_{1(T)}\theta_F} = \frac{150}{1 + (0.05 \times 28)} = 63 \text{ mg/l}$$

The filtered effluent BOD is ~0.3 × 63 ≈ 19 mg/l.

4.8.2.2 Ammonia removal

The series of maturation ponds calculated above for ammonia removal is scarcely economical, but it simply reflects the very low rate of ammonia removal at 5°C. Alternative solutions would include (a) a primary facultative pond (or a septic tank and a secondary facultative pond) followed by a constructed wetland (Chapter 3), and (b) a primary facultative pond followed by an aerated rock filter (Chapter 5).

4.9 CASE STUDY: Combined cw-wsp system at VIDARÅSEN, NORWAY

This Case Study is taken from the paper *Exceeding tertiary standards with a pond/reed bed system in Norway* by Browne and Jenssen (2005).[14] It is included here as it is a high-performance NWT system serving a small rural community located further north than the whole of mainland UK, where the winters are very cold (-5 to -25°C) and the short summers warm (15-25°C); rainfall is 1035 mm per year. The system serves the Camphill community at Vidaråsen (59°N, 10°E), approximately 100 km south of Oslo.

The combined CW–WSP system, which was commissioned in 1998, serves 160 people and receives the effluents from a dairy, a food-processing workshop, a bakery and a laundry. The wastewater flow is ~30 m^3/day and the total p.e. is ~200. The overall area is 10 m^2 per p.e. and the retention time is ~75 days. The treatment train comprises (Figure 4.13):

(a) two primary sedimentation tanks in series (volume = 13 m^3),

(b) two vertical-flow CW in parallel (pump-fed alternately for 7 days; area = 200 m^2), in series with

(c) two gravity-fed VF-CW (area = 100 m^2),

(d) an 'enhanced' facultative pond (300 m^2) with internal circulation with a flowform cascade,

(e) two maturation ponds in series (600 m^2 and 250 m^2), the first with internal cascade circulation,

(f) a VF-CW (90 m^2),

(g) a third maturation pond (200 m^2), and

(h) two subsurface horizontal-flow CW in series (90 m^2 and 100 m^2).

The effluent is discharged to river. Effluent quality is very high, even in winter (Table 4.2): 96 percent P removal, 92 percent total N removal, 99.7 percent ammonia-N removal, 98 percent SS removal, and 94 percent removal of total organic carbon (TOC). The final effluent easily complies in both summer and winter with the discharge consent of $\leq$0.4 mg P/l. Effluent thermotolerant coliforms are <10 per 100 ml throughout the year.

Table 4.2: Influent and effluent concentrations (mg/l) for the various treatment stages in the combined CW–WSP treatment system at Vidaråsen, Norway

Parameter	Influent	VF-CW	Enhanced fac. pond	First mat. pond	Third mat. pond	SSHF-CW [a]
Total P	6.8	3.6	2.2	0.88	0.52	0.25
Total N	49	28	14	6.5	4.4	4.1
NH_4-N	46	11	3.2	0.33	0.24	0.13
TOC	85	19	8	6	5	5
SS	130	39	–	–	5	<3

[a]Final Effuent
Source: Brown and Jennssen (2005)

[14]This publication should be consulted for full details. Only an outline is given here.

5

ROCK FILTERS

5.1 TYPES OF ROCK FILTER

Rock filters (RF) are subsurface horizontal-flow units filled with a coarse granular bed medium (40-200 mm). They are thus similar to SSHF gravel-bed constructed wetlands (Chapter 3) but unplanted and with a larger-size medium. There are two types of RF: unaerated and aerated.

Unaerated RF have been used in the United States for over 30 years mainly to 'polish' maturation pond effluents (i.e., to remove algal SS and BOD) (O'Brien *et al.*, 1973; Swanson and Williamson, 1980; Middlebrooks, 1988, 1995; EPA, 2002). Aerated RF are a recent development in the UK and so far have only been evaluated at pilot scale (Johnson, 2005; Johnson and Mara, 2005; Mara and Johnson, 2006).

In the UK it has been common practice to surround the outlet of each pond in a series with rock, so creating a small in-pond rock filter (Figure 5.1). While this reduces the number of algae leaving the pond, it does little (except in the case of the last pond in the series) to improve final effluent quality as algae grow in the next pond. A more recent development by Iris Water and Design[1] is to follow a facultative pond by a long shallow, partially planted pond containing several 'cross-pond' gravel filters (Figure 5.2). This may be expected to improve effluent quality significantly, but no performance evaluation of this innovation has yet been undertaken.

5.2 UNAERATED RF FOR BOD AND SS REMOVAL

Middlebrooks (1995) compared the performance and costs of RF, SSHF-CW (Chapter 3), intermittent sand filters, macrophyte (duckweed and water hyacinth) ponds and microstrainers for upgrading WSP effluent. He found that, while further development was needed to design RF to produce effluents of a consistent quality, "the advantages of rock filters on a purely cost basis are dramatic": costs were ~50 percent lower than SSHF-CW. Swanson and Williamson (1980) investigated the relationship between RF performance and the applied hydraulic loading rate (HLR, defined as m^3 of wastewater per m^3 of gross RF volume per day; i.e., with units of day^{-1}); their data from the RF treating primary maturation pond effluent in Veneta, Oregon, yield the following equations:

(a) Percentage BOD removal (R_{BOD}):

$$R_{BOD} = 72 - 109(HLR)$$

(b) Percentage SS removal (R_{SS}):

$$R_{SS} = 97 - 137(HLR)$$

In the United States the range of HLR applied to unaerated RF is ~0.1-0.3 day^{-1}, which produces BOD and SS removals of ~40-60 percent and ~55-80 percent, respectively.

[1]Castleton, North Yorkshire.

5.3 AERATED RF FOR AMMONIA REMOVAL

Two of the disadvantages of unaerated RF, because they rapidly become anoxic, are slight odour release due to H_2S and no removal of ammonia. Aeration of the RF (using a dome aerator of the type used in diffused-aeration activated sludge tanks) eliminates H_2S generation, significantly improves BOD and SS removals (Figure 5.3), and provides the conditions for nitrification to occur (Figure 5.4) (Johnson, 2005; Johnson and Mara, 2005; Mara and Johnson, 2006).[2] This work was done on pilot-scale RF at Esholt, Bradford (Figure 5.5) which were fed with facultative pond effluent (rather than maturation pond effluent) at an HLR of 0.3 day^{-1}; the aerated RF 95-percentile effluent quality was 5 mg BOD/l, 6 mg SS/l and 4 mg ammonia-N/l throughout the year.

5.4 COMPARISON WITH CONSTRUCTED WETLANDS

As shown in Figure 3.4, SSHF-CW are not good at removing ammonia during winter. Aerated RF, on the other hand, do remove ammonia in winter. Figure 5.6 shows ammonia concentrations in the effluents of the pilot-scale aerated RF and SSHF-CW (planted with *Typha*) at Esholt, Bradford; the influent to both is effluent from a primary facultative pond loaded at 80 kg BOD/ha day at an HLR of 0.3 day^{-1}. The SSHF-CW is better than the aerated RF in summer, but worse in winter.

5.4.1 Sludge accumulation

Sludge accumulation in the first third of the lengths of the RF and SSHF-CW at Esholt over 24 months was:[3]

Aerated RF:	8 cm
Unaerated RF:	50 cm
SSHF-CW:	50 cm

In the last third of the lengths sludge accumulation was undetectable (<1 cm).

[2]Strictly speaking, aeration of the RF makes the process no longer 'natural'. Aerated RF are included here as they are a land-saving alternative to maturation ponds or constructed wetlands used to treat facultative pond effluents

[3]Unpublished observations, Michelle Johnson, School of Civil Engineering, University of Leeds.

6

NWT TECHNOLOGY SELECTION

6.1 COMPARATIVE COSTS

6.1.1 Europe

Comparative costs of constructed wetlands and waste stabilization ponds in France in 1997 are given in Table 6.1, and in Germany in 1996 in Table 6.2. These tables show that WSP are cheaper than CW (and indeed other treatment processes) in both these countries. Pond desludging costs in France amount to ~€3.20 per person per year on average (range: €0.2-12) (Racault and Boutin, 2005).

In Greece Tsagarakis *et al*. (2003) found that WSP were the least-cost treatment process up to a land price of €28 000 per ha in 1999.[1]

Of course, the fact that WSP are cheaper than CW and other treatment technologies in these countries does not mean that this is necessarily also the case in the UK. However, these European cost data are a reasonable indicator that this might well be so. It is therefore always worth at least considering NWT technologies, especially WSP, for wastewater treatment in small villages in the UK. Added to this is the use of WSP by some small private communities in the UK: why would WSP have been chosen if

Table 6.1: Capital and O&M costs of various wastewater treatment processes for a population of 1000 in France in 1997

Treatment process	Capital costs (ecu per person)[a]	O&M costs (ecu per person per year)[a]
Activated sludge	230	11.50
Trickling filter	180	7.00
RBC	220	7.00
Aerated lagoon	130	6.50
Vertical-flow CW[b]	190	5.50
WSP	120	4.50

[a] Average exchange rate in 1997: 1 ecu = £0.69 (www.oanda.com/convert/fxhistory).
[b] Two-stage VF-CW receiving raw wastewater.
Note: All processes designed to produce effluents complying with French regulations (see Alexandre *et al.*, 1997; Racault and Boutin, 2005).
Source: Alexandre *et al.* (1997) (see also Berland and Cooper, 2001).

[1] Based on a case study in Sana'a, Yemen, Arthur (1983) similarly found that WSP were cheapest up to a land price of USD 50 000-150 000 per ha, depending on the discount rate used (5–15 percent).

Table 6.2: Capital and O&M costs of various wastewater treatment processes for a population of 500 in Germany in 1996

Treatment process	Capital costs (ecu per person)[a]	O&M costs (ecu per person per year)[a]
Activated sludge	230	11.50
Trickling filter	180	7.00
RBC	220	7.00
Aerated lagoon	130	6.50
Vertical-flow CW	*190*	*5.50*
WSP	*120*	*4.50*

[a]Average exchange rate in 1996: DEM 1 = £0.43 = 0.53 ecu (www. oanda.com/convert/fxhistory).

Source: Burka (1996)

they were unable to produce a compliant effluent at lower cost than other treatment technologies?

6.1.2 United Kingdom

In the UK there are no direct comparative costs for CW and WSP, but there are individual costs for the two processes.

6.1.2.1 Constructed wetlands

Severn Trent Water has published its construction costs for subsurface horizontal-flow CW (Green and Upton, 1994, 1995): "the costs of tertiary [SSHF-CW] treatment systems [then 1 m^2 per person] have varied between about £100/head for 100 population to about £40/head for 1000 population. The secondary [SSHF-CW] treatment systems (for complete works) [i.e., primary treatment and 5 m^2 per person for the SSHF-CW] have varied from about £700 to £1600/head" (Green and Upton, 1995). These costs were confirmed by Upton *et al.* (1995), who also gave the costs of the RBCs preceding the tertiary SSHF-CW: ~£400-1000 per person for populations of 200-1000, and ~£500-2400 per person for populations <200. Thus the tertiary SSHF-CW accounted for only ~10-20 percent of the total cost. These 1994 costs can be converted to approximate first-quarter 2005 costs, and hence 2005 costs per m^2, using an index of 1.60 (Davis Langdon, 2006), as shown in Table 6.3.

6.1.2.2 Waste stabilization ponds

The construction costs (excluding land costs) of the privately owned WSP system serving Burwarton Estate and village, near Bridgnorth, Shropshire, were £50 000 in 1994 (Mara et al., 1998). The total pond volume is 5000 m^3, so the construction cost was £10 per m^3 in 1994, equivalent to an approximate first-quarter 2005 cost of £16 per m^3 (using the same cost index as above for CW).

Table 6.3: Conversion of SSHF-CW 1994 costs to 2005 costs

	1994 cost per p.e.	*2005 cost per p.e.*	*2005 cost per m^2*
Secondary SSHF-CW (5 m^2 per person)[a]	£700-1600	£1100-2600	£220-520
Tertiary SSHF-CW (1 m^2 per person)[b]	£40-100	£65-160	£65-160

[a]Cost includes primary treatment.

[b]Current sizing is 0.7 m^2 per p.e.

Table 6.4: Land and construction costs for a primary facultative pond in the UK

Area per person (m^2)	Cost of land (£ per person)[a]	Cost of construction (£ per person)[b]	Total cost (£ per person)
6.25	15	220	235

[a]Cost = (area per person, m^2) × 1.5 (to allow for embankments and access) × (£1.60 per m^2 — ie., allowing for the land purchase price to be twice its market value).

[b]Cost = [(area per person, m^2) × (depth; taken as 2 m to include freeboard) × (£16 per m^3)] + 10%.

Land costs. The price of farmland ('bareland', i.e., without any buildings) in the UK is nearly £8000 per ha (i.e., 80p per m^2) (RICS, 2005). Thus land costs are a relatively small part of total costs — for example, for a primary facultative pond in the UK, they are ~6 percent (Table 6.4).

The land area requirement for a rock filter (A_{rf}) receiving a hydraulic loading rate (HLR) of 0.3 day^{-1} is given by:

$$A_{rf} = \frac{q}{(\text{HLR}) \times D_{rf}} = \frac{0.2}{0.3 \times 0.6}$$

$$= 1.1 \text{ m}^2 \text{ per person}$$

where D_{rf} is the wastewater depth in the RF (taken as 0.6 m).

The area of the RF is thus ~1.5 m^2 per person overall. Taking the 2005 RF cost as ~£100 per m^2 (i.e., the same as that for a tertiary SSHF-CW), the RF cost is ~£150 per person, so the overall cost of a primary facultative pond and a rock filter is of the order of £400 per person, which is very much less than the range given above for a secondary SSHF-CW system (including primary treatment).

6.2 TECHNOLOGY SELECTION

If the selection of an NWT treatment train is to be based as far as possible on rational grounds, then the selection criteria are land area, performance and cost.

6.2.1 Land area and performance

The land area requirements for CW and WSP systems are determined below for two levels of required effluent quality:

(a) ≤40 mg unfiltered BOD and ≤60 mg SS per litre (95-percentile values) (this is commonly required by the Environment Agency at small works in, for example, the Yorkshire Water area); and

(b) ≤15 mg unfiltered BOD, ≤25 mg SS per litre and ≤5 mg ammonia-N per litre (95-percentile values) (this is the strictest effluent quality in Table 3.1 set in the Severn Trent area).

The design parameters are taken as:

Wastewater flow: 200 litres per p.e. per day,

BOD: 50 g per p.e. per day,

Ammonia: 8 g N per p.e. per day,

Winter temperature: <8°C

Thus the BOD is 250 mg/l and the ammonia concentration 40 mg N/l.

6.2.1.1 Constructed wetlands

The area (A_{CW}) of a secondary SSHF-CW is given by equation 3.6 as:

$$A_{cw} = \frac{Q_i (\ln L_i - \ln L_e)}{k_A}$$

where the design value of k_A is 0.06 m/d.

(a) <40 mg unfiltered BOD and ≤60 mg SS per litre (95-percentile values) ("40/60"):

L_i is taken as 150 mg/l (i.e., 250 mg/l less 40 percent removed in, for example, a septic tank), and L_e as 20 mg/l as this is approximately equal to a 95-percentile value of 40 mg/l. Thus:

$$A_{cw} = \frac{0.2 (\ln 150 - \ln 20)}{0.06}$$

$$= 6.7 \text{ m}^2 \text{ per p.e.}$$

(b) ≤15 mg unfiltered BOD, ≤25 mg SS per litre and ≤5 mg ammonia-N per litre (95-percentile values) ("15/25/5"):

The critical part of this effluent quality requirement is the 95-percentile ammonia concentration of ≤5 mg N/l. For a winter temperature of 7°C and assuming that partial ammonification of organic N in the septic tank increases the mean influent ammonia concentration (C_i) to 50 mg N/l, and that a 95-percentile ammonia concentration of 5 mg N/l is equivalent to a mean ammonia concentration of 1 mg N/l (Cooper, 2005b), A_{cw} is given by equations 3.5, 3.7 and 3.8 rewritten as follows:

$$\theta_{cw} = \frac{(\ln C_i - \ln C_e)}{0.126(1.008)^{T-20}}$$

$$= \frac{(\ln 50 - \ln 1)}{0.126(1.008)^{7-20}} = 34 \text{ days}$$

$$A_{cw} = \frac{Q_i\,\theta_{cw}}{\varepsilon\, D_{cw}} = \frac{0.2 \times 34}{0.4 \times 0.6}$$

$$= 28 \text{ m}^2 \text{ per person}$$

This area is extremely large and in practice secondary SSHF-CW would not be used to achieve this degree of ammonia removal. (This also explains, at least in part, why Severn Trent Water's preferred strategy is to use a tertiary SSHF-CW to polish the effluent from a nitrifying RBC.)

6.2.1.2 Waste stabilization ponds

The design loading for facultative ponds in winter in the UK is 80 kg/ha day (= 8 g/m^2 day), so the area of a primary facultative pond is:

$$\frac{50 \text{ g per p.e. per day}}{8 \text{ g per m}^2 \text{ per day}} = 6.25 \text{ m}^2 \text{ per p.e}$$

Assuming the BOD is reduced by 40 percent in, for example, a septic tank to 30 g per p.e. per day, the area of a secondary facultative pond is:

(a) 40/60 effluent quality:

$$\frac{30 \text{ g per p.e. per day}}{8 \text{ g per m}^2 \text{ per day}} = 3.75 \text{ m}^2 \text{ per p.e.}$$

The facultative pond effluent has to be treated in a rock filter. As shown in Chapter 5, an unaerated rock filter receiving facultative pond effluent at an HLR of 0.3 day^{-1} produces a 95-percentile effluent BOD/SS of <40/60. As shown above, its area is 1.1 m^2 per p.e.

(b) 15/25/5 effluent quality:

As shown in Chapter 5, an aerated rock filter receiving facultative pond effluent at an HLR of 0.3 day^{-1} produces a 95-percentile effluent BOD/SS/Amm.N of <10/15/5 mg/l. Its area is thus also 1.1 m^2 per p.e.

6.2.1.3 Area comparison

These land area requirements for CW and facultative ponds and rock filters are summarized in Table 6.5. It is apparent that, to achieve a 40/60 effluent quality, the secondary SSHF-CW requires 38 percent more land than the secondary facultative pond and unaerated rock filter. The CW is unable to achieve a 15/25/5 effluent quality as this quality has to be achieved in both summer and winter and it is unable to produce an effluent with a 95-percentile ammonia concentration ≤5 mg N/l in winter (unless it were excessively large), whereas the secondary facultative pond followed by an aerated rock filter can.

6.2.2 Cost

Cost should be the lowest cost, although a treatment train with the lowest CAPEX may not necessarily have the lowest OPEX and it could be more expensive in net present value terms than one with a higher CAPEX but lower OPEX. However, this latter alternative may be financially more attractive as its higher OPEX is funded from revenue.

As shown above, the CAPEX of a secondary SSHF-CW (including the cost of the associated primary treatment) is at least 175 percent more than that of a primary facultative pond and a rock filter.

6.2.3 Concluding remarks

Strict application of these land area, performance and cost criteria should therefore lead to the selection of either a primary facultative pond and a rock filter (aerated if required to remove ammonia), or a septic tank, a secondary facultative pond and a rock filter (aerated as necessary). Preference may be

Table 6.5: Land area requirements for constructed wetland and waste stabilization pond systems designed to achieve two different effluent qualities

Wastewater treatment system	Land area requirements (m^2 per p.e) for:	
	40/60 effluent quality	15/25/5 effluent quality
Primary facultative pond and unaerated rock filter	7.35	n.a.[a]
Primary facultative pond and aerated rock filter	–[b]	7.35
Secondary facultative pond and unaerated rock filter	4.85	n.a.
Secondary facultative pond and aerated rock filter	–	4.85
Secondary subsurface horizontal-flow CW	6.7	28[c]

[a] Treatment system not able to produce this quality effluent.
[b] Treatment system would not be used to produce this quality effluent.
[c] In practice this treatment system would not be used to produce this quality effluent.

given to constructed wetlands for reasons of familiarity, apparent aesthetics or "politics",[2] but it should be at least recognised that this choice may not be always optimal.

[2] Water companies using CW are often able to deflect criticism from 'green activists' simply by saying that they are using 'green technologies'. The same argument applies, of course, to WSP.

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