Chapter 5

Practical information on design of specific wetland types and typical pitfalls

5.1 INTRODUCTION

Design manuals and guidelines are available from a number of sources worldwide, providing recommendations on all aspects of wetland design, operation and maintenance. The purpose of this chapter is to move away from specific guidelines and provide a summary of collective practical experience with different TW types by practitioners and researchers from around the world. The information is organised based on TW type rather than treatment application, to highlight key elements relevant to each configuration. The TW types covered are:

- VF wetlands
- French VF wetlands
- HF wetlands
- FWS wetlands
- Sludge treatment wetlands
- Aerated wetlands
- Fill-and-drain wetlands
- Floating treatment wetlands
- Willow systems
- Use of reactive media for enhanced P removal
- Multi-stage wetlands.

5.2 VF WETLAND

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5.2.1 Overview of existing design guidelines

The main design parameters of VF wetlands according to the design guides in Denmark, Germany and Austria are summarized in Dotro *et al.* (2017). When VF wetlands are designed according to the guidelines, legal requirements regarding organic matter and ammonia nitrogen removal in these countries can be achieved (Table 5.1). To achieve almost complete nitrification, the Austrian, Danish and German guidelines (Brix & Johansen, 2004; ÖNORM B 2505, 2009; and DWA-A 262E, 2017; Nivala *et al.*, 2018, respectively) require that sand is used for the main layer of the VF filter with a minimum depth of 50 cm. Since 2017, also the Czech wetland design guidelines include VF wetlands requiring a specific surface of 4 m^2 per person and 50 cm main layer of washed sand (0.06–4 mm).

5.2.2 Main factors affecting treatment performance

The main factors affecting the treatment performance of VF wetlands are (e.g. Stefanakis & Tsihrintzis, 2012a):

- Filter material of main layer (grain size of material, filter depth)
- Loading: loading interval, volume of single doses, resting periods
- Loading rate: hydraulic and organic loading rates
- Distribution pipes: number of holes in distribution pipes.

Table 5.1 Comparison of legal requirements for organic matter and ammonia in Austria, Denmark, Germany and Czech Republic.

Parameter	Requirement	Austria[1]	Denmark[2]	Germany[3]	Czech Republic[4]
BOD_5	Max. effluent concentration	25 mg/L	–	40 mg/L	40 mg/L
	Removal efficiency	–	95%	–	–
COD	Max. effluent concentration	90 mg/L	–	150 mg/L	150 mg/L
NH_4-N	Max. effluent concentration	10 mg/L*	5 mg/L	10 mg/L*	20 mg/L
	Removal efficiency	–	90%	–	–

[1]For wastewater treatment plants, i.e. $\leq$50 PE.
[2]For wastewater treatment plants, i.e. $\leq$30 PE.
[3]For wastewater treatment plants $\leq$1,000 PE (organic matter) and $\leq$10,000 PE (NH_4-N).
[4]For wastewater treatment plants $\leq$10 PE and infiltration into groundwater.
*For effluent water temperatures $>$12°C.

The effect of selected parameters on the treatment performance of a VF wetland treating domestic wastewater is shown in the following for identical systems with 50 cm main layer comprising of three different filter materials (based on Pucher & Langergraber, 2019):

(1) Sand, 0.06–4 mm
(2) Coarse sand, 1–4 mm
(3) Gravel, 4–8 mm.

Measured volumetric effluent flow rates for calibration of the water flow model as well as measured influent and effluent concentrations of COD and NH_4-N for calibration of the pollutant transport and degradation model were available. For the VF wetlands with filter materials 0.06–4 mm and 1–4 mm, the wetland models have been calibrated on data described by Canet Martí *et al.* (2018) and Pucher and Langergraber (2018), respectively. For the 4–8 mm gravel system, volumetric effluent flow data were measured at BOKU University Vienna whereas concentration data came from the system as described by Nivala *et al.* (2019a). Table 5.2 summarises the main operational parameters of the VF wetlands for which the wetland models were calibrated.

For each of the calibrated wetland models (i.e., filter materials) simulations for the following operational settings were run for:

- Organic loading rates of 20, 40 and 80 g COD/m^2/d;
- Loading intervals of 1, 3, 6 and 12 hours;
- Number of holes per m^2 in distribution pipes: 0.5, 1, 2, 4; and
- Water temperature: 5, 10, 15 and 20°C.

Thus for each filter material 192 simulations were run (in total 576 simulations for all three filter materials). It has to be noted that not all combinations of operational setting are applicable, i.e., for VF wetlands using sand as filter material in temperate climates an OLR of 40 g COD/m^2/d can only be applied when the systems are operated during the summer months, whereas an OLR of 80 g COD/m^2/d leads to clogging of the system.

The same influent concentrations have been used for all simulations and all VF wetlands (Table 5.3). Thus only the effect on effluent concentration is reported. In the case of changing influent concentrations the design parameters – of course – also influence removal efficiencies.

Table 5.4 show the simulated COD effluent concentrations for the different filter materials and different OLRs. VF wetlands in Table 5.4 were loaded every 6 hours with distribution pipes having 0.5 holes in distribution pipes per m^2 (these are standard design values for VF wetlands using sand with grain size 0.06–4 mm as filter material). Table 5.5 shows simulated NH_4-N effluent concentrations for the same settings.

Table 5.2 Main VF wetland operational parameters of the data sets used for calibration.

Filter Material (mm)	Loading Interval (h)	Organic Loading Rate (g COD/m^2/d)	Number of Openings per m^2	Data for Calibration
0.06–4	6	20	0.5	see Canet Martí *et al.* (2018)
1–4	3	80	1	see Pucher and Langergraber (2018)
4–8	1	80	1	Water flow: measurements at BOKU University Vienna; Concentrations: Nivala *et al.* (2019a)

Table 5.3 Influent concentrations (in mg/L) used for the simulation study (from Pucher & Langergraber, 2019).

Parameter	COD	CR	CS	CI	NH_4-N	NO_2-N	NO_3-N	PO_4-P
Concentration	495	325	163	7	65	0.015	0.4	11.9

CR = readily and slowly biodegradable COD; CS = slowly biodegradable COD; CI = inert COD.

Table 5.4 Median and maximum COD effluent concentrations in mg/L of VF wetlands loaded every 6 hours with 0.5 holes in distribution pipes per m^2 (maximum concentrations in brackets).

Filter Material	OLR	5°C		10°C		15°C		20°C	
(mm)	g COD/m^2/d	Median	(Max)	Median	(Max)	Median	(Max)	Median	(Max)
0.06–4	20	42	(45)	24	(25)	18	(18)	17	(17)
	40*	79	(86)	48	(55)	28	(33)	21	(23)
	80*	115	(136)	82	(108)	55	(81)	39	(58)
1–4	20	56	(65)	32	(38)	21	(24)	19	(20)
	40	97	(117)	63	(85)	40	(57)	27	(39)
	80	131	(149)	100	(130)	67	(107)	43	(86)
4–8	20	58	(67)	33	(40)	23	(26)	20	(22)
	40	99	(119)	65	(87)	43	(58)	27	(43)
	80	139	(148)	104	(127)	73	(106)	47	(86)

*OLR $>$20 g COD/m^2/d is not experimentally verified in temperate climates.

Table 5.5 Median and maximum NH_4-N effluent concentrations in mg/L of VF wetlands loaded every 6 hours with 0.5 holes in distribution pipes per m^2 (maximum concentrations in brackets).

Filter Material	OLR	5°C		10°C		15°C		20°C	
(mm)	g COD/m^2/d	Median	(Max)	Median	(Max)	Median	(Max)	Median	(Max)
0.06–4	20	4.0	(4.0)	1.1	(1.1)	0.4	(0.4)	0.1	(0.1)
	40*	3.1	(3.6)	1.3	(1.4)	0.4	(0.4)	0.1	(0.1)
	80*	1.4	(9.3)	0.2	(5.6)	0.1	(4.5)	0.1	(4.7)
1–4	20	29.8	(29.9)	8.0	(8.1)	2.9	(3.0)	0.9	(1.0)
	40	28.1	(30.3)	7.1	(8.9)	2.9	(3.2)	1.1	(2.1)
	80	34.9	(43.3)	15.6	(30.3)	14.1	(28.7)	16.7	(28.8)
4–8**	20	30.5	(30.6)	7.5	(7.5)	2.8	(2.9)	1.0	(1.0)
	40	27.0	(29.0)	6.1	(8.3)	2.7	(3.0)	1.0	(1.6)
	80	34.8	(41.7)	13.9	(27.8)	13.9	(26.3)	14.8	(26.2)

*OLR $>$ 20 g COD/m^2/d is not experimentally verified in temperate climates.
**Results for NH_4-N: the 4–8 mm main layer could not be fitted well (see Pucher & Langergraber, 2018a).

Table 5.4 and Table 5.5 clearly show the importance of the filter material used for the main layer on the achievable COD and NH_4-N effluent concentrations. The coarser the filter material of the main layer, the higher the effluent concentrations. If operated with the same loading interval, at higher OLR higher single doses are applied. When coarser filter material is used for the main layer this results in higher flow velocities in the filter and thus reduced removal efficiencies. The increase is even more significant for maximum COD and NH_4-N effluent concentrations. A higher increase of the maximum compared to the median COD and NH_4-N effluent concentrations can also be observed at lower temperatures.

Figure 5.1 and Figure 5.2 show the effect of different loading intervals and different numbers of holes in the distribution pipes on simulated COD and NH_4-N effluent concentrations, respectively. The example shows a VF filter with 50 cm main layer of coarse sand (1–4 mm) operated with an OLR of 80 g COD/m^2/d. The removal efficiencies can be increased if (a) the loading interval is decreased (i.e. more doses with less volume per single dose) or (b) the distribution network gets denser (i.e. more openings per m^2). Both measures lead to lower water flow velocities in the filter and thus to higher performance and lower effluent concentrations. The reduction is even more significant for maximum effluent concentrations (dashed lines in Figure 5.1 and Figure 5.2, respectively). If the same loading interval is applied, the difference between maximum and median effluent concentrations gets less by increasing the density of holes in the distribution pipes (i.e. less volume of water per opening and thus lower flow velocities).

5.2.3 Field tests for filter material

The previous section showed the importance of the filter material for the performance of the VF wetland. All design standards for VF wetlands include specifications for the filter material. Most of the time, these include the grain size distribution (e.g., sand, 0.06–4 mm), d_{10} and/or d_{60} (grain size under which 10% and 60%, respectively, of the grains pass [by weight]) and U (the uniformity coefficient). Filter material is usually purchased from gravel pits according to these requirements.

However, it is advisable to test the sand delivered for the main layer before filling the bed. The following field test according to EN 12566-2 has been proven to be adequate if sand will be used as filter material and full nitrification is the treatment target. To carry out the test only a few items are required, i.e., a measuring

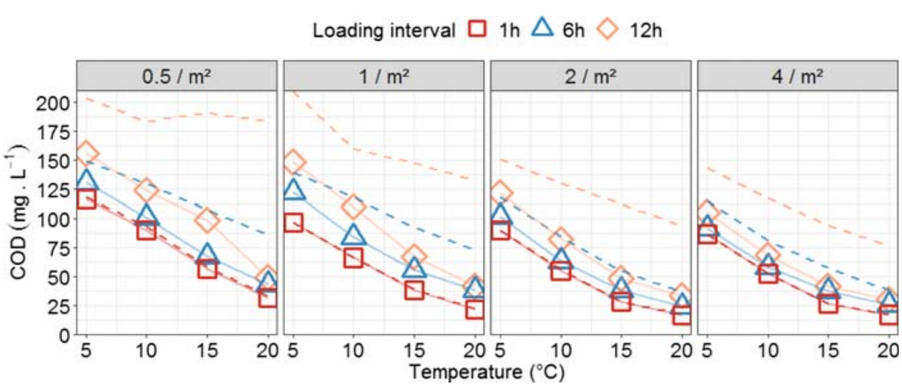

Figure 5.1 Simulated COD effluent concentrations of a VF filter with main layer of coarse sand (1–4 mm) for at an OLR of 80 g COD/m^2/d for different loading intervals and different number of holes in the distribution pipes (median concentrations: symbols; maximum concentrations: dashed lines).

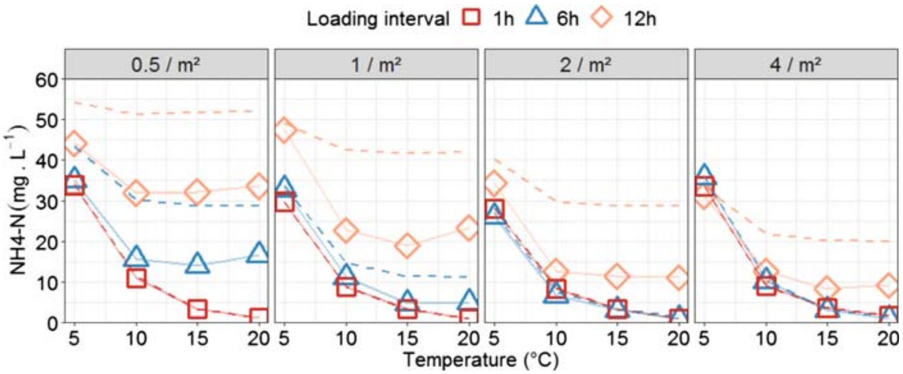

Figure 5.2 Simulated NH_4-N effluent concentrations of a VF filter with main layer of coarse sand (1–4 mm) for at an OLR of 80 g COD/m^2/d for different loading intervals and different number of holes in the distribution pipes (median concentrations: symbols; maximum concentrations: dashed lines).

cup, a stop watch, a metre stick and a thermometer (Figure 5.3). The instructions for the field test are as follows:

(1) Construct a test column of DN 100 mm, length 300 mm
(2) Put gravel (with grain size of 4–8 mm) in a bucket with draining holes on the bottom
(3) Put test column on top of the gravel
(4) Fill in 200 mm of filter sand to be tested, shake or knock on the pipe until filter column is medium densely packed
(5) Add 500 ml clean water without disturbing surface of the sand
(6) At the moment, when water has completely infiltrated start first test with stop watch
(7) At least 5 times in a row fill in 500 ml within 5 sec., stop time for infiltration

Figure 5.3 Items required for the field test of the infiltration capacity of filter material for VF wetlands according to EN 12566-2 (2005).

(8) When infiltration time stays nearly constant use average value out of 5 measurements.
(9) Calculate saturated hydraulic conductivity from:

$$k_s = \frac{l}{t} \ln \frac{h_1}{h_2}$$

where

k_s = saturated hydraulic conductivity (m/s);
l = length of the sand filter column = 0.2 m;
t = average value of 5 measurements of the infiltration time (s);
h_1 = head at the beginning of the infiltration test = 0.263 m (i.e. 0.2 m sand filter column + 0.063 m free water [500 ml] on top); and
h_2 = head at the end of the infiltration test = 0.2 m.

For sand 0.06–4 mm the target value for the infiltration time is I–30 s resulting in a targeted saturated hydraulic conductivity of $k_s = 10^{-3}$ m/s.

The obtained k_s at the air temperature during the test should be adapted to the climatic condition of the site. The final k_s value is acceptable if in the range 10^{-3}–10^{-4} m/s at $T = 10$°C. Values of k_s higher than 10^{-3} m/s limit the proper development of the biofilm and the nitrification processes, values lower than 10^{-4} m/s make the system very prone to clogging and favour too long saturated conditions in the sand layer.

5.2.4 Specific design considerations

Besides basic design recommendations presented by Dotro *et al.* (2017), the specific design considerations for VF wetlands are as follow:

- *Filter material*. The importance of the granularity of the filter material for the main layer has been shown in the previous sections. For sand-based VF wetlands, measuring the hydraulic conductivity onsite has been proven a valuable measure to ensure the hydraulic functioning of the system. Filter material should be mainly siliceous, with a low carbonate content. Besides using sand and gravel, also reactive media such as zeolite has been used to enhance the nitrification capacity of VF wetlands (e.g. Pucher *et al.*, 2017; Stefanakis & Tsihrintzis, 2012b).
- *Layer composition*. In the case 0.06–4 mm sand is used for the main layer, a transition layer of 4–8 mm gravel is required between the main and drainage layers to prevent the washout of the sand. To use different layers of filter materials with different grain size distributions does not have any advantage compared with using a main layer comprised of only one material.
- *Loading interval*. To achieve the maximum treatment efficiency, the time between two loadings must guarantee a complete percolation of the wastewater and complete aeration of the main layer of the VF wetland. This requires that for finer filter materials longer loading intervals are foreseen. For 0.06–4 mm sand the common design guidelines recommend less or equal than 6 loadings per day, i.e., a minimum loading interval of 4 hours.
- *Distribution system*:
  - *Loading with pumps*. In order to guarantee a complete distribution of the water on the sand layer and to ensure the cleaning of the distribution pipes, it is important to ensure brief and consistent loading periods. The single dose should be not less than 2 cm, whereas the pump flow should be decided on the basis of the diameter of the opening holes and in any case not less than 0.2 m^3/h per

m^2 of loaded sector. Minimal velocity in the pipes should be not less than 0.7 to 1 m/s to ensure their self-cleaning.

  - *Loading with siphons*. Similar considerations on flow and velocity are required when siphons are utilized to load the beds. Moreover, depending by the model of siphon, it is important to consider a minimum difference of level between the maximum water level in the siphon tank and the level of the surface of the VF bed. Siphon design has to ensure the maximum durability of the device, considering that this element is critical to allow adequate distribution of the water on the whole surface for years without requiring frequent maintenance interventions. Most of the siphons on the market are instead developed for self-cleaning operation of sewer with clean water and are not suitable for wastewater or frequent activations, resulting in improper functioning of loading operations. Therefore siphons should be tested for this specific use, ensuring a constant flow according with the design requirements and a fast emptying of the siphon tank
  - For *maintenance and operation* of the distribution system, it is advisable to allow their periodic cleaning every 1–2 years of operation, i.e., providing a removable plug at the end of each line.

- *Water-saturated zone at the bottom*. For VF wetlands with a main layer of coarse sand of 1–4 mm or gravel of 4–8 mm, a saturated zone on the bottom of the VF bed below the sand layer can be maintained to improve denitrification. It has been shown that denitrification cannot be enhanced with this measure for VF wetlands with a main layer of 0.06–4 mm sand (due to lack of organic matter for denitrification in the saturated water layer).
- *Shape of the VF beds*. VF beds are not subject to geometry constrains, therefore their shape can be chosen by interacting with landscaping architectural approaches and generate side benefits in terms of aesthetics, leisure and increased chances in finding available space for the realisation of extensive treatment solutions, making them a relevant component of the architectural design itself. When choosing the bed shape the only relevant factor which has to be kept in mind is to be able to make use of every m^2 of surface by an appropriate dosing of the influent over the whole surface, avoiding dead zones where the water will not flow properly. If the terrain where the beds are going to be realised is not too loamy, the side walls should be preferably designed with a 90° shape, in order to minimize the footprint of the system; otherwise a classic 1:1 ratio (45°) for the banks can be advised.

5.2.5 Considerations for the start-up phase

The following points shall be considered during the start-up phase of VF wetlands:

- Low initial applied loading rate, with gradual increase of the applied load in order to reach the design load by the end of the start-up phase.
- Ensure wastewater is distributed uniformly and reaches the most distant holes of the distribution pipes network.
- Secure plant establishment and growth and avoid open areas without vegetation: for this, the water level inside the VF bed can be set at a higher point to allow better growth of plants and gradually lower water level.
- Before start-up, to aid the establishment of the plants, the bed can be flooded with 5–10 cm of water above the surface, except when climate conditions favour algae formation that could partially clog the system at the beginning.
- During the first vegetative season a regular control and removal of weeds is very important for the growing of wetland plants; VFs are more prone than HFs to weeds intrusion.

5.3 FRENCH VF WETLANDS

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5.3.1 Overview of existing design guidelines

French VF wetlands are two-stage and contain alternately operated cells for each stage. Sludge treatment and partial removal of organic matter takes place in the first stage, and nitrification and further removal of organic matter occurs in the second stage. The first stage, which is divided into three parallel filters, is fed with screened wastewater (Figure 5.4). The second stage is divided into two filters. The sludge from the first stage collects at a rate of approximately 2–3 cm per year and needs to be removed every 10–15 years. Sludge accumulation rates may be lower in systems that do not continuously receive the full design load. French VF wetlands are planted with *Phragmites australis* to ensure proper water

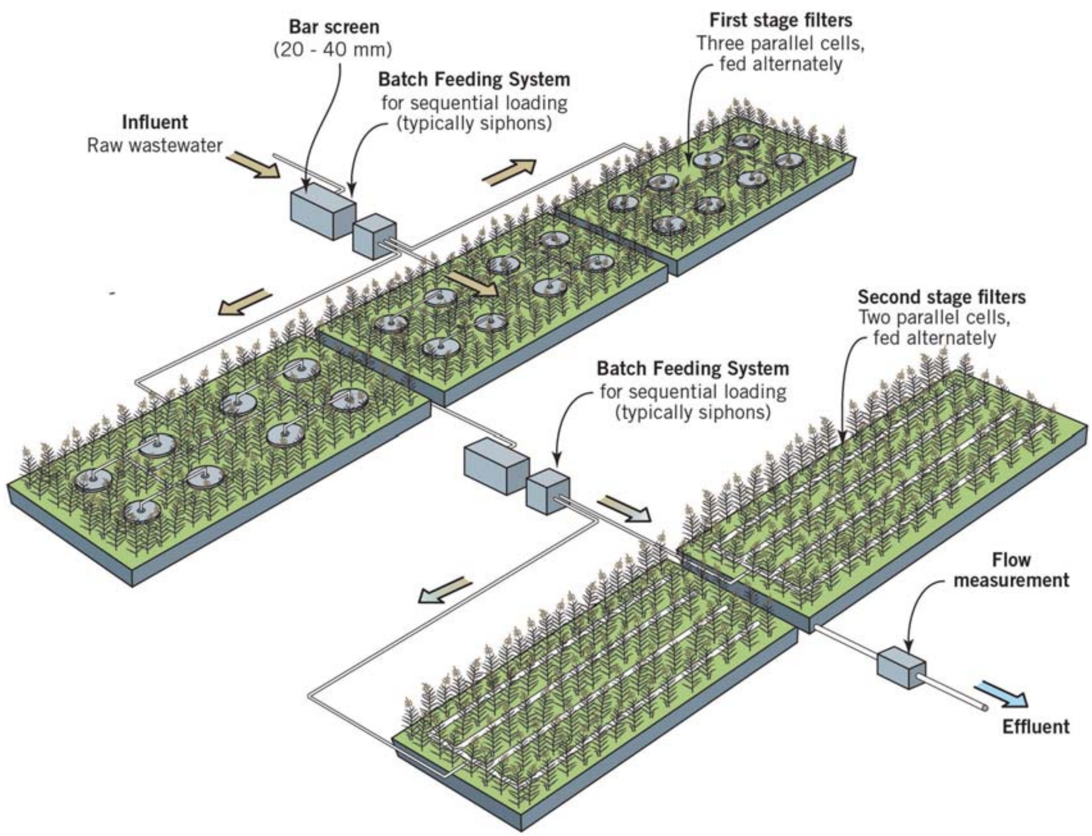

Figure 5.4 Schematic of the classical French VF design (Dotro *et al.,* 2017).

infiltration capacity and passive aeration of the filter. The *Phragmites* stems create small openings in the sludge layer that maintains the infiltration capacity of the filter. This is a critical component in proper functioning of the system. In other countries, other plants have successfully been used (Molle *et al.*, 2015) but it important to test whether other plant species can provide this function and also survive the resting periods without wastewater flow.

Over 4,000 classical two-stage French VF wetland systems have been built in France, with most systems serving populations less than 1,000 PE (Dotro *et al.,* 2017). The design has been adapted and implemented outside of France, specifically in tropical overseas French territories, South America, and other countries in the EU. The information in this technical report is restricted to the application of the classical two-stage French design in temperate European climates.

The maximum design loads for a classical two-stage French VF wetland system are given in Molle *et al.* (2005) and summarized in Table 5.6. For typical situations in France, this leads to a surface area of 1.2 m^2/PE for the first-stage cells and 0.8 m^2/PE for the second-stage cells. The anticipated effluent concentrations for French VF systems treating domestic wastewater are also provided in Table 5.6.

5.3.2 Hydraulic considerations

The cells of a VF French wetland are dosed on an alternating basis (e.g., one filter is dosed while the others are rested). The alternating dosing is a fundamental aspect of proper operation of the French VF system, because it (a) promotes mineralization and stabilization of the accumulated sludge (on the first-stage cells), (b) maintains aerobic conditions in the filter bed itself (both first- and section-stage cells), and (c) ensures that the plants in each cell receive water on a frequent basis (to avoid water stress). First-stage cells are typically loaded for three to four days and rested for seven days; second-stage cells are generally loaded for three to four days and rested for three to four days. This feeding schedule requires that the system operator visits the site twice a week to make these changes manually, unless dosing is performed by a programmable logic controller (PLC) system.

5.3.3 Specific design considerations

Influent distribution is different for first- and second-stage cells. First-stage cells generally use large distribution pipes to distribute the wastewater, with at least one feeding point per 50 m^2, and pipe diameters should be chosen for a flow velocity >0.7 m/s, in order to ensure self-curing. However, in order to avoid blockage, they should not be less than 90 mm in diameter. First-stage distribution pipes are suspended above the surface of the filter in order to allow for sludge accumulation, and a minimum flow of 0.5 $m^3/h \cdot m^2$ per batch is necessary to correctly distribute the water. Second-stage cells are fed

Table 5.6 Maximum design loads and expected effluent concentrations for classical French VF wetland design under dry weather conditions. Values are given per square metre of bed in operation (Dotro *et al.,* 2017).

	Hydraulic Load (m/d)	COD ($g/m^2 \cdot d$)	BOD_5 ($g/m^2 \cdot d$)	TSS ($g/m^2 \cdot d$)	TKN ($g/m^2 \cdot d$)
First stage	0.7	350	150	150	30
Second stage	0.7	70	20	30	15
Final effluent concentration	–	75 mg/L	15 mg/L	15 mg/L	15 mg/L

with pipes that are installed directly on the filter surface. Feeding points are drilled holes and there should at least be one hole for every 2 m^2 of filter surface. The diameters of the pipes and of the holes should be chosen in order to limit differences in flow between any two feeding points to less than 10%, which means minimizing the headloss in the pipes. The diameter of the holes should assure a squirt height of at least 25 cm at the outflow of each hole, but should be at least 8 mm, in order to avoid blockage. The squirt height at the outermost orifices in the distribution pipes on the second-stage cells must be >30 cm. In order to maintain aerobic conditions in the filter, passive oxygenation by the bottom of the filter is necessary. Drainage pipes (minimum diameter 125 mm) contain slots (with a length of one-third of the pipe circumference, and width greater than 8 mm) for at least every 25 cm of pipe length.

Different filter media are used in the first- and second-stage cells. The first-stage cells have a main layer of 2–6 mm gravel, which is coarse enough to avoid problematic clogging but fine enough to support the formation of a sludge layer on the surface of the filter. Below the main layer is a transition layer of larger gravel (5–15 mm) which prevents finer particles from being washed into the drainage layer. The drainage layer consists of a coarse gravel (20–60 mm) which is installed along with drainage pipes on the bottom of each cell.

Second-stage cells use sand for the main layer ($0.25 < d_{10} < 0.4$; uniformity coefficient <5; less than 3% fines). A deeper layer of sand must be used if the sand specifications in Table 5.7 cannot be met. The transition layer (3–12 mm gravel) and drainage layer (20–60 mm gravel) must adhere to the Terzaghi rule ($D_{15}/d_{85} \leq 4$) and permeability criterion ($D_{15}/d_{15} \geq 4$) to ensure that the interface between the filter layers does not produce a decrease in permeability by reducing the local porosity.

Construction of the cells is generally with a side slope of 1:1. The cells are lined with a combination of a plastic liner and geotextile membrane.

5.3.4 Considerations for the start-up phase

During the first year of start-up, excessive growth of weeds in the filter must be avoided. The only way to do this is to manually remove the weeds. It is possible to saturate an individual cell for one or two weeks during the first growing season to kill the weeds and favour establishment and growth of the *Phragmites*. However, do not saturate both first- and second-stage cells simultaneously because this will hinder the nitrification process.

If the system starts operation with a very low hydraulic load, the *Phragmites* that are not located near a feeding pipe can undergo water stress. This does not impact on the performance of the filter but can favour the growth of weeds. Weed removal is a tedious and time-consuming task.

Systems started at the nominal design load will form a sludge layer relatively quickly. If the *Phragmites* are still small, they cannot aid in water infiltration and mineralization of the sludge layer. The sludge

Table 5.7 Filter media specifications for French VF wetlands (Dotro *et al.,* 2017).

	First Stage		**Second Sctage**	
	Depth	**Material**	**Depth**	**Material**
Freeboard	>30 cm		>20 cm	
Main layer	30–80 cm	2–6 mm gravel	30–80 cm	sand $0.25 < d_{10} < 0.4$ and $d_{60}/d_{10} < 5$ and less than 3% fine particles
Transition layer	10–20 cm	5–15 mm gravel	10–20 cm	3–12 mm gravel
Drainage layer	20–30 cm	20–60 mm gravel	20–30 cm	20–60 mm gravel

deposits dry quickly, without mineralization, which can result in unwanted ponding on the surface of the bed. This problem ends when the *Phragmites* stand becomes established.

Storm events during the first year can result in ponding and/or surface clogging on the second-stage cells. This will end once the sludge layer is established on the first-stage cells. To accelerate the establishment of a sludge layer on the first-stage cells, a sludge or compost layer can be applied during start-up of the system.

5.3.5 Routine maintenance

Unless the filters are automatically dosed with a PLC system, the operator must visit the treatment system twice a week to alternate the feeding on the first- and second-stage cells. Feeding should be alternated every three to four days to maintain sufficient oxygen transfer into the bed. Weeds should be removed on a monthly basis and harvested once per year (if necessary). The height of the sludge layer should be checked once per year. The sludge deposit layer must be removed once it reaches a depth of 20–25 cm (generally every 8–15 years), otherwise problematic ponding will occur. Removal is conducted with mechanical machinery, and the sludge can be spread on fields (depending on local regulations). There is no need for a resting period before sludge removal (as opposed to sludge treatment wetlands), and the French VF wetlands can be put back into operation immediately after removal of the sludge layer. The sludge layer generally has a dry matter content of $>25\%$ and an organic matter content of less than 40%.

5.4 HF WETLANDS

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5.4.1 Introduction

Horizontal-flow wetlands have been used for a number of decades around the world. Current design, operation and maintenance guidelines have been summarised by Dotro *et al.* (2017). Many reviews exist in the literature on performance of these systems for treating municipal sewage, agricultural wastewater, industrial effluent, mine drainage, landfill leachate, polluted river and lake water, urban and highway runoff (Vymazal & Kröpfelová, 2008) The use of horizontal-flow wetlands has also been developed in various climate conditions such as cold climate (Wang *et al.* 2006) and tropical climate (Zhang *et al.* 2014). HF wetlands are most effective for removal of organic matter (measured as BOD and COD) and total suspended solids (TSS).

5.4.2 Design considerations

Most guidelines include recommendations on the main factors affecting the treatment performance of HF wetlands. This section summarises the rationale behind the recommendations made.

- *Filter material*. The selection of substrate is a key design parameter because it provides the area for biofilm attachment, rooting medium for the emergent plants, adequate hydraulic retention time and, if required, can react with specific pollutants such as phosphorus or metals. The hydraulic conductivity of the media is considered in current sizing criteria to balance the risk of clogging and contact time between the wastewater and the media (biofilm). Whilst media can be natural, industrial by-products or engineered products, typical material is gravel with sizes of 8–16 mm for the main layer and 50–200 mm for the inlet and outlet zones. Soil has proven to have too low a hydraulic conductivity for the loading rates typically applied and as such is no longer recommended. Whilst checking grading on delivery of media to site can be done with sieves, in reality this is not performed as the range of gravel sizing recommended is broad enough to be less critical if deviations occur, and easy to visually detect.
- *Distribution of wastewater*. Systems are typically loaded along the width of the bed, either with subsurface pipes (secondary treatment) or surface troughs (tertiary treatment). Cleaning access needs to be provided to either type of flow distribution structure as flow velocities from the upstream processes can vary daily and settling can occur within the pipes or troughs. Coarse stones are used to help flow distribution in depth.
- *Upstream treatment processes and loading rates*. The pollutant loads (g/m^2 d) are typically expressed in terms of plan area ($L \times W$), although for clogging considerations the cross-sectional area ($W \times D$) is a critical parameter. Rule of thumb sizing approaches assume typical influent quality and therefore loads applied. For example, areal loading rates of less than 10 g BOD m^{-2} d^{-1},

20 g COD m^{-2} d^{-1}, and 10 g TSS m^{-2} d^{-1} have been shown to enable secondary HF wetlands to operate without surface water ponding for 15 years of operation (Vymazal, 2018). Kadlec and Wallace (2009) suggested key design parameter for HF wetlands a design limit for the cross-sectional loading rate 250 g $BOD_5/m^2/d$. In tertiary systems, similar BOD and TSS areal loading rates have been employed in tertiary systems resulting in refurbishment intervals between 8 and 15 years. The main difference is the quantity of water that passes through the system as tertiary systems with hydraulic loading rates of 0.2–0.4 m/d, as opposed to 0.02–0.05 m/d in secondary systems (Knowles *et al.,* 2011). The capital and operational costs associated with sizing tertiary systems at these high hydraulic loading rates accept the fact that it will result in increased refurbishment intervals, as it is still a lower whole-life cost solution than building a significantly larger system that lasts longer between refurbishments (Dotro & Chazarenc, 2014).

Influent water quality can also affect the predominant wetland processes and require additional management allowed for in the design stage. Strongly anaerobic wetlands like secondary HF beds can generate sulphide and associated odours and a white discharge that will need management. Tertiary HF systems can be carbon limited for denitrification, resulting in low nitrate removal rates.

5.4.3 Potential design and operational issues

As HF systems are inherently passive (no mechanical parts for operation) and the media is fully saturated with water, they are less susceptible to critical failure than other wetland systems. Where systems have encountered major issues these are typically due to poor O&M or significant deviation from design guidelines. Experience from over twenty years of HF systems for secondary and tertiary treatment suggests, as well as following design guidelines (Dotro *et al.*, 2017), a few steps are recommended for operation. These include the protection of plant establishment in the first year of operation and management of preferential flow paths in mature systems.

Plant establishment may need to be protected based on site (region) specific risks. The most common strategies for protecting plant establishment from rodents that feed on reed plantings have included temporary flooding and temporary rabbit fencing. Flooding has resulted in unintentional incorrect operation of HF beds, with operators forgetting to lower the water level once plants have established. Fencing requires additional investment and can have a negative visual impact on the overall system. In many instances, no protection has been employed and the plants successfully established. Therefore, the risk of rodent access and damaging effects should be assessed during the design phase.

Preferential flow paths in mature systems will form as a result of accumulation of inert organics and the decay of biofilm within the bed media, as well as in the surface of HF systems that are surface loaded (Knowles *et al*., 2011). Management strategies have included resting of HF beds (i.e., draining the water and leaving it to dry for a number of weeks) and altering the operational water level in the beds. Both of these solutions have implications for sites with only one operational bed as the ability of the system to continue to provide treatment is impaired during intervention.

5.5 FWS WETLANDS

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5.5.1 Overview of existing design guidelines

FWS wetlands are probably the oldest TW type that has been tested and used, owing to its relative ease of construction, operational simplicity and considering that it resembles most closely a natural wetland. They are widely applied in North America (Kadlec & Wallace, 2009) and Australia (QDNR, 2000), but less so in Europe. FWS wetlands are typically applied as a polishing stage of secondary effluents, e.g., activated sludge, MBR, lagoons, but they have also been used in the treatment of various industrial and agro-industrial wastewaters and also surface runoff/stormwater.

There have been various design approaches over the years. Typically, sizing of FWS wetlands is based on area or volume. Design parameters that have been widely used in the volume-based design are the hydraulic retention time (e.g., 2–3 days per cell; USEPA, 2000), the vegetation porosity (0.65–0.75; Reed *et al.*, 1995), the water depth (0.1–0.6 m) and the length to width ratio (2:1 to 5:1; Economopoulou & Tsihrintzis, 2004). Area-based design considers the pollutant reduction using the overall wetland area. There is some merit to considering the sizing from an areal perspective, rather than volumetric, since increasing the area is the primary means to increase the amount of vegetation, and hence submersed stems supporting the biofilms responsible for treatment. Increasing the wetted depth (to gain more volume) beyond about 0.5 m will lead to a decline in vegetation health and a reduction in stem density. BOD and nitrogen removal rates are typically based on first-order kinetics and on the assumptions of plug flow (Crites and Tchobanoglous, 1998; Reed *et al.*, 1995; USEPA, 1988). The *P–k–C** first-order model is one of the latest models that can be effectively used to size the system and estimate its performance, e.g., for BOD_5 and/or ammonia reduction (Kadlec & Wallace, 2009).

5.5.2 Considerations for the start-up phase

For several reasons, a dense coverage of healthy and vigorously growing wetland vegetation is a particularly important component of a successfully functioning FWS wetland system, especially if the focus is on treatment. The vegetation provides a significant portion of the submersed surface areas for attached growth of biofilms and periphyton responsible for many biochemical treatment processes. For treatment processes such as denitrification, the designer often relies on the productivity and turnover of wetland vegetation to provide organic carbon via the internal photosynthetic conversion of CO_2 into biomass. Patchy vegetation growth can reduce the hydraulic efficiency of the wetland by creating short-circuiting flow paths. In many cases, gaps that develop in plant cover and occur during the initial planting and establishment phase can persist for many years and be difficult to close in an operating system, especially once bird populations establish in the wetland, which can have the effect of actively maintaining or expanding areas of open water. Thus, the initial planting and vegetation establishment period is a critical phase in the construction and commissioning of a FWS wetland.

Several factors need to be considered to optimize the chances for successful vegetation establishment, including appropriate plant species selection and diversity, use of good quality seedlings of an appropriate level of development (not too young, but not too old and root-bound), time of planting (spring is good, while late winter can be detrimental in cold climates that experience frost), suitable and well levelled topsoil, ensuring an adequate supply of suitable quality water immediately after planting, careful management of water levels and flow during the first three months after planting (keep soil moist and raise the water level as the young plants increase in height), management of algae which can quickly smother the soil surface and impose an oxygen stress on young plants in nutrient-rich waters, and management of waterfowl, aquatic wildlife and insect pests which may cause rapid damage to large areas of freshly planted seedlings.

Depending on the climate and nutrient status of the water, and how well the above considerations are managed, it may take anywhere between 6 and 24 months to achieve a dense coverage of well established vegetation. For some treatment processes (e.g., denitrification) which rely on the biofilms and organic matter turnover afforded by a healthy stand of vegetation, the duration of this start-up time should be considered in the project planning, especially if meeting specific treatment performance targets is critical.

5.5.3 Considerations for the construction

Despite the apparent simplicity of the FWS wetland system, there are a series of technical and economic challenges that should be considered in the construction stage, such as:

- Site selection, topography, geology and land availability are the first parameters that will define the economics and the feasibility of the project, considering that FWS wetlands tend to have higher area demands and typically operate via gravity flow.
- Good soil quality for the substrate and local availability are also important for the successful construction and operation of the system. Care should be taken that weed seeds are not introduced with the topsoil.
- Depending on the size of the system, a plastic impermeable liner or a clay soil layer can be used to seal the bottom. However, as the wetland size increases, the use of a plastic liner becomes prohibitive due to high cost, and a natural sealing layer is preferred. Hence, local availability of proper quality and quantity of the required materials is important.
- In terms of earthworks, adequate bund stability, including compaction and proper materials, hydraulic and geotechnical considerations, is also crucial to avoid any damage once the water level within the FWS wetland bed starts increasing. In general, earthworks should aim to balance excavation and filling to avoid buying surplus soil or discarding superfluous soil.
- FWS wetland systems designed for stormwater/runoff treatment may receive high volumes of water within short period time, thus the system should be able to accommodate these volumes and be constructed to withstand the expected flow velocities and erosional forces.
- For larger FWS wetland systems, large number of plants may be required for the plants establishment, which means that proper plant propagation schedule and logistics plans should be in place.

5.5.4 Design and dimensioning

For FWS wetlands designed for water quality improvement purposes, determining the size of the wetland to achieve certain pollutant reduction requirements is usually done using some form of first order concentration reduction model with reaction rates for each parameter of concern calibrated against performance datasets from existing systems. The current state-of-the-art approach for this is the P–k–C^* model, which is essentially a form of the retarded first-order tanks-in-series model derived for conventional wastewater treatment unit processes. An areal (rather than volumetric) approach is generally considered most appropriate for FWS wetlands (Kadlec & Wallace, 2009). Once the required area has been calculated from such a concentration reduction model, several cross-checks should be performed to verify if the predicted performance is in line with the experience base from other systems (e.g., by comparing areal mass removal rates), and to identify if any process limitations may exist which could slow the rate of pollutant reduction (e.g., alkalinity required for nitrification, or organic carbon required for denitrification). Such sanity-checks are particularly important for atypical wastewaters and contaminants or influent concentrations which are beyond the realm of the common performance experience (i.e., tertiary treatment of sewage). In some cases, a pilot study may be wise to gather information on performance rates and limitations.

After the FWS wetland area has been determined, the next critical design step is that of defining the number and configuration of individual wetland cells (in parallel and series) and their dimensioning (length and width). This is largely an iterative process to find the optimal solution with consideration of wetland hydraulics (headloss), site topography and slope, optimizing earthworks quantities (cutting versus filling) and operational considerations (e.g., ability to take cells off-line for maintenance). A key consideration here is the hydraulic design and calculation of headloss from inlet to outlet of a wetland cell, in order to define the maximum allowable length of any individual cell (for the given inflow rate and selected number of parallel cells). The vegetation imposes a resistance to flow through the wetland, which requires head (water elevation) to overcome this resistance. In large-scale wetland systems, the head-loss can be significant, resulting in significantly deeper water at the inlet end of the wetland (inhibiting plant growth) if the hydraulic design is not carefully considered. The power function calculation approach recommended by Kadlec and Wallace (2009), which includes a coefficient to account for the density of vegetation, is the most appropriate method currently available. To a certain extent, bed slope can be used to provide some of the head to overcome the vegetative resistance. However, achieving slight grades accurately during construction adds difficulty. Excessive difference in elevation between the inlet and outlet of a wetland cell, due to bed slope, also creates the risk that the front end of the cell will dry out (threatening vegetation) at low or no flow.

It is also important to consider the water balance for the wetland once the size is defined. In arid climates, evapotranspiration (ET) losses can be substantial, especially if relatively long residence times are required for treatment, leading to problematic salinity concentrations at the outlet or even no outflow during hot summer conditions in the worst case. Conversely, in tropical monsoonal climates which experience more than 3000 mm of rainfall per year, the wet season rainfall captured by the wetland can dominate the water balance, exceeding the influent hydraulic loading rate and resulting in a significant increase in the volume of water exiting the wetland which needs to be managed. As a minimum, a monthly water balance should be compiled to estimate the monthly outflow volumes, considering as a minimum the expected inflow rates, historical rainfall for the site (average and variability), an estimate of evapotranspiration (either from local Class A-pan data, reference ET from a weather station or monthly average potential ET maps that exist for many regions of the world), and assumptions about infiltration/exfiltration rates.

The vegetation selection and planting plans are also very important design considerations. The plant species should be selected based on the locally occurring flora, site conditions (e.g., climate, soil), the water quality (e.g., salinity, nutrient status and organic load), design water depth and considerations such as biodiversity and habitat creation. A high diversity of plant species is recommended to increase the ecological resilience of the wetland, especially with regards to pests and diseases which may threaten the health of the vegetation. The planting density needs to be defined, with consideration of cost (tending towards a lower density) and the desire to establish a dense cover of vegetation in the shortest timeframe and achieve the design treatment performance as soon as possible. Planting densities between 0.5 and 6 plants per m^2 have been used. In some FWS wetland projects, being constructed (or "reinstated") in former wetland sites that may have previously been drained, a sufficient seedbank may exist in the soil to achieve adequate revegetation without planting.

5.5.5 Main factors affecting treatment performance

In summary, the treatment performance of FWS wetlands can be affected by the following main parameters:

- Climatic conditions, i.e., rainfall, temperature variations, evapotranspiration or seepage, if not taken into account during the design stage;
- Inadequate hydraulic retention time and/or hydraulic design (e.g., length to width ratio);
- Higher applied pollutant loads than assumed in design, which exceed the oxygen transfer capacity of the wetland and result in anaerobic conditions, respective nuisance and decline in vegetation health;
- Monocultures, i.e., use of only one plant species, promote insects' development;
- Inadequate plant coverage and large open water areas, which could create algae blooms;
- Selection of plants species not adopted to the specific climate and water quality;
- Lack of vegetation management, overgrowth of plants and increased vegetation porosity, which may change the hydraulic flow patterns, create preferential flow within the system and, thus, affect the transformation/removal processes; and
- Variations in water depth and/or periods without inflow (e.g., in stormwater wetlands), which can result in dry-out and potential risk of releasing pollutants stored in the organic sediments of the bed.

5.6 SLUDGE TREATMENT WETLANDS

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5.6.1 Overview of existing design guidelines

Sludge Treatment Reed Beds (STRBs) or Sludge TWs are designed to dewater and mineralize sludge from Wastewater Treatment Plants (WWTPs) and Water Works. The sludge is passively dewatered by drainage through the filter and by evapotranspiration. Plant and microbial activity contribute to the dewatering, aeration and mineralization, leaving the treated sludge residue layer on top of the filter. The process results in the production of a higher quality biosolid end-product, which can be safely reused and recycled as a fertilizer or soil enricher (Nielsen & Bruun, 2015; Stefanakis *et al.*, 2011).

The main design parameters of STRB according to the design guidelines in various countries have been summarized (Nielsen, 2003; Nielsen & Willoughby, 2005; Nielsen *et al.*, 2018; Stefanakis & Tsihrintzis, 2012c; Stefanakis *et al.*, 2014). Dimensioning of the STRB is based on sludge production (tons of dry solids per year), sludge origin and quality, and climate. Those dimensioning criteria define the process area, the area load (kg DS/m^2/yr), the number of basins, loading and resting periods and finally the capacity of the system and the basins during the emptying period (Table 5.8).

Loading must be planned in such a way as not to inhibit the development of the reeds and to prevent the sludge residue from staying permanently wet and growing so fast that could undermine the reeds growth. In order to achieve the necessary balance between loading and resting periods and meet the requirement for long-term treatment, it is recommended that the systems have a minimum of six to eight basins depending on the climate (Table 5.8). According to the guidelines and the operational strategy, a STRB commonly operates for around 30 years. During this period, two to three operational cycles of 10–15

Table 5.8 Design and dimensioning criteria (*dimensioning in hot climates).

General Guidelines	
Number of basins	8–14 (6–10)*
Area load (kg DS/m^2/yr) – Full scale	30–60 (50–100)*
Area load (kg organic solid/m^2/yr)	20–40
Loading days	3–8
Number of daily loads	1–3
Resting days (older systems)	40–50 (7–21)*
Operation cycle	10–15 years
Feed Sludge	
pH	6.5–8.5
Dry solid (%)	0.3–4%
Loss on ignition (%)	50–65%
Fat (mg/kg DS)	5,000
Oil (mg/kg DS)	2,000

years are completed. An operational cycle consists of four phases: (1) commissioning, (2) normal operation, (3) emptying and final disposal of the sludge residue, and (4) re-establishment of the system (Nielsen, 2003; Stefanakis *et al.*, 2014).

5.6.2 Considerations for the start-up phase

Before a new STRB can become fully operational or a newly emptied bed can be put back to operation, it must undergo a period of 1–2 years depending on the regrowth or replanting and the climate. During an operational cycle, the different beds in the STRB are emptied in shifts to avoid simultaneous emptying and/or commissioning. An operational cycle is completed when all beds have been emptied. When some of the beds are out of operation or receive a reduced sludge volume due to emptying or commissioning, the quota must be raised for the other beds. Therefore, when dimensioning a new STRB, the capacity of the individual beds during the emptying period should be taken into consideration. Some of the older Danish systems are now running with at least one basin out of operation each year.

5.6.3 Pilot systems

Before the design, dimensioning and construction of a system, it is important to determine the sludge quality; in particular, its dewatering characteristics and the ratio between organic and inorganic solids (phase 1). The main goal is to test whether the sludge would be suitable for further treatment in a STRB. Other goals in phase 1 are to find out the following:

- Is it possible to treat and drain the sludge in a STRB system?
- How will the sludge behave (dry/crack up) in a pilot bed?
- Is it possible to get the vegetation grow in the sludge?
- What will be the dewatering efficiency of the sludge (L/s/m^2)?
- Are there any adverse or undesirable effects on reed health/growth rates?

The main goal of the next phase is to test and ascertain the criteria for the dimensioning, number of basins and operation of a full-scale system. In this phase, different loading rates and loading/resting days are tested to define the following:

- What sludge loading (kg DS/m^2/yr) can the pilot bed treat?
- How big a load (m^3, kg DS) can a pilot bed receive?
- How many loads can one pilot bed receive daily and in one quota?
- What is the optimum load and rest program in relation to sludge quality?
- What is the measured dewatering efficiency (L/s/m^2)?

5.6.4 Design and dimensioning

The sludge quality and sludge capacity requirements are very important parameters for planning and dimensioning of a new STRB. Moreover, in order to ensure a sufficient resting period, the number of basins to be established typically should be six to eight or more, depending on the climate, the total annual sludge production and quality. Most STRBs are dimensioned for an area load of 30–60 kg DS/m^2/yr (depending on the climate and the sludge quality), but higher loads (up to 90 kg DS/m^2/yr) have also been successfully applied under warmer climates for aerobic sludge (Nielsen *et al.*, 2018; Stefanakis & Tsihrintzis, 2012c). For sludge with a large proportion of fat, oil, and/or organic matter or a low sludge age ($<$20 days), the recommended area load should be reduced (Nielsen, 2003; 2011). STRBs essentially comprise a series of gravel/sand basins that are planted with wetland plants/reeds and typically 6–12 beds, even up to 24 beds or more depending on the climate. It is very important to consider the climatic conditions as a part of the STRB dimensioning and operation. For this, it is recommended to measure sludge dewaterability and run a pilot STRB before the design and construction of the full-scale STRB.

5.6.5 Climate

The results from a pilot plan in Wacol (SE Queensland, Australia) demonstrated very high sludge dewatering capability and challenges from the implementation of the STRB in a hot, subtropical climate (Nielsen *et al.*, 2018). The climate is characterized by mild, dry winters with mean daily temperatures in the range of 20–25°C, lows of 10°C, and hot and humid summer conditions with daily temperatures above 30°C and even exceeding 40°C several times a year. The hot climate enabled faster and better drying of the sludge residue due to air drying and higher plant evapotranspiration rates allowed for increased total annual loading rates. A sustainable loading rate of 60–70 kg DS/m^2/yr was established for the sludge quality from Wacol WWTP. A similar loading rate of 75 kg DS/m^2/yr was also determined for a pilot STRB operating under the Mediterranean climate, with mild winters and warm summers (Stefanakis & Tsihrintzis, 2012c). However, the climate also imposes the challenge of maintaining plant health in hot and dry conditions. Extended resting periods without loadings lead to water stress conditions for the plants (Nielsen *et al.*, 2018; Stefanakis & Tsihrintzis, 2011). These indicate that water stress may need to be managed more actively in the first few years of operations, when a layer of sludge residue is building up. Once the beds are covered with a sufficient layer of sludge residue of approximately 20–25 cm, it is expected that this layer will retain more moisture than the filter layers, enabling the plants to use capillary-bound water in the sludge residue layer in periods of water depletion between the loading periods. Two main solutions were found, an adapted loading program and a design response to the problem where the basins were designed to fill up with water within the filter below the drying sludge residue (Nielsen *et al.*, 2018). The fast and more efficient drying of the sludge residue allows for shorter resting periods between basin rotations, which can respectively allow for a reduced required number of basins than for systems in cooler climates and, therefore, lower capital cost (Table 5.8).

5.6.6 Main factors affecting treatment performance

In Denmark, Germany, Sweden, France and other countries in Europe the design and dimensioning of STRB systems has been extremely variable during the last 20 years, even if they were treating the same sludge type: the number of basins in different systems was between 1 and 24 basins, basin areas between less than 100 m^2 and over 3,000 m^2 and the area load from 30 to over 100 kg DS/m^2/year. In spite of this variety, all STRBs were more or less designed and dimensioned for 10 years of operation. The overall experience showed that a great deal of the systems had run into operation problems with a low efficiency, i.e., a low dry solid content in the sludge residue. The problems were observed in the vegetation, the low dewatering degree and the fast development of the wet anaerobic residual sludge layer; vegetation becomes stressed, wilted and even non-vegetated areas occurred. The operation problems could be attributed to failure in one or all of the four main categories:

Category 1: Sludge Quality
Category 2: Design and dimensioning
Category 3: Construction
Category 4: Operation

Very often the design and dimensioning were not related to sludge quality. The process areas were too small resulting in high areal load and the system had a small number of basins. The filters were constructed with media with low or none capillarity and finally they were operated with a wrong ratio between loading and resting periods (Nielsen, 2011). On top of that, the sludge quality is a very important parameter, which is often neglected. Even systems with a large number of basins, low area load and long resting periods have been insufficient to ensure healthy reeds, proper dewatering and mineralisation of the sludge, if the sludge quality is not suitable for treatment in STRB. Higher concentrations of organic solids and/or fat and oil result in lower dry solid content and more pronounced anaerobic conditions in the sludge residue (Nielsen, 2011; Stefanakis *et al.*, 2014). Dimensioning, number of basins and system operation must be based on sludge quality analysis (Table 5.8), in particular the dewatering characteristics of the sludge and the ratio between organic and inorganic solids (Nielsen, 2011).

5.7 AERATED WETLANDS

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5.7.1 Introduction

Aerated wetlands are saturated, HF or VF wetlands that rely on a mechanical system (an air pump connected to a subsurface network of air distribution pipes) to introduce air bubbles into the water being treated (Wallace, 2001). The use of an artificial aeration system dramatically increases the oxygen transfer rate compared to passive wetlands (Table 5.9), enabling improved performance for treatment reactions that require oxygen (such as nitrification) or occur more rapidly under aerobic conditions. The aeration system can also be operated intermittently to promote nitrification/denitrification (van Oirschot & Wallace 2014). A simple schematic and description of the process was covered recently by Dotro *et al.* (2017) and the improved treatment performance through aeration of pilot scale systems fitted to the first-order kinetic (*P*–*k*–*C**) model by Nivala *et al.* (2019b).

5.7.2 Design considerations

Standard HF and VF wetland systems rely on passive diffusion of oxygen into the water column. This is a very slow process in saturated-flow wetlands (HF and FWS) and passively improved upon in unsaturated-flow wetlands (VF and French VF wetlands). Mechanically aerating the system allows the amount of air introduced to be independent of the surface area of the wetland, allowing aerated systems to be loaded up to maximum clogging limits, which greatly reduces the area required and associated

Table 5.9 Estimated oxygen consumption in g $O_2/m^2/d$ for different TW types (adapted from Wallace, 2014)

TW Type	Estimated O_2 Consumption	Notes
HF wetland[1]	6.3	50th percentile values from Kadlec and Wallace (2009) assuming aerobic BOD removal and conventional nitrification.
FWS wetland[1]	1.47	
VF wetland (unsaturated)[1]	24.7	
French VF wetland (1st stage)[2]	40–60	Data from France indicates that the first stage of a French VF wetland can sustainable operate at roughly 1.5 m^2/PE
Aerated (HF and VF)	250	Mechanically aerated wetlands can achieve higher oxygen transfer rates, but 250 g/m^2-d is considered an upper $CBOD_5$ limit for clogging; most sustainable designs operate at $<$100 g/m^2-d (Wallace, 2014).

Notes:
[1]50th percentile values from *Treatment Wetlands, Second Edition* (Kadlec & Wallace, 2009); assuming aerobic BOD removal and conventional nitrification.
[2]Data from France indicates that the first stage “French VF” process can sustainably operate at roughly 1.5 m^2/PE (Molle *et al.*, 2005).

Table 5.10 Typical design parameters for aerated wetlands.

Design Parameter	Recommendation	References
Pre-treatment	Primary treatment common (CSO systems typically do not have pre-treatment)	DWA-A262E (2017)
Influent loading (inlet cross-sectional area)	<250 g $CBOD_5/m^2/d$ (maximum)* ≤ 100 g $CBOD_5/m^2/d$ (recommended)	Wallace (2014)
Specific area	$\geq 0.5\ m^2/PE$ $\leq 80\ g/m^2/d\ CBOD_5$	Stefanakis and Prigent (2018)
Influent distribution	$\leq 50\ m^2$ per feed point (unless bed is permanently flooded)	Dotro *et al.* (2017)
Air flow rate	$\geq 0.6\ m^3/m^2/h$	DWA-A262E (2017)
Air distribution	30 cm $\times$ 30 cm	DWA-A262E (2017)
Media size	8–16 mm	DWA-A262E (2017)
Treatment kinetics	pilot testing	Nivala *et al.* (2019b)

*Mechanically aerated wetlands can achieve higher oxygen transfer rates, but 250 g/m^2-d is considered an upper $CBOD_5$ limit for clogging (Wallace and Knight, 2006); most sustainable designs operate at $<100\ g/m^2$-d (Wallace, 2014).

capital cost. Aerated wetlands are generally dimensioned based on clogging, hydraulics, uniform air distribution, and first-order kinetics (Table 5.10).

Aeration of wetlands follows standard wastewater aeration design practices in terms of calculating oxygen demands and air flows based on actual/standard oxygen transfer rates (AOTR/SOTR) protocols (Metcalf and Eddy Inc., 2003). However, the hydrodynamic mixing of the water column induced by aeration is greatly reduced in gravel-bed systems compared to ponds or tanks (Wallace, 2014). This requires that the air distribution in wetland beds be very uniform (Wallace, 2014). Most air diffusers in mechanical treatment systems are high-flow/small-area devices that are poorly suited to uniform distribution, and successful wetland aeration designs have been based on alternative pipes or tubing that can distribute air uniformly. This generally requires empirical testing to determine the air flow vs. air pressure relationship for the product(s) under consideration.

Gravel media used in the system must have pore spaces large enough to allow the passage of air bubbles. Sand is too fine for aerated systems as the air collects and "blows out" in just a few locations. Air bubbles moving through the gravel media can combine and coalesce into larger bubbles (reducing oxygen transfer), however air bubbles follow a tortuous path through the media, slowing their transit time (increasing oxygen transfer). As a result, wetland aeration systems typically demonstrate an oxygen transfer efficiency intermediate between fine-bubble and coarse-bubble diffusers (von Sperling & Chernicharo, 2005; Wallace *et al.*, 2007).

5.7.3 Potential design and operational issues

Since aerated wetlands are high-rate treatment processes, they are sometimes designed very close to clogging limits, especially for HF; if overloaded, they can clog and require resting or refurbishment like other types of treatment wetlands.

During construction, testing of the aeration system to verify proper air delivery is essential. Since the air distribution lines are buried at the bottom of the wetland bed, replacing/repairing air lines after construction is difficult.

Fouling of the air distribution lines has been reported in isolated cases due to iron precipitates forming at the air distribution orifices. Using acid (HCl) to clean fouled air lines has been reported to be a successful quick and low-cost method (van Oirschot & Wallace, 2014).

Although the selection of the appropriate blower for the air distribution network should be based on air requirements, they can sometimes be limited by the smallest available size that a client can accept (based on rigorous health and safety requirements). To illustrate, four systems in the UK used the same size of blower to provide aeration to different size tertiary and secondary systems, resulting in a specific power allocation ranging from 4 W/m^3 of wetland to 26 W/m^3 wetland (Butterworth *et al.,* 2016a). In systems that are over aerated, venting of the air has been necessary resulting in wasted energy and noise complaints from adjacent residents. To minimise this, the selection of the correct aeration equipment should be emphasized to the client.

Stress of plants in both passive and artificially aerated wetlands has been reported in the literature, with chlorosis (yellowing of the leaves) being most predominant (Weedon, 2014) and a downward gradient observed in plant height from inlet to outlet in highly aerobic systems. In an assessment of four full-scale systems, one of the systems struggled to establish the common reed (*Phragmites australis*) whilst its twin bed under equal conditions but without aeration thrived with the same plants (Butterworth *et al.,* 2016a). The other three artificially aerated systems reported normal plant growth. The difficulty experienced with plant establishment in some UK systems did not affect treatment performance. A side-by-side full-scale trial comparing reeds (*P. australis*) to reedmace (*Typha latifolia*) plantings showed both plant species exhibited signs of stress (chlorosis and stunted growth) when grown with artificial aeration. Further controlled trials proved reedmace is proportionally more affected by aeration than the common reed but its higher natural growth rate can offset the true impact of aeration on biomass production (Butterworth *et al.,* 2016b). Plant stress has been attributed to iron deficiency and/or toxicity in aerobic systems. The fact it happens on some systems but not all suggests complex interactions between the biogeochemical conditions in the wetland subsurface and the plants. To illustrate, from 27 aerated wetlands built with expanded clay aggregates as their main media (instead of gravel), there have been no reports of plant stress to date. Recent research suggests observed iron-induced stress in reeds could be related to the plant's genetic code, with an iron foliar spray currently being assessed as mitigation strategy (Ren *et al.,* 2018). In practice, plant species selection for artificially aerated wetlands is typically done by the designer based on previous experience, and a variety of native wetland plants have been used to date.

5.8 FILL-AND-DRAIN WETLANDS

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5.8.1 Introduction

Fill-and-drain wetlands are subsurface-flow wetlands that rely on the alternating filling and draining of wetland cells to move water (during the fill cycle) and air (during the drain cycle) in and out of the wetland cell. These systems are alternately called *tidal flow* or *reciprocating* wetlands (Austin, 2006; Behrends, 1999). Fill-and-drain wetlands are a further development of the contact bed systems developed in late 1800s (Kinnicutt *et al.,* 1910).

In the simplest configuration, a cell is filled using the influent flow and then drained so the next "batch" of water is then treated. Alternate configurations use recirculation so that the fill-and-drain cycle frequency can be adjusted independently of the influent flow rate.

Fill-and-drain wetlands are of interest because through the sequence of filling and draining, they cycle through aerobic and anoxic/anaerobic phases automatically. This makes them especially useful when removal of total nitrogen (TN) is a goal through nitrification/denitrification.

Fill-and-drain wetlands undergo cyclic changes in redox potential, ranging from aerobic (draining + empty phase) to anoxic/anaerobic (filling + full phases). These cycling changes result in distinct treatment mechanisms operating in different phases of the treatment cycle:

- *"Empty" Phase.* Air, having been drawn into the bed during the draining phase, allows rapid oxygenation of biofilms (Behrends *et al.*, 2001). Organic compounds having been previously adsorbed into biofilms are consumed by microorganisms under aerobic conditions, and positively charged ammonium ions (NH_4^+) are converted to negatively charged NO_3^- ions. Once the food supply is exhausted, further microbial activity results in the endogenous respiration, reducing the occurrence of clogging.
- *"Filling" Phase*. As the cell is filled, air is forced out of the system. Water enters first at the bottom of the treatment cell, and thus has the longest contact time. Chemical transformations of the "full" phase begin to occur.
- *"Full" Phase*. As the pore spaces fill with wastewater, oxygen is consumed and the redox potential decreases. Nitrate ions (NO_3^-), formed from previously oxidized NH_4^+, diffuse out of the biofilms into the bulk liquid. The presence of NH_4^+ (from the influent wastewater) and NO_3^- creates conditions suitable for alternate nitrogen transformations such as anammox. NO_3^- can also serve as an oxygen supply for degradation of organic matter and for conventional denitrification; approximately 80% removal of total nitrogen (TN) has been observed (Austin *et al.*, 2003).

  Positively charged NH_4^+ ions in the bulk liquid diffuse into the biofilms. This diffusion process and the overall total adsorption capacity of the bed is enhanced by the cation exchange capacity (CEC) of the bed media (Austin, 2006). Organic compounds are adsorbed into the biofilms, and this process is relatively rapid, taking approximately 5 minutes (Kinnicutt *et al.,* 1910). When NO_3^- and dissolved oxygen are fully consumed, biodegradation of organic compounds can continue under anaerobic conditions.

- *"Draining" Phase*. Water is released from the bed. Rapid drainage times of 30 minutes or less are recommended, as this aids in drawing air into the bed (Dunbar, 1908). Chemical transformations of the "empty" phase begin to occur as the bed is drained, and the cycle begins anew.

5.8.2 Design considerations

Fill-and-drain wetlands are generally dimensioned based on clogging, hydraulics, and the number of fill-and-drain cycles per day, as summarized in Table 5.11. Flow is typically rotated through multiple beds in parallel or series, often using internal pumping to achieve the desired number of fill-and-drain cycles.

Once the wetland cells are dimensioned to avoid clogging, the most important design parameter becomes the number of fill-and-drain cycles per day, as this relates to oxygen transfer (Table 5.12). In many designs, this is related to a "rule of thumb" oxygen consumption rate of approximately 7–10 g O_2/m^3 cycle (Wallace, 2014), with the number of cycles per day determined by the total oxygen demand (carbon + nitrogen) applied to the system. This "rule of thumb" is commonly used because it is simple, but it does not take into account the fact that oxygen is not limited during the "empty" phase, so ammonia removal is actually a function of the amount of ammonia adsorbed by the bed during the "full" phase.

Ammonia removal is related to the ammonia exchange capacity (AEC) of the bed materials, which determines the total amount of ammonia adsorbed during the "full" phase. The AEC is related to the presence of biofilms on the bed media and the cation exchange capacity (CEC) (ASTM D7503-18, 2018) of the material making up the bed media. Standard laboratory procedures for measuring AEC have yet to be developed, and designers typically devise tests specific to the project under consideration. However, ammonia removal in fill-and-drain wetlands clearly improves when materials with a high CEC are utilized (Austin, 2006). Fill-and-drain wetlands with very high rates of ammonia removal have been designed and constructed based on AEC methods.

Overall performance (inlet vs. outlet) of fill-and-drain systems can be described using first-order kinetic rate coefficients (k) (Nivala *et al.*, 2019b), as summarized in Table 5.13. However, the diffusion processes

Table 5.11 Typical design parameters for fill-and-drain wetlands.

Design Parameter	Recommendation	References
Pretreatment	Primary treatment required	Kinnicutt *et al.* (1910)
Influent loading (inlet cross-sectional area)		
BOD_5	$\leq$100 BOD_5 g/m^2/d	Wallace (2014)
TSS	$\leq$100 TSS g/m^2/d	
Influent distribution	$\leq$50 m^2 per feed point	Dotro *et al.* (2017)
Drainage system	$\leq$30 min to drain bed (generally by siphon)	Barwise (1899)
Fill-and-Drain cycles	6–24 per day (6–12 per day common)	Dotro *et al.* (2017) Kinnicutt *et al.* (1910), Austin *et al.* (2003)
Media size	8–16 mm	Kinnicutt *et al.* (1910), Nivala *et al.* (2014)
Number of beds	2–8	Nivala *et al.* (2013a); Austin *et al.* (2003)
Treatment kinetics	Table 5.13 or pilot testing	Nivala *et al.* (2019b)

Table 5.12 Estimated oxygen transfer rates for passive and fill-and-drain wetlands (Wallace, 2014).

Wetland Type	Estimated Oxygen Consumption (g $O_2/m^2/d$)
HF[1]	6.3
FWS[1]	1.47
VF (unsaturated)[1]	24.7
French VF (1^{st} stage)[2]	40–60
Fill-and-Drain[3]	168–240

[1]50th percentile values from Kadlec and Wallace (2009) assuming aerobic BOD removal and conventional nitrification.
[2]Data from France indicates that the first stage of French VF wetlands can sustainably operate at roughly 1.5 m^2/PE (Molle *et al.*, 2005).
[3]Roughly estimated at 7–10 g O_2/m^3 per fill-and-drain cycle; at up to 24 cycles per day and a 1 m bed depth (Wallace, 2014). However, this also depends on the cation exchange capacity (CEC) of the media.

that occur during each phase of the treatment cycle appear to be rapid and occur more quickly than the beds can be physically filled and drained. As a result, each individual phase of the fill-and-drain treatment cycle have not yet been described using kinetics.

5.8.3 Potential design and operational issues

Loading fill-and-drain wetlands above recommended limits (Table 5.11) can result in clogging of the beds due to excess production of microbial biomass. Fill-and-drain wetlands are normally designed to receive primary-treated wastewater so that solids loadings on the beds are minimized. If primary treatment is problematic or is not provided, coarser bed materials (>75 mm) are required in the first treatment stage (Kinnicutt *et al.,* 1910). The use of coarser bed materials to reduce the potential for clogging also lowers treatment performance, so more than one treatment stage is employed (Barwise, 1899; Dunbar, 1908), with the first stage essentially acting as a roughing filter.

Fill-and-drain wetlands operate with a variable water level, and the gravel used does not have the same capillary action as fine-grained soils. This can be an issue during vegetation establishment, especially in arid climates. When plants are fully established, the root systems will extend throughout the bed down to the minimum water level. When plant root systems are still shallow, they can lose contact with the water,

Table 5.13 *P–k–C** model fit parameters for passive and fill-and-drain wetland systems at Langenreichenbach, Germany (Nivala *et al.*, 2019b).

	$CBOD_5$			NH_4-N			TN		
Wetland Type	**P**	**C* mg/L**	**k_{A20} m/yr**	**P**	**C* mg/L**	**k_{A20} m/yr**	**P**	**C* mg/l**	**k_{A20} m/yr**
HF	2.5	14.6	35	5	19	2.9	2.5	19.1	3.9
VF	2	0.6	315	2	1.5	176	2	11.5	40
Fill-and-Drain	2	0.3	672	2	0.1	450	2	4.4	123

increasing the risk of drought stress. This may require temporary irrigation systems during the plant establishment phase.

Treatment in the system is dependent on the number of fill-and-drain cycles per day. Designs that depend only on the influent flow rate to regulate the fill-and-drain cycle can have very slow cycling during low flows, consequently many designs use flow recirculation to increase the cycle frequency. Generally, it is best to design the system to support the maximum number of cycles desired, as it is much easier to slow down the cycle frequency than it is to speed it up.

5.9 FLOATING TREATMENT WETLANDS

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5.9.1 Introduction

Floating Treatment Wetlands consist of a pond or basin containing emergent macrophytes growing on a floating mat or raft, with the roots and associated biofilms hanging in the water column beneath (Figure 5.5). Water receives treatment as it passes through this hanging root-mat via biological, chemical and physical processes.

5.9.2 Overview of existing design guidelines

While some of the earliest full-scale deployments of Floating TWs for wastewater treatment were conducted in the late 1990s (see for example the acid mine drainage applications of Smith & Kalin, 2002), their broader application was relatively limited until the past 5–10 years. The recent and rapid development of the technology has also been across a broad range of applications, including urban stormwater, sewage effluent, eutrophic lakes and streams, airport de-icing runoff and various industrial applications. Thus, there is currently a lack of consistent and rigorous design guidelines for Floating Treatment Wetlands. It is fair to say that our understanding of the key treatment processes and functions that occur in Floating TWs is still in its relative infancy, albeit progressing rapidly with performance data from a growing number of studies each year. Pavlineri *et al.* (2017) provide a review of the performance data from published Floating TW studies. Based on results at that time, Headley and Tanner (2012) calibrated first-order areal removal rates (k) for BOD_5, NH_4-N, TN and TP within the P–k–C^* model structure of Kadlec and Wallace (2009). However, much of the available data is still from pilot or lab-scale systems

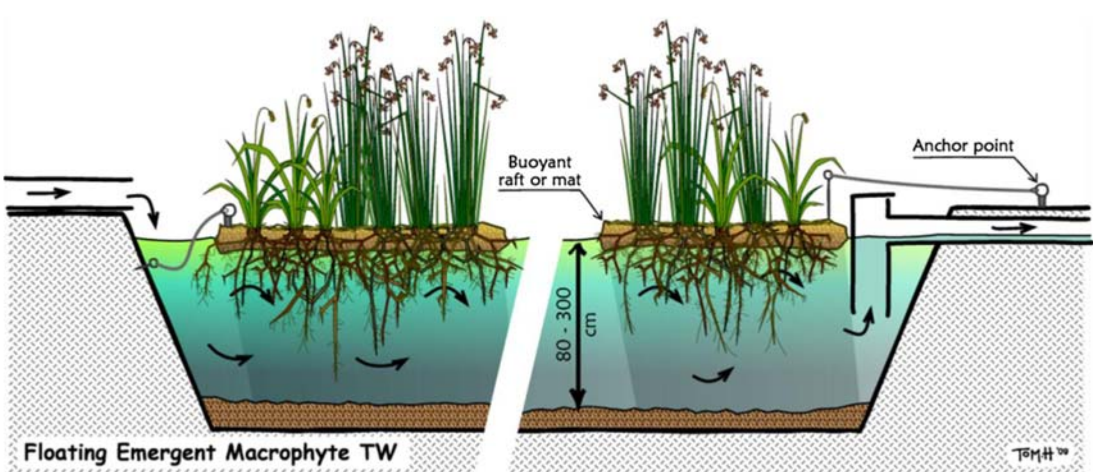

Figure 5.5 Schematic section through a typical Floating Treatment Wetland. Reprinted with permission from Headley and Tanner (2012).

and needs to be verified by long-term studies of full-scale systems. Arguments can be made as to whether it is more appropriate to take an areal or volumetric first-order approach to the sizing. However, for many contaminants (e.g. BOD, nitrogen species, fine suspended particles and associated metals) the rate of their removal will be fundamentally governed by the amount of hanging root-mat and associated biofilms that are present in the Floating TW. Given that the quantity of root-mat will be primarily a function of the area of floating mat, while the density of roots will decrease with depth, we advocate that it is more rational to consider the sizing of Floating TWs on an areal rather than volumetric basis.

5.9.3 Main factors affecting treatment performance

Aside from the usual universal biogeochemical processes which occur in other treatment wetland designs, the main factors affecting the treatment performance in Floating TWs are

- *Wetland vegetation*. especially the depth and extent of the hanging root-mat that develops beneath the Floating TW, which ultimately determines the specific surface area (m^2 root-mat per m^3 of water volume) of biofilm available for biochemical treatment processes and for trapping fine suspended particles.
- *Percentage areal coverage of the pond with floating mat*. This directly affects the amount of root-mat and associated attached-growth biofilm in the Floating TW reactor, versus suspended growth processes in open water. Also, dissolved oxygen tends to be consumed under a Floating TW, while the rate of passive oxygen transfer will be higher in open water areas due to diffusion, wind-induced turbulence and algal photosynthesis in the presence of sunlight.
- *Hydraulic Residence Time*. A minimum contact time may be required for some treatment processes.
- *Water depth and variability*. If the design water depth exceeds the depth of the hanging root-mat, then a certain portion of the flow will bypass beneath the root-zone with limited exposure to treatment. If the water depth is too shallow, the residence time may be compromised and there is a risk that the roots will embed in the benthic substrate causing operational problems, especially if the water level increases at some time.
- *Hydraulic efficiency and the configuration of the floating mats*. A given total area of floating mats on a Floating TW pond can be arranged in a multitude of ways to achieve the desired ratio of floating cover to open water (e.g. Walker *et al.,* 2017; Winston *et al.*, 2013). The configuration of the individual floating units can be used to manipulate the overall hydraulic efficiency of the Floating TW, with the aim of minimising the risk of short-circuiting paths, dead-zones and maximising the interaction between water and the hanging root-mats. Open water without root-mats will provide less resistance to areas with a dense root-mat beneath the floating rafts. Thus, water will preferentially flow through these open water zones. If possible, transverse bands of floating mats with complete connectivity from one side of the basin to the other, oriented perpendicular to the flow direction, are preferable.
- *Media used in the floating mat*. In some cases, media with specific properties have been trialled to promote certain processes, such as adsorption of compounds. However, the effectiveness and long-term feasibility of such approaches is yet to be verified at full-scale. In other cases, selection of inappropriate planting media in the Floating TW can leach nutrients or organic matter into the water column, leading to eutrophication or oxygen crash in the water column beneath. Care should also be taken to ensure that any plastic products used in floating mats are stable against UV degradation and do not degrade to release microplastic particles into the water body.
- *Aeration of the water column*. In some situations, where the oxygen demand of the water is relatively high, active aeration has been employed to overcome oxygen transfer limitations and enhance the efficiency of oxygen-consuming treatment processes.

5.9.4 Specific design considerations

Aside from the questions around sizing of the Floating TW to achieve given water quality improvement targets, there are several practical aspects to be considered in the design:

- *Techniques and materials for constructing the floating rafts*. Several options exist, with varying degrees of compromise between cost, longevity and convenience of deployment. Headley and Tanner (2012) provide a comprehensive overview of the main approaches adopted to date for construction of the floating structures.
- *Anchoring the floating structures* to the edges and/or bottom of the basin. The floating rafts should be secured sufficiently to prevent that they drift excessively with wind or wave action. Sufficient allowance should be made to for rising and falling of the floating rafts with changing water level in ponds with variable water depth, such as stormwater systems.
- *Minimum and maximum allowable water depths*. Careful consideration needs to be given in the design to the operating water depths of the Floating TW system. It is recommended that the minimum water depth be greater than the expected depth to which most of the plant roots develop, to avoid the roots imbedding in the benthic sediments or that the root-mat becomes damaged at low water levels. The design of outlet water control structures is often key in ensuring the desired water depths are achieved. In some cases, it may be important to consider design allowances to ensure a permanent pool and depth of water can be maintained during low or no-flow conditions.
- *Planting media*. Use of a planting substrate may be necessary to establish the plants in the floating rafts. Media that are lightweight, low in nutrients and will not impose a high oxygen demand when saturated with water, while providing a good substrate for root development are preferred. Coir fibre or peat moss materials are suitable options. In some cases, plants are established without growth media.
- *Hydraulic design*. Consideration should be given to the dimensioning of the system (length, width and depth) and configuration of the floating rafts from the perspective of flow velocities to avoid scouring of sediments or biofilms attached to the hanging root-mats.

5.9.5 Considerations for the start-up phase

Establishment of the wetland vegetation in Floating TWs can be approached in two ways:

- *In situ planting and establishment*. the rafts are planted while floating on the pond;
- *Onshore planting and establishment*. the rafts are planted out of the water (e.g., on the shore or at a remote location) and transferred onto the Floating TW at some stage thereafter, often when the plants have at least partially established.

The benefits of *in situ* planting are that the Floating TW rafts do not need to be relocated once the vegetation is established, with the risk of damaging the stems and roots in transport. It can also be physically challenging to transfer large floating rafts from the shore into a pond with the added weight of vegetation and waterlogged media. However, it can be difficult to control birds and conduct weed maintenance during plant establishment if the Floating TW rafts are floating on the pond. In this regard, the early care and maintenance of the vegetation can be easier with onshore planting, with the added benefit that the Floating TW commissioning and start-up period can be minimised if rafts are deployed already with established vegetation.

5.10 WILLOW SYSTEMS

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5.10.1 Introduction to willow systems and existing design guidelines

Willow systems are TWs dominated by willows. They are currently in use for *onsite* wastewater treatment and reuse in Scandinavian and Baltic countries, Ireland, England, China and Poland, and pilot systems are operating in France, Greece, Spain and Slovenia. Willow systems are mostly designed to treat all the influent water though evapotranspiration, so there is no outflow from the system. They are also called evapotranspirative systems as they combine two separate processes removing water from the soil surface by evaporation and from plants via transpiration.

Additionally, there are also willow systems that combine seepage with evapotranspiration or that are designed as flow-through systems that produce some outflow, which is used for the recovery of resources, such as nutrients and reclaimed water for irrigation.

National design guidelines for willow systems are only available in Denmark (Miljøstyrelsen, 2003a, b). The Guidelines comprise systems of up to 30 PE in two construction and operation versions:

- *Zero-discharge systems*. These systems are specifically designed and established to evapotranspire all the influent water (both wastewater and precipitation). Therefore, the bed must be lined using impermeable foil to prevent infiltration of water to groundwater. Since all the water is evapotranspired, these systems are intended to be built in locations with high water-quality discharge standards, where no effluent is an option. The treatment area demanded for such system will be the balance between the influent water and the plant's potential evapotranspiration in the geographical location.
- *Systems with infiltration*. For these systems the bed will not be lined, so some infiltration is possible. Such systems are intended for the locations with clayish soils where natural infiltration is already low. Calculating the area needed will also include the soil infiltration capacity.

According to the Danish national guidelines the surface area of the system depends on local climatic conditions and the amount of wastewater to be treated. In Danish climatic conditions, a single household willow system covers between 200 and 300 m^2. When properly dimensioned, willows should evapotranspire all the inflow wastewater and rainfall (Gregersen & Brix, 2001).

Willow systems enable wastewater treatment, evaporation of water and recycling of the nutrients through the willow biomass. They are most appropriate for the sites where standards for wastewater discharge are strict and where soil infiltration is not possible. In such areas, other treatment technologies like compact wastewater treatment plants or TWs may not reach the desired outflow concentrations, or the upgrading of the technology to meet the discharge limits is economically unfeasible.

Willow system produce a significant amount of biomass that can be used for energy purposes (see Chapter 4.7 Biomass production); therefore, the construction of willow systems should go along with establishment of a chain of biomass processing, combustion and end users of the produced heat.

5.10.2 Main factors affecting dimensioning and performance

In general, the amount of the wastewater that can be loaded into the system is equal to the difference between evapotranspiration of the system and precipitation; however not all falling precipitation enters the system on a yearly basis since a significant part of precipitation is captured in the tree canopies or system's surface from where it evaporates back to the atmosphere or is blown by the wind to adjacent areas (Istenič *et al.*, 2018). While evapotranspiration and precipitation vary throughout the year, the loading with wastewater is constant. According to the amount of wastewater to be treated per year, a net area of the system can be calculated (Brix & Arias, 2011). Existing willow systems have been designed in a way that the water loss in the system is at least twice the potential evapotranspiration. Potential evapotranspiration (ET_P) is calculated from local meteorological data and depends on the plant species. Namely, the ET_P is a product of reference evapotranspiration (ET_0) and crop coefficient (k_c). ET_0 data can be obtained from the nearest climate station and are given for a reference crop (short grass) (Penman–Monteith equation).

When designing a willow system its orientation on site is of crucial importance. To increase evapotranspiration, willow systems should be long and narrow (up to 50 m), placed in an open landscape and perpendicular to the prevailing wind direction, sheltered and shaded locations must be avoided (not surrounded by tall vegetation and/or buildings). When dimensioned correctly, the so-called clothesline and oasis effect significantly increase the evapotranspiration. Clothesline effect is caused by the fact that willow trees in the system are higher than surrounding vegetation acting as a line of drying clothes – the evapotranspiration is increased due to broadsiding of wind horizontally into the taller vegetation. The wind brings heat from the surroundings and increases the air turbulence in the canopy that enables transport of vapour away from the canopy. Similarly, the oasis effect is caused by the difference in temperature of the system compared to the surroundings. Namely, the vegetation has higher soil water availability than the surroundings. Solar radiation and heat provide the energy needed for transformation of water from liquid to vapour state. Due to this endothermic process, the air in the system is cooled causing a difference in air pressure and as a result, the warmer air from the surrounding blows into the system, increasing evapotranspiration. In this way, willow systems also affect the local microclimate.

The willow system must also be positioned to enable access for all the machinery during construction as well as for harvesting.

5.10.3 Operation and maintenance

The willows are harvested in one- to three-year cycles to maintain healthy vegetation with high biomass production. To maintain high evapotranspiration, not all the system is harvested at every cycle but only half or a third (depending on the length of the cycle), which in case of a three-year cycle results in three sections of the bed with harvested, one-year and two-year plants (Gregersen & Brix, 2001). To reach higher biomass production, the number of shoots has to be as high as possible; therefore, the first year all shoots are cut in order to stimulate their propagation. The harvest should be done during the dormant period.

After a certain period of operation of the evapotranspirative system, the media may become saturated with nutrients. The nutrient-rich media can be used as compost or fertilizer in agriculture (Gregersen & Brix, 2001), which enables the return of nutrient to the food chain.

5.10.4 Specific design considerations

The willow system consists of a septic tank as defined for other TWs, pumping well, and soil bed planted with willows (Figure 5.6). The bed is 1.8 m deep, in case of zero-discharge system, lined with high-density polyethylene or equivalent membrane and filled with original soil from the site up to 1.5 m level. A 0.3 m basin dike protects the basin against intrusion of water from the surroundings and enables accumulation of

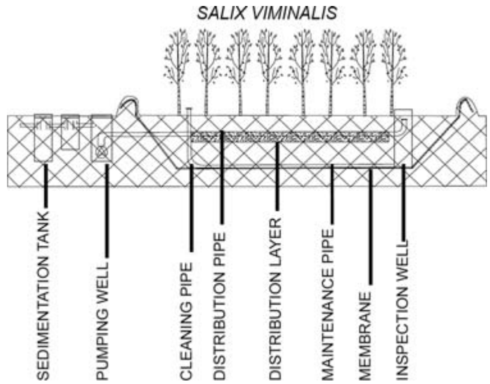

Figure 5.6 Cross section through a zero-discharge willow system. Reprinted with permission from Brix and Arias (2011).

the water on the surface during the winter or at high precipitation. The distribution pipes are placed on a 0.25 m thick and 0.20 m wide layer of 16–32 mm gravel or some other material, running in the middle of the bed through all its length to allow proper distribution of the influent wastewater. The distribution pipe must be covered with 0.6 m of soil or dug into the bed in order to prevent frost. The wastewater is pumped to the system according to the wastewater production.

Besides the distribution pipe, willow systems also include the maintenance pipe and inspection well. Maintenance pipe enables potential washing of the system with freshwater when the salinity is increased to the level to reduce the willow growth. The inspection well enables monitoring of water level in the bed and pumping out the water after the cleaning process.

Willow systems can be planted with species and clones of willows that are fast growing and have high biomass yield. Mainly selected clones of *Salix viminalis*, and *Salix alba* have been used (e.g. Curneen & Gill, 2014; Istenič *et al.*, 2017). Indigenous willows taken from nature are not appropriate due to lower biomass yield. Ideally different clones or varieties should be planted in parallel rows of a system to prevent spread of diseases though all the system. Willows are planted as 20–30 cm cuttings ideally in early spring after the last frost. Cuttings are gained from a year-old shoots during a dormant period (December–March in a temperate north latitude climate). If immediate plantation of cuttings is not possible, they should be stored at −2 to −4°C (can last for several weeks). Willows are planted in rows 1 m apart; the spacing along the rows is 0.5 m. Every three rows there is a wider gap of 1.5 m (Brix & Arias, 2011).

5.10.5 Considerations for the start-up phase

- The cuttings must be prepared correctly and planted at prescribed time of the year as described above.
- During the establishment year, weeds must be removed mechanically; willows must be controlled for general health and harmful pests must be eliminated.
- The winter after planting, willows must be cutback to 10 cm above ground which will induce multi-stem growth in the next season, resulting in a dense canopy that will significantly reduce the light penetrating to the ground and thus prevent weed growth.
- During the establishment year, evapotranspiration of the system is lower (according to Istenič *et al.*, 2018, half of the designed rate), thus it is recommended that the system is not fully loaded.

5.11 USE OF REACTIVE MEDIA FOR ENHANCED PHOSPHORUS REMOVAL

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5.11.1 Introduction

A common adaptation to both conventional HF and VF wetlands is the replacement of inert media (sand or gravel) by reactive media (e.g., steel slag, apatite), or the addition of a separate treatment step, to achieve sustained phosphorus removal. There are strong regulatory drivers for efficient removal (and recovery) of phosphorus from wastewater. During the last decades, a vast amount of research has been invested to tackle the problem (Dotro *et al.,* 2017). This section summarises the current knowledge and state of the art from collective experience on the design and operation of such systems from long-term studies (>2 years) at both pilot and full scale.

5.11.2 Overview of existing design guidelines

After decades of research on reactive media for P removal, the best available information on design and operation of reactive-media wetlands is based on long-term trials rather than national standards (Table 5.14). There is currently no literature reporting on full-scale operation of reactive-media wetlands reaching capacity and removal (or recovery) of the exhausted media. Lifetime predictions are based on retention capacities from various scales and operational conditions, with limited consistency between studies making it difficult to draw general design guidelines. On a recent report from three full-scale systems designed, built and operated with the same criteria in the UK the removal pattern was inconsistent among sites with no clear link to alkalinity or influent phosphorus concentration, the two commonly accepted key parameters to consider for the design of reactive-media systems for P removal (Fonseca, 2018).

5.11.3 Design considerations

The main factors affecting the treatment performance of reactive media wetlands are:

- *Reactive media type and size*. Phosphorus removal is mainly associated with the physical–chemical properties of the media. Chemical composition of the media is of importance, specifically its content of Ca, Al or Fe, three elements that can react with P under different environmental conditions. Media with small granulometry, higher porosity and larger specific surface area would be the best option. However, the smaller grain size is also associated with higher clogging risk (biological, physical and chemical clogging), low hydraulic conductivity and therefore, an optimal size, according to the special characteristics of the media and the expected water quality must be determined.
- *Reaction kinetics*. P retention kinetic has to be determined in real environment sampling water at different retention time within the filter. The use of simple models (i.e. P–k–C* or K–C*) is generally accurate enough to fit P concentration evolution within the media. As different

Table 5.14 Summary of existing design recommendations for reactive media.

Reference	Delgado *et al.* (2018)	Kõiv-Vainik *et al.* (2016)	de la Varga *et al.* (2017)	Fonseca (2018)
Target pollutant	o-PO_4	TP	TP	TP
Mode of operation	Saturated VF or HF	HF	VF	HF
Plants used	None or *Phragmites australis*	*NA*[1]	*NA*[1]	*Phragmites australis*
Influent wastewater range (mg/L)	5–15 (PO_4-P)	2–36 (PO_4) 4–36 (TP)	18–25 (TP)	4–7
Target effluent concentration (mg/L)	1.5–2.5	1–2 (TP)	1.5 (TP)	1–2
Media	Granulated apatite – Phosclean	Hydrated oil shale ash	Calcite based	Steel slag
Media size (mm)	2–6	4–16	4–8	10–14
Hydraulic residence time (hours)	6–48	144	48	24
Expected effluent pH	9–11 at the beginning then 7–8	8–13	7–9	10–12
Effluent pH management	Chemical acidification	Dilution (not solved)	Not needed	Blending (not solved)
Design life	5–6 years	Not determined	1 year	Not determined
Fate of material at breakthrough	Potential use as long-time release fertilizer	Potential use as slow release fertilizer (Kõiv *et al.*, 2012)	Fertilizer	Proposed as fertilizer but not currently done

[1]The systems in Denmark and Estonia are based on contactors complementing a wetland system rather than wetlands retrofitted with reactive media.

mechanisms can operate according to saturation state (adsorption–precipitation) and environmental conditions (pH, alkalinity), kinetics can evolve with time. The one measured in a commissioning period can differ from those after years of functioning. Designers should define kinetic evolution when long-term P retention is targeted.

- *Pollutants load to the reactive media*. All reactive filter materials are vulnerable to insufficient or lack of pre-treatment. As P retention relies on surface mechanisms, an excessive biomass growth will hinder access to the media surface. Therefore, it is recommended to locate the filter after effective biological treatment steps (i.e. tertiary treatment).
- *Alkalinity of influent wastewater*. When P retention mechanisms are linked to Ca–P bonds or precipitation, alkalinity of the influent can impact kinetics or the type of Ca–P retention (stable or not – competition with carbonates). In some cases, Ca addition can be necessary (by the use of calcite gabion) to increase alkalinity, pH and Ca concentration and favour P retention.
- *Temperature*. Some studies show that the seasonal variability in wastewater temperature affects the phosphorus removal efficiency in alkaline reactive media. In some cases, P removal efficiency

improved with increasing temperature, because this affected the rates of CaO-slag dissolution and Ca-phosphate precipitation (Barca *et al.*, 2013). However, the effect of the high temperatures can be opposite in case of higher influent organic pollutants content that can result with biofilm growth inside the media. The reaction kinetics must be defined at different temperatures prior to set up of the full-scale system to take into account possible seasonal variations in performance.

- *Hydraulic residence time*. Hydraulic residence time has to be set up, taking into account the porosity of the material, in accordance with outlet P concentration required and retention kinetic measured on the media. When long-term removal is targeted, kinetics with high saturation levels have to be used for design. Sorption capacity measured in batch tests decreases at real hydraulic residence time (Arias *et al.*, 2003). Removal performances will be higher at the commissioning period.

  The hydraulics of the filter (water distribution and collection) have to be carefully designed to avoid short-circuiting and dead zones that could impact on the efficiency of the filter. The residence time is calculated including the porosity of the media, which typically ranges from 0.35 to 0.5. For reactive media using calcium (e.g. steel slag, hydrated oil-shale ash, apatite), there is usually a direct link between HRT and effluent pH. Thus, the retention time must be carefully selected to avoid too high pH in the effluents (>9) as a result of excessive CaO-slag dissolution and rapid chemical saturation by secondary carbonate precipitates (Barca *et al.*, 2013; Liira *et al.*, 2009).
- *Necessity of pilot trials before full-scale application*. A pilot trial should be utilised before scaling up, testing the media with the target wastewater (i.e., no synthetics or surrogates) and operate it for at least a full year (preferably until media saturation with P) to enable results to be translated to full-scale systems. Particular consideration should be given to the hydraulics of the reactor on scaling up.

5.11.4 Potential operational issues

Secondary pollution

One of the main issues when using Ca-rich media is high pH of the effluent. One recent study dealing with high effluent pH of the slag filters shows that effluent neutralization with CO_2-enriched air from an upstream bioprocess could be a solution in some cases (Bove *et al.*, 2018). However, the addition of dosing strategies defeats one of the key benefits of using reactive media (i.e., no chemical dosing onsite). Other studies have suggested effluent dilution, polishing ponds, neutralising filters with acidic media (bark, peat, sand). Media-specific problems have also been identified on vanadium leaching from industrial by-products (steel slag; Fonseca, 2018), and chromium and radiation concerns from engineered media (apatite; Fonseca, 2018).

Role of vegetation

When alkaline reactive media are used in a separate treatment unit then the general recommendation is to avoid vegetation and any other biological activity inside of the filter. There have been mixed reports from planted reactive-media filters where plants (*Typha latifolia*) have established without issue or have struggled under similar pollutant loading conditions (Fonseca, 2018). However, because alkaline materials are vulnerable to air CO_2 (resulting in formation of Ca carbonates) the reactive filters are sometimes covered, with an insulation/cover media amenable to planting where root development can be contained above the reactive media.

Planning for maintenance and clogging management

In addition to the standard risks of clogging associated with the particular wetland configuration, chemical reactions within the media make clogging more likely to occur in these types of systems. Precipitates in alkaline media can result in cementing of media, which significantly limits porosity as the bed ages. Strategies to minimise this include isolating the media from air and choosing the right media size taking into consideration the removal mechanisms that will dominate in the system.

5.12 MULTI-STAGE WETLANDS

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5.12.1 Overview

Multi-stage wetland systems (also known in literature as hybrid systems) can be defined as the combination of different TW types, in order to exploit the advantages of the different systems and to obtain better results in comparison with the use of a single stage. The definition is quite broad; indeed, every combination of HF, VF, or FWS wetlands can be considered a hybrid system. Due to the advantages and the provided removal processes, the hybrid systems are well suited for various applications, some of which are summarised in Table 5.15. A multi-stage approach can be valuable in case of particular wastewater to be treated. Two examples are the treatment of industrial wastewater with high influent organic loads (e.g., winery wastewater, see Chapter 4.11.5) and landfill leachates (see Chapter 4.10).

Strict guidelines on multi-stage systems are not available, due to diverse operating conditions and applications. Therefore, this section gives an overview of the possibilities using multi-stage wetland systems, thus allowing a proper consideration of hybrid instead of single-stage solutions when the use of TWs for wastewater treatment is under discussion. Moreover, this section provides suggestions for designers and practitioners to properly design multi-stage wetland systems as a function of different targets.

As a general observation, the design methods for sizing TWs which are based on datasets acquired at single-stage or single-typology treatment plants can be unsuitable for the design of "stages" where the performance in removing or retaining pollutants has to be decided in an often-tailored way for every specific realisation.

5.12.2 Nutrient removal

Multi-stage systems are often adopted when removal of nutrients is required.

Total nitrogen (TN) removal is a typical case in which the application of a hybrid TW leads to successful results (Gajewska *et al.*, 2015; Masi *et al.*, 2013). Indeed, the complete cycle of N removal is possible combining the efficient nitrification process of VF (aerobic conditions) with the denitrification promoted by either HF or FWS (anoxic/anaerobic conditions). The design of TN removal with hybrid wetlands can be done as follows:

- The VF nitrification stage needs to be sized for the desired effluent $NH4^+$-N concentration, according with methods and guidelines summarised in Chapter 4 of the *Treatment Wetlands* textbook (Dotro *et al.*, 2017), such as the oxygen balance method of Platzer (1999). It is important to fully nitrify the wastewater in order to provide an efficient denitrification in the next stage and/or meet legal discharge requirements on effluent $NH4^+$-N. The oxygen transfer rate for VF is reported in literature with a wide range, varying from 23 to 92 $gO_2/m^2/d$ (see Nivala *et al.*, 2013b). Therefore, it is better to be conservative in the selection of the oxygen transfer rate for the sizing of the VF for proper TN removal. In case of stringent limit for effluent $NH4^+$-N concentrations, a subsequent nitrifying stage can be added (e.g., VF + HF + VF) to ensure complete nitrification.
- The HF or FWS denitrification stage, or both (VF + HF; or VF + FWS; or VF + HF + FWS), can be designed according to well known kinetic formulations, which are summarised in Chapter 2 of the *Treatment Wetlands* textbook (Dotro *et al.*, 2017), such as the P–k–C* model of Kadlec and

Table 5.15 Examples of multi-stage systems for different applications.

Application	Multi-Stage Scheme	Size	Location	Reference
Municipal wastewater – Secondary treatment	HF + VF + HF + FWS	3,500 inhabitants	Dicomano (Italy)	Masi *et al.* (2013)
	HF + VF	190 pe	Sarbsk (Poland)	Gajewska *et al.* (2011)
	HF + VF + HF	150 pe	Wiklino (Poland)	Gajewska *et al.* (2011)
	HF + HF + VF + HF	600 pe	Darżlubie (Poland)	Gajewska *et al.* (2011)
	HF + VF + HF	500 inhabitants	Chorfech (Tunisia)	Masi *et al.* (2013)
	French VF (1st stage) + VF + FWS + Infiltration pond	1,000 pe	Castelluccio di Norcia (Italy)	Rizzo *et al.* (2018b)
Municipal wastewater – Tertiary treatment	HF + FWS	60,000 pe	Jesi (Italy)	Masi (2008)
Winery wastewater	HF + VF + HF + FWS + Pond + Sand filter	42 m^3/d	Bolgheri (Italy)	Masi *et al.* (2015a)
	French VF (1st stage) + HF + FWS + Sand filter	100 m^3/d	Castellina in Chianti (Italy)	Masi *et al.* (2015a)
	Hydrolytic upflow sludge bed + VF + HF	7 m^3/d	Pontevedra (Spain)	Serrano *et al.* (2011)
Landfill leachate	VF + HF	10 m^3/d	Liubljana (Slovenia)	Griessler Bulc (2006)
	Pond + FWS + Pond	473 m^3/d	Escambia County, FL (USA)	deBusk (1999)
Combined sewer overflow	VF + FWS	maximum treated flow 640 l/s	Gorla Maggiore (Italy)	Masi *et al.* (2017a)

Wallace (2009). Since the denitrification is planned to occur after a previous nitrifying stage, the risk of limited denitrification for carbon deficit needs to be checked. Kadlec and Wallace (2009) reports 0.7–1.1 g C/g N or a 5:1 C:N ratio for uninhibited denitrification in wetlands. The carbon deficit issue for denitrification is particularly relevant for influent wastewater showing strongly unbalanced C:N, such as swine wastewater (Masi *et al.*, 2017c). On the other hand, a carbon deficit can be overcome by nature-based solutions itself. Indeed, the more natural environment can provide endogenous C sources to fuel denitrification, for instance root exudation (Zhai *et al.*, 2013) and decayed plant biomass (Hang *et al.*, 2016). Indeed, FWS systems as tertiary stages for denitrification purposes (Masi, 2008) and multi-stage wetlands in general have shown the capability to efficiently remove TN with BOD_5/TN influent ratios (1.5–2.5) lower than other biological treatment methods (4–5) (Gajewska *et al.*, 2015). Alternatively, recirculation can be adopted in case of limited area, allowing effluent rich in nitrate and poor in carbon to be mixed in the first stage with an incoming wastewater rich in C (Saeed & Sun, 2012). For instance, Brix and Johansen (1999) propose a

HF+VF hybrid scheme with recirculation option to enhance TN removal. In case of recirculation, the HRT needs to be carefully checked, in order to avoid reducing too much the retention time due to the recirculation, compromising the denitrification process.
- Since no detailed guidelines are available for the design of multi-stage systems, it is always suggested to check the sizing with available similar systems and results reported in literature (e.g., Gajewska *et al.*, 2015; Masi & Martinuzzi, 2007; Vymazal, 2007).

Phosphorous is removed in TWs mainly via sorption processes (Kadlec & Wallace, 2009). Therefore, the adoption of multi-stage wetlands favours the removal of TP, since each stage removes TP as function of available sorption sites (i.e., sorption material). An option to improve the TP removal is to adopt a final "sacrificial" unplanted stage, which needs to be refurbished when the TP sorption capacity decays. The "sacrificial" filter need to be filled with highly adsorbing material, either natural or commercial (Kasprzyk *et al.*, 2018; Vohla *et al.*, 2011, see also Chapter 5.11 Use of reactive media for enhanced phosphorus removal). In the design phase of the sacrificial P filter, the reduction of adsorbing capability due to low-functioning HRTs must be considered (Arias *et al.*, 2003; Brix *et al.*, 2001). Alternatively, dosing of iron salts can enhance the TP precipitation (Dotro *et al.,* 2015; Kim *et al.*, 2015).

5.12.3 Enhanced disinfection with nature-based solutions and wastewater reuse

The use of tertiary treatment for disinfection purposes in multi-stage systems is a common practice, especially using FWS systems (Wu *et al.*, 2016). Typically, hybrid systems are used when treated wastewater reuse is a goal. The following points need to be checked in the design of multi-stage systems aimed to reuse treated wastewater:

- It is fundamental to check legislation limits for reuse, which can differ in different countries (Jeong *et al.*, 2016). If legislated limits are not available, an effluent safe pathogen water quality standard needs to be set by the designer as a function of differently aimed wastewater reuse, as suggested by WHO guidelines (Jeong *et al.*, 2016; Licciardello *et al.*, 2018).
- In case of particularly strict pathogen removal requirements, a cost–benefit analysis should be done to understand if it is better to oversize nature-based solutions only for water quality targets linked to disinfection or if it would be better to implement a technological disinfection unit (such as UV lamps, e.g. Álvarez *et al.*, 2017) as final disinfection stage. This is particularly true for applications in arid climates, in which the high evapotranspiration rate characteristic of extensive wetlands could reduce the amount of recovered water and decrease the dilution coefficient (i.e., higher effluent concentration with the same mass removal obtained in temperate countries).
- As function of the needed pathogen removal, it can be valuable to check the possibility to reduce the nitrogen removal to recover nutrients (Zurita & White, 2014), and build sustainable circular economy loops (Masi *et al.*, 2018).

5.12.4 Exploitation of different ecosystem services

Multi-stage design can be designed to exploit the additional ecosystem services provided by nature-based solutions (particularly FWS system) such as biodiversity increase (Hsu *et al.*, 2011), flood mitigation (Rizzo *et al.*, 2018a), and social benefits (Ghermandi & Fichtman, 2015; Liquete *et al.*, 2016). Therefore, hybrid TWs enhances the possibility to integrate ecosystem services in a multi-purpose design (e.g., Liquete *et al.*, 2016; Masi *et al.*, 2018).

From the point of view of urban runoff management, multi-stage wetlands have great potential to be integrated in green–blue infrastructures, following new city design concepts such as Sustainable Drainage Systems (SuDS – Woods-Ballard *et al.*, 2015), Water Sensitive Urban Design (WSUD – Wong *et al.*, 2009), Low Impact Development (LID – Dietz, 2007) or Sponge cities (Li *et al.*, 2017). To this aim, it is interesting to highlight that some nature-based solutions proposed in these approaches can coincide with TW classifications: bioretention systems (Liu *et al.*, 2014) can be considered an application of VF wetlands, while the wetlands reported in SuDS and LID manuals are what the TW experts call FWS wetlands. Following this analogy, it is interesting to observe how the “Treatment Chain” concept proposed by the SuDS Manual (Woods-Ballard *et al.*, 2015) is analogous with the concept of multi-stage wetlands, i.e., different stages in series, each one promoting different functions. An example is the use of bioretention systems for urban water quality improvement followed by a floodable wetland as detention basin, which is conceptually the same scheme (VF + FWS) proposed by Masi *et al.* (2017a) for CSO treatment. Therefore, a deep knowledge of the two “worlds”, i.e., SuDS (or WSUD, sponge city, etc.) and TWs, is mandatory for a successful integration of multi-stage systems in green–blue infrastructures.