How to treat raw sewage with constructed wetlands: an overview of the French systems

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Abstract The development of vertical flow constructed wetlands treating raw wastewater in France has proved to be very successful over the last 20 years. In view of this a survey was carried out on more than 80 plants in order to study their performance and correct the design if necessary. This study shows that such systems perform well in terms of respecting the goals of both low level outlet COD and SS and nitrification. Pollutant removal performance in relation to the loads handled and the specific characteristics of the plants were investigated. Nitrification is shown to be the most sensitive process in such systems and performance in relation to sizing is discussed. Such systems, if well designed, can achieve an outlet level of 60 mg L^{-1} in COD, 15 mg L^{-1} in SS and 8 mg L^{-1} in TKN with an area of 2–2.5 $m^2.PE^{-1}$. The sludge deposit on the first stage must be removed after about 10–15 years.

Keywords Vertical flow constructed wetlands; raw sewage; data collection; design; performance

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Introduction

Among the different constructed wetlands systems treating domestic wastewater the two-stage vertical flow constructed wetland (VFCW) is the most common design found in France. The special feature of this system is that it accepts raw sewage directly onto the first stage allowing for easier sludge management in comparison to dealing with primary sludge from an Imhoff settling/digesting tank. The use of this system, developed by Cemagref more than 20 years ago (Liénard, 1987), really took off when applied by the SINT company during the 1990 s. With the passing of time this system has gained a good reputation for small community wastewater treatment. Now it is well developed and several companies offer this process. The sizing of such a system is rather empirical, based on the knowledge gained by Cemagref over years of laboratory studies and have been full-scale experiments on attached growth culture. General guidelines proposed (Boutin *et al.*, 1997; Liénard *et al.*, 1998) to avoid poor conceptual design which could have damaged development of the system. The sizing of the reed bed filters is based on an acceptable organic load expressed as a filter surface unity per Person Equivalent (PE). Current recommendations are 2 stages of filters, the first of which is divided into 3 filters and the second into 2 filters. Filter configuration and media profile can be seen in Figures 1 and 2.

Each primary stage unit receives the full organic load during the feeding phase, which often lasts 3 to 4 days, before being rested for twice this amount of time. These alternating phases of feed and rest are fundamental in controlling the growth of the attached biomass on the filter media, to maintain aerobic conditions within the filter bed and to mineralise the organic deposits resulting from the SS, contained in the raw sewage which are retained on the surface of the primary stage filters (Liénard *et al.*, 1990a, b). Then effluent is sent to the second stage to complete treatment and, in particular, nitrification.

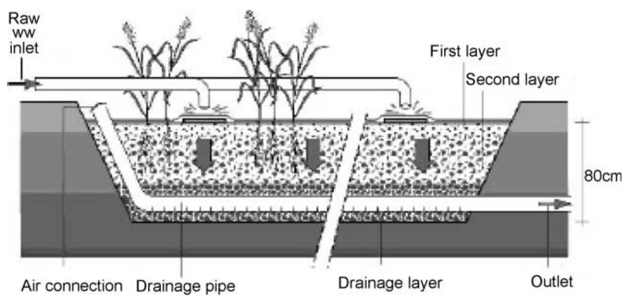

Figure 1 A typical 1st stage RBF

The surface recommended per stage, which could be adapted according to the climate, the level of pollutant removal required by water authorities and the hydraulic load (HL) due to for example, the amount of clean water intrusion into the sewerage network (even though VFCWs have mostly been recommended for separate networks until now), can be expressed as: a total area of $1.2\,m^2$ per PE, divided over 3 identical alternately fed units on the first stage (i.e. an organic load of $\approx 300\,g\ COD\,m^{-2}.d^{-1}$, $\approx\ 150\,g\ SS\ m^{-2}.d^{-1}$, and $\approx\ 25-30\,g\ TKN\ m^{-2}.d^{-1}$ and a HL of $0.37\,m.d^{-1}$ on the filter in operation), and $0.8\,m^2$ per PE divided over 2 identical alternately fed units for the second stage. This design is based on a ratio of $120\,g\ COD.PE^{-1}$, $60\,g\ SS.PE^{-1}$, $10-12\,g$ of $TKN.PE^{-1}$ and $150\ L.PE^{-1}$ as most often observed for small communities in France.

Wastewater is supplied to the filters in hydraulic batches (by a storage and high capacity feeding system) to ensure an optimum distribution of wastewater and SS over the whole available infiltration area and improve oxygen renewal. When the difference in height between the inlet and outlet of the plant is sufficient, the plant operates without an energy source thanks to self-priming siphons. This configuration is known to allow significant removal of COD, TSS and almost complete nitrification (Boutin *et al.*, 1997). In view of the popular success of VFCWs for small communities, and knowing that design recommendations have not yet been fixed, the recently created French Macrophytes Group (2003) initiated an investigation to give an overview of the number of plants, their design, their efficiency and the problems which could occur. The aim was to correct design deviations that might have occurred. This paper relates the overview given by the survey and the design lessons that can be drawn from it.

Materials and methods

Data collection in relation to the national situation was carried out by sending questionnaires to the local technical services for wastewater treatment plants (SATESE), in order to ascertain the different design characteristics and behaviour of VFCW plants in

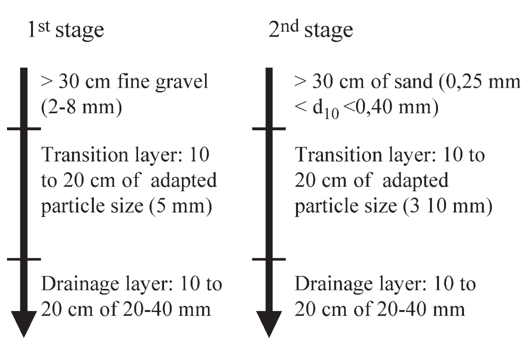

Figure 2 Particle size profiles

operation in France. Using data, a sample of 72 plants was chosen to assess pollutant removal efficiency and increase the database on wetland efficiency. The sample chosen is representative of the national situation. 60% of the plants are 4 to 6 years old, 60% treat only domestic wastewater and the average plant capacity is 410 PE (median 325 PE). We took care to evaluate the situation at different altitudes (between 0 and 1000 m).

The efficiency study was done by a 24-hour flow-composite sampling at different times of the year (summer and winter). As far as possible each stage of the treatment plant was evaluated for COD, BOD, SS, TKN, $N\text{-}NH_4$, TP and $P\text{-}PO_4$ according to French standard methods. Flow was measured by venturi channels or by measuring the functioning time of pumps, if present. Knowing that the percentage of N in the SS of raw sewage is about 3 to 5% and about 0.7% in the sludge deposit (Molle, 2003), the TKN removal observed is assumed to be due to nitrification only. Such an approximate calculation is considered more reliable than those based on nitrate concentration because of the difficulty of assessing nitrogen balance due to nitrate leaching during the rest period. All removal efficiencies are calculated as kg of pollutant removed. Statistical analysis of the data is necessary for comparison of efficiency in relation to design characteristics as a number of sources of uncertainty can affect the quantitative measurement (different operators and methods in some cases). Analyses of variances and mean comparison were performed at $p = 0.05$ by the Fisher F-test and the Student t-test, taking the samples two at a time. The confidence interval (95% of the values) is determined by $\pm 2(SD/\sqrt{N})$, where N is the number of values and SD the standard deviation.

Results and discussion

VFCW situation

Over 400 plants are actually in operation and more than 100 plants were built in the year 2003 (Figure 3). The results are not complete because only 61 out of 95 departments answered the questionnaire. Nevertheless, it indicates that VFCWs have become popular for small communities. The survey revealed 300 plants in France treating wastewater with VFCW (65% $<$ 300 PE) with a design close to the one recommended by Cemagref with some deviations (min – max on the 1st stage: $0.1\text{–}4.7\,m^2.\ PE^{-1}$; 2nd stage: $0.1\text{–}3.6\,m^2.\ PE^{-1}$). Differences in surface sizing result from adaptation to influent characteristics (presence of clear water for example). About 70% of these plants treat wastewater from separate network systems, 10% wastewater from separate networks with clear water intrusion and 20% wastewater from combined sewer systems. Feeding systems mainly use gravity (60% by siphon on the first stage and 75% on the second stage) and thus avoid the necessity of an electrical supply to the plant.

Removal efficiency was studied by 233 assessments on 81 plants (Table 1). We focused our analysis on the vertical + vertical design fed with raw wastewater.

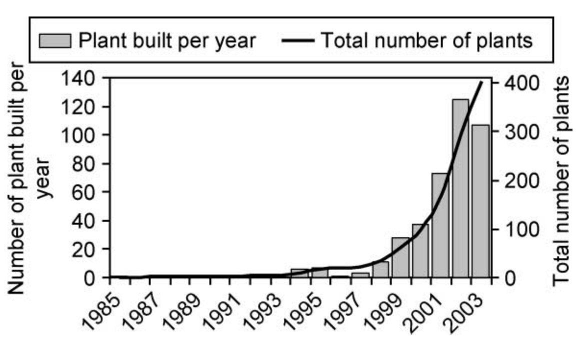

Figure 3 Development of vertical flow CW over time

Table 1 Types of plants evaluated

Type of plant	Number of plant	Assessment number	Plant age (y) at the assessment
V + V	53	134	0–7.0
V + H	2	33	1.2–8.0
V + SF	7	11	0.4–2.0
V	5	5	0.6–4.6
V + P	3	12	0.2–2.5
V + V + H	1	9	Start up
V + V + P	1	6	11.6–15.0
V + H + H	2	3	0.6–2.3
V + H + P	2	3	1.2
V + P + V	1	2	1.6–8.5
V + H + V	1	1	2.6
P + V	1	9	0–1.0
P + V + V	1	3	?
H + V + H	1	2	3.5–4.0

V: vertical; H: horizontal; SF: sand filter; P: pond

This meant that all the first stage vertical filters fed with raw sewage were taken into account in order to focus our analysis on the performance of this first stage whatever the following stages. The performances of second stage vertical flow systems are examined separately.

Global efficiency

For the typical design of two stages of vertical flow filter fed with unsettled wastewater, large variations were observed on the filter in operation at the first stage in hydraulic load ($Mean_{HL}$ = 0.37 m.d^{-1}; SD = 0.38; min-max = 0.03–3.9 m.d^{-1}) and organic load ($Mean_{COD}$ = 223 g.m^{-2}.d^{-1}; SD = 260; min-max = 17–1680 g.m^{-2}.d^{-1}). Consequently, and also because of differences in design, age of the plant etc., removal efficiency varied. If cases of abnormally high hydraulic load resulting in very diluted influent are excluded, the potential of the filters for good pollutant removal can be observed. Table 2 shows the removal efficiency and outlet concentration for plants with hydraulic loads lower than 0.75 m.d^{-1} on the filter in operation on the first stage (2 times the dry weather HL). Globally systems are able to achieve good effluent quality for all but phosphorus removal and denitrification (denitrification is limited due to the enhanced aerobic conditions, and mean P removal is about 40%).

Very often improvement in treatment is mentioned by operators with sludge deposit evolution on the first stage over the first years of operation. This effect is not observed over the two stages of treatment where no significant differences are observed between newer and older plants. The second stage of filters ensures treatment efficiency. Some

Table 2 Removal and outlet pollutant concentration of two-stage VFCWs for hydraulic loads < 0.75 m.d^{-1}

Plant age (y)	COD		SS		TKN	
	% Removal	Outlet concentration mg L^{-1}	% Removal	Outlet concentration mg L^{-1}	% Removal	Outlet concentration mg L^{-1}
2–6						
Mean (N)	**91 ± 3** (48)	**66 ± 13** (49)	**95 ± 2** (49)	**14 ± 5** (49)	**85 ± 5** (49)	**13 ± 5** (49)
SD	10.2	45.5	5	17.5	17.1	17.5
< 2						
Mean (N)	**90 ± 2** (43)	**65 ± 15** (51)	**94 ± 4** (43)	**15 ± 6** (51)	**85 ± 6** (43)	**12 ± 5** (49)
SD	7.1	51	12.2	19.7	18.4	15.7

limitations can be observed for nitrification due to its sensitivity to oxygen presence and competition with COD removal. Several parameters such as flow distribution, batch frequency, type and depth of media etc. can influence oxygen renewal. This probably explains the nitrification variation performance observed. These variations in our samples can be correlated to the surfaces used but not to hydraulic or COD load. For comparable HL ($0.20\,m.d^{-1}$) and inlet concentrations ($TKN_{inlet} = 80 \pm 15\,mg\,L^{-1}$), TKN outlet concentrations differ significantly according to surface per PE. Outlet concentrations clearly show the limitation in using total surfaces of below $2\,m^2.PE^{-1}$ (see Table 3). It does not seem necessary to design plants with an area greater than $2.5\,m^2.PE^{-1}$ for better nitrification but $2\,m^2.PE^{-1}$ is a prerequisite in order to achieve $8\,mg\,TKN.L^{-1}$ ($6\,mg\,N\text{-}NH_4.L^{-1}$).

First stage of treatment

Fourty six assessments were used to evaluate the performance of first stage treatment As plant design, hydraulic and organic load vary, it is not easy to estimate the precise impact of design on removal performances. Nevertheless it can be observed that the first stage of treatment concerns mainly SS and COD removal, though TKN removal is not negligible (see Table 4).

High SS removal performance is obtained in the first stage due mainly to the deposit on the filter surface. This deposit layer is of great importance in limiting the infiltration rate and thereby the hydraulic flow that can pass through the filter. The effect of this restricting factor, which influences the hydraulic load which can be accepted whilst allowing enough surface aeration time, is reduced by reed growth over the year (Molle, 2003). Nevertheless no significant differences in pollutant removal were observed over the year even with hydraulic loads of up to two times the dry weather flow.

Figures 4 and 5 present the removal performances in relation to the organic load (100% removal represented by the dotted line). Even for organic loads greater than those allowed for in the design, COD and SS removal are acceptable. For low hydraulic loads, a greater variation in COD removal is observed (80 ± 6%; N = 15). This can be related to the fact that during poor loading water distribution, and therefore the sludge deposit, is not

Table 3 outlet TKN concentration of two-stage VFCWs according to size

Total surface area	$1.5-2\,m^2.p.e^{-1}$	$2-2.5\,m^2.p.e^{-1}$	$2.5-3\,m^2.p.e^{-1}$
TKN outlet ($mg.L^{-1}$) (N)	16 ± 8 (28)	6 ± 2 (20)	5.6 ± 3 (10)

Table 4 Removal and outlet pollutant concentration of the first stage of VFCW for hydraulic load < $0.6\,m.d^{-1}$

	COD		**SS**		**TKN**	
	% Removal	**Outlet concentration $mg\,L^{-1}$**	**% Removal**	**Outlet concentration $mg\,L^{-1}$**	**% Removal**	**Outlet concentration $mg\,L^{-1}$**
All assessments						
Mean (N)	**79 ± 3** (54)	**131 ± 20** (54)	**86 ± 3** (54)	**33 ± 6** (54)	**58 ± 5** (54)	**31 ± 5** (54)
SD	10	71	12	19	17	17
520 < COD < 1400 (mean 840) $mg\,L^{-1}$						
Mean (N)	**82 ± 3** (34)	**145 ± 24** (34)	**89 ± 3** (34)	**33 ± 7** (34)	**60 ± 6** (34)	**35 ± 7** (34)
SD	7	70	7	19	16	18

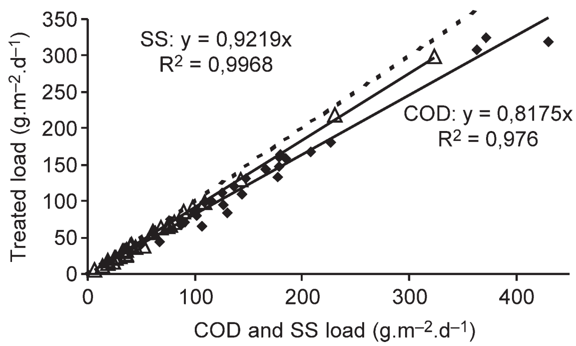

Figure 4 Treated COD and SS for COD concentrations between 520–1400 mg.L^{-1}; 0.15 < HL < 0.6 m.d^{-1}

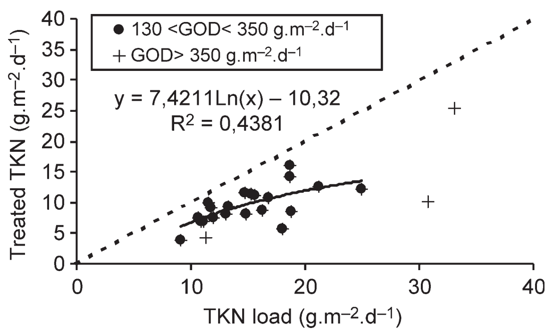

Figure 5 Treated TKN for plants > 1 year, 0.15 < HL < 0.6 m.d^{-1}

homogeneous. Heterogeneity in distribution can lead to some deficiencies in COD removal due to flow short-circuiting. COD removal is sensitive to infiltration rate (Molle, 2003). SS removal appears relatively stable and efficient. This is not the case for nitrification (see Figure 5). Nitrification for nominal TKN loads (25–30 g.m^{-2}.d^{-1}) can be expected to be about 50%. Variations in nitrification efficiency cannot be correlated with plant design, plant age or media depth, for global oxygen demand (GOD = DCO + 4.57 ∗ TKN) of between 40 and 110% of the nominal load, and hydraulic loads of between 40 and 160% of the nominal load. In fact we observed a tendency to improve TKN removal over the year. The winter period, with lower mineralisation of sludge deposit and low temperature, is the worst for biological activity. Nitrification is probably the first to be affected by these limiting conditions. Moreover, the period from January to April is affected by a longer period of accumulation of sludge (from November) which remains wet leading to poor mineralisation. This contributes to a limitation in infiltration rate and oxygen renewal.

Second stage of treatment

As shown in Table 5, the second stage of treatment has mainly a nitrification contribution. Because of low inlet concentrations in COD and SS (about 140 and 40 mg L^{-1}, respectively), this second stage has only a polishing effect on these parameters (figures 6 and 7).

No correlation was observed between removal rate and size of the filter. For COD removal high hydraulic loads seem to decrease efficiency (Figure 6). This was shown by Molle (2003) in laboratory column experiments. COD removal is sensitive to the hydraulic retention time. There was no correlation between outlet COD concentration and hydraulic load. Hydraulic overload decreases COD removal but is compensated for by the dilution effect. Therefore outlet concentration remained low. More information would

Table 5 Removal and outlet pollutant concentration of the second stage of VFCW for hydraulic load < 0.6 m.d^{-1}

	COD		**SS**		**TKN**	
	% Removal	**Outlet concentration (mg L^{-1})**	**% Removal**	**Outlet concentration (mg L^{-1})**	**% Removal**	**Outlet concentration (mg L^{-1})**
All assessments						
Mean (N)	**56 ± 12** (44)	**51 ± 7** (44)	**65 ± 10** (44)	**11 ± 3** (44)	**71 ± 7** (44)	**7 ± 2** (44)
SD	38	23	34	9	23	6
80 < COD < 280 (mean 140) mg L^{-1}						
Mean (N)	**60 ± 8** (28)	**55 ± 8** (29)	**72 ± 7** (28)	**11 ± 4** (29)	**78 ± 7** (28)	**6 ± 2** (29)
SD	21	21	19	9	18	5

be necessary in order to analyse in what way design characteristics contribute to changes in removal efficiency. In our study media depth and characteristics were not always noted or not precise enough to allow observations of media influence on removal levels. The overall flow distribution on the filter surface is of great importance. This information (on flow of feeding systems) would have been invaluable in order to define quality of distribution over the surface. Nevertheless we can observe that globally nitrification has the same rate of efficiency as that observed by Molle (2003) in a study performed under better controlled conditions. Some assessments (9 out of 53) carried out by SATESE

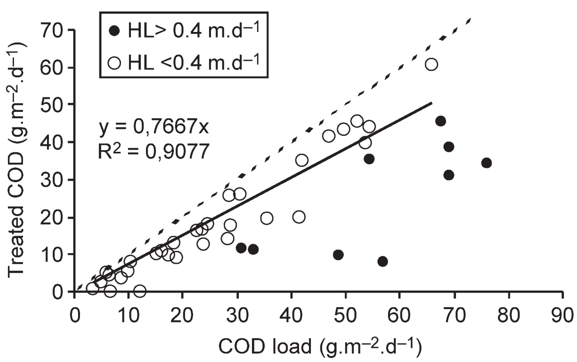

Figure 6 Treated COD on 2nd stage

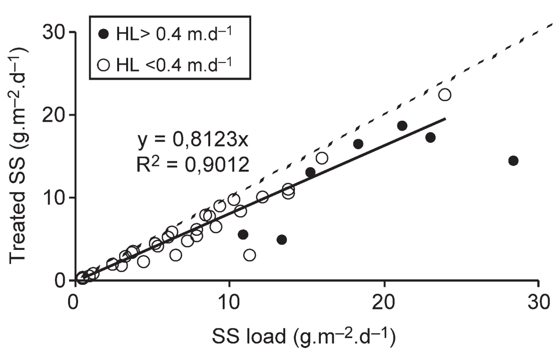

Figure 7 Treated SS on 2nd stage

deviate from this correlation for low TKN loads (Figure 8). No clear nor general reason could be established because of the numerous different conditions that could have affected nitrification rates (low inlet concentration due to diluted effluent, presence of industry, use of natural soil as a medium etc.). Most importantly, it would seem that good flow distribution is essential. More precise studies need to be done to evaluate in what way distribution and batch frequency can modify nitrification rate.

Nevertheless we found that nitrification met the treatment objectives for loads below $15\,g.m^{-2}.d^{-1}$ of TKN on the second stage filter in operation. For loads above this level, a decrease in nitrification rate was observed. This is of no importance in respect to outlet levels if the high load is due to a high hydraulic load of diluted influent because outlet concentration will meet quality objectives. However if it is due to the small surface area of the unit it could be difficult to achieve 90% nitrification over the whole plant.

Sludge accumulation and handling

Up to now, sludge removal has only been carried out on one plant designed for 1600 PE and composed of 8 VFCWs prior to 3 WSPs (dimensioned at $5\,m^2.PE^{-1}$). The plant was put in operation in 1987. Sludge removal was required in 1996, not because of deterioration in effluent quality, but because there was an unequal height of sludge causing distribution problems and little remaining freeboard with risk of spillover in winter. The poor distribution was due to an insufficient flow rate of the pumping station a long way away from the plant and an unsuitable distribution channel. The average sludge height was estimated to be 13 cm (minimum 6 cm, maximum 27 cm). In 1999, after this sludge was removed from 6 filters, the pump and distribution system were changed to give a better distribution of SS over the surface area.

In March 2001 the accumulated sludge on the 2 filters which had not been removed since the beginning of operation (June 1987) had reached approximately 22.5 cm over the entire surface of each filter and the freeboard was not sufficient to guarantee treatment of daily hydraulic peaks. From these measurements, it can be confirmed that in this plant sludge height increases at about 15 mm per year. Several samples of the different layers of sludge were analysed in order to determine their degree of mineralisation (Table 6). Because of hydraulic experiments and wet weather, just before sludge removal, drying conditions were not optimal. Nevertheless, the dry matter content was always greater than 20%, except at the top where the deposits were most recent. Mineralisation which occurs over time induces DM and OM gradients over the sludge height. Analyses confirm a relatively high DM content in relation to the wet conditions prevailing at the time.

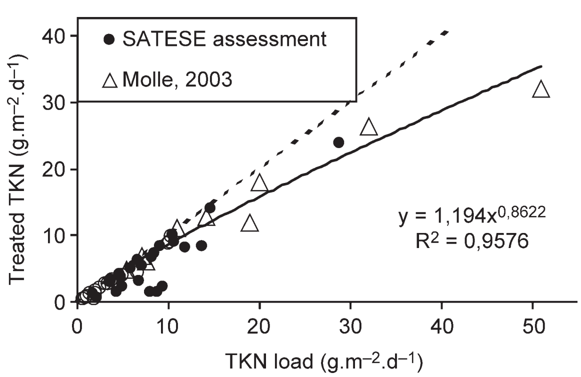

Figure 8 Treated TKN on 2nd stage ($0.05 < HL < 2.2\,m.d^{-1}$)

Table 6 Quality of the sludge on the two filters at Gensac la Pallue in 2001

		Dry Matter ($g.kg^{-1}$)		Organic matter (% of DM)	
Filter 7	Top layer	181.0		61.2	
	Middle layer	205.0	Mean = 261.8	54.9	Mean = 42.96
		214.5		51.5	
		365.9*		22.5	
	Lower layer	291.6		39.8	
	Removed sludge**	284.0		34.3	
Filter 6	Top layer	154.0		54	
	Middle layer	213.2		48.3	
	Lower layer	218.1	Mean = 264.3	45.3	Mean = 41.5
		310.5		37.8	
	Removed sludge**	217.8		49.2	
	Sludge stored since the first withdrawal in 1996	583.0		10.4	

*This large amount can be explained by the location of this sample, at the end of the filter, very little fed before 1999, because of distribution device failure as mentioned previously
**Made up of several mixed sludge samples taken out during the withdrawal from one filter

Probably the mineralisation provided a structure to the sludge which allowed rapid percolation of water and prevented it from staying too wet.

Based on a daily SS load of 16.3 kg and a SS removal rate of 90%, the mass balance of SS input on these 2 filters over 14 years can be calculated to be 75000 kg SS. The evacuated mass (mean height 22.5 cm, DM content 25% and surface area of the 2 filters 520 m^2) is estimated to be 29000 kg SS, which represents almost 39% of the SS introduced with the wastewater. Thus, the mineralisation rate attained was 61% and is similar to a previous estimation of 65% Boutin *et al.* (1997). This aerobic mineralisation, as evidenced by the presence of many *Lumbricus* earthworms, can also be explained by the fact that, once roughly dewatered (i.e. 15% SS content), the SS retained on the deposit surface represents a height not exceeding 1.5 mm per week before any mineralisation process has occurred. Such a thin layer is in direct contact with the atmosphere most of the time. Bacteria in the sludge layer, which are in optimum hygrometry and protected from UV by the shade of the reeds, can easily start their aerobic activity.

This deposit layer becomes part of the biologically efficient media and tends to increase the removal rates of COD, TSS and TKN. Increase in deposit layer does not drastically affect the hydraulic capacity of the filter. In fact, due to the mechanical role of reeds (Molle 2003), it is only the thin layer of newer deposits which is hydraulically limiting. The sludge withdrawal did no affect the regrowth of the reeds from the rhizomes. Metal analysis of sludge (Molle 2003) showed that its use for agricultural purposes is possible as long as no agro-industries have been connected to the sewerage network (for example copper from vineyard treatments).

Conclusion

This study gives an overview of the performance of the many various design and functioning characteristics of VFCW in France. Globally, this system is very appropriate for small communities because treatment is extremely efficient (>90% for COD, 95% for SS and 85% for nitrification) despite variations in organic and hydraulic loads (15% of the assessments showed organic loads higher than the nominal COD load and 25% hydraulic loads higher than the nominal load). The first stage of treatment operates an COD and SS removal while nitrification is variable and about 50% of inlet TKN. The second stage of treatment secures carbon removal (COD and SS) and completes the nitrification. The effect of design on pollutant removal rate (size, material characteristics etc.) cannot be proved statistically. Nevertheless, as it is more sensitive to oxygenation and functioning conditions, nitrification is a suitable parameter for observation of the appropriateness of the plant design and/or functioning in pollutant removal performance. In this study, design data were either not obtained or not precise enough in terms of material depth, material size distribution, siphon volume, pump flow etc. to determine how nitrification could be improved by design or optimal management. Nevertheless we can state that:

- An overall surface for both stages of $2\,m^2.PE^{-1}$ is a prerequisite in order to attain sufficient nitrification. Sizes greater than $2.5\,m^2.PE^{-1}$ do not appear to improve nitrification.

Performance of each stage in relation to organic, and in some cases hydraulic, loads allow the potential of the system to be more clearly defined. For nominal loads we can state that:

- $1.2\,m^2.PE^{-1}$ on the first stage and $0.8\,m^2.PE^{-1}$ on the second stage allow outlet concentrations of $60\,mg\,L^{-1}$ in COD, $15\,mg\,L^{-1}$ in SS and $8\,mg\,L^{-1}$ in TKN to be reached.
- Hydraulic overloads can affect COD removal (observed on the second stage of treatment) but outlet concentration is maintained due to the dilution effect.
- In relation to the removal rate observed for each stage, nitrification could be improved by increasing the first stage sizing to $1.5\,m^2.PE^{-1}$ to obtain an outlet concentration of about $6\,mg\,L^{-1}$. However, this would lead to more wastewater distribution problems. In fact, flow feeding of the first stage is of great importance to ensure an overall distribution of water onto the filter to use the whole reactor.

More studies need to be done to accurately determine the optimal conditions for feeding (flow, volume, frequency) in order to improve nitrification, but in our experience, it seems that a feeding flow of $0.6\,m^3.m^{-2}.h^{-1}$ is the minimum. This flow would ensure a satisfactory distribution for the first feeding after a rest period, when infiltration rates can be greater than $1.4\ 10^{-4} m.s^{-1}$ (Molle, 2003). Such a feeding flow would ensure a good sludge and water distribution on the filter.

The deposit layer on the first stage limits the infiltration rate and improves water distribution. It also supplements the biologically active layer. Mineralisation (60%) leads to an increase in sludge of about 1.5 cm per year which must to be removed once it attains a maximum of 20 cm i.e. about every 10–15 years. Sludge can be used for agricultural purposes as long as no industries are connected on the sewerage network.

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References

Boutin, C., Liénard, A. and Esser, D. (1997). Development of a new generation of reed-bed filters in France: First results. *Wat. Sci. Tech.*, **35**(5), 315–322.

Liénard, A. (1987). Domestic wastewater treatment in tanks with emergent hydrophytes: latest results of a recent plant in France. *Wat. Sci. Tech.*, **19**(12), 373–375.

Liénard, A., Boutin, C. and Esser, D. (1990a). Domestic wastewater treatment with emergent hydrophyte beds in France. In: *Constructed Wetlands in Water Pollution Control (Adv. Wat. Pollut. Control no 11)*, Cooper, P.F. and Findlater, B.C. (eds), Pergamon Press, United Kingdom, pp. 183–192.

Liénard, A., Esser, D., Deguin, A. and Virloget, F. (1990b). Sludge dewatering and drying in reed beds: an interesting solution? General investigation and first trials in France. In: *Constructed Wetlands in Water*

Pollution Control (Adv. Wat. Pollut. Control no 11), Cooper, P.F. and Findlater, B.C. (eds). Pergamon Press, United Kingdom, pp. 257–267.
Liénard, A., Boutin, C. and Esser, D. (1998). France. In: *Constructed wetlands for wastewater treatment in Europe*, in Vymazal, J., Brix, H., Cooper, P.F., Green, M.B. and Haberl, R. (eds). Backhuys Publishers, Leiden, Netherlands, pp. 153–168.
Molle, P. (2003). Subsurface flow constructed wetlands: Phosphorus retention and hydraulic limit of vertical subsurface flow CWs. PhD Thesis, University of Montpellier, France, 267 (In French).