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Methodology for an ecological solution of subsurface flow constructed wetlands used in the treatment of greywater

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ABSTRACT

Hammams or public baths continue to consume substantial quantities of drinking water up to 120 m^3/day and discharge equivalent quantities of greywater. These hammams thus become an important source of this greywater, which can be easily treated using constructed wetlands (CWs). In this context, the present study proposes to practitioners a general method for sizing subsurface flow (SSF) CWs for the treatment of greywater discharged from hammams. It is oriented to simple applications such as irrigation, car washing and toilet flushing. Due to the complexity of quantifying the evapotranspiration (*ET*) of the treated water at the CWs, a practical and flexible method is presented here to calculate *ET*. In the end, a case study of a Moroccan hammam has been treated and discussed. It provides the designers of SSF CWs with a concrete example of the application of the proposed methodology.

Key words: evapotranspiration, greywater, methodology, public bath, subsurface flow constructed wetlands, water treatment

HIGHLIGHTS

- Design method for constructed wetlands with the subsurface flow for greywater treatment.
- Method for calculating water evapotranspiration in constructed wetlands with the subsurface flow.
- Practical reference for designers of constructed wetlands with the subsurface flow.
- Contribution to facilitate the implementation of eco-systems for water reuse for non-potable purposes, thus reducing the inappropriate demand for treated drinking water.

INTRODUCTION

Hammams or public baths are essential sanitary establishments in the social life of North African and Middle Eastern countries. In Morocco, hammams are part of the cultural heritage, with a number of units of about 12,000 distributed throughout the country (AMEE 2019). They consume enormous quantities of water and obviously discharge equivalent quantities of wastewater, such that the average consumption varies from 60 to 120 m^3/day. Moreover, the water used in these hammams can be drinking water or well water (Hajji *et al.* 2008; Chakri *et al.* 2019). On the other hand, about 844 million people still need a basic supply of drinking water and about 159 million others use it directly from surface water resources (Bajpai *et al.* 2019). This dangerous situation, therefore, requires a practical, economical and ecological solution for the reuse of wastewater discharged from hammams, especially since this water is grey and rich in biodegradable organic matter (Chakri *et al.* 2019), which reflects a low degree of pollution compared to black water.

Different technologies exist in the field of decentralized greywater treatment. Membrane systems such as the membrane bioreactor (MBR) represent a small footprint solution that is characterized by a high treatment load and very good effluent quality, but it is limited by high investment, maintenance and energy consumption costs. Another technology close to the previous one is the biomass concentrator reactor (BCR), which uses high porosity filters, consumes less energy and produces better-treated effluents than the MBR. However, this technology is in development for commercial use (Capodaglio *et al.* 2017; Bajpai *et al.* 2019). An improvement in membrane technology is the use of hybrid systems that combine MBR and BCR to control filter fouling by introducing electrical fields (Bajpai *et al.* 2019). Small-scale wastewater treatment plants (WWTPs) are also adopted in greywater treatment using anoxic or aerated tanks with some sedimentation, which makes them quite sensitive to transient hydraulic loads and changes in sludge quality. Although they give a moderate effluent quality, anaerobic treatment systems (UASBs) provide a biological treatment of greywater that allows the production of methane and biomass, as they are usually followed by another treatment system in order to obtain the desired effluent quality (Capodaglio *et al.* 2017).

In general, hammam owners cannot implement decentralized treatment systems such as the MBR because they are too expensive and consume a lot of electricity. Compared to conventional wastewater treatment technologies, constructed wetlands (CWs) are inexpensive, robust, simple to use and can be constructed from locally available materials (Wallace & Knight 2006). Therefore, an efficient decentralized treatment can be ensured using CWs to obtain effluents that can be used in the irrigation of certain types of crops, toilet flushing, fire protection and other applications (Bajpai *et al.* 2019).

Subsurface flow CWs (SSF CWs) are effective for total suspended solids (TSS) removal and biological oxygen demand (BOD) due to the relatively low flow velocities and high fluid surface area (US EPA 2000), which coincides with the nature of greywater discharged from hammams. Vertical flow CWs (VF CWs) are much more aerobic than horizontal flow CWs (HF CWs) and offer suitable conditions for nitrification. On the other hand, VF CWs do not provide any denitrification (Vymazal 2005). Compared to HF CWs, VF CWs require less land, typically 1–3 m^2/PE, while HF CWs require 5 m^2/PE (Vymazal 2010). In general, VF CWs are expected to perform better than HF CWs for BOD reduction because VF CWs are intermittently loaded and have an unsaturated flow resulting in higher oxygen transfer to the filter media compared to HF CWs (Zhang *et al.* 2014).

Several models exist in the literature for the dimensioning of CWs, which are often considered black boxes and therefore difficult to model their behaviors (Rousseau *et al.* 2004). The most common design approaches are ground rules, regression equations, influent loading graphs, *k-C** model, *P-k-C** model and oxygen demand (Platzer 1999; Rousseau *et al.* 2004; Wallace & Knight 2006; Dotro *et al.* 2017). Sometimes a combination of two or three approaches can be adopted, as in this work, where the influent loading graphs are combined with the *P-k-C** model to calculate the required surface of the HF CW. Thus, they were also combined with the oxygen demand approach to determine the required area of the VF CW. However, Kadlec (2000) has shown that the *k-C** model is inadequate for the design of CWs, resulting in reduced use by engineers. The basic rules and regression equations are not used here, as they were developed under specific conditions such as site climate, pretreatment technology, wastewater type and other factors (Dotro *et al.* 2017).

There is a lack of practical studies that provide engineers and practitioners with a methodology for designing SSF CWs. This implies a disruption in the design methods, as each one has been developed based on local parameters such as the nature of the greywater source and the climate where the system will be implemented. This will lead to a big difference in the results obtained from local conditions compared to others. The lack of this type of study ultimately results in poorly designed systems that do not meet expectations and have poor treatment performance. Hence the novelty of the present study, which sets up a practical and flexible methodology that can be standardized in any climate and location, regardless of the nature of the greywater source.

This work proposes a practical method for sizing VF and HF CWs using a hybrid system for the treatment of greywater discharged from hammams (public baths), including suggestions and feedback from experts in the field. The quantification of water evapotranspiration (ET) represents an essential step in the design of CWs in SSF, but it has become ignored by most designers due to the complexity of establishing it theoretically and especially in the cases of new installations where it is impossible to measure it experimentally according to the real data that characterize the implemented system. This has a negative impact on the calculation of the necessary surfaces of the SSF CWs, which comes mainly from the water balance that does not take into account the ET. For these reasons, the quantification of ET will not be ignored here, as a semi-empirical method will be presented to quantify the water ET through SSF CWs. A calibration will thus be ensured at the water balance level which leads to a reliable and close-to-reality design. The main objective of this paper is:

- To provide a practical and flexible methodology for designers of SSF CWs intended for the treatment of greywater, especially those discharged by hammams.
- To solve the ET quantification problem for SSF CW design.
- To provide designers with a concrete case study using this methodology.

METHODOLOGY AND MODELING

To generate the present methodology, a revision of previous works was elaborated taking into account those characterized by the practical theme of designing SSF CWs intended for greywater treatment (Brix & Arias 2005; Kadlec & Wallace 2009; Dotro *et al.* 2017). Deficiencies have been detected in the integrity and flexibility of the proposed methodologies, such as they are sometimes limited only to the phase of establishing the required surfaces of the CWs without taking into account the ET, while others are based on specific conditions of the climate and the source system of the generated methodology that can never be used in other conditions. A new flexible and integrated methodology has been developed by combining and improving the previous ones by collecting relevant recommendations from experts in design practice. The different steps of the proposed methodology will be presented in the following article.

Characterization of the site

The first step in designing CWs is to characterize the site where the treatment system will be installed. For this purpose, it is essential to define the area reserved for the installation in order to compare it with the calculated area. Indeed, the feasibility of the treatment system can be well discussed. The location of the site is also necessary to know the type of climate and the meteorological data. The water losses and gains at the CW level can be quantified through the water balance, as well as the choice of vegetation adapted to the climate of the site.

Characterization of the influencers

The characterization of the influents discharged by hammams represents the basis of all that follows. The objective is to know the nature of these waters as well as the daily water flow. The type of SSF CW (VF or HF or both) most suitable for the measurement results of the physico-chemical and biological characteristics of the wastewater can be chosen.

Several studies have focused on the characterization of greywater, including water discharged from private and public baths (Eriksson *et al.* 2002). In Morocco, a study was carried out on greywater discharged by 15 Hammams in the Dar Bouazza region. Table 1 presents the results of the measurements obtained and the limit values imposed by the water quality standards for some countries.

Table 1 | Characteristics of the influents of 15 hammams in the region of Dar Bouazza in Morocco with the standards intended for irrigation

				Water standards for irrigation						
	Measurement results (Chakri *et al.* 2019)			S.E.E.E (2007)	Jeong *et al.* (2016)					
Parameters	**Min**	**Max**	**Average**	**Morocco**	**US EPA**	**Cyprus**	**Italy**	**France**	**Spain**	**Greece**
T (°C)	27.3	35.5	30	35	–	–	–	–	–	–
Conductivity (μS/cm)	–	–	>2,669	12,000	–	–	3,000	–	–	–
BOD_5 (mg/L)	10	410	167	–	30[a]	30[b]	20	–	–	25[c]
COD (mg/L)	64	836	385.8	–	–		100	60[d]	–	–
TSS (mg/L)	35	724	431	100	30[a]	45	10	15[d]	35[b]	35[c]
TKN (mg/L)	2.8	86.9	16.22	–	–	–	–	–	–	–
Phosphate (mg/L)	–	–	–	–	–	–	5	–	–	–
Sulfate (mg/L)	–	–	–	250	–	–	–	–	–	–

[a]Processed food crops.
[b]Crops for human consumption.
[c]Restricted.
[d]Unrestricted.

In general, the limit value of water intended for irrigation is 30 mg/L concerning the parameters BOD_5, COD and TSS. Most of the water discharged by hammams is grey with traces of black water. The ratios COD/BOD_5 and BOD_5/COD reflect the richness in organic matter as well as the biodegradable nature of hammam wastewater (Chakri *et al.* 2019), but unfortunately, Chakri *et al.* (2019) did not indicate the concentrations of phosphate and sulfate even though they are important parameters in the analysis of greywater quality.

Treatment system configuration

The choice of treatment system configuration using CWs depends primarily on the nature of the water being treated, geographic location, cost, land availability and treatment objective (Horner *et al.* 2011). For hammam discharge, a hybrid system combining VF and HF CWs is the best solution for organic matter and nitrogen removal, which allows a higher treatment effect to be achieved using the advantages of each system individually (Vymazal 2005). Generally, VF CWs are followed by HF CWs (Vymazal 2005; Kadlec & Wallace 2009), the configuration adopted here. Because of the constraint of needing rest periods for the VF CWs, three VF CWs in parallel ensure a continuous operation of the overall system, such that each bed operates for 2 days and rests for 4 days, as well as good system flexibility. Several experiments have shown that the performance of VF CWs is strongly dependent on the pretreatment (Kadlec & Wallace 2009). Before feeding the VF CW, an efficient primary treatment is, therefore, necessary to remove solid particles in order to avoid filter clogging (Dotro *et al.* 2017). In addition, the range of characteristics presented in Table 1 confirms the use of pretreatment. For SSF CWs, the use of lagoons is not the preferred method for primary treatment (US EPA 2000). The pretreatment achieved by the septic tank is important to ensure the success of the secondary treatment achieved by the CWs and to avoid clogging of the filters (Brix & Arias 2005). More than 45% of the final treatment can be done in the septic tank (Bounds 1997) (Figure 1).

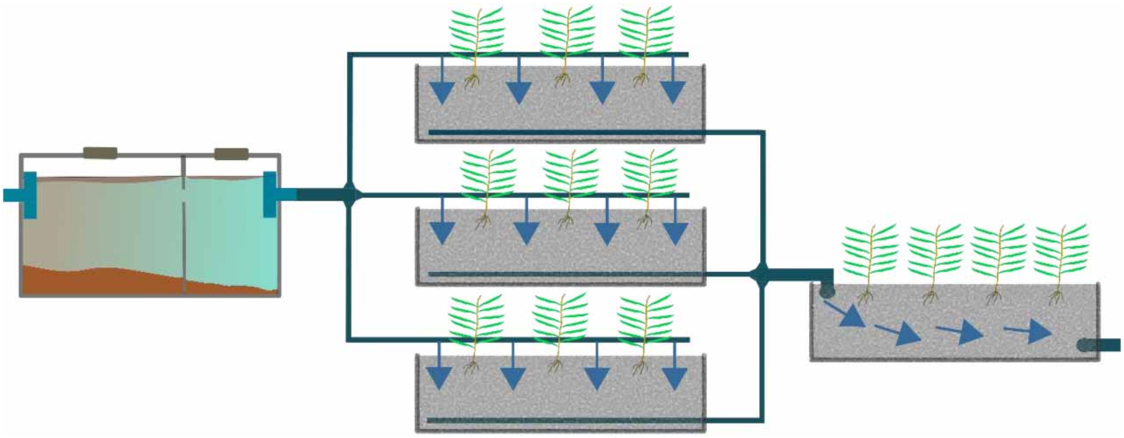

Figure 1 | Synoptic diagram of the proposed treatment system configuration.

Septic tank

Generally, the septic tank is sized with two compartments (UN-HABITAT 2008). The first of which is larger and allows enough time for the sludge to settle to the bottom and a scum layer to float to the surface, while the second is designed to settle a small amount of solids. A baffle is used to separate the two compartments with an opening in the center through which water passes from the first compartment to the second (Adhikari & Lohani 2019). The following are typical recommendations for septic tank design (Bounds 1997):

- The path of travel between the entrance and exit must be longer than the width or depth.
- The ratio of length-to-depth (L:D) of liquid varies from 1:1 to 3:1 (1.5:1 to 2.5:1 is the most common).
- The typical height-to-width (H:W) ratio is 1:1.

The volume of the septic tank is given by the following equation (UN-HABITAT 2008):

$$V_{ST} = HRT_{ST} \times Q_T \tag{1}$$

where HRT_{ST} is the hydraulic retention time (h) and Q_T is the water flow to be treated (m^3/day). The minimum value of HRT_{ST} is 0.5 days.

Water balance

In order to size any type of CWs, water balance must first be performed to account for all flows into and out of the CW. The balance equation is written as follows (Kadlec & Wallace 2009):

$$\frac{dV_w}{dt} = Q_i + Q_c + Q_{sm} - Q_o - Q_b - Q_{gw} + (PR - ET)A_w \tag{2}$$

where V_w is the volume of water in the wetland (m^3), t is the time (d), Q_i is the incoming wastewater flow (m^3/day), Q_c is the catchment runoff rate (m^3/day), Q_{sm} is the snowmelt flow rate (m^3/day), Q_o is the outflow of wastewater (m^3/day), Q_b is bank loss rate (m^3/day), Q_{gw} is infiltration rate through the soil (m^3/day), PR is precipitation (m/day), ET is evapotranspiration (m/day) and A_w is the area of the wetland (m^2). Water balance can be simplified by assuming equilibrium conditions (no water accumulation) some terms can be neglected. In practice, designers use Equation (3) (Wallace & Knight 2006; Kadlec & Wallace 2009), but most SSF CWs are lined (no seepage through the soil), which allows the seepage term to be neglected as shown in Equation (4) (Dotro *et al.* 2017) (Figure 2).

$$Q_o = Q_i - Q_{gw} + (PR - ET)A_w \tag{3}$$

$$Q_o = Q_i + (PR - ET)A_w \tag{4}$$

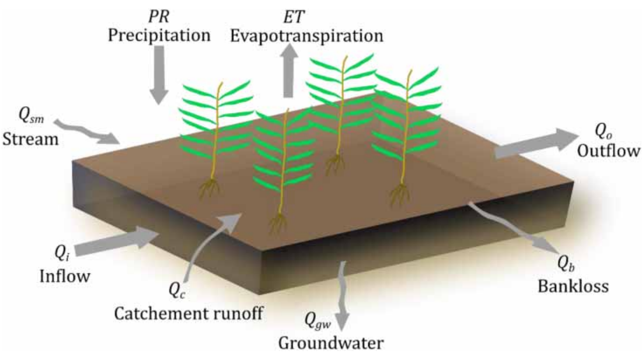

Figure 2 | Illustrative diagram of different water flows involved in the water balance.

The main objective is to size CWs using a wastewater flow that takes into account the effects of water losses and gains (*PR* and *ET*). The average flow between the inlet and outlet of the CW will therefore be given by design and calculated by Equation (5). The VF CW outlet flow also represents the inlet flow to the HF CW.

$$Q_m = \frac{(Q_i + Q_o)}{2} \tag{5}$$

The *ET* increases the concentration of pollutants due to the loss of pure water. It temporarily lowers the water level, increases the hydraulic retention time (*HRT*), and decreases the hydraulic loading rate (*HLR*) (US EPA 2000; Wallace & Knight 2006). For this reason, it is important to quantify the *ET* but it is difficult to calculate it accurately in small SSF CWs (Dotro *et al.* 2017), which is the case in this study. L'*ET* is highly dependent on plant type and density, climatic conditions of the site, height/width ratio (US EPA 2000), orientation relative to the general wind direction and the growing season of the plants (Dotro *et al.* 2017).

Among the most commonly used methods for estimating *ET* in wetlands is the crop coefficient method K_c (Kadlec & Wallace 2009):

$$ET = K_c ET_0 \tag{6}$$

K_c is the dimensionless crop coefficient and ET_0 is the reference evapotranspiration (mm/day). The FAO-56 Penman–Monteith method is commonly used to calculate the ET_0. The free software ET0, provided by FAO-56, is used to calculate the ET_0, which corresponds to the CW site. Several analytical methods exist in the literature to estimate K_c and each has advantages and disadvantages. The method used in this study takes into consideration the type of crop, the variation of crop growth periods, the length of each period, the crop height and the climatic conditions of the site. According to FAO-56, each plant has its own crop coefficients appropriate for each growing period (Bos *et al.* 2009). Table 2 lists some of the plants used in CW.

The K_c values provided by FAO-56 are measured in a subhumid climate with a minimum relative humidity of 45% and a wind speed of 2 m/s. In effect, these values must be converted. The conversion of $K_{c\ mid}$ (and $K_{c\ end}$ if it is greater than 0.45) is given by the following equation (Bos *et al.* 2009):

$$K_c = K_{ctable} + [0.04(u_v - 2) - 0.004(RH_{min} - 45)]\left(\frac{h_c}{3}\right)^{0.3} \tag{7}$$

Table 2 | Seasonal crop coefficients by plant type (Bos *et al.* 2009)

Plants	Initial period		Mid-season period		End of season period	
	$K_{c\ ini}$	L_{ini} (d)	$K_{c\ mid}$	L_{mid} (d)	$K_{c\ end}$	L_{end} (d)
Cattails, Bulrushes, killing frost	0.30	180	1.20	90	0.30	35
Cattails, Bulrushes, no frost	0.60	–	1.20	–	0.60	–
Short veg., no frost	1.05	180	1.20	90	1.10	35
Reed swamp, standing water	1.00	–	1.20	–	1.00	–
Reed swamp, moist soil	0.90	–	1.20	–	0.70	–

K_{ctable} is the crop coefficient provided by FAO-56, u_v is the wind speed (m/s), RH_{min} is the minimum relative humidity (%) and h_c is the height of the plant (m). Generally the adjustments of K_c are made using the average values for u_v and RH_{min} during the mid-season period (and also at the end of the season if $K_{c\ end} > 0.45$) as well as a maximum value of the plant height to have conservative results. $K_{c\ ini}$ is corrected using graphs that take into account ET_0, soil type and wetting frequency (Bos *et al.* 2009) (Appendix 1, Supplementary material).

Generally, there is no problem in estimating precipitation (*PR*) because it is derived from weather data provided by the weather stations closest to the study site.

Vertical flow constructed wetland

In general, BOD_5, COD and ammonia nitrogen are readily degraded aerobically using intermittently loaded VF CWs (Kadlec & Wallace 2009; Dotro *et al.* 2017). For this reason, oxygen consumption-based models represent the best design approach for VF CWs. The required surface area can be calculated according to Equation (8) (Kadlec & Wallace 2009):

$$A_{VF} = \frac{OD}{OTR} \tag{8}$$

OD is the oxygen demand (g O_2/day), which can be calculated by Equation (9) of Platzer (1999) taking into account attenuation of input concentrations through the septic tank; *OTR* is the oxygen transfer rate (g O_2/m^2/day). Generally, the determination of *OTR* is based on empirical observations, such as $23 < OTR < 64$ g O_2/m^2/day (Platzer 1999), $7.9 < OTR < 58.6$ g O_2/m^2/day and $5 < OTR < 100$ g O_2/m^2/day (Tyroller *et al.* 2010; Nivala *et al.* 2013). Cooper (1999) recommends using a typical value of 30 g O_2/m^2/day.

$$OD = Q_i(0.7\ \eta\ COD_{in} + 4.3\ TKN_{in} - 0.29\ NO_{3\_}N_{in}) \tag{9}$$

Q_i is the wastewater input rate (L/day), η is the *COD* removal efficiency (%) (Platzer (1999) measured 85%), COD_{in} is the chemical oxygen demand of the influents (g/L), TKN_{in} is the influent total Kjeldahl nitrogen concentration (g/L) and $NO_{3\_}N_{in}$ is the influent nitrate concentration (g/L).

The next step is to use the influent loading rate figures to verify the effluent outlet concentration using the previously calculated area. We recommend using relative conservatism of 50% as a basis for evaluation (Appendix 2, Supplementary material). If the degree of conservatism is low (less than 50%), the *OTR* should be adjusted by decreasing it to increase the required surface area, and therefore the effluent outlet concentration will decrease while the degree of conservatism will increase. On the other hand, if the degree of conservatism is very high (greater than 90%), it must be decreased by increasing the *OTR* to decrease the required surface area. It should be noted that VF CWs generate lower effluent concentrations than those shown in the influent loading rate figures (Wallace & Knight 2006).

VF CWs are typically constructed with a pre-specified bed depth (*h*). Several recommendations exist in the literature; such as $50 < h < 80$ cm (Kadlec & Wallace 2009) and $30 < h < 70$ cm (Stefanakis *et al.* 2014). The maximum value of the bed depth can be up to 100 cm (Brix & Arias 2005).

Horizontal subsurface flow constructed wetland

For HF CWs, the *P-k-C** model represents the most recent kinetic model for predicting pollutant output concentrations that exhibit first-order removal (Dotro *et al.* 2017; Gajewska & Skrzypiec 2018). The basic equation is written as follows:

$$C_o = C^* + \frac{C_i - C^*}{(1 + (k_V\tau/P))^P} = C^* + \frac{C_i - C^*}{(1 + (k_A/Pq))^P} \tag{10}$$

where C_o is the output concentration (mg/L) provided by the standard that corresponds to the desired end use, C_i is the input concentration (mg/L), which represents the VF CW output concentration plus 10 mg/L to ensure conservative results, C^* is the background concentration (mg/L), k_V is the first-order volumetric rate coefficient (1/day), k_A is the first-order areal rate coefficient (m/day), τ is the HRT (d), q is the HLR (m/day) and P is the apparent dimensionless number of tanks in series. The relationship between the coefficients k_V and k_A is given as follows (Kadlec & Wallace 2009):

$$k_V = \frac{k_A}{\varepsilon V/A_{HF}} \tag{11}$$

where ε is the porosity, which represents the fraction of water volume in relation to the total volume of the CW, V is the volume of the CW (m^3) and A_{HF} is the surface area of the wetland (m^2). The required area is calculated as follows:

$$A_{HF} = \frac{PQ_m}{k_A}\left(\left(\frac{C_i - C^*}{C_o - C^*}\right)^{1/P} - 1\right) \tag{12}$$

If we replace the average wastewater flow with its expression as a function of ET and PR (Equation (5)), the required area becomes:

$$A_{HF} = \frac{PQ_i(((C_i - C^*)/(C_o - C^*))^{1/P} - 1)}{k_A - 0.5P(PR - ET)(((C_i - C^*)/(C_o - C^*))^{1/P} - 1)} \tag{13}$$

The area expression contains the term (PR–ET) that varies over the year, so the required area (A_{VF}) will vary, and therefore the solution sought is the largest area that corresponds to the worst case situation. The coefficients k_A, P, C^* are derived from a database of actual wetland performance, indicating how conservative these values are in percentage terms. However, k_A and C^* are given at a wastewater temperature of 20 °C (Kadlec & Wallace 2009). Since water temperature influences the reaction rates of most pollutant degradation, a correction of these two coefficients must be made. For this purpose, the effects of temperature can be described by the Arrhenius temperature equation (Dotro *et al.* 2017):

$$K_T = K_{20}\theta^{T-20} \tag{14}$$

$$C_T^* = C_{20}^*\theta^{T-20} \tag{15}$$

where K_T is the reaction coefficient at water temperature, K_{20} is the reaction coefficient at a water temperature of 20 °C, C_T^* is the background concentration at water temperature, C_{20}^* is the background concentration at a water temperature of 20 °C, θ is the Arrhenius temperature factor (dimensionless) and T is the wastewater temperature (°C). Table 3 summarizes the input parameters for the P-k-C^* model for a 50% conservatism level.

Table 3 | *P-k-C** model input parameters corresponding to the 50th percentile (Kadlec & Wallace 2009)

Pollutants		k_A (m/year)	C^* (mg/L)	P	θ
BOD_5 (C_i in mg/L)	$0 < C_i < 30$	86	1	3	0.981
	$30 < C_i < 100$	37	5		
	$100 < C_i < 200$	25	10		
	$C_i > 200$	66	15		
TN		8.4	1	6	1.005
NH_4-N		11.4	0	6	1.014
NO_x-N		42	0	8	–
TKN		9.1	1	6	1.001

In practice, most treatment CWs are designed to remove multiple pollutants (Dotro *et al.* 2017). For this reason, we recommend performing the calculations for all pollutants of interest, the designer then selects the resulting surface that will remove all pollutant targets.

After calculating the surface area, it must be ensured that the effluent concentration is below the limit value using the influent loading rate graphs. If the degree of conservatism is low (less than 50%), the reaction coefficient should be adjusted k_A should be adjusted by increasing its conservatism to increase the area required to achieve the required output concentrations at a conservatism level of 50% or greater.

Several recommendations exist in the literature for the depth h of HF CWs, such as $27 < h < 60$ cm (Wallace & Knight 2006; UN-HABITAT 2008), $50 < h < 70$ cm (Dotro *et al.* 2017) and $60 < h < 80$ cm (Vymazal 2018).

Hydraulic loading rates and retention times

Generally, *HRT* is calculated as the volume of water in the CW divided by the wastewater flow (Dotro *et al.* 2017). However, the volume of water in a SSF CW is difficult to measure (US EPA 2000). Equation (16) is widely used to calculate *HRT* (Wallace & Knight 2006; Kadlec & Wallace 2009):

$$HRT = \frac{\varepsilon\, A\, d}{Q_m} \tag{16}$$

where ε is the porosity of the filter medium, A is the surface area of the CW (m^2), d is the depth of the CW bed (m) and Q_m is the average wastewater flow (m^3/day). Porosity varies with time and is rarely measured and almost never accurately known (Dotro *et al.* 2017). In HF (saturated) CWs, the volume of water in the CW may only be 30–45% of the nominal volume of the CW basin (Kadlec & Wallace 2009). In contrast to VF (unsaturated) CWs dosed by intermittent pulses, the empty spaces in the bed are occupied by air in the time interval between two feeds. Therefore, Equation (16) is not applicable in VF CWs to be able to calculate *HRT*.

Another important parameter in the hydrology of CWs is the *HLR* (Kadlec & Wallace 2009):

$$HLR = \frac{Q_m}{A} \tag{17}$$

it is given in (m^3/m^2/day), Q_m is the average wastewater flow (m^3/day) and A is the surface area of the CW (m^2). The *HLR* concept can be used for both VF (unsaturated) and HF (saturated) CWs without problems. In cold climates, lower loading rates should be applied; for example, 0.05–0.10 m^3/m^2/day. In warmer climates, these values can be increased to 0.15 m^3/m^2/day (Stefanakis *et al.* 2014).

Length/width ratio

For a VF CW, the aspect of L/W ratio is not as important as for a HF CW, considering that the power supply is done from the top of the filter. However, for a HF CW, the designer is confronted with two contradictory design objectives (Kadlec & Wallace 2009):

- The desire to spread the influent over a very large flat area, this delays the clogging time.
- The desire to spread the influent as evenly as possible over the vertical cross-section improves flow distribution.

As the L/W ratio increases from 1/1 to 4/1, the removal efficiency of COD and ammonia nitrogen (NH_4–N) becomes higher, while that of sulfate sulfur (SO_4–S) becomes lower (Sanchez-Ramos *et al.* 2017). Several recommendations of typical ratios are proposed in the literature. In practice, it is advisable to avoid a ratio that exceeds 4/1 because it can cause hydraulic problems (US EPA 2000).

Clogging

When sizing SSF CWs, the process kinetics may not correctly size a CW if the clogging kinetics are not quantified (Austin *et al.* 2007). A simple method to verify that the proposed design does not lead to bed clogging is to calculate the organic loading rate (*OLR*) and ensure that it is below a predefined limit. For HF CWs, the $OLR_{c,HF}$ calculated from Equation (18) should be less than 250 g BOD_5/m^2/day, provided that the filter material size is 4–6 mm (Wallace & Knight 2006).

$$OLR_{c,HF} = \frac{Q_{i,HF} BOD_{5,in}}{A_{c,HF}} \tag{18}$$

$Q_{i,HF}$ is the wastewater flow rate in the HF CW (L/day), $BOD_{5,in}$ is the input concentration of BOD (g/L), and $A_{c,HF}$ is the cross-sectional area of the HF CW ($W \times D$) (m^2). Vymazal (2018) recommends using the total area of the HF CW ($L \times W$), such that OLR is less than 10 g BOD_5/m^2/day, 10 g TSS/m^2/day and 20 g COD/m^2/day.

For VF CWs, the $OLR_{c,VF}$ calculated by Equation (19) must be less than 20 g COD/m^2/day with a maximum value of 27 g COD/m^2/day, provided that the filter material is sand with a size of 0–4 mm; the bed depth must be greater than 50 cm with one distribution point on each m^2 of the bed (Dotro *et al.* 2017).

$$OLR_{c,VF} = \frac{Q_{i,VF} COD_{in}}{A_{c,HF}} \tag{19}$$

$Q_{i,VF}$ is the wastewater flow entering the VF CW (L/day), COD_{in} is the chemical oxygen demand input concentration (g/L) and A_c is the cross-sectional area of the VF CW ($L \times W$) (m^2). If these conditions are not validated, the bed cross-sectional area must be adjusted. For HF CWs, the bed width and/or depth is increased, and for VF CWs the bed width and/or length is increased. Also proposed is another method, which consists in evaluating the Damköhler number *(Da)*, such that $Da < 0.09$ indicates that there is a high probability of clogging the wetland. In contrast, $Da > 0.09$ represents less of a tendency for the filter to become clogged. This method can be applied in HF and VF CWs (Austin *et al.* 2007; Stefanakis *et al.* 2014), but it requires experimental data to apply it.

Water distribution and collection system

In HF CWs, the goal is to achieve a uniform distribution over the entire cross-section of the inlet end of the bed. The most common method of water distribution is the use of perforated plastic pipes, usually reinforced polyethylene with a buried location in the inlet area and the surface of the inlet area that is leveled with a filter bed. The holes should be at least 3 cm in diameter to avoid uneven water distribution due to clogging of the holes, and they should be regularly spaced at a distance approximately equal to 10% of the width of the cell. Effluent collection in HF CWs can be achieved by a simple perforated pipe buried at the bottom of the outlet area. This leads to a sump located just after the HF CW to control the water level in the HF CW either by a socket pipe, a swivel elbow, or flexible pipes that can be held in position by a chain or a rope (Vymazal & Kröpfelová 2008; Kadlec & Wallace 2009).

Most VF CWs are pulse fed. This method involves storing influent water and then rapidly dosing it onto the bed surface, either through the use of pumps, or through gravity drains (if there is an adequate drop on site) or siphons. The design goal is to rapidly flood the bed in less than 15 min with shorter dose intervals; the 5–10 min range is more desirable. There is no rational design method for volume per dose (Kadlec & Wallace 2009). For VF CWs, one of the most common distribution methods is a series of perforated influent distribution pipes placed above the surface of the CW bed, with orifices at least 8 mm in diameter and spaced 0.4–0.7 m (Brix & Arias 2005; Dotro *et al.* 2017). Water collection is done through a network of perforated drainage pipes placed at the bottom of the bed. This network is connected to the atmosphere via vertical aeration pipes, which extend above the bed and allow a better aeration of the deepest parts of the bed, as well as the distance between the pipes, which is usually less than 5 m but it depends on the size of the bed (Stefanakis *et al.* 2014). The collection system is usually oversized to allow passive aeration of the VF CW bed. They are most commonly designed as free-draining systems, and water level control is possible and may be desirable, but is not mandatory for these systems (Kadlec & Wallace 2009).

Vegetation selection

When selecting the vegetation type for any CW type, designers should consider the following criteria:

- The plants chosen must be available on the local market, and adapted to the climatic conditions of the site.
- They must be tolerant to a variety of pollutants present in the wastewater (organic matter, nitrogen, phosphorus…) with a high removal capacity (direct absorption or indirect by providing adequate conditions such as oxygen transfer).

In SSF CWs (VF or HF), emergent macrophytes are generally used extensively, such as *Phragmites, Typha, Scirpus and Schoenoplectus,* which represent the most frequently used plant species (Stefanakis *et al.* 2014).

Media selection

Finer materials may result in better treatment, but may cause hydraulic problems and clogging. On the other hand, coarser materials allow water to flow easily through the wetland bed (Wallace & Knight 2006). The type of wastewater can also be an important factor in the selection of media size, which increases from greywater through secondary to primary influents (Kadlec & Wallace 2009). For HF CWs, using coarse material in the first part of the HF CW bed and finer material closer to the outlet provides the best treatment with ease of flow. The minimum media size is 4 mm, while the media in the outlet and inlet areas should be between 25 and 50 mm in size (Wallace & Knight 2006). Vymazal (2018) recommends higher sizes ; between 5 and 20 mm in the main layer, while the size of the media in the inlet and outlet areas is between 50 and 200 mm. Usually, different layers of gravel with an increasing grain size from top to bottom are used for VF CWs. In general, the bed is divided into three layers; a sand configuration, an intermediate layer of medium-sized gravel, and a drainage layer of larger gravel (Stefanakis *et al.* 2014).

Bed slope

The bed slope allows for the collection of treated water and drainage to the outlet of the area. However, there is a paucity of research to determine the optimal slope (UN-HABITAT 2008). In general, sloped HF CWs have shown relatively high treatment efficiency compared to non-sloped HF CWs (Hua *et al.* 2021). A slope of 0.5–1% is recommended for ease of construction and proper drainage (US EPA 2000).

Treatment efficiency

To account for water losses and gains due to ET and PR, the treatment efficiency must be calculated using the inlet and outlet flows:

$$E = \frac{Q_i C_i - Q_o C_o}{Q_i C_i} \tag{20}$$

where E is the treatment efficiency (%), C_i and C_o are the inlet and outlet concentrations (mg/L), respectively, and Q_i and Q_o are the inlet and outlet flow rates (m^3/day), respectively.

Figure 3 summarizes the different steps in the design of the proposed treatment system combining between VF and HF CWs.

APPLICATION

In order to provide the designers of SSF CWs with a practical example of the application of the proposed methodology, a case study of a Moroccan hammam will be treated following the steps cited in Figure 3. As shown in Figure 4, the hammam site is characterized by a net evapotranspiration (PR-$ET < 0$) over the whole year due to the nature of the arid climate that characterizes the city of Marrakech. This allows the application of higher HLR, and therefore treat more wastewater.

The treatment system configuration will be as recommended in this methodology as it fits within the framework of treating greywater discharged from public baths.

In order to quantify the flow of wastewater to be treated, Figure 5 presents the results of the comparison between the total surface area required for each flow during the month of January when the peak of PR will be. The objective is not to exceed the available land area which is equal to 210 m^2. Then, as shown in Figure 5, the flow that can be treated must not exceed 6 m^3/day. The surfaces of the CWs are the largest obtained after the realization of the water balance using Equation (13) on the basis of the history of the PR and ET in the site of installation for each flow entering chosen in notice.

Table 4 shows the calculated septic tank dimensions using 24 h as the HRT to ensure effective TSS removal.

The surface area of the VF CW was calculated by applying the Platzer model, such that a conservative percentage of attenuation in the septic tank was taken into account; 30% of organic matter (COD and BOD_5) and 50% of TSS, however the attenuation of TKN and NO_3-N which represents denitrification was eliminated to have a conservative calculated surface area. The input parameters and calculation results are summarized in Table 5.

The influent loading graph confirm the validity of the calculated VF CW area (40.5 m^2). This provides sufficient enough output concentrations of BOD_5 and TSS as an intermediate treatment step, such that the removal efficiencies are 66.15 and 73.07%, respectively, while the resulting output concentration at the TKN loading rate is higher

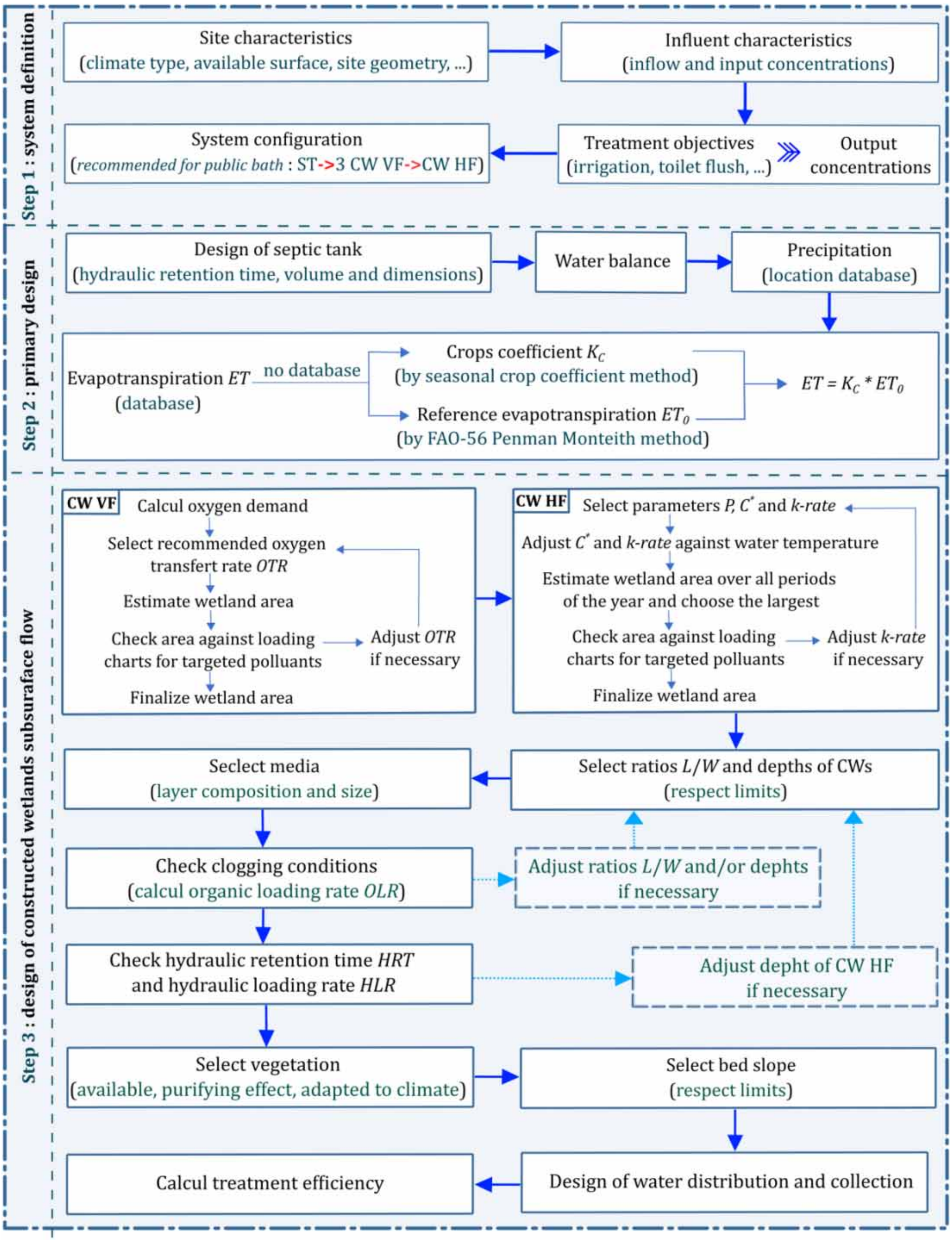

Figure 3 | Summary of the different steps in the design of the treatment system.

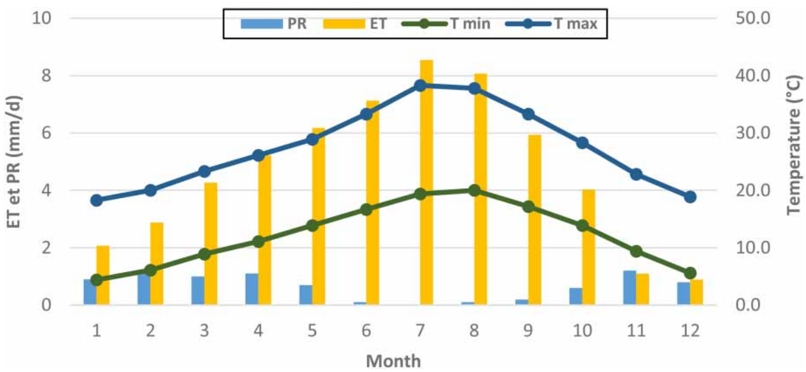

Figure 4 | Climatic conditions of the site.

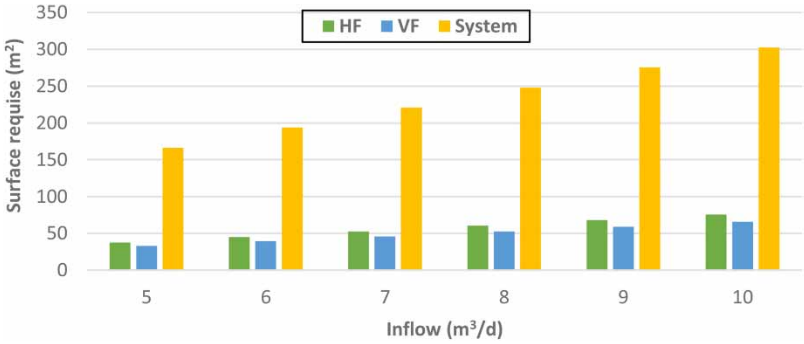

Figure 5 | Variation of the total surface of the treatment system according to the water flow to be treated.

Table 4 | Septic tank calculation results

Input parameters	Output parameters
Flow of wastewater to be treated: $Q_T = 6\ m^3/d$	Total volume: $V_{ST} = 6\ m^3$; $l \times w \times d = 2m \times 1.5m \times 2m$
Hydraulic retention time: $HRT_{ST} = 24\,h$	Volume of the room 1: $V_{R1} = 4\ m^3$; $1.33m \times 1.5m \times 2m$
	Volume of the room 2: $V_{R2} = 2\ m^3$; $0.66m \times 1.5m \times 2m$

Table 5 | Calculation result of the vertical flow constructed wetland

Input parameters	Output parameters
– Inflow rate: $Q_{i,VF} = 6000\ L/d$ – $COD_{in} = 0.2702\ g/L$ – $BOD_{5,in} = 0.0120\ g/L$ – $TKN_{in} = 0.0162\ g/L$ – $TSS_{in} = 0.2155\ g/L$ – Removal efficiency of COD: 70% – Oxygen transfer rate: $OTR = 30 g/m^2/d$	– Oxygen demand: $OD = 1183.6\ g/d$ – Area required for each bed: $A_{VF} = 40.5\ m^2$ (exact value is 39.45 m^2) – Ratio $L : W = 2 : 1$ (9m : 4.5m) – Depth: $d_{VF} = 0.8m$. – Media: main layer of sand (20 cm); intermediate layer of medium gravel (40 cm); drainage layer of coarse gravel (20 cm). – Total surface of the beds: $A_{VF,T} = 121.5\ m^2$ – Average hydraulic loading rate: $HLR_{VF} = 14.2 cm/j$ – Organic loading rate: $OLR_{COD} = 40.03$ g $COD/m^2/j$; $OLR_{BOD_5} = 17, 32$g $BOD_5/m^2/j$; $OLR_{TSS} = 31.93$g $TSS/m^2/j$; $OLR_{TKN} = 2.403$ g $TKN/m^2/j$ – $COD_{out} = 81.06$ mg/L – Influent loading graph (50th percentile) : $BOD_{5,out} = 40.62$ mg/L ; $TSS_{out} = 58.04$ mg/L; – Vegetation: *Typha latitofa* – Slope: 1%

than the input (41.38 > 16.22 mg TKN/L). The OLR_{COD} is higher than 27 g COD/m^2/day, this may cause a risk of bed clogging. However, the presence of three VF CW in parallel avoids this problem, as two beds can be fed at the same time, and the OLR_{COD} will be reduced to half its value.

Figure 6 shows the calculated area of the HF CW based on the heat balance equation and the history of *PR* and *ET*. The required area is the largest area in November (45.31 m^2) because it can absorb the peak *PR*. Table 6 shows the input and output parameters applying the first-order *P-K-C** model with BOD_5 as the main calculation parameter.

The output concentrations of BOD_5 and TSS determined from the influent loading graphs (26.86 and 29. 17 mg/L) are lower than the limit value of irrigation water (30 mg/L), as well as the calculated $OLR_{BOD5,C}$ cross-sectional *OLR* (120 g BOD_5/m^2/day) is much lower than 250 g BOD_5/m^2/day, it means that the proposed design achieves the desired effluent quality, as well as it is much far from the risk of bed clogging.

As shown in Figure 7, the *HLR* had the lowest values in July; 11 cm/day for HF CW and 13.4 cm/day for VF CW, however the peak was in November; 12.7 cm/day for CW HF and 14.8 cm/day for VF CW. This peak is always lower than 15 cm/day which is the maximum value in arid climates. At the *HRT* level, the calculation

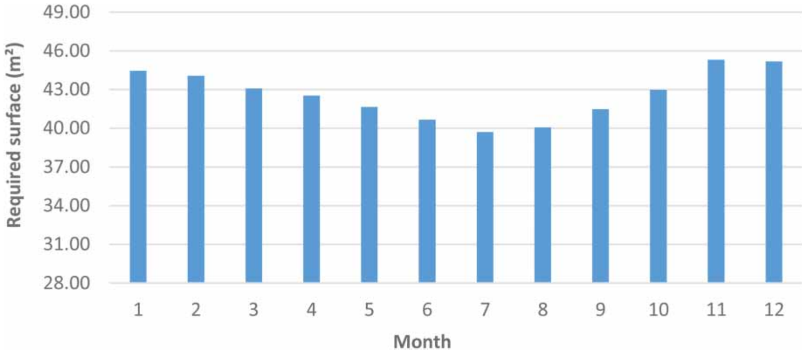

Figure 6 | Variation of the HF CW surface during the year.

Table 6 | Design result of the horizontal flow constructed wetland

Input parameters	Output parameters
– Average input rate: $Q_{i,HF} = 5,735$ L/d – $BOD_{5,in} = 0.050$ g/L – $TSS_{in} = 0.070$ g/L – $COD_{in} = 0.082$ g/L – $P = 3$ – $k_A(20°C) = 37$m/years – $C^*(20°C) = 5$ mg/L – $\theta = 0.981$	– $k_A(30°C) = 30,542$ m/years – $C^*(30°C) = 4.127$ mg/L – Surface required: $A_{HF} = 47.5$ m^2 (exact value is 45.31 m^2) – Ratio $L:W = 1.9:1$ (9.5m :5m) – Depth: $d_{HF} = 0.5$ m – Media: processing area 10 mm; input and output areas 30 mm. – Average hydraulic retention time: $HRT_{HF} = 1.69$ d – Average hydraulic loading rate: $HLR_{HF} = 11.9$ cm/d – Cross-sectional organic loading rate: $OLR_{BOD_5,C} = 120$ g BOD_5/m^2/d – Organic loading rate: $OLR_{BOD_5} = 6.310$ g BOD_5/m^2/d ; $OLR_{TSS} = 8.833$ g TSS/m^2/d – $COD_{in} = 45.53$ mg/L (by Equation (10)) – Influent loading graph (50th percentile): $BOD_{5,out} = 26.86$ mg/L ; $TSS_{out} = 29.17$ mg/L; – Vegetation: *Typha latitofa* – Slope: 1%

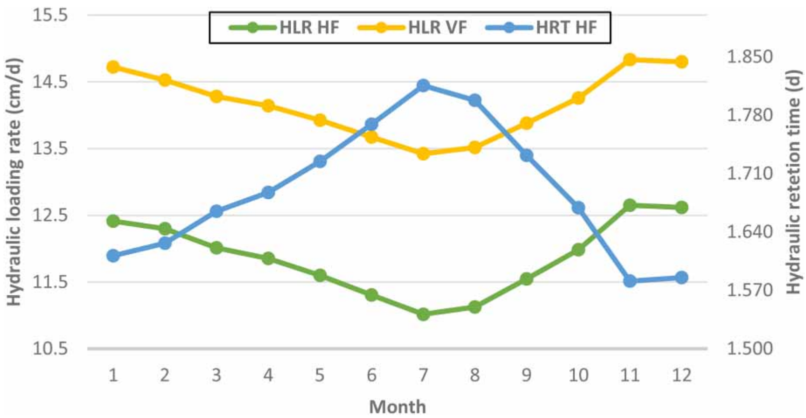

Figure 7 | Variation in retention times and hydraulic loading rates during the year.

was done only for HF CW because there is no practical formula to determine the *HRT* of VF CW with intermittent loading. In contrast to the *HLR*, the *HRT* had the highest value in July (1.815 d) and the lowest value in November (1.581 d) with an average of 1.69 d. These results are in line with the normal operation of HF CWs and confirm the validity of the adopted design.

The arid nature of the climate results in a net *ET* throughout the year (with the exception of November). This generates an output of the hybrid system that is always lower than the input, such that the peak of losses was in the month of July with a percentage of 16.17% which corresponds to 970.2 L/day. While a water gain of 0.19% (11 L/day) was in the month of November where there is net *PR*. The average water loss over the whole year is 7.65% which corresponds to 459 L/day (Figure 8).

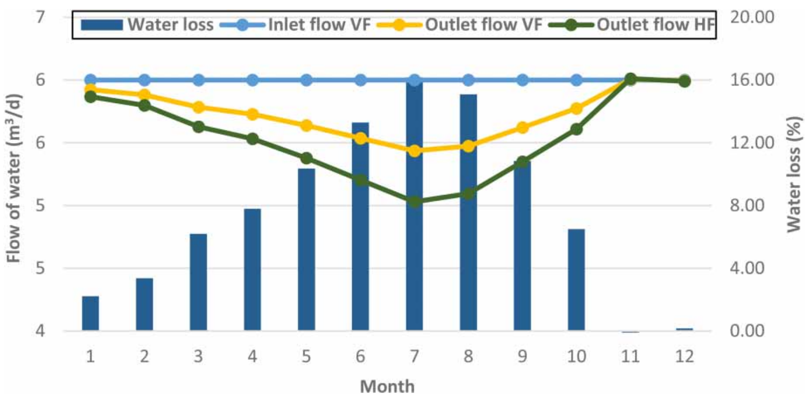

Figure 8 | Variation of different flows of the hybrid system with the percentage of water loss during the year.

The sized hybrid system achieves an outlet BOD_5 concentration below 30 mg/L throughout the year, as it varies from 26.99 mg/L (in the month of July) to 28.88 mg/L (in the month of November) with an average concentration of 28 mg/L. While the treatment efficiency varies from 82.67% (in November) to 86.45% (in July), with an average of 84.5%. This confirms the positive effect of net *ET* ($ET > PR$) on pollutant output concentration and treatment efficiency, provided that the calculation of HF CW area should be performed in the month of November when there is net precipitation. These results are not contradictory with the fact that *ET* increases the concentration of pollutants through the loss of pure water, because the surface area of HF CW with which these concentrations are calculated (47.5 m^2) is greater than that required to ensure 30 mg of BOD_5/L at the system outlet, such that this difference in surface area can go up to 7.79 m^2 in the month of July. Figure 9

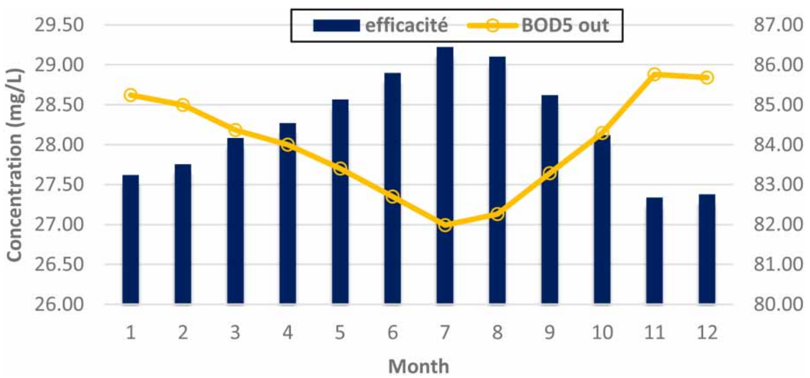

Figure 9 | Variation in the hybrid system treatment efficiency and BOD_5 output concentration over the year.

Table 7 presents some results of studies of SSF CWs using hybrid systems like the case of this paper. Figure 10 illustrates the different geometric parameters of the treatment system studied in the demonstration application presented earlier. Note that the nature of the terrain in this case study allows for gravity flow through the elements of the system, such that there is a natural level difference of 3.2 m between the VF and the HF CWs, thus avoiding energy consumption. A drainage network consisting of perforated PVC pipes (nominal diameter 110 mm) was used as a water collection system at the bottom of the VF CW such that six pipes are spaced at a distance of 0.5 m and meet in a manifold in the outlet area of the bed. This network is connected to the atmosphere by vertical aeration pipes, which allow better aeration of the deeper parts of the bed. For the HF CW, water collection is achieved through a 1% slope that ensures water flow through the cross-sectional area of the bed. A buried perforated PVC collector (nominal diameter of 160 mm) located in the outlet area collects the water and

Table 7 | Performance level of the CWs treating wastewater using hybrid systems

Country	Type of wastewater	SSF CW configuration	Removal efficiency % TSS	BOD_5	COD	Plantation	Reference
Morocco	Greywater (public bath)	3 parallel VF CW +1 HF CW	86.5	84.5	83.1	*Typha latitofa*	the present study
Bangladesh	Municipal waste	Hybrid unit	–	97	94.4	*Macrophytes*	Saeed *et al.* (2014)
India	Greywater	VF CW + HF CW	41	75	36	*Phragmites australis*	Baskar *et al.* (2009)
Bangladesh	Industrial waste	VF CW + HF CW	95	87	83.2	*Canna indica*	Saeed *et al.* (2018)
Bangladesh	Textile waste	VF CW + HF CW	–	96.6	89.3	*P. australis and Dracaena sanderina and Asplenium platyneuron*	Saeed *et al.* (2012)
Italy	Piggery waste	3 parallel VF CW +1 HF CW	–	–	79	*Canna india, Symphytum officinale, P. australis*	Borin *et al.* (2013)
Tunisia	Domestic	VF CW + HF CW	–	93	89	*P. australis, Typha sp.*	Kouki *et al.* (2009)

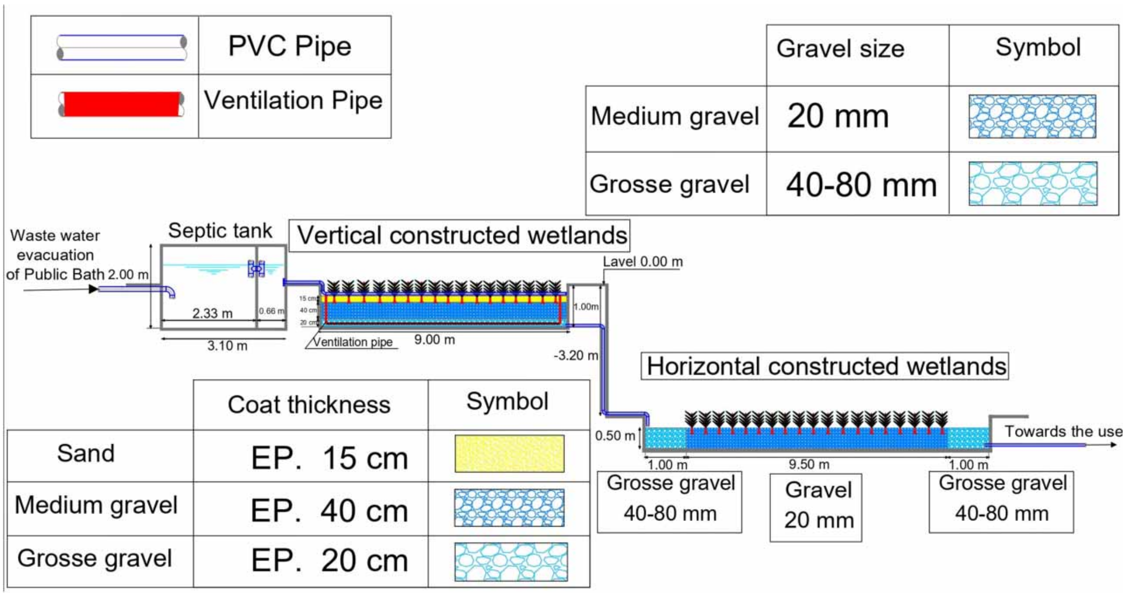

Figure 10 | Summary diagram of the treatment system studied.

brings it to a sump located just after the HF CW to control the water level in the bed through a flexible tube. Then the water goes to the treated water storage tank.

CONCLUSION

This work proposes a practical and easy-to-apply method for sizing subsurface flow constructed wetlands (SSF CWs). It is applicable for all sources of greywater, especially those discharged from public baths. Crop ET is determined using the crop coefficient (K_c) method. Knowing that the works that have addressed the design problem of SSF CWs have not provided any analytical method to determine K_c. This work allows practitioners to quantify it by adopting the FAO 56 method that takes into account the type of plant, its growth period, the climatic conditions of the site, the media size and the frequency of wetting. The results obtained by the case study show the flexibility of the proposed methodology, such that the removal efficiency of TSS, BOD5 and COD was in accordance with the standards of irrigation waters (the desired application in this case). Therefore, the surfaces obtained from the SSF CWs will be compatible with the need for treatment avoiding any kind of oversizing of the system that implies larger surfaces or undersizing that causes treatment efficiencies that do not achieve the desired objective. This accounting to the needs of the treated water application allows to have a control of the design of the SSF CWs.

This work contributes to the progress of the use of decentralized treatment systems, especially CW. It encourages owners of public baths to implement this type of ecosystem, which provides ecological treatment and only requires local equipment that is easy to handle. We hope that this study will help local communities to more easily implement local water reuse programs for non-drinking purposes (irrigation, fire protection, toilet flushing and so on) and reduce the inappropriate demand for treated drinking water.

DATA AVAILABILITY STATEMENT

All relevant data are included in the paper or its Supplementary Information.

CONFLICT OF INTEREST

The authors declare there is no conflict.

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