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Long term experiences with dimensioning and operation of vertical flow constructed wetlands in warm climate regions of South America

Short title: Experiences with vertical flow constructed wetland in South America

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Abstract
The paper gives an overview of about 15 years of practical experience with different kinds of vertical flow wetlands in South America. In specific, it shows that the oxygen design approach for VFCW is directly valid for warm climates and the dimensioning of oxygen consumption and hydraulic load (< 200 mm/d) are typically the imitating factors for the design. The organic loading should not exceed 32 g BOD_5/m^2,d. The paper discusses a wide range of influent concentrations for small treatment systems and gives numbers about influent concentrations. Eight full-scale wetland cases are discussed, comprising of 3 greywater systems with reuse, 4 systems for secondary treatment and 1 for raw water treatment inspired by the *French System*. Key aspects involved in the design of the pretreatment systems are discussed, as appropriate design largely influences the performance of the wetland. Finally, the paper alerts to some deviations currently occurring in South America, where some wetlands are substantially sub-dimensioned and posing developmental challenges to this technology.

Keywords: **case study, dimensioning, subtropical climate, operation, vertical flow CW**

Introduction:

In South America, reuse of waste streams is becoming increasingly important and a key driver to inform decisions and investment in wastewater treatment. This is set against a raised awareness of the water stress futures, placing fortuitous significance in the strategic planning of water resources and the public authorities, private companies and other institutions which render water services.

Wetlands have proven itself innumerous times as a suitable technology for decentralized wastewater treatment. Langergraber (2013) gives a broad overview about the possible applications.

Unplanned urban growth in emerging countries renders the use of decentralized, small or individual wastewater treatment systems as crucial enablers to support the development of new residential- and commercial areas in the cities, as well as for single shopping malls, factories, hospitals, etc. Consequently, the specific characteristics of the various effluents can differ significantly from mixed communal wastewater types (von Sperling & Chernicharo, 2005).

Constructed wetlands (CW) represent an ideal solution for many of these situations, especially considering reuse aspects. Unfortunately, the market place has a few dubious or inexperienced suppliers for CWs on offer subscribing projects with far too small area approaches, incorrect hydraulic design, wrong filter material, impossible efficiency promises or inadequate primary treatment. This presents the risk that CWs be discredited in South America, similarly to what happened during the early development of the technology in Europa. Reputational damages were finally overcome by dissemination of credible data and shared experiences within the regulatory environment.

Against this background, purpose of the paper is to share knowledge and practical applications with regard to design and operation of VFCW in warm climates in order to contribute to correct the most common errors which are observed. The paper focuses on vertical flow CW, due to the lower area need in comparison to horizontal flow CW, as this is an important argument especially for use in urban situations. It is based on the more than 15 years of experience gained in construction, operation and monitoring of wastewater treatment in decentralized situations in South America, which includes vertical- and horizontal subsurface flow constructed wetlands.

Materials and Methods: Design approach for VFCW in warm climates

Vertical flow constructed wetlands (VFCW) for secondary treatment uses a sand filter bed. The design approach f**or all case studies in this paper is based on the oxygen demand model developed by Platzer (1999)** and the adaption to subtropical conditions as required for competitive wetland designs (Platzer *et al.*, 2007).

This model is based on the oxygen requirements for aerobic processes (oxidation of COD and nitrification). Partial denitrification decreases the need of oxygen, according to:
Equation 1: Requirement $O_{2\ TOTAL} = O_2$ ***Use for oxidation*** $_{COD+NITRI}$ ***– Recuperation of*** $O_{2\ DENI}$

In case of the VFCW, oxygen transfer is possible by convection (depends on superficial effluent discharge, 4 to 8 times/ day with the given volume) and diffusion (continuously, decreases only when the pumped effluent accumulates on the surface, 4 to 8 times x 10 - 30 min / day:
Equation 2: Supply $O_{2\ TOTAL}$ ***= Entry*** $(O_{2\ CONVECTION} + O_{2\ DIFFUSION})$

For the dimensioning, the offered oxygen supply must be equal or exceeding the oxygen demand:
Equation 3: Supply $O_{2\ TOTAL}$ ***- Requirement*** $O_{2\ TOTAL} \geq 0$

Platzer (1999) reported that a larger surface is needed in colder climates, than would be necessary for the oxygen demand, where the risk of clogging determines the dimensioning. The result was quite different in warmer climate, where Platzer *et al.* (2007) operated a pilot plant (surface 4.44 m², 80 cm sand with specification: d10 = 0,3 mm; d60 = 1,5 mm; U = 4.83; kf = 9 x 10^{-4} m/s) under subtropical conditions of southern Brazil (average annual temperature 20°C, precipitation 1200 mm).

The pilot plant was fed during 8 month intermittently every 6 hours (4 times/ day) with pretreated (septic tank) communal wastewater in two phases with increasing loads:
- 1.^{phase} 28 g BOD_5/m².d; 146 l/m².d (conc.: 387 mg COD/L; 189 mg BOD_5/L; 72 mg TKN/L)
- 2.^{phase} 35 g BOD_5/m².d; 205 l/m².d.(conc.: 357 mg COD/L; 170 mg BOD_5/L; 57 mg TKN/L)

Figure 1 shows the variability in the influent concentration during the first 4 month, in terms of COD, COD, alkalinity and TKN. Figures 2 illustrates the reduction in BOD, ammonia, alkalinity and nitrates over time. COD was removed in both phases by 78-79%. The efficiency of BOD_5 oxidation was higher in the 2.^{phase} (92% in relation to 85% in the first phase). Nitrification ($NTK_{influent}$: NH_4-$N_{effluent}$) also was higher in the 2.^{phase} (89% in relation to 84% in the 1.^{phase}). Certain part of Nitrogen was used for plant grow, especially during the 1.^{phase} (Figure 3) were the total Nitrogen loss ($TKN_{influent}$: $\Sigma\ N_{effluent}$) was higher (37% in relation to 27% in the 2.^{phase}).

A typical characteristic for wastewater, where surface water is used as drinking water source, is low alkalinity (Figure 1) which is reduced by the nitrification process (Figure 2) and as a critical result the pH value lowered during the second phase to pH 4,2. The partial recuperation of alkalinity at the end of the 2.phase can indicate a parallel occurring denitrification processes, or reduced nitrification. However, slightly elevated ammonia concentrations in the effluent (1.phase 7,6 mg NH_4-N/L and 2.phase 5,4 mg NH_4-N/L) may be ascribed to the fact that the oxygen supply was limited.

Based on these results, the relation between oxygen supply and requirement was calculated, using the factors (cited by Platzer 1999)

- Oxidation of BOD_5: 1.2 g O_2 / g BOD_5; Nitrification: 4.3 g O_2 / g NTK
- Convection: 300 mg O_2 per liter of air entering after each L of effluent
- Diffusion: 1 g O_2 / m² .h for 22 hours / day (4 times/day ½ hour for discharge) (Platzer, 1999)

Table 1 compares the resulting oxygen supply for the 1.phase and 2.phase with:

A) Theoretically requirement of oxygen for total oxidation (100%) of BOD_5 and Ammonia;

B) Real oxidation with special consideration of a probable denitrification.

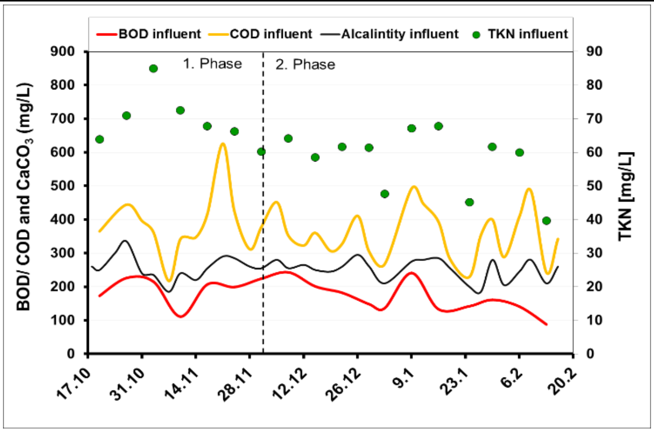

Figure 1: Influent concentration of Pilot plant (Source: Platzer et al., 2007)

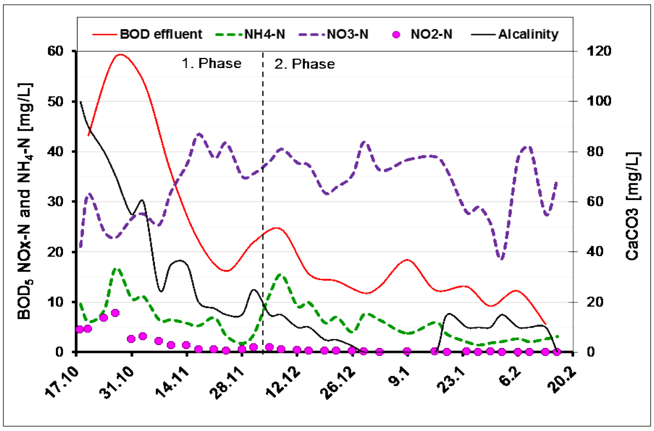

Figure 2: Effluent concentration of Pilot plant (Source: Platzer eta al., 2007)

Figure 3: Plant growth in Pilot plant during firs 4 month in southern Brazil

Calculation	1.phase g O_2/d	2.phase g O_2/d
Supply O_2 CONVECTION	195,0	273,0
Supply O_2 DIFFUSION	97,7	97,7
Supply O_2 TOTAL	**292,7**	**370,7**
Requirement TOTAL O_2 BOD	161,5	199,4
Requirement TOTAL O_2 NITRI	171,3	225,4
A. Requirement O_2 TOTAL	**332,8**	**424,8**
Requirement REAL O_2 BOD	145,2	186,3
Requirement REAL O_2 NITRI	147,0	203,5
Recuperation REAL O_2 DENI	0	19,8
B. Requirement O_2 REAL	**292,2**	**370,0**

Table 1: Oxygen supply and requirement (Source: Platzer et al., 2007)

The results prove that the oxygen demand for the real processes (B) combined perfectly with the oxygen offer calculated by the model. Denitrification, other nitrogen loss or oxygen recovery in the higher loaded 2.phase is possible. The demand for a 100% oxidation (A) would be higher, which means that the normal aerobic processes would be limited by the oxygen offer. Nevertheless, no clogging was observed, even under extended operations of a further 4 month period without systematic sampling.

The authors concluded therefore that the organic load below 32 g/m²,d is not the critical aspect in designing VFCW in warm climates. Sezerino (2006) recommend, for the same climatic conditions as in this study, a full nitrification with maximum loads of 41 g DQO/m².d and 15 g TS/m².d.

Based on these investigations, the presented cases were designed with organic loads between 15 and 32 g BOD_5/m².d; with consideration of a maximum hydraulic (<200 mm/d) and oxygen balance. In contrast to cold climates, the most common cause for clogging of CW is inadequate solid retention in the pre-treatment, caused by inadequate or poorly operated pre-treatment facilities. Other challenges for CW design and operation are also encountered, as described in the following case studies.

Results and Discussion: Findings of full-scale case studies

After a discussion of the special challenges presented for small Wastewater Treatment Plants (WWTP), 8 case studies of full-scale VFCW are presented, operated between 5 to 8 years in coastal areas of Peru and southern Brazil. Most of the analyses and observations resulted from operational field work and did not include for specific scientific research and data collection, only 3 systems were included in temporal studies by universities UFSC (Brazil) and UNALM (Peru).

Wastewater composition in decentralized situations and consequences for CW design

Small or individual WWTP are characterized by a wide range of concentration of wastewater constituents and flow rates. Figure 1 represents typical wastewater concentrations of a residence area, whereas Figures 4 - 7 represent influent concentrations to WWTP for individual commercial institutions in Brazil. Only Figure 7 is an influent to a CW system. VFCW are used for same influent ranges as documented in all figures, but further analysis is hampered by the lack of data.

Wastewater Concentration: The influent in Figure 1, 5 and 6 were analyzed using grab samples at a point after the pre-treatment stage (grease trap and septic tank). The effluent of the shopping mall (Figure 4) is characterized by high- and varying concentrations of BOD, COD, nitrogen and phosphate. In comparison, the effluent without contribution of restaurants (Figure 5) typically has very low organic concentrations, but high ammonia- and elevated phosphate concentration. High concentration of organic matter is mostly caused by fat and organic remains from restaurants, whereas high concentrations of ammonia and phosphate are caused by a relatively high contribution of urine (toilet use). Phosphate is also found in cases where detergents are used.

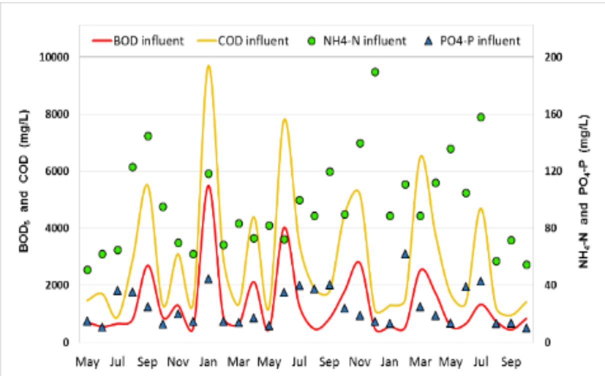	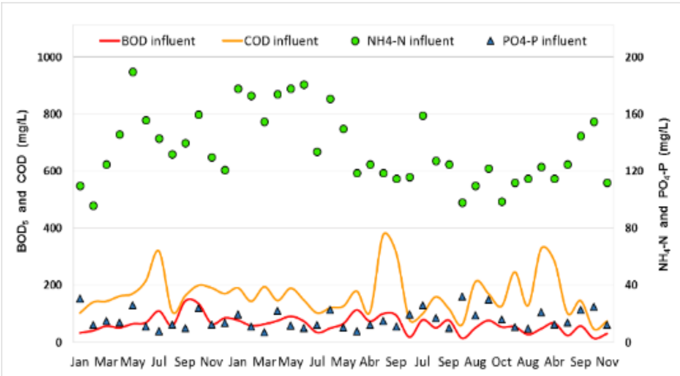
Figure 4: Wastewater composition of a Shopping mall (public toilets and food court)	**Figure 5:** Communal wastewater of a company (bathrooms for employees, without restaurant)

The average concentration between both examples (Figures 4 and 5) differ by factor 1,5 (N; P) to factor 20 (BOD_5/COD):
- Shopping mall: BOD_5 1.250 mg/L; COD 3.000 mg/L, Ammonia 90 mg N /L; Phosphate 24 mg P/L
- Company: BOD_5 65 mg/L; COD 160 mg/L; Ammonia 140 mg N/L; Phosphate 16 mg P/L

Flow rate: In case of the pilot plant (Figure 1), the daily influent was constant and regulated by mechanical pumping. In the full-scale applications, the flow rate differed in residential areas, and were even higher in commercial areas depending on activities during the week (Figures 6, 7). Flow rate also varied depending on holiday periods (factories, campus) or seasons (hotels, tourism areas). Figure 7 demonstrates differences during Christmas time (black line), low commercial season (blue line) and high season (red line). A typical problem are increasing wastewater volumes by new activities as shown for the supermarket in Figure 6 and the commercial center in Figure 7, where the new restaurants not only increased the flow rate but also the organic load during working days up to 400%. The results for the existent VFCW are presented in case study Ww2.

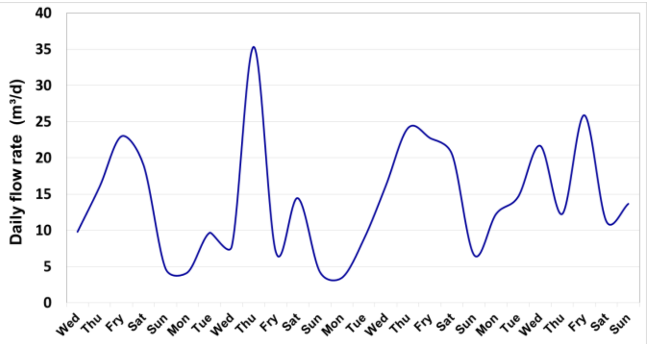

Figure 6: Flow rate of a supermarket designed for 8 m³/day, increasing water use because of later decision for own bakery and fast food restaurants

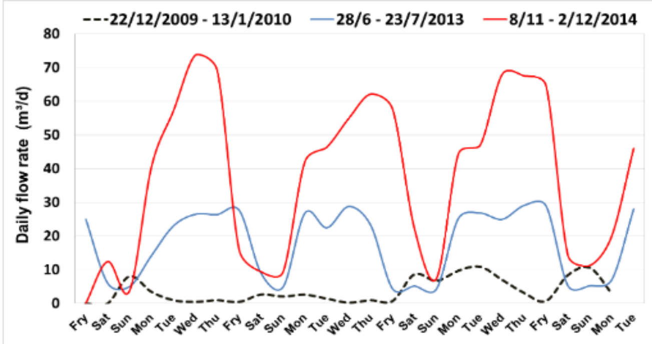

Figure 7: Flow rate of an commercial center, designed for 20 m³/d, increasing water use because of installation of a lot of restaurants

Discussion points for CW design

As the characteristics of CW equilibrate peak flows and loads, VFCW are seen to offer a very stable technology in all cases compared to other technologies. The case studies below show that substantial variations in inflow qualities and loading did not have a substantial impact on the effluent quality:

- If shower or laundry causes a high flow rate, there is no influence on organic load. Different to other technologies hydraulic loads do not reduce efficiency of the treatment, in the case of VFCW hydraulic design loads of up to 200 L/m² are applicable, by that being able to work with significant hydraulic peaks (much higher than common dimensioning) (Hoffmann *et al.* 2011).
- Treatment of effluents with high ammonia concentrations (Figure 5) and low alkalinity (surface water as drinking water sources) in VFCW have the risk of a pH reduction, as nitrification is naturally occurs in aerobic treatments in warm climates. In contrast to other technologies, low pH is not critical for CW operation (see experiences Figure 2). When regulatory requirements limit the pH values (CONAMA, 2011) and presence of ammonium in effluent is permissible, or even desirable for irrigation reuse, the nitrification in VFCW can be suppressed by higher water levels.
- The most critical aspect that impact on CW performance, is high organic loads (Figure 4). This is mostly found where restaurants or food processing activities contribute significantly to the wastewater character and quality. Separated greywater from kitchen sinks (dark greywater) also has high concentration of organic matter. In all these cases, appropriate pre-treatment with adequate sludge retention is indispensable for a sustainable operation of CWs.

Experiences with greywater treatment in VFCW

Separation of greywater and treatment in CW is considered as an economic and safe approach for effluent reuse, especially in regions where water is scarce. All VFCWs in the presented 3 case studies (Greywater; Gw 1-3) were implanted in desert situations of Lima/Peru exclusively for reuse purposes:

Gw1: Private School, 70 PE (Lima, Peru 2008)
Project: Treatment of 1,5 m³/d greywater from bakery and school kitchen in 1m³ settling tank, level regulated pump, 16 m² VFCW (32 g BOD_5/m².d). Filter material comprised of a very fine sand. The treated effluent is used directly for irrigation of the school court.
Current: Organic load on working days is up to 25% higher, because of higher BOD_5 concentration and sludge loss from the settling tank (manual operation). But no clogging is observed. Drying on weekends and holidays might help to avoid clogging.

Figure 8: Photo case study Gw1

Gw2: Guest House, 160 apartments (Lima, Peru 2011)
Project: Treatment in 2 separate 50 m² VFCWs each for 6 m³/d greywater (32g BOD_5/m².d) from restaurant and showers; pre-treatment (each) by grease trap, settling tanks and tank with level regulated pump.
Current: organic load is 25 g BOD_5/m².d (only 60% of flow rate but 25% higher BOD_5 concentration), excellent effluent quality (table 2). Reuse after disinfection for toilet flush in apartments and irrigation of the park.

Figure 9: Photo case study Gw2

Table 2: Case study Gw2 results of 6 monthly samples by UNALM (unpublished) for one VFCW

	BOD_5 mg/L	COD mg/L	TSS mg/L	Turbidity NTU	$Coliforms_{Total}$ NMP/100 ml	$Colif._{Thermotolerant}$ NMP/100 ml
Raw greywater	390	550	132	163	9.9×10^5	3.9×10^5
Influent VFCW	381	516	103	144	3.9×10^5	1.8×10^5
Effluent VFCW	10	25	2,6	3,5	2.5×10^3	4.3×10^2

Gw3: Housing project (Lima, Peru 2011)
Social project, first stage for 288 families with green area of 4.350 m², which needs 9 m³/d water for irrigation. Greywater collected from showers of 108 departments (very low water consumption in this social class).
Treatment system: Sedimentation tank with screen, 60 m² VFCW divided in 3 parts, two parts are charged 6 times a day (by pump and timer), the third stays dry for one week drying (rotating system), total hydraulic load is 150 L/m²,d.

Figure 10: Photo case study Gw3

Discussion about CW design for greywater treatment (Gw1-3)
VFCW systems are able to treat greywater with high efficiency. Unfortunately, sampling and analysis are not adequately performed, but the visual appearance of the treated greywater was excellent. No refusal of the effluent for reuse application was noted between the different clients. Experiences are:

- Volume of greywater often is over-estimated but concentration of organic matter is underrated, especially regarding kitchen effluents, but an insufficient number of samples were available to prove the finding on a scientific basis. The results (table 2) for mixed greywater show the same range of concentrations as for domestic wastewater. In case study Gw 1 (only kitchen, bakery) a single sample resulted in 1.200 mg DBO_5/L. Hernández Leal et al (2007) found much higher organic concentrations and lower volumes in greywater as usually reported as well. Therefore, the authors found it imperative to emphasize the need of being careful in dimensioning.
- Greywater solids (sand, soap, fat and food) have to be removed by a settling tank. Grease traps do not serve for settling and beside this, the grease traps often fail in practice, due to: a) insufficient retention in peak time, b) hot flushing water from kitchen use, or c) lack of operation - grease and scum are not removed with adequate frequency.
- Temporarily high organic loading (> 32 g BOD_5/m^2,d) cause a need for resting phases of the surface area to prevent clogging (weekend, holidays, or, division in sectors for resting).
- The appearance of the treated greywater was odorless, totally transparent. Temporary turbidity was caused once by a dammed up wetland (case study Gw3) and in case study Gw2 by exaggeration of the use of detergents in the first month of operation. That problem was overcome with time (and probably bacteria adaption). Use of biodegradable detergents is recommendable but not always possible.
- Presence of Coliform bacteria (table 1) cannot be excluded for greywater and even if VFCW are able to reduce bacteria efficiently, the safe reuse requires additional disinfection.

Experiences with VFCW for wastewater treatment and special consideration of pre-treatment

In cases of small communal wastewater treatment systems, CW usually compete with other technologies which require a lesser footprint. Therefore, CW offers are often in a conflict between dimensioning to the extreme smallest area use possible or a secure effluent quality. Many offers come to impossible design approaches, the authors have the knowledge of wetlands in southern Brazil (some up to several 1.000 pe.) designed with less than 0,2 m^2/pe.

Under these competitive conditions, adapted pre-treatment technologies are essential for an economic CW design. Four cases are discussed (wastewater Ww 1-4) four VFCW implanted in Southern Brazil. Case studies Ww1/2 are designed with a septic tank as pre-treatment and Ww3/4 have an ABR (Anaerobic Baffled Reactor) as described by Sasse, (1998) with 3 compartments.

Figure 11: Photo case study Ww1

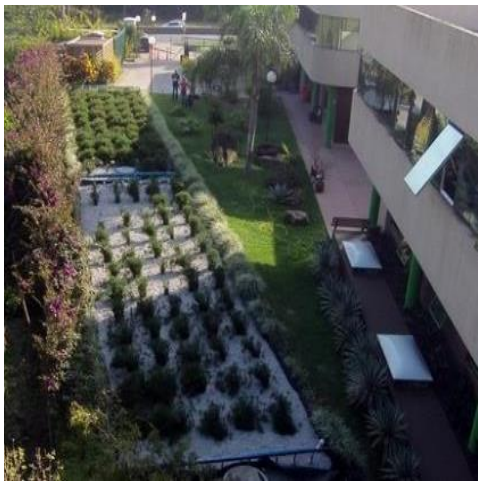

Figure 12: Photo case study Ww2

Ww1: Company 300 employees, only bathrooms without restaurant (2009)
Project: use of existent septic tanks, VFCW - 160 m² (30 g BOD_5/m².d), post-precipitation ($FeCl_3$) and disinfection (Chlorine) in two separate tanks for effluent of the VFCW.
Until now (2016) there are only 150 employees, the organic load is 5 g BOD_5/m².d but the nitrogen load already came to 60% of design. Effluent quality, average: 12 mg BOD_5/L; 50 mg COD/L; 20 mg NH_4-N/L (not required by law); 3,6 mg P/L (< 4 mg/L is required by law).

Ww2: Commercial center designed for 20m³/day (2009)
Flow rate development shown in Figure 7, wastewater composition similar to Figure 4;
Project: Septic tank, 160 m² VFCW (32 g BOD_5/m².d), disinfection and reuse for irrigation.
In 2014, the VFSW load on working days was up to 50 g BOD_5/m².d; even weekly grease and sludge collection could no longer prevent clogging. As VFCW expansion was not possible, the pre-treatment was upgraded into SBR (for 35 m³/d) and the VFCW into an aerated wetland as parallel treatment to absorb the high effluent peaks especially during decantation period of the SBR.

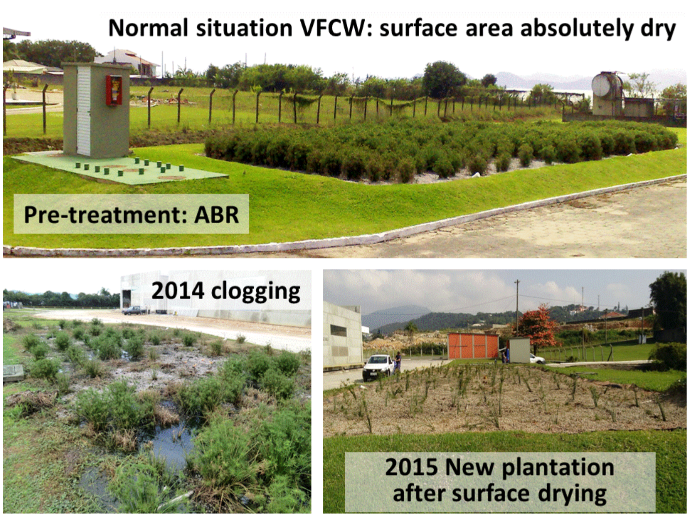

Figure 13: Photo case study Ww3, VFCW 2010, 2014 clogging and

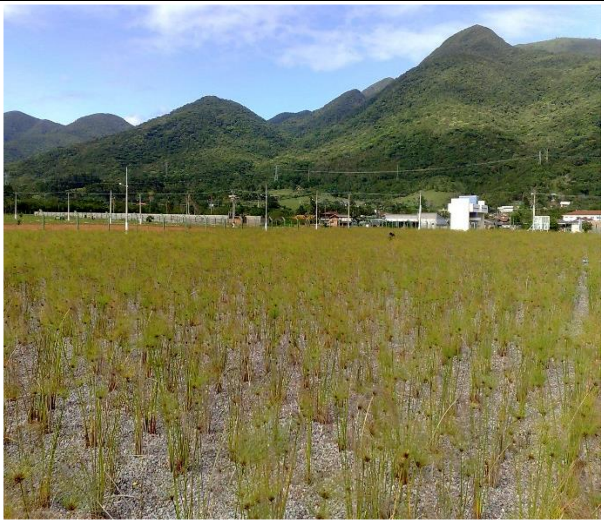

Figure 14: Photo case study Ww4 (6 month after implantation)

Ww3: Company 200 employees, bathrooms and restaurant (2009)
Project: 1st treatment ABR, 2nd treatment 190 m² VFCW (15 g BOD_5/m².d), final disinfection.
Between 2013-2014, Trein *et al.* (2015) documented for ABR a 16% (BOD_5) efficiency due to sludge overflow until the last compartment. As result, the load of VFCW increased temporarily to > 30 g BOD_5/m².d (table 3: 22 month average 22 g BOD_5/m².d; 6,3 g TSS/ m².d). Additional critical points were pumping frequencies of 18 times/ day due to rain water influence in the sewer and superficial overflow from clogged VFCW areas to the area in a resting period. Clogging was controlled in 2015 by frequent sludge collection of ABR and improvement of resting of the 4 VFCW areas.

Table 3: Case study Ww3; results of monthly samples (22) 2013-14 (source: Trein *et al.*, 2015)

	BOD_5 mg/L	COD mg/L	TSS mg/L	NH_4-N mg/L	NO_3-N mg/L	P mg/L	*E.coli* NMP/100 ml
Influent ABR	390 +/-190	724 +/-395	134 +/-126	103 +/-37	-	27 +/-11	1.0×10^8
Effluent ABR	329 +/-134	603 +/-108	109 +/- 75	98 +/-35	-	-	1.4×10^8
Effluent VFCW	48	179	22	54	20	10	1.4×10^7

Ww4: Residence park close to a beach, designed for 2.200 habitants (2009)
Project: 1st treatment ABR, 2nd treatment 3.140 m² VFCW (15 g BOD_5/m².d), disinfection with chlorine. Operation of own WWTP was the condition for commercialization of the housing area.
In 2014, Trein *et al*. registered only 100 habitants (5%) and relatively low concentrated wastewater because (table 4) of relatively high water consumption. The efficiency of the system is excellent; ABR arrives 47% BOD_5 and 50% TSS reduction without emptying during 5 years; in VFCW has complete removal of organic load and nutrients. The most critical point for operation was the maintenance of *Cyperus papyrus* vegetation because of low total hydraulic load.

Table 4: Case study Ww4; results of monthly samples (20) 2013-14 (source: Trein et al., 2015)

	BOD_5 mg/L	COD mg/L	TSS mg/L	NH_4-N mg/L	NO_3-N mg/L	P mg/L	*E.coli* NMP/100 ml
Influent ABR	158 +/- 48	276 +/-138	54 +/-51	51 +/-18	-	15 +/- 8	3.3×10^7
Effluent ABR	83 +/- 28	157 +/- 60	27 +/-18	43 +/-18	-	-	2.4×10^6
Effluent VFCW	5	18	3	3	11	1	2.2×10^4

Discussion about CW design for communal wastewater treatment (Ww1-4)
The different case studies, which part in a similar dimensioning approach as mostly no specific numbers are available beforehand, provide some observations regarding the design of VFCWs:

- The ABR (Anaerobic Baffled Reactor) is a more efficient treatment before CW as a septic tank because its special hydraulic layout improves solid retention and anaerobic degradation. In the presented case, the ABRs were designed for a 40 to 50% organic load retention with hydraulic retention time about 1 day and division only into 3 compartments. The experiences shows that ABR have more capacity to resist high hydraulic peak loads than a Septic Tank, but it is necessary to adopt the design to the real peak loads (in special consideration of pump operation) as excessive upstream velocities could lead to a washout of sludge.
- ABR and Septic tanks need both a regular sludge collection. Depending on the specific wastewater characteristics, sludge has to be removed before if overflows to the last compartment. This happened in case study Ww2 (additionally provoked by hydraulic peak loads due to rain

water influence). In extreme cases, were grease scum comes into the pre-treatment facility, monthly emptying service can be required to prevent clogging of the CW.

- Only in special cases, the hydraulic design load of CW should exceed 200 mm/d, this ensures enough treatment capacity even in extreme hydraulic situations. Main problem of very frequent loadings (case study Ww2) is that the surface does not dry out, therefore the oxygen introduction is reduced (Platzer, 1999). Very high hydraulic loading needs higher storage volume or a division in different loading areas (rotating daily or between loads); minimum interval of 3 hours between two loadings never should be reduced.
- As already pointed out for greywater, temporarily high organic loading (> 32 g $BOD_5/m^2.d$) need adequate resting phases to prevent clogging. The plants in the case studies are divided into 2 or 4 sections and operated in a monthly rotating system. The possibility of overflow between the sectors has to be prevented; but it is not necessary to separate the beds down to the bottom.
- The case studies prove that well designed and -operated VFCWs even in critical situations exceed legal requirements on wastewater treatment; in Brazil (CONAMA, 2011) 120 mg BOD_5/L and in Peru (MINAM; 2010) 100 mg BOD_5/L.
- Even if law might not require Nitrification or Nitrogen reduction, it is an important indicator for the treatment efficiency in VFCWs, as the non-complete nitrification might hint to an oxygen limitation. The high loaded VFCW (table 3) for instance has only 48% Ammonia reduction and 28% reduction of $N_{total,}$ whereas the low loaded VFCW (table 4) has 97% Ammonia reduction and 73% reduction of N_{total}. Investigations about the involved processes would be interesting, which includes analyses of TKN concentration for correct N balance.
- Disinfection of treated wastewater is no longer required by national law in Brazil (CONAMA, 2011), but often is required by local authorities and it is important for reuse options, which do not exclude human contact (as toilet flush or irrigation of food plants or play grounds). The most common disinfection in developing countries is still chlorine application. Well designed- and operated CWs offer good conditions for this disinfection without the danger of by-products.

<u>Experiences with raw wastewater application in wetlands (French system)</u>

In search for a more efficient and sustainable pre-treatment for VFCW, the first "*French System*" in South America with a gravel bed for raw water treatment has been constructed in Chincha/Peru (2011). The design used basic design data from Molle *et al.*, 2005 but adopted the design for local conditions (annual temperature 20°C, annual precipitation 5-20 mm). The plant is designed for a retirement home with 60 persons (7,5 m^3/d) with the purpose of reuse of treated wastewater in the garden.

Ww5: *French System* (2011)

1st stage: different to Molle *et al.* (2005) divided only into 2 loading areas (3 day alimentation with 2 separate grinder pumps), hydraulic load for total area: 125 $l/m^2.d$ (1 m^2/pe). Gravel material also is different, from surface to bottom:

- 60 cm gravel with 6.4 mm (¼ ") diameter
- 25 cm gravel with 12.7 mm (½") diameter
- 15 cm drainage layer 15 cm gravel 19.1 mm (¾ ").

2nd stage: VFCW (1 m^2/pe), sand d10 = 0,33 mm; d60 = 0,63 mm; U= 1,9), dimensioned as a "normal" secondary treatment, due to a lack of experience with efficiency of the gravel filter in warm climate and because of reuse intention.

Table 5: Case study Ww5; results of 3 to 5 samples in 2nd. year of operation (Hoffmann et al., 2015)

	BOD_5 mg/L	COD mg/L	Turbid. NTU	NH_4-N mg/L	NO_3-N mg/L	*Giardia* 1L	*Entamebia* 1L	*Helminten* (N° *ova*)/L	*Coliformes* 100 ml
Raw Ww.	656	1960	470,0	23,0	-	75	30	< 1	4,3 x 10^7
1st stage effl.	23	74	7,5	5,7	10,6	10	0	30	5,1 x 10^6
2nd stage effl	4	16	1,3	< 0,2	8,4	0	0	< 1	9,4 x 10^3

Discussion about French system for wastewater treatment in warm climates (Ww5)

The use of the gravel filter for primary treatment of raw wastewater has been a new experience and was very convincing. In comparison with common pre-treatment technologies (Septic Tank, ABR, Imhoff Tank, UASB), the gravel filter is very efficient and reliable without any significant operational requirements as the interchange between the two beds was automatized. As primary treatment before a secondary CW it presents an interesting option, especially in areas with no- or unreliable fecal sludge removal. Some findings include the following:

- The first stage performed an unexpectedly high organic removal (96% BOD_5 and COD) and also a nitrification (Table 5). Therefore the first stage could be sufficient for many purposes and restrictions in warm climates. The visual appearance of effluent showed a constant reduction of turbidity within the first 6 month of operation.
- During 2nd year of operation WHO (2006) guidelines for effluent reuse could not be fulfilled just by the first stage (gravel filter), but were attained by using both stages (gravel and sand filter). This is important as disinfection (Chlorine application or UV radiation) inactivates *Coliform bacteria* but not safely *Helminth ova*, *Entamebia*, *Giardia* or *Cryptosporidium*. The last one was analyzed with negative result in all cases but it could be present, as other pathogens as well, in case of infected persons.
- Despite higher hydraulic load as reported by Molle *et al.* (2005) under given climate conditions no buildup of a compost filter (mineralized fecal sludge) at the surface of the gravel filter could be observed, even after 5 years of operation. Molle *et al.* (2005) report a buildup of 1-1,5 cm/ year.
- The plants used on the gravel filter are Vetiver (*Vetiveria zizanioides*) and Papyrus umbrella *(Cyperus alternifolius)*. After 2 years of operation the Vertiver has been overgrown by Papyrus.

In the next years, more publications on the subject can be expected as the Agricultural University of Lima (UNALM) has commissioned a test plant with the same dimensions.

Conclusions:

The case studies demonstrate a high potential for application of vertical flow constructed wetlands for decentralized wastewater treatment in warm climates regions. It is crucial that the design of the entire system is adapted to local conditions and especially to specific wastewater characteristics and for verified dynamics.

The authors promote the following parameters as minimum requirements for design of the VFCW beds: a positive oxygen balance, a hydraulic load below 200 mm/d and a maximum organic load of 32 g BOD_5/ m^2.d.

Additional critical points for sustainable projects are: a) Adaption of the treatment components of the whole system on the specific requirements regarding the effluent quality; b) Choice and design of the pre-treatment technology adapted to the specific wastewater characteristics; c) Selection of correct filter material, which includes, up to a certain level, the foresighted adaption of design to the locally available filter materials; d) consideration of the hydraulic of the whole system; e) anticipation of resting areas and adapted resting schedules for organic and/or hydraulic overloads.

In warm climates, primary treatment processes (especially anaerobic treatment) can be highly efficient (von Sperling & Chernicharo, 2005) and in the experience of the authors, the use of the given potential plays an important role in the economic design and sustainable operation of CWs. Under certain conditions, the ABR is an interesting alternative to a septic tank system, because it is more efficient and compact. But the highest efficiency by lowest operational requirements is seen in the first stage of French system (planted gravel bed).

Special consideration of the role of plants was not the focus of this paper, but it needs mention that clients always prefer Papyrus species, even if not native in South America. *Cyperus papyrus* is decorative but difficult to control, *Cyperus alternifolius* is resistant to wastewater but not so common and *Cyperus haspan* is less resistant against dryness and clogging. Vetiver *(Vetiveria zizanioides)* is often used in warm climates for erosion protection, it is not a typical wetland plant, but it seems to be highly resistant to wastewater application.

Certain kinds of reuse require final disinfection of CW effluent before reuse as a resource. The effluent of VFCW can be treated with a limited use of chlorine and without danger of by-products. Yellowish color of CW effluents can provoke rejection of certain types of reuse (toilet flushing), but it is only due to esthetic reasons. The color, caused by humid acids, sometimes can be observed in case of domestic wastewater treatment in CW; it occurs less in case of greywater treatment.

It is the intention of the authors to motivate more scientific analysis of full scale constructed wetlands in order to disseminate solid experiences and therefore further explore the potential of CWs in warm climates. The uptake of simple and sustainable systems which can deliver a high quality of treated effluent for reuse, are worth further research and vital to inform decision making and investment.

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