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BEHAVIOUR EVALUATION OF VERTICAL FLOW CONSTRUCTED WETLANDS FOR TREATMENT OF DOMESTIC SEWAGE AND SEPTIC TANK SLUDGE

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Thesis submitted to the post-graduation programme in Sanitation, Environment and Water Resources at the Federal University of Minas Gerais as a partial requirement for obtaining a Doctorate degree in Sanitation, Environment and Water Resources

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RESUMO

Na presente pesquisa, procurou-se avaliar o desempenho e comportamento de *wetlands* construídos de fluxo vertical (WCFV) para o tratamento de esgotos domésticos e lodo de tanques sépticos. As unidades de tratamento foram construídas e plantadas com *Cynodon dactylon Pers* na cidade de Belo Horizonte, Minas Gerais - Brasil, obedecendo ao primeiro estágio típico de um sistema Francês, conforme as recomendações e especificações do CEMAGREF. Em relação ao tratamento de esgoto, unidades plantadas e não plantada foram avaliadas, funcionando com uma vazão afluente no sistema de 13 $m^3.d^{-1}$ e uma Taxa de Aplicação Hidráulica (TAH) em todo o sistema de 0,22 $m^3.m^{-2}.d^{-1}$. No concernente ao tratamento de lodo de tanques sépticos, três estratégias operacionais foram implementadas e avaliadas. A *primeira estratégia operacional (OS1)* foi implementada com a alimentação da unidade de lodo e posterior envio do percolado para a unidade de pós tratamento. A *segunda estratégia operacional (OS2)* operou com a retenção do percolado durante 7 dias, seguido do envio do efluente para a unidade de pós tratamento. A *terceira estratégia operacional (OS3)* foi implementada com a retenção do percolado durante 7 dias, seguido do envio do efluente para a unidade de pós tratamento, onde ficava retido durante 7 dias. Em relação ao tratamento de esgotos, as medianas das eficiências de remoção obtidas foram de 72%, 76%, 85%, 63% e 49% para DBO_5, DQO, SST, NH_4^+-N e NTK respectivamente. Os resultados demonstraram que a remoção de DQO e SST diminui com o aumento da TAH. Tratando lodo de tanques sépticos, as medianas de eficiências de remoção obtidas com a *primeira estratégia operacional* foram de 73%, 81%, 42%, 55%, 66%, 67% para DBO_5, DQO, ST, STV, NH_4^+-N, NTK respectivamente e 0,2 unidades log de *E. coli* removidas. Implementando a *segunda estratégia operacional,* foram obtidas medianas de eficiências de remoção de 97%, 90%, 73%, 73%, 75%, 78%, para DBO_5, DQO, ST, STV, NH_4^+-N, NTK respectivamente e 2,3 unidades log de *E. coli* removidas. Em relação à *terceira estratégia operacional* obteve-se medianas de eficiências de remoção de 90%, 94%, 68%, 80%, 68%, 87%, para DBO_5, DQO, ST, STV, NH_4^+-N, NTK respectivamente e aproximadamente 4 unidades log de *E. coli* removidas, tendo sido a melhor estratégia para a remoção de poluentes. Ocorreu desaguamento do lodo acumulado na superfície da unidade de lodo e foi observada uma estabilização e mineralização satisfatória do material.

Palavras-chave: Wetlands construídos de fluxo vertical, tratamento de esgotos domésticos, tratamento de lodo de tanques sépticos, estabilização e mineralização do lodo.

ABSTRACT

The research aimed to evaluate the performance and behaviour of Vertical Flow Constructed Wetlands (VFCW) for treating sewage and septic tank sludge. The facility is comprised of a typical first stage of the French system, built and planted with *Cynodon dactylon Pers* in Belo Horizonte - Brazil, according to the specifications and recommendations of CEMAGREF French Institute. Regarding sewage treatment, planted and unplanted units were evaluated operating with a daily flow of 13 $m^3\ d^{-1}$ and a Hydraulic Loading Rate (HLR) on the full system of 0.22 $m^3\ m^{-2}\ d^{-1}$. In relation to septic tank sludge treatment, three operational strategies were implemented and evaluated. In *operational strategy 1 (OS1)* the sludge unit received septic tank sludge without any retention of the percolate, followed by sending the effluent to the post-treatment unit. During *operational strategy 2* (OS2), the percolate was retained during seven days. After that, the effluent was sent to the post-treatment unit. *Operational strategy 3 (OS3)* was implemented, where the sludge was discharged onto the sludge unit and the percolate was retained during seven days. The percolate from the sludge unit was then sent to the post-treatment unit for retention during seven days. In terms of sewage treatment, the medians of removal efficiencies obtained were of 72% for BOD_5, 76% for COD, 85% for TSS, 63% for NH_4^+-N and 49% for TKN. The removal efficiencies of COD and TSS decreased with an increase in HLR. Treating septic tank sludge based on *operational strategy 1,* the following median removal efficiencies were obtained: 73% for BOD_5, 81% for COD, 42% for TS, 55% for TVS, 66% for NH_4^+-N, 67% for TKN and 0.2 log units for *E. coli* removal. During *operational strategy 2*, the system increased its performance leading to global median removal efficiencies of 97% for BOD_5, 90% for COD, 73% for TS, 73% for TVS, 75% for NH_4^+-N, 78% for TKN and 2.3 log units for *E .coli* removal. Implementing *operational strategy 3,* the global median of removal efficiencies obtained were of 90% for BOD_5, 94% for COD, 68% for TS, 80% for TVS, 68% for NH_4^+-N, 87% for TKN and approximately 4 log units for *E. coli* removal. The dewatering of the accumulated sludge on the top occurred well and satisfactory sludge stabilization and mineralization were observed.

Key words: Vertical flow constructed wetlands, Sewage treatment, Septic tank sludge treatment, Stabilization and mineralization of septic tanks sludge.

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LIST OF ABBREVIATIONS

BOD	Biochemical Oxygen Demand
CePTS	Centre for Research and Training in Sanitation UFMG/Copasa
CERH-MG	Council of Water Resources of Minas Gerais
COD	Chemical Oxygen Demand
CONAMA	National Environmental Council (Brazil)
COPAM	Environmental Policy Council (State of Minas Gerais)
COPASA	Water and Sanitation Company of Minas Gerais
CU	Control Unit
CW	Constructed Wetlands
DNA	National Water Directorate
DO	Dissolved Oxygen
Effluent 1	Percolate from sludge unit
Effluent 2	Effluent from post-treatment unit
FS	Faecal Sludge
FWS	Free Water Surface Constructed Wetlands
HSFCW	Horizontal Subsurface Flow Constructed Wetlands
HLR	Hydraulic Loading Rate
HFB	Horizontal flow bed
MDG	Millennium Development Goals
NH_4^+-N	Ammonium
OS1	Operational Strategy 1
OS2	Operational Strategy 2
OS3	Operational Strategy 3

TKN	Total Kjeldahl Nitrogen
pH	Hydrogen ionic potential
PU	Planted Unit
PU1	Planted Unit in Phase 2
PVC	Polyvinyl Chloride
SDRBs	Sludge Drying Reed Beds
SL	Solids Load
SLR	Solids Loading Rate
ST	Septic Tank
STDRB	Sludge Treatment Drying Reed Beds
TKN	Total Kjeldahl Nitrogen
TS	Total Solids
TSS	Total Suspended Solids
TVS	Total Volatile Solids
Unit 1	Sludge Unit
Unit 2	Post-treatment Unit
UWEP	Urban Waste Expertise Programme
VFB	Vertical flow bed
VFCW	Vertical Flow Constructed Wetlands

1 INTRODUCTION

1.1 Background

The research is written in English because it is part of a programme funded by the Bill & Melinda Gates Foundation under the framework of “Sanitation for the Urban Poor” project (Stimulating Local Innovation on Sanitation for the Urban Poor in Sub-Saharan Africa and South-East Asia). There are eight international universities from Africa, Europe, Asia and South America as partners involved in this research. Since the author of this thesis is from Mozambique, most of the comments given in this Introduction aim at giving a characterization of the sanitation reality in this country. However, as it will be seen throughout the text, the experimental studies have been conducted in Brazil, at approximately the same latitude of Mozambique.

The lack of adequate infrastructure for Faecal Sludge (FS) and wastewater management are related to various environmental and health problems in developing countries. Many studies have showed limited coverage of sanitation and hygiene in Africa. Cross and Coombes (2014) reported that, in Sub Saharan Africa, 70% of the population remain without access to basic sanitation. According to them and AMCOW/WHO UNICEF (2012) currently, only eight African countries are on track to meet the sanitation Millennium Development Goals (MDG) target. Figure 1.1 shows the coverage trends and projection of sanitation in Africa. From Figure 1.1, it is possible to observe that a very small proportion of the sub Saharan Africa population has improved sanitation, while North Africa has a better sanitation coverage. Therefore, it is necessary to create and innovate sustainable systems to promote low cost treatment, decentralized sanitation for urban and peri urban poor areas of sub Saharan Africa.

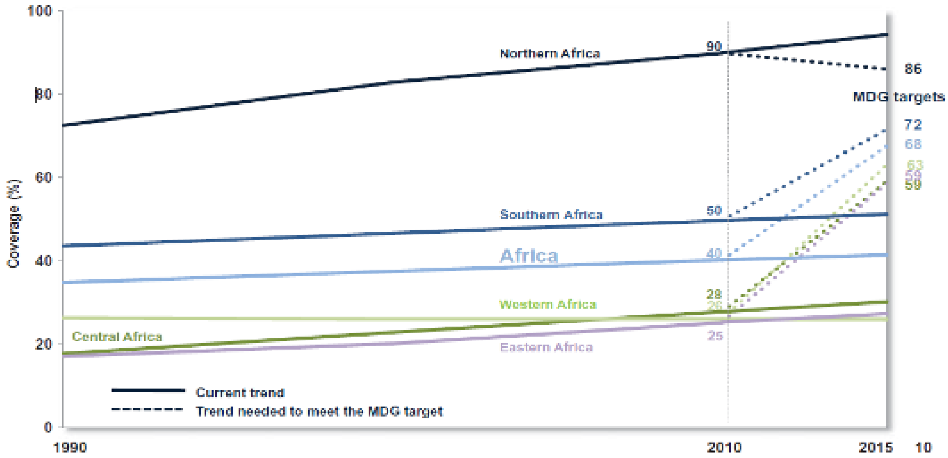


Figure 1.1**.** Coverage trends and projection in sanitation and hygiene in Africa.

Source: AMCOW/WHO UNICEF (2012)

In Sub-Saharan countries like Mozambique, the sludge resulting from sanitation facilities is discharged in an uncontrolled manner to the environment, leading to pollution and health problems, as shown in Figure 1.2. In Mozambique, most households in the urban poor areas have no infrastructure for sanitation needs; this has mostly happened in urban poor areas and can be seen as a critical indicator of their limited willingness and ability to pay for these facilities.

Figure 1.2. Truck discharges content of septic tank sludge from households to stabilization pond in Maputo.

In 2002/2003, a survey conducted by DNA (National Water Directorate) on the distribution of sanitation facilities in urban and rural areas indicates that in Mozambique more than 75% of the population live in urban areas and on the outskirts of urban centres, where urban sanitation infrastructures are poor and about 27% of the households have no sanitation infrastructure (DNA, 2003). Table 1.1 shows distribution of households by type of sanitation in Mozambique.

Table 1.1: Distribution of households by type of sanitation in Mozambique.

Sanitation type	**Number of households**					
	Urban areas	**%**	**Rural areas**	**%**	**Total**	**%**
Septic tanks	290944	18.8	24556	0.7	318505	6.3
Improved latrine	603555	39	438507	12.5	1041460	20.6
Non improved latrine	456535	29.5	1340076	38.2	1794749	35.5
Open space	196542	12.7	1701406	48.5	1895861	37.5
Total	1547577	100	3508053	100	5055630	100

Source: INE (2015)

In Mozambique, there are many communities with mixed solutions of wastewater treatment, sewerage systems, septic tanks and storm water. By observing Table 1.1, the predominant sanitation facility is open space. About 37.5% of households mainly in rural areas do not have any facility to deposit the excreta.

In 2015, INE reported that from 2008 up to 2015 the number of households without any facility to deposit excreta was reduced by 24%. Only 6.3% of households are using septic tank and most of them are living in urban areas. The results presented in Table 1.1 shows the critical situation concerning sanitation facility in Mozambique. Besides, only 20.6% of households are served by improved latrines with majority in urban areas (39%) and 35.5% of households fall under the category of those that are served by non-improved latrine.

In 2003, DNA reported that about 60% of the inhabitants, mainly in urban areas, are using improved and non-improved latrine to deposit the excreta because of poor sewerage system in the country, and waterborne diseases proliferate in the country. The incidents of these diseases are significantly high to require local and appropriate solutions on sanitation in developing countries like Mozambique.

In Sub-Saharan African countries like Mozambique, very little research has been conducted to identify suitable treatment schemes for faecal sludge. In Mozambique, faecal sludge collection and haulage are faced with great challenges. Several trucks of sludge management companies have no access to pit latrines for emptying. Many dwellers are not willing to pay to empty their latrines.

The need to produce an adequate decentralized wastewater treatment system has become critical. Most of the sanitation solutions in the country are based on latrines. The utilization of simple systems such as vertical flow constructed wetlands (VFCW) as part of a decentralized treatment of urban wastewater is considered a good alternative. However, very few wetlands have been implemented in Sub-Saharan Africa, while pulse-loading vertical flow systems are being widely implemented in some European countries and are gaining increasing acceptance in North America (Kadlec and Wallace, 2008). Constructed wetland technology for wastewater treatment is used in many countries serving relatively small settlements (Gikas and Tsihrintzis, 2012). Recently, constructed wetlands have been used for decentralized sanitation in developing countries. Undoubtedly, wastewater treatment using constructed wetlands appears as an efficient alternative for decentralized sanitation in developing countries. This type of sanitation system is characterized by low cost construction, maintenance and operation, as well as a high efficiency in the retention and removal of contaminants.

There are experiences that utilize VFCW for the treatment of the domestic sewage and sludge from septic tanks around the world. One of the experiences can be found in Belo Horizonte city, capital of the state of Minas Gerais, in Brazil. The VFCW in Belo Horizonte is located at the Centre for Research and Training in Sanitation (CePTS), situated at the sewage treatment plant of the Arrudas River in Belo Horizonte and is a result of a partnership between UFMG and the Company of Water and Sanitation of Minas Gerais (COPASA). The construction followed the recommendations of the first stage of the French System developed by CEMAGREF and consists of three vertical units, two planted, and one unplanted as a control unit. In addition, the system was designed to treat raw domestic sewage operating with pulse loading in alternating phases, with periods of resting and feeding in order to control the growth of biomass and to guarantee aerobic conditions.

1.2 Problem statement

Faecal Sludge Management (FSM) includes storage, collection, transport, treatment and safe disposal of the sludge (Strande et *al.,* 2014). It is well known that in developing countries the excess of sludge and domestic wastewater are often discharged in an uncontrolled manner to the environment. Normally in most regions of developing countries, domestic sewage and sludge from septic tanks are not treated and disposed safely due to the lack of basic sanitation services, the lack of effective participation of local stakeholders in the FSM and the lack of trained personnel in FSM.

However, the treatment of raw sludge and its final disposal is a great challenge in many regions of developing countries. The planning process of FSM and domestic wastewater treatment requires an integrated systems approach and the final end use or disposal option for the sludge should first be determined (Strande et *al.*, 2014).

There is little consolidated scientific evidence regarding the dynamics of vertical flow constructed wetland system in terms of treatment of domestic sewage and sludge coming from septic tanks, hence in this research it is expected to contribute to the performance evaluation of the organic matter and microbial removal in a planted vertical flow wetland.

2 RESEARCH OBJECTIVES AND HYPOTHESES

2.1 General objective

To evaluate the behaviour of VFCW French system for the treatment of domestic wastewater and for the treatment of sludge from septic tanks, both operating under variable loading rates.

2.2 Specific objectives

- To evaluate the effluent concentrations and removal efficiencies of vertical flow constructed wetlands in terms of removal of organic matter, total suspended solids, total Kjeldahl nitrogen and *Escherichia coli* for raw domestic sewage and septic tank sludge treatment.
- To compare the performance of planted and unplanted units of vertical flow constructed wetlands for raw domestic sewage treatment.
- To determine hydraulic and solids loading rates of sewage and sludge that allow a satisfactory operation of vertical flow wetlands for organic matter, total suspended solids and total Kjeldahl nitrogen removal.
- To evaluate the performance of the VFCW system in terms of dewatering, stabilization and mineralization of the applied septic tank sludge.

2.3 Hypotheses

H1- The first stage of the French VFCW system can be adapted for the treatment of sludge from septic tanks and the treatment of domestic sewage in warm regions.

H2- The first stage of French system of VFCW allows different loading regimens for treatment of raw domestic sewage and septic tank sludge in warm regions.

H3- There are differences in terms of removal efficiencies between planted and unplanted units of VFCW treating domestic sewage.

H4- The performance of vertical flow constructed wetlands treating sludge from septic tanks is influenced by the applied hydraulic and solids loading rates.

H5- Vertical flow constructed wetlands for the treatment of sludge from septic tanks allows dewatering, stabilization and mineralization of the sludge in the top deposit layer of the bed.

2.4 Significance of the research

Domestic wastewater and sludge from septic tanks must be treated and adequately disposed due to the high concentration of non-stabilized organic matter, as well as elevated content of pathogenic organisms (Suntti *et al.*, 2011). Nowadays, the utilization of VFCW for domestic sewage treatment and for dewatering, mineralization and stabilization of sludge from septic tanks is increasing. VFCW is considered one of the sustainable solutions for treatment of domestic wastewater and sludge from septic tanks.

Nowadays, it is well known the valuable contribution of the first stage of the vertical flow constructed wetlands treating domestic sewage, mainly in Europe. Many studies carried out by CEMAGREF/Irstea, with emphasis on several publications by Pascal Molle and his group, related with VFCW treating different types of the effluents have demonstrated a valuable contribution of these systems concerning pollutants removals and the increase of scientific information about its operational dynamics. However, there is little consolidated scientific knowledge regarding the dynamics of first stage of the VFCW treating domestic sewage operating with only two units and another unit dedicated to septic tank sludge treatment. Additionally, there is little scientific information about these systems treating ST sludge under variable hydraulic and solids loading rates, as well as the retention of the percolate inside of the units for percolation and drying. In this sense, more scientific information regarding the utilization and dynamics of VFCW system can be an important contribution for improving its performance.

Taking into account the importance of VFCW in terms of raw domestic sewage and septic tank sludge treatment, the present research aims to evaluate the behaviour of vertical flow wetlands based on the first stage of the French system for treatment of domestic wastewater and sludge from septic tanks. Furthermore, the objective of the research is to evaluate different flow rates or loads, in order to verify the removal efficiencies of organic matter, solids and other pollutants. Thereby, it is important to determine at what extent these systems provide a suitable treatment of domestic wastewater and sludge from septic tank.

3 LITERATURE REVIEW

3.1 Faecal sludge management

The world has become urbanized. In recent years, rapid growth of urban and peri urban areas has been observed. There is no doubt that the population growth should be associated with the increase of infrastructures for sanitation supply and suitable faecal sludge management. Nowadays, one of the challenges, especially in developing countries, is the provision and management of basic sanitation services. Koottatep *et al.* (2002) reported that the collection of FS from on-site sanitation installations and their haulage in large cities in Asia are faced with immense problems and most of the FS in developing countries are disposed of untreated and indiscriminately into lanes, drainage ditches, onto urban spaces and into water bodies resulting in enormous health risks.

In Mozambique and other developing countries, the excess of sludge and pollutants are often discharged in an uncontrolled manner to the environment, into the rivers, drains, canals, lakes and seas; leading to pollution and health problems. A major challenge in the implementation of sustainable decentralized and low cost sanitation is related to the suitable FS management in poor areas. Most people in poor areas are using pit latrines facilities, which are an on-site sanitation solution employed in many countries in the world.

The proper management of FS, mainly in urban areas of developing countries can avoid problems related with environmental pollution and human health. Nevertheless, efforts have been made and actions have been taken to promote municipal sanitation planning, on-site sanitation facilities and desludging.

Faecal sludge is raw, partially digested, slurry or semisolid, and results from collection, storage or treatment of combinations of excreta and black water, with or without grey water (Strande *et al.*, 2014). Faecal sludge is produced from onsite sanitation technologies, for example, pit latrines and septic tanks, and is the result of the collection, storage or treatment of combination of excreta and black water, with or without grey water from these technologies (Kengne *et al.*, 2014). Therefore, faecal sludge management includes the storage, collection, transport, treatment and safe disposal.

According to Strauss *et al.* (2010) in Sub-Saharan Africa, more than 75% of houses in large cities are served by on-site sanitation facilities. Strauss *et al.* (2003) reinforced that in Sub-Saharan Africa and Asia, about 65-100% of urban dwellers and 20-50 % of urban dwellers in Latina America are served by on-site sanitation systems. The authors commented that in these regions the predominant systems include septic tanks, public toilets and pit latrines.

In urban areas of developing countries, on-site sanitation systems predominate over water borne sewered sanitation, based on septic tanks and public toilets (Koottatep *et al.,* 2005). One of the most important elements of implementing cost effective and sustainable wastewater system designs is early planning and coordination of the overall project plan (Partner, 2010)**.** In fact, the success of faecal sludge management depend on planning the process.

According to Koottatep *et al.* (2005), the concept of FS management should be based on the assessment of:

- Existing sanitary infrastructure and trends;
- Current FS management practices and their shortcomings;
- Stakeholders customs, needs and perceptions regarding FS management and use;
- Environmental sanitation strategy; and
- The general urban development concept.

One of the major challenges nowadays is related to the implementation of sustainable and low cost technologies for wastewater collection and treatment, mainly in developing countries. Sludge treatment and its final disposal are complex and expensive taking into account economical, technical, environmental, health and legal aspects that are involved.

Nowadays, it is estimated that between 2.1-2.6 billion people in low and middle-income countries rely on onsite technology that produce tons of untreated faecal sludge every day (Strande *et al.,* 2014). The amount of faeces produced per capita per day may vary for different regions. The implementation of low cost technologies for sludge treatment in developing countries became an important factor to reduce the quantity of untreated sludge in the world.

Koné and Strauss (2004) pointed out that the choice of a FS treatment option depends primarily on the characteristics of the sludge generated in a particular town or city and the treatment objectives. The authors emphasized that FS characteristics vary widely within and between cities, based on the types of on-site sanitation installations used and on the emptying practice.

3.2 Septic tanks

Septic tank is defined as a cylindrical or rectangular prismatic tank with horizontal flow for sewage treatment by sedimentation, flotation and digestion (Prosab, 2009). A septic tank is key for the septic system, a small-scale system common in areas with no connection to main sewage pipes provided by local government or private corporations. Septic tanks are a watertight chamber made of either concrete, fibreglass, PVC or plastic for the storage and treatment of black water and grey water. Through settling and anaerobic process the solids and organics are stabilized, but treatment is only moderate (Tilley *et al*., 2008).

The system is appropriate when there is a way of dispersing or transporting the effluent. In several areas of developing countries, mainly sub-Saharan Africa, most people use pit latrines than septic tank. Many households in sub-Saharan Africa do not have the ability to pay for adequate sanitation services which implies the utilization of pit latrines rather than septic tank. Differently to the septic tank, the input of a pit latrine can include urine, faeces, flush water and dry cleansing materials.

The availability of septic tanks for the treatment of sludge from septic tanks is a priority mainly in developing countries. In Brazil, it is estimated that about 38% of urban population, equivalent to 68 million people, and 64% of rural population, equivalent to 12 million people, are served by septic tank facilities for sewage treatment (Prosab, 2009). Regarding septic tank availability, the situation in Mozambique is more critical where only 7.4% of households in rural areas and 0.2% of households in urban areas are served by septic tanks for sewage treatment (DNA, 2005).

Hoffmann and Platzer (2010) reported some principles about septic tanks, which include the following points:

- The system is widely used for decentralized (on-site) domestic wastewater treatment.
- The system is common in many developing countries;
- The removal efficiency for organic matter is typically 30%; in subtropical climates, septic tanks are designed with 24 to 48 hours hydraulic retention time, and in cold climates up to five days may be needed.

These systems are mostly used for primary treatment purposes and are the most common, small-scaled, decentralized system used worldwide. There is no doubt that it is a system with simplicity in construction, operation and maintenance. In septic tanks, basically the sludge is stabilized by anaerobic digestion, where dissolved and other suspended particles leave the tank partially treated. Figure 3.1 presents a simplified illustration of a septic tank used for residential-scale primary treatment.

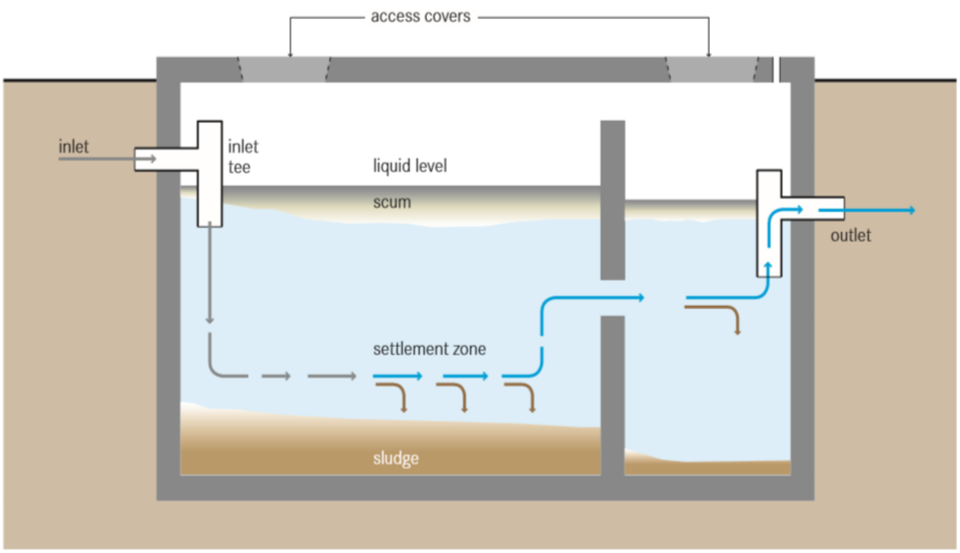


Figure 3.1: Basic elements of a septic tank.

Source: Tilley *et al.* (2014)

The literature successfully describes different types of septic tanks used in different regions of the world and constructed by different materials for primary treatment. By observing Figure 3.1 which illustrates a septic tank, it can be seen elements that there are inlet and outlet pipes, compartments constituted by sludge in the lower layer and scum in the first compartment.

According to Parten (2010), primary treatment using septic tanks can achieve the following level of treatment:

- About 60% to 80% suspended solids removal;
- About 50% to 60% BOD_5 removal;
- Fats, oils, and grease removal to about 80%; and
- Minimal nutrient removal.

Many of the households in developing countries are served by septic tanks and pit latrines as their main type of sanitation. However, the effluent from septic tanks requires complementary treatment and the sludge requires adequate disposal.

Klingel *et al.* (2002) reported that it is irresponsible to promote septic tanks without providing at the same time solutions for regular desludging of the facilities and for adequate disposal of the sludge. The Septic Tank (ST) settling and anaerobic process reduce solids and organic matter. In terms of treatment process, Tilley *et al*. (2008) commented that inside the ST most of the solids settle in the first chamber. The baffle, or the separation between the chambers, is to prevent scum and solids from escaping with the effluent. Liquid flows into the tank and heavy particles sink to the bottom, while scum floats to the top. Simultaneously, the solids that settle to the bottom are degraded anaerobically.

In addition, after preliminary treatment of sludge from septic tanks among various alternatives of treatment, VFCW have been used for final treatment of septic tank sludge in several regions of the world.

3.3 Characteristics of faecal sludge in different places

The characteristics of faecal sludge constitute a greater factor that influence the choice or selection of the appropriate technology or process for sludge dewatering, stabilization, mineralization and pollutant removal. In addition, the characteristics of faecal sludge indicate the physical-chemical conditions, the level of biochemical degradation, the FS source and among others. Strande *et al.* (2014) reinforced that currently there is a lack of detailed information on the characteristics of FS and research is being conducted in this field. Recently, there is an increase in the literature indicating more information regarding characteristics of faecal sludge based on research carried out in different regions of the world. Bassan *et al.* (2013) emphasized that the availability of FS characteristics is very limited in contrast to wastewater characteristics. Another important factor of the choice or selection of the treatment technology is the type of FS.

Authors like Koottatep *et al.* (2005), Strauss *et al.* (2000) and Strande *et al.* (2014) have successfully classified FS in two types according to the source or origin. First, ***type "A***" (high-strength); FS from public toilet or bucket latrine sludge, on the other hand, characterized by high concentrations of total solids, mostly fresh and stored for a day or weeks only. Second, ***type "B"*** FS from septic tanks, characterized by low concentrations of total solids, usually stored for several years and more biodegraded than type "A".

Table 3.1 lists FS characteristics in the world, taking into account many studies developed by different authors. Strande *et al.* (2014) pointed out that parameters that should be considered for the characterization of FS include solids concentration, chemical oxygen demand (COD), biochemical oxygen demand (BOD), nutrients, pathogens, and metals. The characteristics of faecal sludge differ widely by location, from household to household, from city to city, from district to district (Koottatep *et al.* 2005).

Table 3.1: Main characteristics of the sludge from septic tanks in different parts of the world.

Region	FS source	pH	TS (mg.L^{-1})	TVS (mg.L^{-1})	TSS (mg.L^{-1})	BOD (mg.L^{-1})	COD (mg.L^{-1})	TN (mg.L^{-1})	NH_4^+-N (mg.L^{-1})	References
EUA	Septic Tank	5.4	40000	25000	15000	7000	15000	700	150	Strauss (1995)
Brazil	Septic Tank		12880	3518	7091	2434	6895	120	89	Meneses *et al* (2001)
Brazil	Septic Tank		7186	3413	2064	1087	6199	--	58	Belli Filho *et al.* (2007)
Brazil	Septic Tank		9267 (745 – 44472)	4868 (304 – 21445)	--	1863 (499 – 4104)	9419 (1363 – 25488)	---	---	Leite *et al.* (2006)
Thailand	Septic Tank	6.7 – 8.0	5700- 28000	4000-21000	3150-21600	600-5500	5400-34500	370-1500	200-590	Koottatep *et al.* (2008)
Thailand	Septic Tank		15350 (2200 – 67200)	73% (TS)	---	2300 (600 – 5500)	15700 (1200 – 76000)	1100 (300 – 5000)	415 (120 – 1200)	Koné and Strauss, (2004)
Jordan	Septic Tank				2600	1600	5750			Strauss (1995)
Indonesia	Septic Tank		47000				24000	644		Strauss (1995)
Philippines	Septic Tank		31000	19000		5500	12800		209	Strauss (1995)
Norway	Septic Tank		54000	31600	45000	10300	42550	793	113	Strauss (1995)
Ghana	Septic Tank		12000	7080		840	7800		330	Koné and Strauss, (2004)
Ghana	Public toilet sludge		52500	35700		7600	49000	---	3300	Koné and Strauss, (2004)
Cameroon	Not defined	7.50	3.7%	65.4%	27600		31100	1100	600	Kengne *et al.* (2010)
Burkina Faso	Septic Tank		19000	8930	-	2.240	13500	2100	-	Koné and Strauss, (2004)
Burkina Faso	Septic Tank		8984	5121	7077	1453	7607	-	-	Bassan *et al. (*2013)
Burkina Faso	Pit latrines		13349	7742	10982	2126	12437	-	-	Bassan *et al. (*2013)
Argentina	Septic Tank		6000-35000	3000-17500		750-2600	4200	190	150	Koné and Strauss, (2004)
France	Septic Tank		30000	21300	23000		42000	1423	287	Vicent *et al.* (2010)
Europe	Septic Tank	5.2 – 9.0	200 – 123860	160 – 65570	5000 – 70920	700 – 25000	1300 – 114870	150 – 2570	-	Koottatep *et al.* (2008)
Tropical countries	Public toilet or bucket latrine		≥ 35000		≥ 30000		20000 – 50000		2000 – 5000	Koottatep *et al.* (2008)
Tropical countries	Septic Tank		< 30000		7000		< 15000		< 1000	Koottatep *et al.* (2008)

Source: Adapted from Prosab (2009), Strande *et al*. (2014), Koottatep *et al.* (2008)

By observing the data presented in Table 3.1, it can be seen that faecal sludge differs greatly for each origin. The analysis shows greater variability of FS taking into account the origin and type of FS. Koottatep *et al.* (2008) emphasized that the quality of sludge is subject to high variations, likely due to differences in storage duration, temperature, groundwater intrusion, and tank emptying technology and pattern. On the other hand, it should be noted that no information was found related to the storage duration of each evaluated source. In addition, it can be seen that the FS in the world is characterized by greater variability between countries, based on the type of on-site sanitation installations. Koné and Strauss (2004) observed FS variability between cities, also based on the type of on-site sanitation installations in use and on the emptying practice. Nevertheless, not enough information was found on sludge from public toilets or bucket latrines, except researches of public toilet sludge conducted in Accra, Ghana.

By comparing two types of FS cited previously and classified by Koottatep *et al.* (2005), the data show higher concentrations of sludge originating from public toilets than from septic tank, demonstrating more stability of the sludge from septic tanks due to longer storage periods (Koné and Strauss, 2004). Considering two types of classification suggested by Strauss *et al.* (1997), from the data presented, it can be observed that the FS characterized fall under the category of ***type "B"*** or low-strength. Besides, the authors found that it has been complicated to establish strict classification of FS, because FS can often be associated with one of two categories, high and low-strength sludge (Koottatep *et al.*, 2005).

Nevertheless, Strande *et al.* (2014) emphasized that the origin of the sludge is important when the drying sludge reed beds is used for faecal sludge treatment. From the origin, the solids loading rate, frequency of FS application, etcetera can be deduced. It is known that septic tanks are considered to contain a lower specific sludge resistance for dewatering; hence, higher total solids loading rates can be applied.

3.4 Constructed wetlands

3.4.1 Definition and historical perspective

Constructed Wetlands (CW) are engineered systems that utilise natural resources involving wetland vegetation, a filtering medium and their associated microbial assemblages to assist in treating wastewaters (Vymazal, 2010). Hoffmann and Platzer (2010) defined CW as engineered systems designed and constructed to utilise the natural functions of wetland vegetation, soils and their microbial populations to treat contaminants in surface water, ground water or waste streams. Therefore, constructed wetland systems are a man-made complex that mimics the structure of natural wetlands to serve as a wastewater filter. The treatment in constructed wetlands is the result of complex interactions between the compartments. Biological and physical interactions determine the level of pollutant removal which include filtration, decomposition, nitrification and denitrification, adsorption, precipitation, degradation, settling and plant uptake (Hoffmann and Platzer, 2010; Brix, 1997).

The first experiments about constructed wetlands for sewage treatment were carried out in 1950 by Kathe Seidel in Germany using wetland plants (Vymazal, 2008). The technology originated from the research conducted at the Max Plank Institute in West German, starting in 1952. Since 1985, the research and implementation of wetland technology around world has been accelerating (Kadlec and Wallace, 2008).

3.4.2 Applications, advantages and disadvantages of constructed wetlands

Many applications of constructed wetlands are reported, mainly in developed countries. The system is designed to take advantages of many of the processes that occur in natural wetlands. Table 3.2 summarises different applications of constructed wetlands in the world.

Table 3.2: Various applications of CW in the world.

- Domestic wastewater treatment.
- Septage treatment in tropical regions;
- Treatment of household wastewater;
- Grey water treatment;
- Decentralized wastewater treatment;
- Pre-treatment of raw wastewater;
- Tertiary treatment of effluents of conventional wastewater;
- Industrial wastewater treatment;
- Storm water treatment and retention;
- Natural treatment of polluted rivers and lakes;
- Sludge dewatering and mineralisation;
- Natural treatment of water from swimming pools without chlorine.
- Provide habitat for many wetland organisms;
- Can be built to fit harmoniously into the landscape;
- They provide numerous benefits in addition to water quality improvement, such as wildlife habitat and the aesthetic enhancement of open spaces, and
- They are an environmentally sensitive approach that is viewed with favour by the general public.

Source: Koottatep *et al.* (2008), Hoffmann & Platzer (2010), Nguyen *et al.* (2007), Molle (2014), Gkika *et al.* (2014), Brix (1997), Haberl *et al.* (2003)

From Table 3.2 it can be seen that constructed wetlands have a wide range of applications, including treatment of domestic wastewater, domestic sludge, agricultural and industrial wastewater and pollutants removal as well. Table 3.3 summarizes the main advantages and disadvantages of constructed wetlands.

Table 3.3: Main advantages and disadvantages of constructed wetlands.

Advantages	Disadvantages
• Low investment and running costs compared with conventional treatment systems. • It fits harmoniously into the land scape therefore its environmental sensitive approach is viewed in favour by the general public. • Resource recovery. • Decentralized wastewater treatment. • More area requirement compared with conventional systems. • Lower cost of construction, operation and maintenance. • Does not require energy use. • Does not require qualified personnel. • Good performance in terms of organic. • matter removal, total suspended solids, nitrogen and phosphorus. • Constructed wetlands do not produce sludge. • Constructed wetlands could be combined in order to achieve a higher treatment efficiency. • Higher biomass productivity which can be used as food for animals. • Does not require the addition of chemical products and mechanical equipment • Promotion of green areas, natural habitats and recreational areas. • Great potential for its applicability in developing countries, peri urban and urban areas, rural and small communities. • Can be built and repaired with locally available materials.	• May facilitate mosquito breeding. • In some cases as Horizontal Subsurface Flow Constructed wetlands pre-treatment is required to prevent clogging. • CW requires expert design and supervision and dosing system requires more complex engineering. • Efficiencies are according to climate conditions. • Occurrence of clogging. • The biological components are sensitive to toxic chemicals like ammonia and pesticides. • The accomplished coliform removal is not always sufficient, so that it may require subsequent disinfection process. • In colder climates, in winter time, the rate of removal of BOD, NH_3 and NO_3 is reduced.

Source: Vymazal (2010); Tilley *et al.* (2014); Koottatep *et al.* (2005)

By observing Table 3.3 it can be concluded that constructed wetlands offer a number of potential advantages. Due to their greater economic and social advantages when compared to disadvantages, this technology is widely accepted and implemented in practice for decentralized sanitation. In addition, it is characterized by more advantages than disadvantages and has been seen as promising and sustainable tool for wastewater treatment in developing countries.

Nowadays, constructed wetlands have been used for domestic wastewater treatment and for sludge dewatering. The category of constructed wetlands used for sludge dewatering are also called planted drying beds, vertical flow constructed wetlands or sludge drying reed beds. Planted drying beds have been increasing in use in Europe for dewatering and stabilization of wastewater sludge over the last 30 years (Uggetti *et al.* 2010 & Sonko *et al*. 2014). Moreover, this system has gained acceptance and applicability in different regions of the world including Asia and Africa, treating sludge from onsite sanitation technologies (Strande *et al.* 2014).

3.4.3 Classification of constructed wetlands

There are several types of constructed wetlands, which could be distinguished according to several criteria such as presence/absence of free water surface, macrophytes used or direction of flow (Vymazal, 2008). For the purpose of this study, the classification of wetland is based on direction of flow. Different types of constructed wetlands may be combined with each other (so-called hybrid systems) in order to exploit the specific advantages of the different systems (Hoffmann and Platzer, 2010).

Classification of constructed wetlands according to the life form of the dominant macrophyte:

- Free-floating macrophyte-based systems;
- Submerged macrophyte based systems; and
- Rooted emergent macrophyte flow systems.

Classification of wetland according to the water regime and subsurface flow:

- Horizontal Subsurface flow systems;
- Vertical Subsurface flow systems; and
- Hybrid Systems.

Figure 3.2 shows different types of constructed wetlands proposed by Kadlec and Wallace (2008).

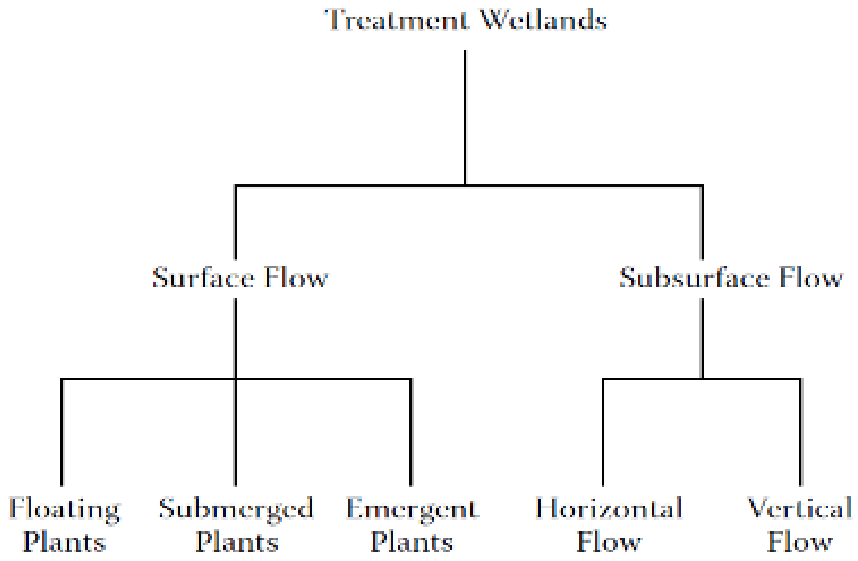


Figure 3.2: Treatment wetlands types.

Source: Kadlec and Wallace (2008)

Vymazal (2010) suggested a classification form which includes the presence of dominant macrophytes and different water flows. Figure 3.3 shows constructed wetland classifications as proposed by the author.

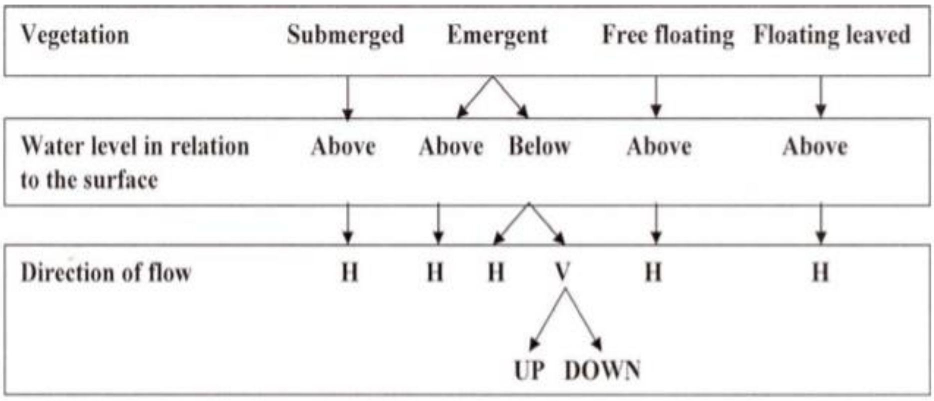


Figure 3.3: Basic elements of different types of Wetlands.

Source: Vymazal (2010)

The classification proposed by Vymazal (2010) is very simple and only considers the following three elements:

i) Type of vegetation;

ii) Water level in relation to the surface; and

iii) Direction of flow of water.

3.4.3.1 Free water surface constructed wetlands

The Free Water Surface (FWS) wetland technology started in North America with the ecological engineering of natural wetlands for wastewater treatment at the end of the 1960s and beginning of the 1970s (Vymazal, 2008). A typical FWS CW with emergent macrophytes is a shallow sealed basin or sequence of basins, containing 20-30 cm of rooting soil, with a water depth of 20-40 cm (Vymazal, 2010). FWS CW have areas of open water and are similar in appearance to natural marshes and these wetlands contain areas of open water, floating vegetation and emergent plants, either by design or as an unavoidable consequence of the design configuration (Kadlec and Wallace, 2008). The authors reinforced that the FWS wetlands are suitable in all climates and TSS removal are more effective under winter conditions than in summer conditions. Figure 3.4 is a simplified illustration of a Free Surface Constructed Wetland proposed by Kadlec and Wallace (2008).

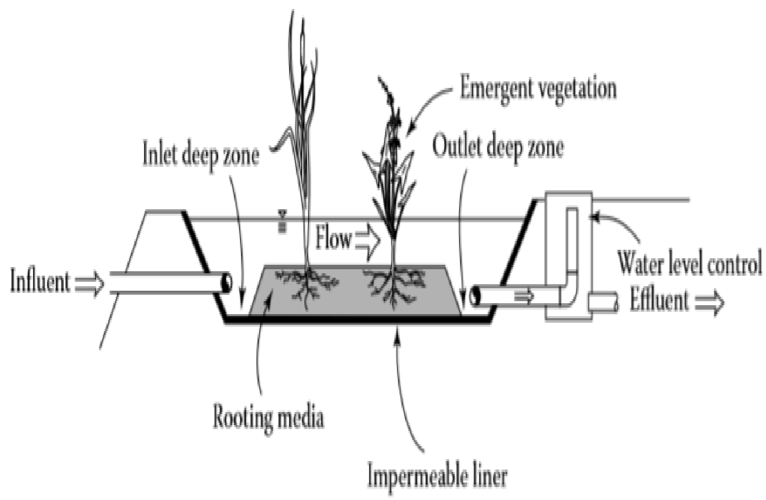


Figure 3.4: Schematic cross section of a FWS.
Source: Kadlec and Wallace (2008)

From Figure 3.4, it can be observed that FWS consists of the inlet of the influent and outlet of the effluent. Shown in Figure 3.4 is the water flow direction above the surface layer, the presence of emergent vegetation, the rooting media and the presence of impermeable liner. The flow is directed into a cell along a line comprising the inlet. Many FWS are widely applied in different regions of the world treating municipal and industrial wastewaters.

3.4.3.2 Horizontal flow constructed wetlands

Horizontal Flow Constructed Wetlands (HFCW) consists of gravel or soil beds planted with wetland vegetation. They are typically designed to treat primary effluent prior to either soil dispersal or surface water discharge (Kadlec and Wallace, 2008). In horizontal wetlands the wastewater flows slowly through the porous medium under the surface of the bed in a more or less of a horizontal path until it reaches the outlet zone (Hoffmann and Platzer, 2010). According to Kadlec and Wallace (2008), in this system the wastewater is expected to stay beneath the surface of the media and flow in and around the roots and rhizomes of the plants, and they are commonly used for secondary treatment for single-family homes or small cluster systems (Wallace and Knight, 2006; Kadlec and Wallace, 2008). Figure 3.5 shows the schematic of the cross-section of a Horizontal Flow Constructed Wetland.

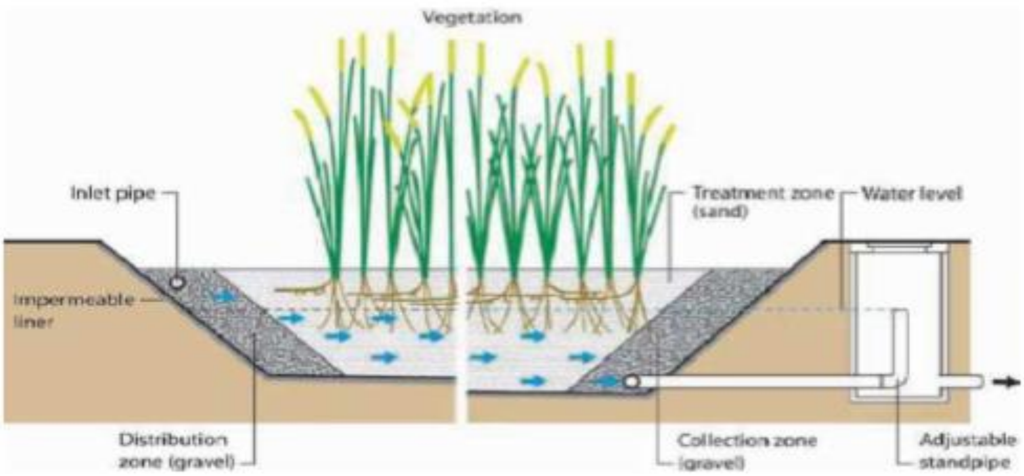


Figure 3.5: Basic elements of Horizontal Flow Constructed Wetlands.
Source: Hoffmann and Platzer (2010)

By observing Figure 3.5, it can be seen that wastewater is fed in the inlet and flows along beneath the surface, crossing the porous media following a horizontal direction. The basic elements of the system are constituted by vegetation, sand and gravel zones, outlet pipe and collection zone. Presently, technology of wastewater treatment with Horizontal Flow Constructed Wetlands represents the most commonly used around the world, (Vymazal, 2008). Comparing this system with FWS, the HFCW is smaller than FWS, the water flow is beneath the surface and the FWS is characterized by shallow water depths.

3.4.3.3 Vertical flow constructed wetlands: the French system

Vertical flow constructed wetlands can adequately treat raw domestic sewage in several regions of the world, including developing countries. The French/Cemagref systems are known to be efficient and economic, since they are able to receive raw sewage and require no sludge disposal over more than ten years of operation (Molle *et al.*, 2005). These systems are efficient for wastewater treatment and are expected to properly treat domestic wastewater and sludge from septic tanks, thus being compatible with various Brazilian and African communities. Hoffmann and Platzer (2010) reported that constructed wetlands can be used as part of a decentralized wastewater treatment system, due to their characteristics of being “robust” and “low-tech” systems with no moving parts (pumps) and relatively low operational requirements.

Vertical flow constructed wetlands treating domestic sewage require less land than horizontal units, but require more operation and maintenance. On the other hand, the systems are an effective alternative for the treatment of septic effluents in small villages. Compared with conventional treatment systems such as, activated sludge and disinfection process, wetlands can be installed in the same place where the wastewater is produced and can be maintained by untrained personnel, have relatively low energy requirements and all in all are considered low cost systems (Ciria, 2005). Comparing conventional activated sludge and subsurface flow constructed wetlands, Stovall (2007) revealed that natural systems are thought to be inferior to conventional systems, but the naturally treated water quality is equal to or better than those of the systems.

Several authors like Molle *et al*. (2005) and Platzer and Hoffmann (2011) found high removal efficiencies for vertical flow constructed wetland application in terms of organic matter and other contaminants treating domestic wastewater.

The use of VCFW to treat wastewater has increased greatly in France, Asia and Africa. The system are among the most suitable technologies for wastewater treatment in small-decentralized settlements. Molle (2014) reported that currently more than 2.500 plants are in operation for the treatment of domestic wastewater. Concerning sludge drying reed beds about 500 systems are in operation. This shows that VFCW are sustainable and important low cost sanitation technology for decentralized sanitation and wastewater treatment. Nevertheless, the vertical flow wetlands can be adequate for different social and economic conditions.

VCFW with vertical flow were originally designed by Seidel as pre-treatment units before wastewater treatment in horizontal flow beds (Vymazal, 2008). This type of system functions at intermittent loading. In vertical flow constructed wetlands, wastewater is intermittently pumped onto the surface and then drains vertically down through the filter layer towards a drainage system at the bottom (Hoffmann and Platzer, 2010). Besides, Vymazal (2008) argues that this kind of feeding provides good oxygen transfer and hence the ability to nitrify (Cooper *et al*., 2008; Vymazal, 2008; Kadlec and Wallace, 2008).

Koottatep *et al*. (2008) emphasized that the performance of vertical flow constructed wetlands as part of a decentralised wastewater system is due to their characteristics of low operational requirements and because they can adequately treat raw domestic sewage in several regions of the world, including developing countries. Vertical flow constructed wetlands have proven to be an effective treatment system while typically providing a good removal of organic matter and suspended solids, but usually providing nitrogen removal (Kadlec and Wallace, 2008).

The most common vertical flow constructed wetland system is described by Molle *et al.* (2005), called the French system. According to Hoffmann *et al.* (2008) the system comprises of two-stages. The first stage consists of three parallel alternating units of a vertical flow bed filled with gravel and designed for treatment of raw wastewater; while the second stage is a vertical flow bed, which can be used to operate in parallel in order to rest one of the filters or to alternate the operation. The operation works through pulse loading (intermittent feeding). Figure 3.6 shows the schematic of the cross-section of a Vertical Flow Constructed Wetland.

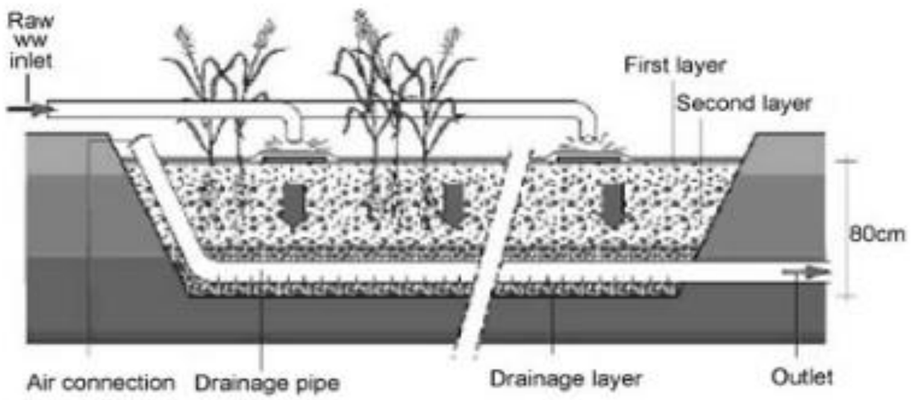


Figure 3.6: Treatment of raw wastewater in the first stage of the French System.

Source: Molle *et al.* (2005)

For vertical flow constructed wetlands (French system), Molle *et al.* (2005) reported that for high loadings, the gravel bed removed on average 90% of COD (Chemical Oxygen Demand), 96% of TSS (Total Suspended Solids) and 85% of TKN (Total Kjeldahl Nitrogen), which is more efficient than several other treatment systems. The French system has great potential for domestic wastewater treatment plants and can be useful in Sub-Saharan African countries like Mozambique. Their configuration traditionally encompasses two stages in series (three parallel units in the first stage and two units in the second stage), but it is expected that most conversion processes can take place in the first stage in warm-climate regions, thus not requiring the second stage, what can bring about considerable savings. By applying only the first stage, land requirements are approximately only 60% of those for the full two-stage French system.

Molle (2012) reinforces the need for seeking different configurations in order to reduce costs. In this context, one alternative could be having only two units (instead of three) in the first stage (without the second stage), which could lead to a land requirement of approximately only 40% of the full French system, resulting in only around 0.6 to 0.7 m^2/inhabitant.

Research done by Koottatep *et al.* (2008) about the potential of vertical flow constructed wetlands for sludge treatment in a tropical region showed satisfactory performance with respect to total suspended solids (TSS), total chemical oxygen demand (TCOD) and total Kjeldahl nitrogen (TKN) removal efficiencies of 80%, 96% and 92% respectively. Probably, considering the first stage of the French system the results can be satisfactory for the usual requirements of most developing countries. Therefore, in tropical regions this system is mainly used for sludge and wastewater treatment.

3.5 Planted sludge drying reed beds

From the literature review, it was observed that many studies about constructed wetlands are related with domestic wastewater rather than faecal sludge treatment. Recently, there is a growing interest in the use of vegetated sludge drying beds with macrophytes, also known as planted sludge drying beds, planted sludge reed beds or vertical flow constructed wetlands. They are a cost-effective and technically feasible approach for sludge dewatering, stabilization and humification (Koottatep *et al*., 2005; Kengne *et al*., 2011). The authors reinforced that sludge from septic tank can be treated using vegetated beds with macrophytes, thus achieving high removal efficiency.

Planted drying reed beds for sludge treatment might be a powerful tool to rise the quality of life of local communities in developing countries (AWEP, 1999). However, the use of these systems are not common in developing countries. Nevertheless, authors like AWEP (1999) revealed reasons for the relatively slow spread of this variant of VFCW:

- Experts from industrialised countries are sometimes unable to transfer their conceptual thinking to the realities and cultures of the developing countries;
- There has been a tendency to translate northern technologies to tropical environments instead of assisting developing countries to develop their technologies; and
- Experts from developed countries are largely educated in the conventional technologies and have only limited access to information and knowledge on new technologies.

In order to promote constructed wetlands technologies treating domestic wastewater and faecal sludge, it is essential to know the advantages and disadvantages concerning the implementation of these systems. The vertical flow constructed wetlands to treat domestic sewage and planted drying beds treating sludge have large advantages. Table 3.4 presents the advantages and disadvantages of the VFCW treating domestic wastewater as well as advantages and disadvantages of VFCW or planted drying beds treating faecal sludge.

By observing Table 3.4 it is possible to conclude that these options of decentralised domestic wastewater and sludge treatment constitute a sustainable tool for low cost sanitation treatment and appropriate for different realities.

Table 3.4: Main advantages and disadvantages of vertical flow constructed wetlands and planted sludge drying reed beds.

Vertical Flow Constructed Wetlands treating domestic sewage	
Advantages	**Disadvantages**
• Lower cost of construction, operation and maintenance. • Does not require energy use. • Does not require qualified personnel. • High reduction in BOD, suspended solids and pathogens. • Requires less space than a Free-Water Surface constructed wetland. • Does not have the mosquito problems of the Free-Water Surface constructed Wetland. • Less clogging than in a Horizontal Flow Constructed Wetland.	• Social and cultural issues may be a weakness in the utilization of the system. • Not all parts and materials may be available locally. • Dosing regimen requires more complex engineering.
Planted Sludge Drying Beds	
Advantages	**Disadvantages**
• Low investment. • Minimized sludge removal costs. • Simplicity and economy. • Small volumes of well-composed bio solids. • Low maintenance, construction and operation. • Very low power consumption (just for feed pumps). • A simple, robust and reliable method. • Silent operation. • Few odours. • Infrequent attention required. • High-solids content in the dried product. • Final product can be safely used for agricultural purposes.	• Large space required. • Insects and potential odours. • Longer period of bed adaptation. • Requires more studies about operation and maintenance. • Effect of climate changes on drying characteristics.

Source: Adapted from Vymazal (2010); EAWAG, (2008); Koottatep *et al.* (2005), Melidis (2009), Stefanakis and Tsihrintzis (2009). Uggetti (2009); Metcalf & Eddy (2003), Brix (2014)

Various studies about VFCW treating wastewater or septic tank sludge have successfully appointed these systems as very suitable technologies for wastewater treatment in small-decentralized settlements, due to their easy adaptation, operation and maintenance costs, and limited generation of by-products (Gkika *et al.*, 2014). Therefore, it is a technology with great acceptance by local communities and authorities.

In recent years, the system is largely used in developing countries for domestic wastewater and septic tank treatment. The French system of VFCW takes advantage of raw domestic wastewater treatment comparing with others. One of the considered advantages is that the VFCW requires less land as compared to horizontal flow systems (Kadlec and Wallace, 2008).

3.6 Operational regimen of VFCW

Research carried out by Lana (2013), evaluating the pollutants removal of the same VFCW units, showed a substantial removal efficiency of organic matter, total suspended solids and others pollutants, farther the operational regimen adopted allowed the nitrification and denitrification process.

Three aspects determine the performance of constructed wetlands, the first is related with the constructive aspects, which include the design, the dimensions and the employed material, the second aspect is concerning the characteristics of the sewage and the third is the operational parameters (Pidre *et al.,* 2010).

In terms of domestic wastewater treatment, several authors have successfully developed studies about operational studies. Certainly, authors have seen that the operational regime for VFCW is one of the imperative aspect that affect the system performance in terms of pollutant removal (Tang and Jong, 2014; Molle *et al.*, 2005). Therefore, different feeding strategies can affect the overall behaviour of the system and define at what point the system is operating efficiently. Authors have demonstrated that there is a difference in the system performance when various hydraulic loading rates, solids loading rates and frequencies of applications are applied.

Molle *et al.* (2006) reported about the effect of the VFCW and feeding operations under hydraulic overloading. Indeed, studying hydraulic loads in the VFCW, the author observed that higher hydraulic loading, with less loading times, and longer periods allowed for renewal of dissolved oxygen, leading to better efficiency of the system. However, for the same hydraulic load, it is crucial the choice between volume or batches feeding which influence the process of oxygen diffusion in the system. Table 3.5 presents different operational strategies for VFCW applied to treat domestic wastewater in various regions of the world.

Table 3.5: Different operational regimen of VFCW treating domestic sewage.

Type of wastewater	Type of System	Country	HLR ($m^3.m^{-2}.d^{-1}$)	Frequency (Feeding and resting/week)	Inflow ($m^3.d^{-1}$)	Per capita Surface Area ($m^2.hab^{-1}$)	References
Domestic Wastewater	VFCW-French System	Brazil	0.38	2.5 days feeding and 4.5 days resting	11	0.9	Moraes (2013)
Domestic Wastewater	VFCW-French System	Brazil	0.45	7 days feeding and 7 days resting	13	0.6	Lana (2014)
Domestic Wastewater	Pilot VFCW	Brazil	0.04, 0.08, 0.15	3 days feeding and 2 days resting	0.04	1	Silva (2007)
Domestic Wastewater	Pilot VFCW-French System	Malaysia	0.0875, 0.0175	For and eight daily dosing			Tang and Jong (2014)
Domestic Wastewater	VFCW-French System	French	0.1875, 0.400	24-hour flow-composite samplings	12.5	1.2 for 1st stage and 0.8 for 2nd Stage	Molle *et al.* (2005) & Boutin *et al.* (2010)
Municipal Wastewater	VFCW-French System	Nepal	0.2, 0.08, 0.04	1, 2.5 and 5 days (Theoretical retention time)			Pandey *et al.* (2010)
Domestic wastewater	VFCW	Denmark		8-12 pulses/day when half of the effluent is recirculated 16-24 pulses/day	Without recirculation (0.16-0.24) With recirculation (0.32-0.48)	3.2	Brix and Arias (2005)
Municipal Wastewater	VFCW- French System	Spain	0.045, 0.046	Hydraulic retention time 3.9	14.3, 13.3		Pidre *et al*. (2010)
Domestic Wastewater	Pilot VFCW-French System	Turkish	0.100		3	0.5	Korkusuz *et al.* (2004)
Waste Stabilization Pond wastewater effluent	VFCW	France	0.15- 0.20 0.35 – 0.40 0.75 – 0.80	3-4 days feeding and 7 days resting			Torrens *et al.* (2009)

As it can be seen in Table 3.5, there are different strategies used in the VFCW to treat domestic wastewater. The criteria of selection of the appropriated strategy is related with population equivalent, area, social and local conditions, among others. The feeding strategies used by Lana (2013) and Pidre *et al.* (2010) were the same as used in this research. In order to reduce the area, the cost and find out the optimal point of operation, authors like Molle *et al.* (2014) suggested implementation of different configurations of VFCW. Thus, more studies about operational strategies may be implemented in order to find out the optimal point that allows satisfactory operation of the system.

Researchers have carried out several experiments using Vertical Flow Constructed Wetlands in order to find out the best configuration and operational strategy. In some cases, there is no consensus about optimal operational strategy that allows for satisfactory operation of the system in terms of septic tank treatment. Authors like Jong and Tang (2014) believe that the operational regime for constructed wetlands is one of the imperative aspects that influence the system performance in terms of pollutant removal. From this analysis, various operational strategies can be implemented in VFCW. The operational strategy to be implemented depends on the design, the characteristics of the sewage, the size of the filters and equivalent population in a particular location.

In recent years, VFCW treating sewage has been used to evaluate the effect of hydraulic loading rates in terms of removal efficiencies for pollutants. The results have shown that the percentage of removal efficiencies of impurities decreased with an increase in hydraulic loading rates. This is associated with less time of oxygen diffusion, lesser time for various removal process as sedimentation, filtration and adsorption to act effectively (Molle *et al.,* 2006; Pandey *et al.,* 2010). Thereby, in VFCW one of the most important factors of removal efficiencies of the pollutants is oxygen transfer. Thus, it is crucial to maintain a sufficient flow of oxygen in the system through the control of hydraulic loading rates and frequencies of feeding and resting.

One of the best characteristics of the VFCW is its capacity to remove organic matter and nitrify the filter influent (Molle *et al.,* 2006; Torrens *et al.,* 2009). Thereby, the authors reinforce that to achieve efficient nitrification, the filters should function in full aerobic conditions and these aerobic conditions exist when the filter medium is not saturated, therefore a good drainage on the system is necessary, which requires a batch feed system and distribution network. Thus, it is important to find out the best feeding and resting frequencies. Torrens *et al.* (2009), studying the impact of design and operation variables of VFCW, concluded that the dosing regime and resting period duration affect the hydraulic performance and purification efficiency of the filters.

Potential risk of clogging is an important aspect of VFCW (Hoffmann and Platzer, 2010). However, in designing these systems, clogging and soil aeration must be considered. Hoffmann and Platzer (2010) recommend the following design aspects concerning a dose regimen:

- Wastewater needs to be pumped onto the VFCW intermittently about 4-12 times daily.
- A uniform distribution of the wastewater is required.

3.7 Vertical flow constructed wetlands treating septic tank sludge

Vertical flow constructed wetlands also known as sludge drying reed beds (SDRB) or planted sludge drying beds have become the main systems implemented for treating sludge from septic tank, mainly in developed countries. Sludge drying reed bed resemble vertical flow constructed wetlands where sludge is introduced periodically and becomes dewatered by percolation, transpiration and evaporation (Melidis *et al.,* 2009). The literature review mention that there are many systems of SDRBs in operation as a cost-effective and feasible approach for sludge dewatering, stabilization and mineralization (Kengne *et al.,* 2010).

Molle (2014) pointed out that about 500 systems of SDRBs from 500 to 27.000 person equivalents are in operation nowadays. The author reinforced that these systems (SDRBs) are implemented to treat sludge from activated sludge systems, primary sludge and sludge from emptying septic tank. Undoubtedly, this low cost technology becomes a sustainable and important low cost sanitation technology for decentralized sanitation and wastewater treatment in various parts of the world.

According to Kengne *et al.* (2014), the treatment effectiveness and plant growth using VFCW depends on the hydraulic loading rate. The Hydraulic Loading Rate ($m^3.m^{-2}.d^{-1}$) expresses the relation between inflow rate and the surface area. This is an important parameter for VFCW dimensioning. However, the operational regime associated the hydraulic loading rate, sludge loading rate, frequency of sludge application, the size of VFCW or SDRBs can affect the performance in terms of pollutant removal, dewatering, stabilization and mineralization of sludge (Kengne *et al.,* 2014; Jong and Tang, 2014; Vicent *et al.* 2010). Molle *et al.* (2006) demonstrated that at high hydraulic loads and high volumes of batch feeding, associated with long resting periods is good for oxygen diffusion.

3.7.1 Solids loading rate

The mass of solids applied in the VFCW or reed beds determines the performance of the system in terms of sludge dewatering and mineralization. Solids loading rate represents the mass of solids (kg of dry matter) applied on one m^2 of bed per year (Strande *et al*., 2014). Literature review have pointed out that several factors can influence sludge drying reed bed efficiency such as solids loading rate, loading strategies, characteristics of the sludge, climate, number of units, among others (Vicent *et al.,* 2010). Table 3.6 presents a summary of various experiences of the sludge reed beds or vertical flow constructed wetlands including solids loading rate, frequency of application, type of sludge and type of system studied in different regions of the world.

3.7.2 Frequency of faecal sludge application

The frequency of faecal sludge application represents the interval between pulse loadings of sludge on the surface. This factor can influence the performance of the system in terms of dewatering, stabilization and mineralization. The dewatering, stabilization and mineralization of the sludge in the SDRB depends on a variety of factors such as the media type and size, the type of plants, the maturity of the beds, climatic factors and sludge characteristics, as well as operational factors. (Strande *et al.,* 2014). Molle *et al.* (2006) found out that long resting periods allow for oxygen renewal.

According to the studies carried out by Koottatep *et al.* (1999) in Bangkok, parameters of constructed wetlands treating sludge from septic tank were suggested. The design parameters include a septage production rate ranging from 0.7 – 1.0 L $person^{-1}$ day^{-1}, TS content ranging from 8.000 – 18.000 mg L^{-1}, solids loading rate ranging from 125 – 250 kg TS m^{-2}.$year^{-1}$, the septage application frequency of one or two times per week and percolating ponding period of two up to six days. However, literature review addresses various operational strategies used in VFCW to treat septic tank sludge in several parts of the world. Table 3.6 shows different operational strategies of constructed wetlands treating ST sludge, taking in account the geographic location.

Table 3.6: Different operational strategies of STRB treating septic tank and sewage treatment excess sludge.

Type of sludge	**Type of System**	**Country**	**HLR (m^3 $m^{-2}.d^{-1}$)**	**SLR (kg TS m^{-2} $.year^{-1}$**	**Frequency of FS application**	**Inflow (m^3 d^{-1})**	**Per capita surface area (m^2 hab^{-1})**	**References**
Raw FS	Pilot-scale VF CW beds	Thailand		80 - 125 – 250	Once-a-week	8		Koottatep *et al.* (2005)
Septic Tank Sludge	Pilot-scale VF CW beds	Brazil		80 – 250	Once a week			Philippi *et al.* (2013)
Raw FS	VFCW	Cameroon		100 - 200 – 300	Once-a-week			Kengne *et al.* (2011)
Raw FS	Reed bed pilot plant	Italy		50	3 days sludge application. Interval between 15 days			Bianchi *et al.* (2010)
Excess sludge from conventional activated sludge	Reed drying beds	Italy		67	2 weeks from autumn to spring and every week during summer			Peruzzi *et al.* (2010)
Excess sludge from activated sludge plant	Sludge treatment reed bed	Denmark		30 – 50	5 – 7 days feeding 40 – 50 days resting	100 – 150		Nielsen *et al.* (2014)
Sludge from septic tank	Pilot-scale planted drying beds	Senegal		200	1X, 2X and 3X times per week			Sonko *et al.* (2014)
Sludge from septic tank	Pilot and full-scale PDBs	Senegal		50 – 200	1 – 2 times weekly			Dodane *et al.* (2011)
Raw septage	Pilot-scale VFCW	Thailand		140 – 360		8		Koottatep *et al.* (2005)
Sludge from septic tank	Pilot-scale planted drying beds	Senegal		200	1X, 2X and 3X times per week			Sonko *et al.* (2014)
Sludge from septic tank	Pilot-scale planted drying beds	Senegal		200	1X, 2X and 3X times per week			Sonko *et al.* (2014)
Sludge from septic tank	Pilot and full-scale PDBs	Senegal		50 – 200	1 – 2 times weekly			Dodane *et al.* (2011)
Raw septage	Pilot-scale VFCW	Thailand		140 – 360		8		Koottatep *et al.* (2005)

Activated sludge		France		50	Feeding/resting period 3.5/17.5			Liénard *et al.* (2008)
Activated sludge	Pilot-scale VFCW	France		30 – 40	5 days feeding/24 days resting			Vicent *et al.* (2010)
Septage and Septage + aerated sludge	Pilot-scale beds	France		30	Feeding/resting period 3.5/17.5 and 3.5/31.5			Troesch *et al.* (2010)
Activated sludge	Pilot-scale beds	Greece		30 -60 - 75	7 days feeding and 21 days resting (winter); 7 days feeding and 14 days resting (summer)			Stefanakis and Tsihrintzis, (2009)
FS from septic tank + Public toilets	Pilot-scale VFCW	Cameroon	0.05, 0.1, 0.15	200	Once a week			Kengne *et al.* (2014)
Septic tank sludge	Full-Scale VFCW	Spain		51 – 55 - 125	2 days periods of feeding and 2 days period of resting and not a regular feeding pattern for 125 Kg/m2.yr	2, 3, 4.5	0.495, 0.54, 0.116	Uggetti *et al.* (2009)
Settled urban sludge	VFCW	Greece		284	20 days feeding by gravity	2.16		Melidis *et al.* (2009)

The data presented in Table 3.6 indicates that there are various operational strategies used for septage or sludge treatment. Nevertheless, it can be seen that several authors have carried out experiments with different design and operation in order to find out the best one. A literature review on the use of vertical flow constructed wetlands treating sludge or sludge drying reed beds for septage treatment shows that the operational strategies and sludge solid rates varies a lot according to the type of sludge, climate and social conditions. On the other hand, studies have showed good removal efficiencies in warmer climates than colder climates. Experiences show that in colder climates the temperature is a limiting condition, where the biological activity can be limited by temperature. However, in general, the social conditions define the willingness to pay or not for improved facility mainly in developing countries.

However, the source of sludge associated with climate conditions and the characteristics of the sludge generated are key aspects that enable the selection of the appropriate faecal sludge treatment option (Koné and Strauss, 2004; Troesh *et al.,* 2010; Vicent *et al.,* 2010; Koottatep *et al.,* 2005).

Few studies carried out by researchers have successfully illustrated that the geographical context defines the solids loading rate as shown in Table 3.6. For example in Cameroon, Kengne *et al.* (2014) found a solids loading rate of 200 Kg TS $m^{-2}.yr^{-1}$ with sludge application once a week as optimum for the type of septage treated. In addition, the authors demonstrated that VFCW can operate efficiently with Hydraulic Loading Rates (HLR) of 0.05, 0.1, 0.15 $m^3.m^{-2}$ d^{-1}. In Denmark, Nielsen *et al.* (2014) suggest a solids loading rate between 30 – 50 kg TS $m^{-2}.yr^{-1}$ with 5 – 7 days of feeding and 40 – 50 days of resting treating activated sludge.

Analysing data from Table 3.6, it can be observed that solids loading rates can be divided in two groups according to geographic context. Therefore, in terms of sludge treatment from activated sludge, it is possible to indicate solids loading rates ranging from 100 to 200 kg TS $m^{-2}.yr^{-1}$ in tropical regions and solids loading rates ranging from 30 to 50 kg TS $m^{-2}.yr^{-1}$ in temperate climates, like that in Europe. On the other hand, Dodane *et al.* (2011) recommend loading rates of 200 kg TS $m^{-2}.yr^{-1}$ in tropical regions, applied in a cycle of one day per week, with six days for percolation and drying. These findings are similar to those obtained by Vicente *et al.* (2010) and Troesch *et al.* (2010).

By observing the data presented in Table 3.6 it can be seen that several factors influence the solids loading rate applied in the systems. For example, in Greece, Melidis *et al.* (2009) suggested a solids loading rate of 284 Kg TS $m^{-2}.yr^{-1}$ applied in total area of 140 m^2 treating settled urban sludge, while Troesch *et al.* (2010) in France recommend 30 Kg TS $m^{-2}.yr^{-1}$ applied in total are of 2 m^2 at pilot scale treating septic tank sludge. In addition the results obtained by several authors, have showed that temperature, the characteristics of sludge and operational strategies determine solids loading rates applied in the system.

Taking into account the results obtained from the literature review about SDRBs and/or VFCW, it can be concluded that this technology is largely consolidated in some countries in Europe and Asia. In Brazil and other South American and African countries, the technology may be further implemented given its operational simplicity, low cost of construction, operation and maintenance and suitable for various local realities.

Regarding STRB, Brix (2014) commented that the dimension and design of the reed bed systems depend on the sludge production (tonnes of dry solids/year), sludge type, quality and regional climate. On the other hand, when the STRBs is correctly designed and operated, the sludge can reach a dry matter content of 20-30% and in optimal conditions up to 40% (Nielsen, 2007).

3.8 Role of plants in constructed wetlands

The plants are an integral part of constructed wetland systems; therefore, they play important roles in constructed wetlands for their capability of degrading and removing nutrients and other pollutants (Cui, 2008). Plants utilise nitrate, and ammonium and their decomposition processes releases nitrogen back to the water. According to Hoffmann and Platzer (2010), the plants also have an important role in the treatment process. They provide an appropriate environment for microbial growth and significantly improve the transfer of oxygen into the root zone, which is part of the filter bed.

Studies have shown satisfactory removal efficiency in terms of pollutants removal in planted units of constructed wetlands. However, many studies indicate that there is no statistical significant difference between planted and unplanted beds in terms of pollutant removal efficiencies. Thus, there is no doubt that the performance is higher when plants are present (Kadlec and Wallace, 2008). Haberl *et al.* (2003) mention that the most important effects of the plants in relation to the treatment process are physical effects. The authors reinforce that the roots provide surface area for attached microorganisms and maintain the hydraulic properties of the substrate. In terms of sludge treatment, Ingallinella *et al.* (2002) pointed out that the advantage of planted over unplanted sludge drying beds is that the root system of some plants like cattails creates a porous structure in the beds and thus enables them to maintain prolonged permeability of the filter body.

Brix (2014) commented that plants in the reed beds have several important functions. One of the functions is related with the root system and stem; these help keep the filter open so that water can drain from the sludge to the drainage system. Another important function of the plants is to remove capillary-bond water in the sludge by plant uptake and transpiration.

Regarding nutrients removal, the plants used in constructed wetland have a significant role. According to Hoffmann and Platzer (2010), plant growth can contribute to reducing nutrients and more importantly for nitrogen removal through nitrification/denitrification by bacteria. In vertical flow constructed wetland with sufficient oxygen supply, ammonium can be oxidised by autotrophic bacteria to nitrate (nitrification). VFCW which are meant to achieve nitrification have to be designed using oxygen consumption and soil aeration. Table 3.7 summarizes the roles of different parts of plants in constructed wetlands.

Table 3.7: Roles of plants in constructed wetlands.

Effects of root structure	Tissue of plants	Aerial part	Other functions of plants
• Filtering effect; • Velocity reduction; • Prevention of medium clogging; • Promotion of sedimentation; • Improved hydraulic conductivity; • Provision of surface of microbiological attachment; • Erosion prevention; • Oxygen release, organic matter degradation and nitrification; • Nutrients absorption; • Release of antibiotics; • Release of gas and exudates; • Excretion of carbon increase denitrification; • Aerobic dynamics; • Provide surface of microbial attachment.	• Storage and uptake of nutrients; • Metal phytoremediation; • Salt phytoremediation; • Microorganisms attachment; • Promotion of Filtering; • Gas exchange (O_2, CO_2, NO_2, H_2S).	• Aesthetic potential; • Storage of nutrients; • Attenuation of solar radiation=Is othermal effect.	• Light attenuation reduces algal growth; • Insulation from frost in the winter; • Insulation from radiation in the spring; • Reduces the wind velocity; • Stabilization of sediments surface; • Elimination of pathogens; • Insect and odour control; • Increase the wildlife diversity; • Wastewater gardens; • Bioindicators.

Source: Kadlec and Wallace (2008); Tanner (2001); Shelef *et al.* (2013)

Kadlec and Wallace (2008) pointed out that the wetland nitrogen cycle is very complex, and control of even the most basic chemical transformations of this element is a challenge in ecological engineering. However, the major comprehension of the nitrogen cycle in wetlands has great ecological and environmental importance as one of the principal constituents in wastewater and faecal sludge treatment because of their role in eutrophication. On the other hand, Metcalf and Eddy (2003) reported that the chemistry of nitrogen is complex, because of the several oxidation states that nitrogen can assume and the fact that changes in the oxidation state can be brought about by living organisms. Nitrogen removal is often required before discharging treated wastewater to sensitive water bodies in order to prevent eutrophication.

The most important inorganic forms of nitrogen in wetlands treating municipal or domestic wastewater are ammonium (NH_4^+-N), nitrite (NO_2^-), nitrate (NO_3^-), nitrous oxide (N_2O), and dissolved elemental nitrogen or dinitrogen gas (N_2) (Kadlec and Wallace, 2008). This type of system has aerated zones, especially near the water surface because of atmospheric diffusion, and anoxic and anaerobic zones in and near sediments.

All biological nitrogen-removal processes include an aerobic zone in which biological nitrification occurs (Metcalf and Eddy, 2003). Authors like Hoffmann and Platzer (2010) observed that in vertical flow constructed wetlands with sufficient oxygen supply, ammonia can be oxidised by autotrophic bacteria to nitrate (nitrification), but unfortunately, the process of denitrification is limited.

According to Kadlec and Wallace (2008), many vertical flow wetlands are designed with the express purpose of oxidising organic and ammonia nitrogen. In terms of nitrogen removal, Koottatep *et al*. (2008) observed highest nitrogen removal efficiency at the solids loading rate of 250 Kg TS $m^{-2}.yr^{-1}$. On the other hand, the authors observed that the variation of solids loading rate subjecting to the N_2 loading did not cause any adverse effect of TKN and NH_4^+-N removal and vertical constructed wetland having longest ponding periods of six days, achieved the highest TKN and NH_4 removal.

Vertical flow constructed wetlands typically provide a good removal of organics and suspended solids, but these systems typically provide little denitrification. Consequently, removal of total nitrogen in these systems is limited (Kadlec and Wallace, 2008).
Macrophytes are plants that grow in aquatic conditions and can be used in constructed wetlands for wastewater treatment. Brix (1997) define macrophyte as larger aquatic plants growing in wetlands. There are 3 types of macrophytes and Silva (2012) describe diversity of plant species that can occur in each type of macrophyte:

Emergent: *Cynodon dactylon Pers, Typha angustifolia, Typha latifolia, Juncus spp, Phragmites spp, Schoenoplectus validus, Carex spp, Scirpus lacustris.*

Free Floating: *Eichhornia crassipes, Lemna, Hidrodictyon, Reticulatum l, Scenedesmus acuminatus, Salvinia molesta, Pistia stratiotes.*

Submerged: *Elodea nutali, Egéria densa, Elodea Canadensis Ceratophyllum demersum, Hydrilla verticillata, Cabomba Caroliniana, Miriophyllum hetrophyllum hetrophyllum, Paramogeton spp*

Several authors indicate the principal characteristics of macrophytes used in constructed wetlands (Hoffmann and Platzer, 2010). According to them the macrophyte that is to be used in constructed wetlands should have the following characteristics:

- Local indigenous species;
- Plant with an extensive root and rhizome system;
- Plant species which grow in natural wetlands;
- Fast growing under diverse conditions;
- High transpiration capacity;
- Tolerance to different water levels and drought conditions;
- Tolerance to extreme of pH and salinity;
- Deep growing rhizome and root system; and
- Ability to build new roots on the nodes when they become encased in sludge; and readily available, indigenous and non-invasive.

Table 3.8 summarizes plants species that can be used in constructed wetlands for raw domestic sewage and ST sludge treatment.

Table 3.8: Plants species for raw domestic sewage and ST sludge treatment in CW.

Plant	**Local of use**	**Type of Wetlands**	**References**
Cynodon dactylon Pers	Brazil	VFB and HFB	Matos *et al.* (2010); Lana, (2014); Matos (2010)
Chrysopogon zizanioides		VFB	Hoffmann and Platzer, (2010)
Typha angustifolia	Thailand; Brazil; Tropical regions	VFB	Koottatep *et al.* (2005); EAWAG (2008)
Typha latifolia	South America, Africa, India and East Asia; Spain	HFB	Hoffmann and Platzer, (2010); Ciria *et al.* (2005)
Phragmites australis	Europe	Planted Drying Beds VFB; HFB	Barbera *et al.,* (2009); Vymazal, (2010); Brix and Arias, (2005); EAWAG (2008)

Typha sp.	Europe	Planted Drying Beds	IWA (2005)
Cyperus papyrus	South America, Africa, India and East Asia	VFB	Hoffmann and Platzer, (2010) Strande *et al.* (2014); Kengne *et al.* (2011)
Pennisetum purpureum	South America, Africa, India and East Asia	HFB	Hoffmann and Platzer, (2010) Matos *et al.* (2010)
Echinochloa pyramidalis	Cameroon; Senegal	VFB; Planted Drying Beds	Kengne *et al.* (2011); Sonko *et al.* (2014); EAWAG (2008)
C. albostriatus	South America, Africa, India and East Asia		Hoffmann and Platzer (2010)
Cyperus alternifolius	South America, Africa, India; China and East Asia	VFCW	Hoffmann and Platzer (2010) Cui *et al.* (2009)
C. haspens	South America, Africa, India and East Asia		Hoffmann and Platzer (2010)
Scripus grosuss		HFB	
Phalaris arundinace	Czech Republic	HFB	Vymazal (2010)
Iris pseudacorus	Czech Republic	HFB	Vymazal (2010)
Bassia indica			Shelef *et al.* (2013)
Canna indica	China	VFB	Cui *et al.* (2010)

Undoubtedly, observing Table 3.8 a wide application of different plant species can be seen regarding the treatment of domestic wastewater and sludge using constructed wetlands. Moreover, several authors like Shelef *et al.* (2013) pointed out that the mechanisms by which macrophytes affect water treatment in Constructed Wetlands are still being debated. Several factors contribute for good performance of plants in constructed wetlands: the type of wetland, quality and quantity of the wastewater loads, climate, plant species, plant management, harvesting time; medium type, among others (Shelef *et al.* 2013).

For this research, the vertical flow constructed wetlands were planted with emergent tropical macrophyte called *Cynodon dactylon pers or* Tifton 85 as it is called in Brazil. Therefore, the characteristics of the *Cynodon dactylon pers* are described below.

Nowadays forage species such as *Cynodon dactylon pers* are used for wastewater treatment. *Cynodon dactylon pers* is a tropical plant with important forage characteristics, as the capacity to uptake nutrients from wastewater and produce high amounts of forage. In terms of wastewater treatment, the plant *Cynodon dactylon pers* has showed very good adaptability to different hydraulic loads and good biomass productivity in constructed wetland treating effluent (Matos *et al.,* 2008).

Cynodon dactylon pers was developed by Burton *et al.* (1993) in the Coastal Plain Experiment Station (USDA-University of Georgia) in Tifton, south of Georgia State (Oliveira *et al.,* 2000). The authors mentioned that Tifton 85 was obtained as a result of a crossing between Tifton 68 and South African introduction (PI 290884). Experiments conducted in the United States indicated that Tifton 85 had higher potential of dry matter of the high digestibility. *Cynodon dactylon pers* is widely cultivated in warm climates. It is also found in the United States mostly in the southern half of the country and in warm climates.

Lana (2012), in the same units of the VFCW, evaluated the capacity of growth of the plant Tifton 85 planted in two units of the VFCW. Two operational modes with feeding and resting periods were evaluated. The plant Tifton 85 showed very good adaptability. The average value of biomass production observed in both phases was 2.9 kg of dry matter $m^{-2}.year^{-1}$ = 29 ton dry matter $ha^{-1}.year^{-1}$. Matos *et al.* (2009) observed good adaptability of Tifton 85 when cultivated from a horizontal flow constructed wetland operated with continuous loadings treating dairy effluent. The grass showed good rooting, high productivity, and a very good capacity for nutrient removal (N, P and K) and Na from dairy effluent.

4 MATERIAL AND METHODS

4.1 Treatment system - Description of the experimental site

The study was conducted at the Centre for Research and Training in Sanitation (CePTS) of the Federal University of Minas Gerais (UFMG) and the Water and Sanitation Company of Minas Gerais (COPASA), in Belo Horizonte, Brazil. Belo Horizonte is the capital and largest city in the Brazilian State of Minas Gerais, located in the south-eastern region of the country. CePTS is located in the Arrudas Wastewater Plant Treatment (WWTP), one of the biggest WWTP of the country, on latitude 19°53' S, longitude 43°52' W, altitude 852 m, in Cfa or Cwa humid subtropical climate according to the Köppen classification. The area of COPASA and CePTS is shown in Figure 4.1 with a special detail on the system investigated.

Figure 4.1: Localization of COPASA and VFCW in the CEPTS

Source: Adapted from http://wikimapia.org

The vertical flow constructed wetland system was built in 2007 to treat domestic wastewater and was designed for a population of 100 inhabitants (around 1 m^2 hab^{-1}) according to the first stage of vertical flow constructed wetlands (VFCW) of the French system configuration (Cemagref/Irstea), with a mean design flow of 13 m^3 d^{-1}. The start-up of the system was carried out from July to December 2009, and operation started in January 2010.

The VFCW consists of three units in parallel, each with 3.1 m in width and 9.4 m in length. The total area of each unit is 29.1 m^2. The unit has 0.30 m of free board in the top layer. The VFCW units are filled with a media filter with three layers, a total height of 0.70 m (fine, medium and coarse gravel as seen in figure), porosity of 40% and were planted with *Cynodon dactylon pers.* A view of schematic cross section of each unit in VFCW is shown in Figure 4.2.

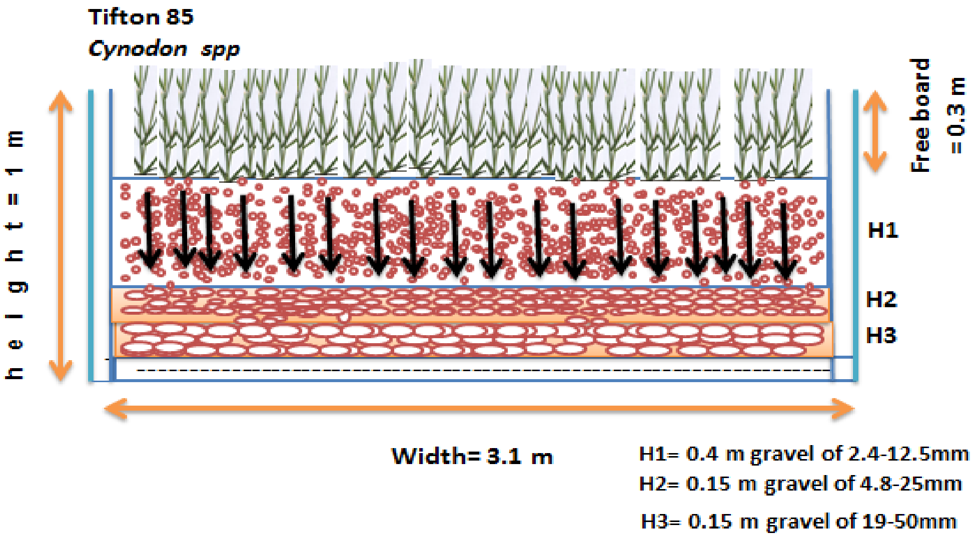


Figure 4.2: View of schematic cross section of each unit.

During 2007 until October 2013, the system was operated according to the first stage of the French system, but with two planted beds and one unplanted bed, as shown in Figure 4.3. In each unit of the VFCW, two ventilation pipes were installed, totalling 6 for the system. PVC tubes with 14.5 cm in diameter constitute the ventilation pipes. Figure 4.3 shows two planted units and one control unit of the VFCW and Figure 4.4 shows a view schematic of the two planted and one unplanted units.

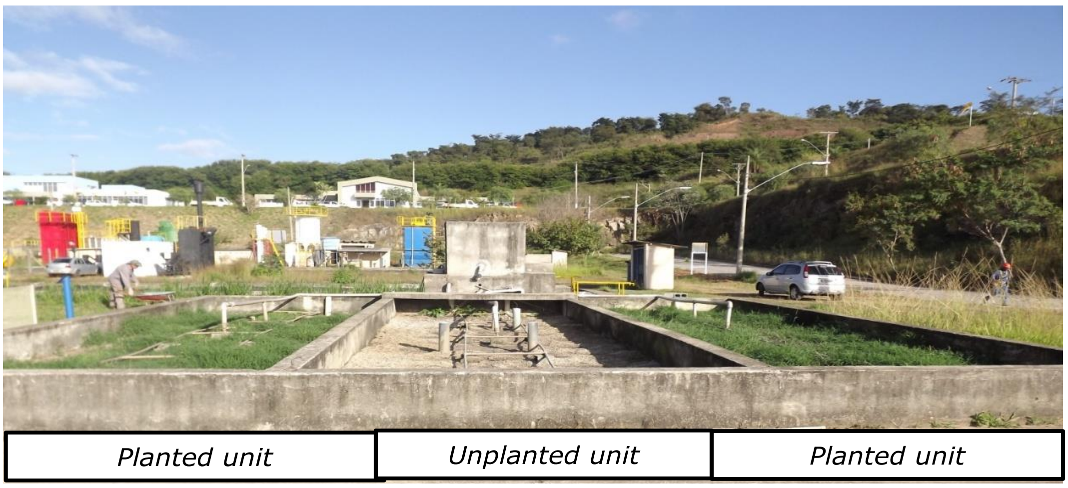


Figure 4.3: View of planted and unplanted units of VFCW.

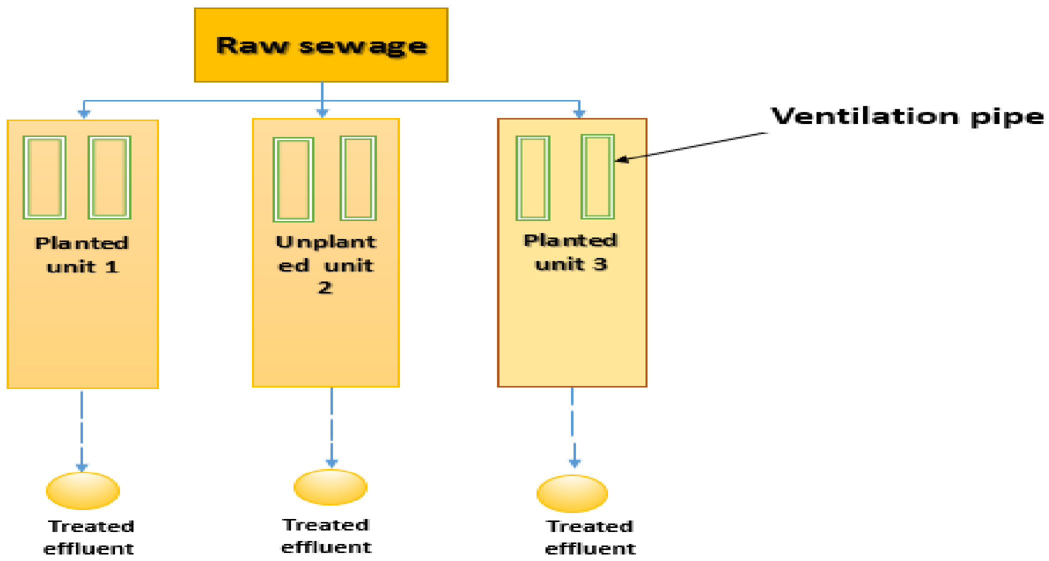


Figure 4.4: View schematic of the beds treating raw domestic wastewater.

Two units of VFCW were planted with Tifton 85 (*Cynodon dactylon pers*), while the middle unit was unplanted as shown in Figure 4.3. Lana (2013) evaluated the same systems with the same three bed unit (two planted units and one control unit) for her master's degree research, entitled pollutant removal in VFCW. On the other hand, Cota (2011) for her PhD research used the same unit beds to evaluate the hydrodynamic and performance of VFCW. The operational mode used for the previous authors were different of which was investigated in the present research.

Concerning the first stage of VFCW one of the possibilities is that in warm climate regions the area of the treatment units can be small compared with cold climate. It is expected that most conversion process can take place in the first stage mainly in warm climate regions. This brings about considerable land and construction cost savings. Nowadays, research have been conducted in order to find out appropriate operation conditions that allow satisfactory performance of the system. Taking into account these explanations, for the present research the area of the system was reduced to 2/3 of the total area of the traditional first stage of the French system. Thus, different operational strategies were investigated and the system was adapted to treat raw domestic wastewater and sludge from septic tank. Thereby, the third unit was dedicated for treating septic tank sludge. It is important to emphasize that the system kept the same physical characteristics, including the filter media, size of units and same type of plant species. All units were planted and are in operation nowadays, units 1 and 2 are treating domestic sewage alternately and unit 3 is receiving septic tank sludge.

4.2 Operational conditions of the system

Three units of vertical flow constructed wetlands (VFCW) were studied to evaluate the performance and behaviour of the system for treating domestic wastewater and septic tank sludge. The facility is comprised by a typical First Stage of the French System, built according to the specifications and recommendations of CEMAGREF French Institute. Molle *et al.* (2005) and AERMC (2005) have published articles about the first stage of the French system. Initially, the system was built to treat raw domestic sewage, but for this research, the system was adapted to treat septic tank sludge and raw domestic sewage in units operating in parallel. In this context, one planted unit of the three VFCW units was dedicated to receive sludge from septic tanks. Thus, the area of the VFCW treating domestic wastewater was reduced to 2/3. Currently the facility is treating domestic sewage and sludge from septic tanks.

In order to achieve the main objectives of this research, it was divided into four different phases, described as follows:

The four phases mentioned describe the main stages of the research related with behaviour evaluation of VFCW for treatment of domestic sewage and septic tank sludge. It is important to mention that, the domestic sewage treatment took place during Phases 1, 2 and 3. On the other hand the first operational strategy (OS1) of the sludge treatment was implemented during Phase 2, the second operational strategy (OS2) was implemented during Phase 3 and third operational strategy (OS3) was implemented during Phase 4.

- **Phase 1.** This phase of operation was for a period of 8 months, from February 2013 to October 2013, when the system operated with two beds, planted and unplanted bed, with the latter serving as control. During this period, both units received only raw domestic sewage.
- **Phase 2.** The second phase of operation was for a period of 17 months, from October 2013 to March 2015. During this phase, the unplanted unit became planted and both planted units kept the same frequency of feeding and resting weekly, with the same inflow of raw domestic sewage and hydraulic loading rate. For this phase in September of 2013, the third unit of VFCW (sludge unit 1) started operation receiving only sludge from septic tank. In addition, the two beds started to receive the percolate sent from sludge unit during the rest period once a week in an alternating form. The two beds served as post-treatment unit 2. In terms of septic tank sludge treatment, this operation was called *operational*

strategy 1 (OS1). This part of research was also carried out by Andrade (2015) and Calderón-Vallejo (2015) in the framework of their Master's degree researches.

- **Phase 3.** The third phase of operation started in April of 2015 up to November of 2015. In this phase, the vertical wetland received sludge from septic tanks where the percolate was retained in the sludge unit (unit1) during a period of seven days for drying and percolation. The percolate was retained 10 cm beneath the filter and after seven days of retention was sent from sludge unit to post-treatment unit (unit 2) during the rest period. This operation was called *operational strategy 2 (OS2)*. Similarly the previous phases, in phase 3 both planted units kept the same frequency of feeding and resting weekly, with the same inflow of raw domestic sewage and hydraulic loading rate.
- **Phase 4.** This phase of operation started in November of 2015 up to March of 2016. In this phase the percolate from sludge unit was retained for seven days inside this unit for initial treatment, after which it was sent to post-treatment unit 2 for final treatment, remaining there for another period of seven days. This part of the research was carried out by Rodrigo Lopez (2016) in the framework of his master degree research- this operation was called *operational strategy 3 (OS3).*

Figure 4.5 shows the summary of the main phases and objectives of this research.

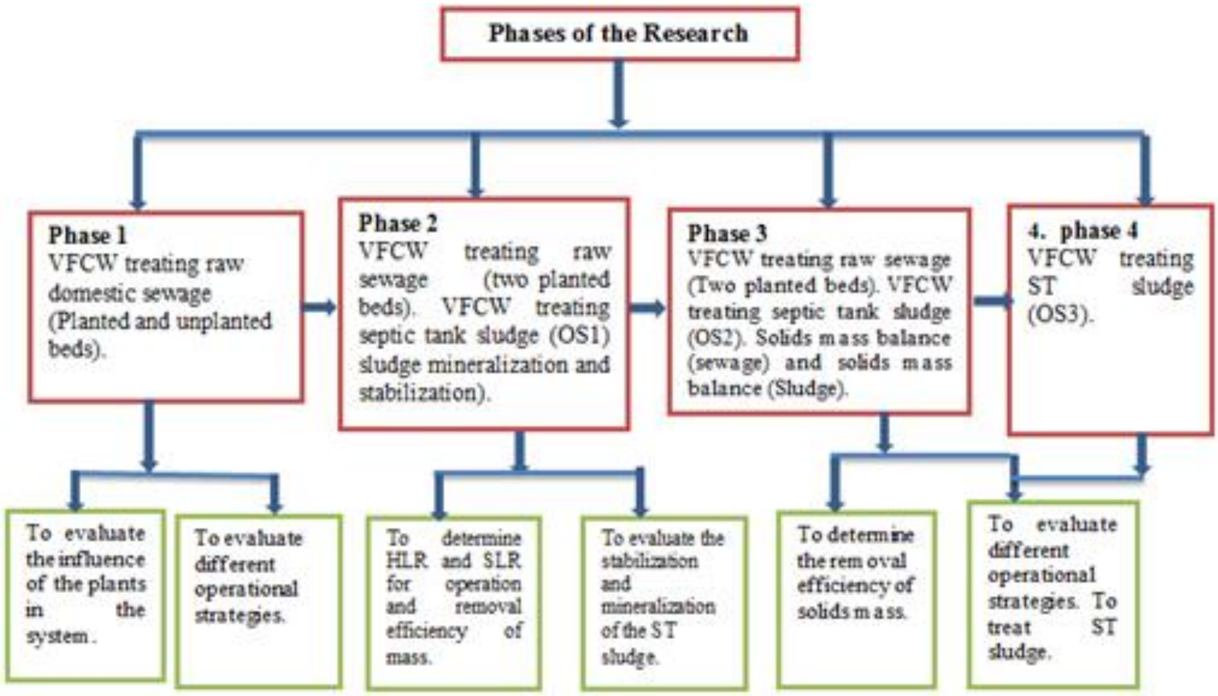


Figure 4.5: Main activities and objectives of the present research.

All experiments proceeded with full scale units and the research was conducted in the city of Belo Horizonte, capital of the Minas Gerais State in Brazil. Monitoring of the system took place at the Centre for Research and Training in Sanitation (CePTS) and all physical chemical analysis were done at the UFMG laboratory. During the second phase of research two master degree students, one from Brazil, Andrade (2015) evaluating the treatment of septic tanks sludge in VFCW using a full-scale unit and pilot scale and another from Colombia, Vallejo (2015), joined in this research. Their subjects were VFCW treating septic tank sludge with full and pilot scale unit. Recently, one PhD Student, from Cuba also joined in this research especially for solids mass balance and oxygen diffusion research in the same units of VFCW.

4.3 Operational characterization of each phase

4.3.1 Phase 1 of operation

The first phase of operation was carried out during a period of eight months, from February up to October 2013. For this period, the system operated with two units, planted and unplanted, working in parallel (only 2/3 of the total area of the traditional first stage of French system). The system operated in fortnight cycles, with the two units alternating in feed and rest periods. Each unit was fed during seven days and then rested during the following seven days. The daily inflow of raw sewage was 13 $m^3.d^{-1}$, spread over and area of 29.1 m^2, resulting in a Hydraulic Loading Rate (HLR) of 0.45 $m^3.m^{-2}.d^{-1}$ for the working bed, or 0.45/2 = 0.22 $m^3.m^{-2}.d^{-1}$ for the overall system. Figure 4.6 shows the planted unit in the left and the unplanted unit in the middle treating raw domestic sewage during the first phase of operation. Figure 4.7 shows a schematic view of planted unit in the left and the unplanted unit in the middle treating domestic sewage during Phase 1. During this phase, the planted unit showed in the right was kept for ST sludge treatment.

Figure 4.6: Planted unit in the left and unplanted unit in the middle treating raw domestic sewage.

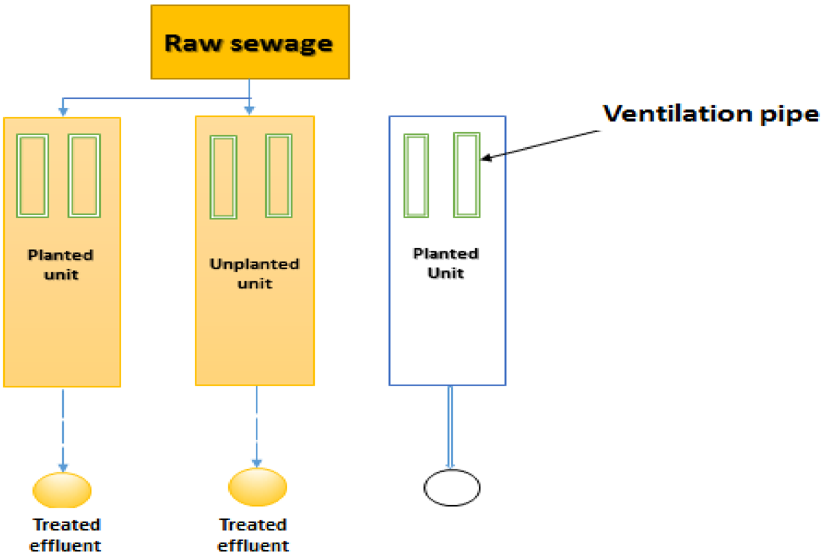


Figure 4.7: Schematic view of VFCW treating domestic sewage during Phase 1.

4.3.2 Phase 2 of operation

The second phase of operation was for a period of seventeen months, from October 2013 to March of 2015. During this phase, the unplanted unit became planted and both planted units kept the same frequency of feeding and resting, the same inflow of sewage and the same hydraulic loading rates (HLR). This configuration allowed for savings of 1/3 of the area of the typical first stage of the French system. Figure 4.8 shows three planted units, one on the left and the other one in the middle treating raw domestic sewage during the second phase of operation. The third unit (sludge unit) received only sludge from ST.

Treating raw domestic sewage through of VFCW the working beds were fed in batch manner. Each batch was discharged as a pulse, every hour (24 batches per day, each batch with a total volume of 0.540 m^3 of raw domestic sewage, while total daily flow was of 13 m^3. The volume of 0.540 m^3 applied hourly in the system was found as the appropriate for the best performance of the system (Lana, 2013). In phase 2, the planted beds started to receive the percolate from the septic tank sludge unit once a week in an alternating form.

In terms of faecal sludge treatment in Belo Horizonte, septic tank sludge management companies have been emptying septic tanks and removing sludge around Belo Horizonte city and disposing it to WWTP Arrudas - Copasa for treatment. The companies use different truck volumes and carry different types of sludge, with varying characteristics (the sources of the sludge are unknown) and all volumes transported by the trucks were discharged in the sludge treatment unit. For this research, initially the first truck to arrive in the morning of the monitoring day transporting septic tank sludge was indicated at the entrance of the Arrudas WWTP to go to CePTS in order to empty its contents in the unit dedicated to sludge treatment on a weekly basis (Wednesdays). Given the difficulty of getting a truck transporting septic tank sludge, in June of 2014, an informal agreement with a Sludge Management Company called ***JM Desentupidora*** was conceived. Thus, from June of 2014, the company supplied sludge from Septic Tank (ST) once a week (every Wednesday).

The sludge unit was fed with ST sludge without any strict control of hydraulic loading and solids loading rates. This reproduces real conditions. This means all the volume of ST sludge transported by companies through trucks were discharged in the sludge unit without previous characterization of the sludge, as a form to reproduce real conditions for a specific region or location. This phase of operation with ST corresponds the operation strategy 1 (OS1) among three operational strategies investigated. Figure 4.8 indicates two planted units treating raw domestic sewage and one planted unit receiving ST sludge with the sending of the percolate from sludge unit to post-treatment units during the rest period. Figure 4.9 shows a schematic view of 3 units of VFCW treating raw domestic sewage and septic tank sludge.

Figure 4.8: Two planted units one in the left and another one in the middle treating domestic wastewater.

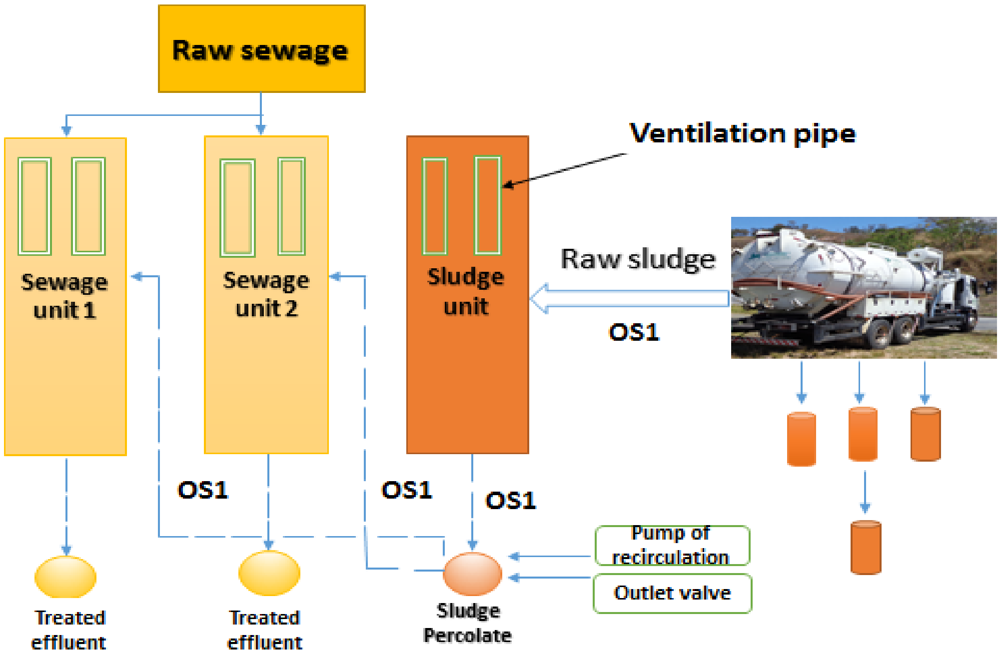


Figure 4.9: Schematic view of VFCW treating septic tank sludge and raw domestic sewage during Phase 2.

4.3.3 Phase 3 of operation

During Phase 2, the sludge unit was fed and the percolate as well as the final effluent after the post-treatment were collected without any retention of the percolate. In Phase 3 during operational strategy 2, in order to test different operational strategies concerning ST sludge treatment, this operation was changed by retaining the percolate in the VFCW unit for a retention period of seven days. This operational strategy is very similar to those carried out by Koottatep *et al.* (2008). This research follows the methodology developed by Koottatep *et al.* (2008). The operational strategy 2 treating ST sludge started on the 1st of April 2015 up to November of 2015.

At the end of operational strategy 1 in March of 2015, the sludge unit was accidentally fed by a toxic sludge load which resulted in clogging and high plant (*Cynodon dactylon Pers)* mortality. The sludge unit was prepared (renewed), in this context, garbage and solid waste were also removed in order to allow a good sludge distribution on the top of the surface bed. Therefore, it required a start-up period which took place during three weeks. The sludge unit was replanted by the same plant species. During the start-up, the plants were watered weekly with domestic wastewater. Thereby, the monitoring period started three weeks after the start-up period and the plant species showed robustness and good adaptability. Figure 4.10 illustrates the sludge unit during the start-up period.

Figure 4.10: Sludge unit during start-up period for operational strategy 2.

Such as in operational strategy 1, in terms of sludge stabilization and mineralization, the operational strategy 2 was designed to reproduce the real conditions where the sludge unit was fed weekly by ST sludge transported by truck without strict control of hydraulic loading rate and solids loading rate.

During operational strategy 2 the sludge unit was being fed by septic tank sludge with different volume, from 5 m^3 to of 8 m^3 $week^{-1}$ and the percolate was retained about 10 cm below the top layer in order to allow the mineralization and stabilization. The percolate was retained in the sludge unit by closing the outlet valves according to the period of feeding and resting. Figure 4.11 shows the outlet valve and the pump for sending the percolate from sludge unit 1 to post-treatment unit 2. The outlet valve was closed weekly in order to receive ST sludge and was opened after seven days to collect and to resend the percolate to post-treatment unit.

Figure 4.11: Pump and outlet valve for sending the percolate from sludge unit to percolate unit.

At the same time, two planted units (sewage unit 1 and sewage unit 2) were being fed by raw domestic sewage, while one was working during seven days, the other was resting during five days, and on the sixth day it received the percolate of septic tank sludge retained in the sludge unit during the past six days. Figure 4.12 shows a view schematic of VFCW treating septic tank sludge and raw domestic sewage during Phase 3. Figure 4.13 illustrates the sludge unit being fed by septic tank sludge during operational strategy 3.

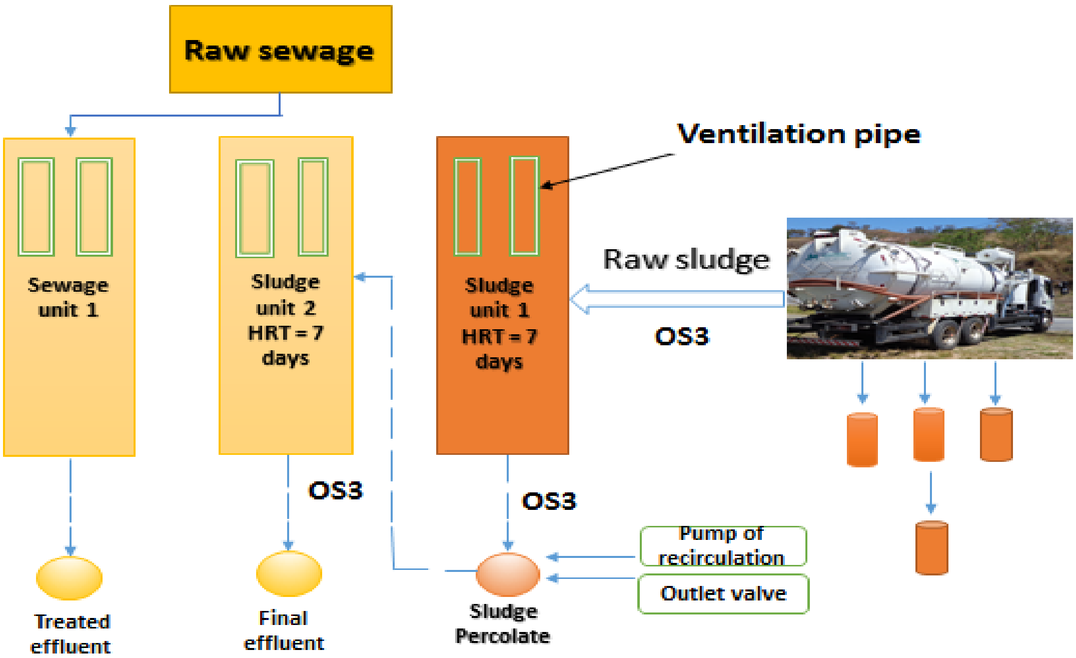


Figure 4.12: Schematic view of VFCW treating septic tank sludge and raw domestic sewage during Phase 3.

Figure 4.13: Sludge unit being fed by septic tank sludge during operational strategy 2.

4.3.4 Phase 4 of operation

In Phase 4, operational strategy 3 of VFCW treating ST sludge was investigated. During operational strategy 3 the truck transporting septic tank sludge discharged the ST sludge onto the sludge unit for retention during seven days. A representative sample of the sludge was taken. After seven days of retention, the outlet valve from sludge unit was opened and a representative sample of the percolated volume was taken, then it was pumped and retained in the post-treatment unit for seven more days. At day fourteen, at the end of the retention period, the outlet from post-treatment unit was opened and a representative sample was taken.

Figure 4.14 shows the operational mode of the VFCW treating sludge from septic tank during operational strategy 3.

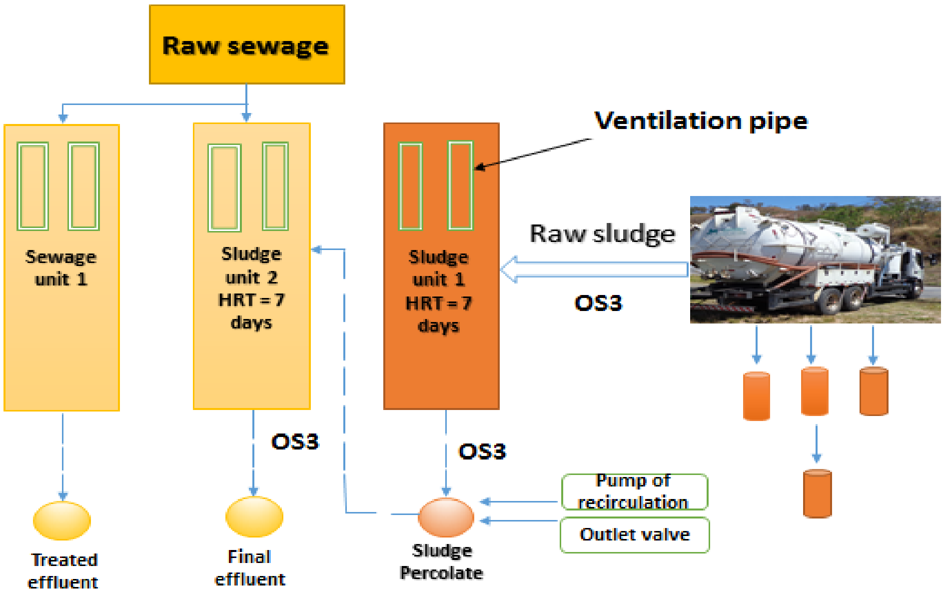


Figure 4.14: Schematic view of the VFCW treating ST sludge during Phase 4.

All Phases of operation carried out in the present research with different operational strategies are described in Table 4.1.

Table 4.1: Summary of the four phases of vertical flow constructed wetland treating raw domestic sewage and septic tank sludge.

Phases	**Phase 1**	**Phase 2**		**Phase 3**		**Phase 4**
Parameters	**Raw sewage**	**Raw sewage**	**ST Sludge (OS1)**	**Raw sewage**	**ST Sludge (OS2)**	**ST Sludge (OS3)**
Number of Units	2	2	1	2	1	2
Area of each filter	29.1 m^2	29.1 m^2	29.1 m^2	29.1 m^2	29.1 m^2	29.1 m^2
Bed depth	0.70 m	0.70 m	0.70 m	0.70 m	0.70 m	0.70 m
Flow	13 $m^3.d^{-1}$	13 $m^3.d^{-1}$	3.5 - 16 m^3/week	13 $m^3.d^{-1}$	4 – 8 m^3/week	4 – 8 m^3/week
Hydraulic Loading Rate (HLR)- Total	0.22 $m^3.m^{-2}.d^{-1}$	0.22 $m^3.m^{-2}.d^{-1}$		0.22 $m^3.m^{-2}.d^{-1}$		
Hydraulic Loading Rate (HLR) - working bed	0.45 $m^3.m^{-2}.d^{-1}$	0.45 $m^3.m^{-2}.d^{-1}$	0.259 m^3dis/m^2weeky	0.45 $m^3.m^{-2}.d^{-1}$		
Operational Cycle	7 d feed; 7 d rest	7 d feed; 7 d rest	Once a week	7 d feed; 7 d feed	Once a week with HRT= 7 days	Once a week with HRT= 14 days
Number of batches per day	24	24	1 batch weekly	24	1 batch weekly	1 batch weekly
Per capita surface area	0.6 m^2/inhabitant	0.6 m^2/inhabitant		0.6 m^2/inhabitant		

4.4 Monitoring of the system

4.4.1 Influent and effluent monitoring of sewage treatment units

Influent and effluent samples were collected weekly. After the pulse loading of 0.540 m^3 from the distribution box, three aliquots of two litres with five minute intervals of effluent were collected, totalling six litres in volume. After that, samples were uniformly mixed and a litre was taken for analyses. The parameters evaluated in the field were: Dissolved Oxygen (DO), Temperature (T), Hydrogenionic potential (pH). These parameters were measured using the *Hach HQ40D portable meter* (see Figure 4.15), which is a multiparameter probe for pH, conductivity, temperature, dissolved oxygen and oxidation reduction potential (ORP) measurements.

Figure 4.15: Probes used for measurement in the field.

A litre of the sample was also taken to the laboratory of the Department of Sanitary and Environmental Engineering (DESA) from UFMG and the parameters analysed were: Ammonium (NH_4^+-N), Total Kjeldahl Nitrogen (TKN), Total Suspended Solids (TSS), Total Volatile solids (TVS), Chemical Oxygen Demand (COD), and Biochemical Oxygen Demand ((BOD_5); Nitrate (NO_3^--N) and Nitrite (NO_2^--N). All analysis were done according to the Standard Methods (APHA/AWWA/WEF, 2012).

4.4.2 Influent and effluent monitoring of sludge treatment unit

Regarding ST sludge treatment, the sludge unit received ST sludge varied from 3.5 to 16 m^3 on a weekly basis, from September of 2013 to March of 2015. Direct discharge of the sludge from septic tank onto the surface of the sludge unit worked well and was effective in achieving the same hydraulic distribution. In order to get representative sample of raw FS, composite sampling were implemented. Three samples of ten litres each were successfully collected at the beginning, middle and end of the feeding process, discharging a total of thirty litres of raw ST sludge. After uniformly mixing, two litres were sampled from each six litres of composite sample. After uniformly mixed, a litre was taken from six litres for laboratory analysis. Parameters measured were temperature, oxygen and pH while other parameters were analysed at the UFMG/DESA laboratory. After the sludge was discharged in the sludge unit of VFCW, three samples of two litres of the percolate from sludge unit were collected between ten minute intervals and three samples of the effluent coming from the post-treatment unit were also collected between ten minute intervals. After that, only a litre of the percolate and a litre of the post-treated effluent were taken for laboratory analysis.

In OS2, the same methodology concerning influent and effluent monitoring was followed. Trucks transporting ST sludge arrived in the treatment units and composite sample from the ST sludge were implemented. After the retention period of seven days, every Tuesday the outlet valve was opened and a litre of the percolate from sludge unit was collected between five minute intervals during fifty-five minutes totalling eleven litres. At the same time, the percolate was sent to the post-treatment units and a litre of the post-treated effluent was also collected between five minute intervals totalling eleven litres. Only a litre of the percolate and a litre of the final effluent were taken for laboratory analysis. During OS2, also 100 ml was taken to laboratory for the microbiological analysis.

In OS3, after ST truck arrived, in order to take a representative sample from the total septic truck volume, three sample were taken. Using three plastic buckets, composite sample was also taken directly from the hose at the beginning, the mid and the end of the discharge. The sampling times were approximated according to the remaining volume observed inside the truck. Subsequently nine litres of influent collected were mixed in another plastic bucket, thus the representative sample was obtained. Then, only a litre from the representative sample was collected in a plastic bottle for the physical-chemical analysis and 100 ml in a smaller one for the microbiological analysis. In the sludge unit, after the retention period of seven days, the outlet register was opened and representative sample was taken for laboratory analysis.

On the other hand, the submerged pump located at the bottom of the register was switched on to move the percolated volume to the post-treated unit. After the retention period of seven days, the outlet register was also opened in the post-treated unit and representative sample was taken for laboratory analysis.

The samples of the influent and effluent were analysed weekly for Total Solids (TS), Total Volatile Solids (TVS), Biochemical Oxygen Demand (BOD), Chemical Oxygen Demand (COD), TKN and Ammonium ($N\text{-}NH_4$). During discharging, composite samples from the beginning, middle and end was implemented. Table 4.2 indicates the main parameters of sewage and ST sludge analysed during monitoring.

Table 4.2: Physicochemical parameters analysed and monitoring program.

Parameter		**Acronym**	**Unit**	**Frequency**	**Analytic Method**	**Method number**
Hydrogenionic potential	Sewage and sludge	pH	-	Weekly	Potentiometric	4500 H^+
Temperature	Sewage and sludge	T	º C	Weekly	Fields methods	2550 B
Dissolved Oxygen	Sewage and sludge	DO	mg L^{-1}	Weekly	Electrometric	4500 O
Biochemical Oxygen Demand	Sewage and sludge	BOD	mg L^{-1}	Weekly	5-Day BOD test	5210 B
Chemical Oxygen Demand	Sludge	COD	mg L^{-1}	Weekly	Closed reflux method, Colorimetric method	5220 C 5220 D
Chemical Oxygen Demand	Sewage	COD	mg L^{-1}	Weekly	Closed reflux method, Titrimetric Method	5220 C 5220 C
Total solids	Sludge	TS		Weekly	Gravimetric	2140 B
Total Suspended Solids	Sewage	TSS	mg L^{-1}	Weekly	Gravimetric	2540 D
Total Volatile Solids	Sewage and sludge	TVS	mg L^{-1}	Weekly	Gravimetric	2540 G
Ammonium	Sewage and sludge	NH_4^+-N	mg L^{-1}	Weekly	Titrimetric Method	4500 C
Total Kjeldahl nitrogen	Sewage and sludge	TKN	mg L^{-1}	Weekly	Titrimetric Method	4500 C
Nitrate	Sewage and sludge	$N\text{-}NO_3$	mg L^{-1}	Weekly	Ionic chromatography	4500 B
Nitrite	Sewage and sludge	$N\text{-}NO_2$	mg L^{-1}	Weekly	Ionic chromatography	4500 B
E. coli			MPN/100 mL		Colilert	9223

4.4.3 Data analysis

The determination of removal efficiency of all parameters analysed was based on the influent and effluent concentrations. Notwithstanding, in VFCW was not possible to determine the effluent outflow during all monitoring period.

The formula used to determine treatment efficiency of all parameters analysed was described as:

$$E = \left(1 - \frac{Ce}{Ci}\right) 100 \tag{4.1}$$

Where:

Ci = influent concentration (mg.L^{-1})

Ce = effluent concentration (mg.L^{-1})

Statistical analysis were performed using the Mann-Whitney U-test at 5% significance level for comparing median effluent concentrations and removal efficiencies in phase 1 between the planted and unplanted units. The same statistical test were performed for comparing the performance of the system between HLR of 0.22 m^3.m^{-2}.d^{-1} and 0.15 m^3.m^{-2}.d^{-1}. In phase 2 of operation, the same statistical test was performed for comparing the median in terms of effluent concentrations and removal efficiencies between two planted units. In phases 3 and 4 the test was also performed at 5% significance level to compare removal efficiencies in the sludge unit and post-treatment unit between OS2 and OS3. In order to determine the best operational strategy, Mann-Whitney U-test, were also used for comparing removal efficiency between OS1 and OS2 regarding ST sludge treatment**.** Figure 4.16 summarizes different statistical analysis performed in all phases of the research.

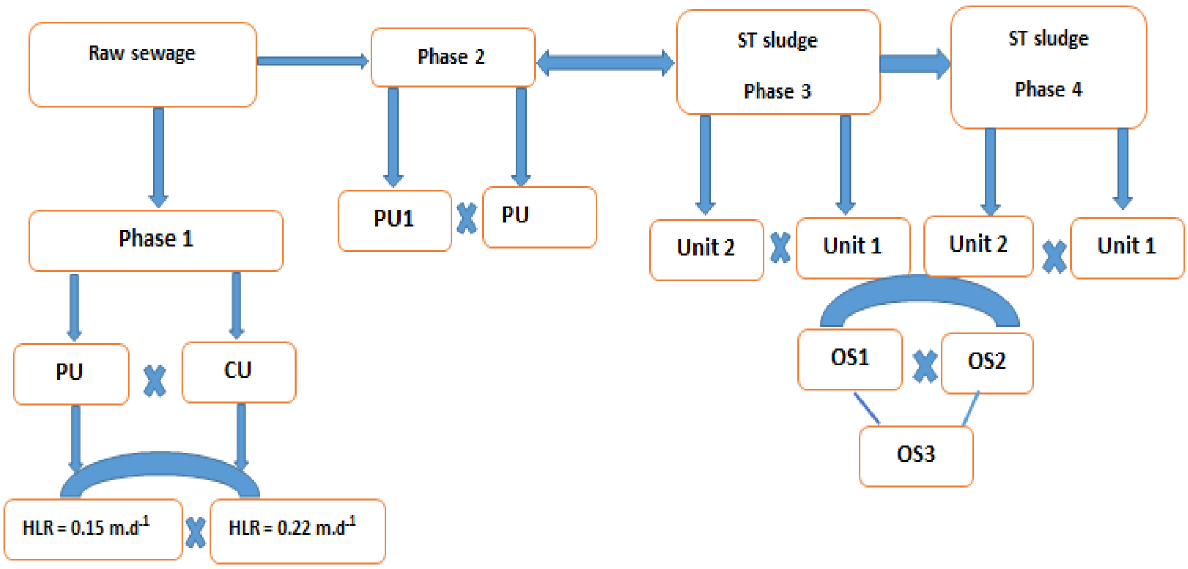


Figure 4.16: Statistical analysis performed during the phases of research.

HLR- Hydraulic Loading Rate; Unit 1- Sludge Unit; Unit 2- Post-treatment Unit; PU- Planted Unit; PU1- Planted Unit in Phase 2, CU- Control Unit; OS1- Operational Strategy 1, OS2- Operational Strategy 2, SO3- Operational Strategy 3

4.4.4 Vegetation management

For the treatment of raw domestic sewage the plant Tifton 85 (*Cynodon dactylon pers*) was harvested between forty five day periods to allow plant rejuvenation, improve the plant micronutrient uptake, prevent clogging, better hydraulic conductivity and maintain aerobic conditions in the system.

In terms of ST sludge treatment, during Phase 2, samples of the plant in the sludge unit were collected with the objective of evaluating the plant species in terms of nutritional analysis, green mass, water content and dry matter. The samples were collected in a triplicate experimental randomised block design with areas of 1 m^2/each.

4.4.5 Sludge mineralization and stabilization

In order to evaluate the sludge unit in terms of sludge stabilization and mineralization, soil medium samples at the beginning, middle and end of the unit were collected to determine the relation between volatile solids and total solids. Samples of the sludge mineralized in the sludge unit were collected using PVC tubes. The PVC tubes were inserted about 2 cm in depth inside the deposit layer, and samples of the sludge were removed. The samples of sludge were taken to the DESA/UFMG laboratory for total solids and moisture analysis.

4.5 Hydraulic behaviour and solids mass balance

4.5.1 Inlet/outlet flows

In order to determine the optimal hydraulic loading rate of sewage that allows satisfactory operation of VFCW for organic matter removal, inlet and outlet flow of raw domestic sewage were measured in the system. The VFCW was intermittently pumped onto the surface by pulse loading of 0.540 m^3 $hour^{-1}$. The duration of batch loading took approximately three minutes. The measurement of variation of the effluent outflow with time was done in the units one and two during a period from November 2014 to August 2015 according to the schedule presented in Table 4.3. These measurements were done in two consecutive batch loading.

Table 4.3: Program of measurements.

Measurement	Date	Unit evaluated	number of batches
1	07/11/2014	Planted unit	1
2	14/11/2014	Planted unit	2
3	25/03/2015	Planted unit	1
4	01/04/2015	Unplanted unit	2
5	08/04/2015	Planted unit	2
6	15/04/2015	Unplanted unit	2
7	13/08/2015	Planted unit	2
8	25/08/2015	Planted unit	2

Thus, the evaluation of effluent outflow in the system was carried out based on the direct method, where the chronometric and graduated bucket were used simultaneously. Thereby, after pulse loading of 0.540 m^3 using the chronometric and graduated bucket the effluent outflow and TSS concentration over time were measured. The graphics illustrating the variation of effluent outflow profile over time, variation of TSS concentration profile over time, solids load profile over time and outlet cumulative volume profile over time were done using *Microsoft Excel* 13. Each curve of all evaluated parameters indicates only the results correspondent to one batch loading and was compared to planted and unplanted units.

Regarding ST sludge, the hydraulic behaviour and solids mass balance were also determined. All volume of ST sludge transported was discharged onto the sludge unit. On 01/10/2014 a single batch of 8 m^3 of the ST sludge was discharged, while on the 08/10/2014 approximately 12 m^3 was discharged. Differently, from raw domestic wastewater, in this case, the hydraulic behaviour and solids mass balance were measured only in the sludge unit.

In both cases (raw domestic sewage and ST sludge evaluation) the effluent outflow, TSS concentration over time, solids load over time, the outlet cumulative volume over time, as well as the percentage of volume applied in the system were measured.

The total volume collected from the outlet can be calculated through the following equation:

$$V = \sum_{t=1}^{T} R * \Delta T \qquad (4.2)$$

Where:

V= total volume collected (L)

R= Outlet flow rate (L s^{-1})

$\Delta T =$ Difference between second and first time of the outflow (s)

The percentage of volume applied in the system was determined between the relation of outlet cumulative volume and the total volume applied in the system. This is an important parameter for determining the total volume recovered from the outlet. This parameter can be calculated through equation 4.3 presented below.

$$\%VA = \frac{AC}{TV} \tag{4.3}$$

Where:

%VA= Percentage of volume applied

AC= Outlet cumulative volume over time (L)

TV= Total volume applied in the system (L)

4.5.2 Inlet/outlet solids

Mass balance is defined as a measure or model for pollutant reduction in wetland treatment (Kadlec and Wallace, 2008). In VFCW the model is an important parameter to determine the percentage of solids mass removed. In order to determine the Total Suspended Solids (TSS) concentration and solids load of raw domestic sewage that allow satisfactory operation of VFCW, the inlet and outlet of TSS concentrations were measured by a *TSS portable handheld measurement instrument for turbidity and solids (HACH Company).* The instrument stores the recorded data under the corresponding calibration curve. Thereby, the sensor was calibrated for the measurement of solids in the TSS concentration of raw domestic sewage in the inlet and outlet over time. It was previously determined at the laboratory the TSS concentration of two samples for the solids calibration curve. The calibration curve was obtained through two points, from the lowest and highest TSS concentration. The TSS portable instrument was calibrated for raw domestic sewage and septic tank sludge.

For raw domestic sewage, the probed measured the average TSS influent in the system from the distribution box and the solids mass of the influent in the system was calculated (=Volume (540 litres) x TSS concentration). Thereafter, the effluent outflow was measured using the bucket and chronometric technique, as well as measuring the TSS concentration over time in two consecutive batch loading. The TSS probe recorded and stored TSS concentration values over time.

Regarding ST sludge, the procedure for solids mass balance measurement was similar with raw domestic sewage. After the effluent outflow was measured, the TSS concentration over time was also measured. Before sludge was discharged onto the sludge unit, eleven litres of the sludge at the beginning, eleven litres of the sludge in the middle and eleven litres at end was collected. Composite sample of two litres each were collected totalling six litres. Thereafter, the influent for TSS concentration was measured using the probe. The solids mass applied was the result of the multiplication of all volume transported and discharged and its respective TSS concentration. Figure 4.17 illustrates measurement of the effluent outflow over time and TSS concentration by probe.

Figure 4.17: Measurement of the effluent outflow and TSS concentration over time.

In addition, effluent outflow and TSS concentration over time, the parameters of solids load over time (mg s^{-1}), outlet total solids mass, outlet cumulative mass, percentage of mass applied and percentage of outlet mass were determined. These parameters appear as important variables to determine the total solids mass in the system.

The solids load expresses the solids mass of the effluent over time. This parameter can be determined by the multiplication of the outlet TSS concentration and outflow. Total solids mass express the total mass in the effluent over time and it was determined by multiplying solids load and time. The total solids mass in the effluent was obtained from the cumulative mass over time. Nevertheless, the percentage of mass reduction efficiency enables the determination of the efficiency of the system in terms of solids mass removal. Thus, performance of the VFCW in terms of solids mass removal was determined using the equation 4.4 presented below.

$$\%\ \text{Mass reduction efficiency} = 100 \times \left(\frac{mi - mo}{mi}\right) \tag{4.4}$$

Where:

mi = inlet mass loading (kg)

mo = outlet mass (kg)

5 RESULTS AND DISCUSSION

5.1 Hydraulic loading rate - VFCW treating domestic sewage

5.1.1 Characterisation of raw domestic sewage

Two phases of VFCW treating domestic sewage were investigated. In Phase 1, planted and unplanted units were tested and in Phase 2, two planted units were tested. The main results found in Phase 1 concerning the two units comprising the first-stage of the French system (planted and unplanted units) treating raw domestic sewage are shown below. From the results obtained, it was observed the influence of plants in raw domestic sewage treatment. Table 5.1 presents physical-chemical characteristics of raw domestic sewage determined during the monitoring period.

Table 5.1: Physical-chemical characteristics of the raw domestic sewage.

Parameters	n	Mean	Median	Minimum	Maximum	Coefficient of variation
pH	34	7.4	7.4	6.6	8.0	0.05
Temperature ºC	51	24	24	22	29	0.06
TSS (mg L^{-1})	54	218	203	74	484	0.49
NH_4^+-N (mg L^{-1})	44	34	38	6	49	0.35
TKN (mg L^{-1})	45	40	43	14	55	0.30
BOD_5 (mg L^{-1})	30	229	175	40	848	0.78
COD (mg L^{-1})	50	561	603	209	989	0.37

n: number of samples; **TSS:** total suspended solids; **NH_4^+-N:** Ammonium; **TKN:** total Kjeldahl nitrogen; **BOD:** biochemical oxygen demand; **COD:** chemical oxygen demand

The quality of raw domestic sewage determined during the monitoring period was characterized by a great variability of the influent concentrations, mainly for BOD_5 and TSS, as expressed by their coefficient of variation. The influent concentrations of NH_4^+-N, TKN and COD also showed expressive variability, except for pH and temperature. From the influent concentrations obtained, it can be seen that the VFCW is treating different characteristics of raw domestic sewage. However, the values of influent concentrations determined in this research were very similar to those obtained by Lana (2013) at the same facility. The influent concentrations of the parameters determined in this research during the monitoring period were compared with the range of physical-chemical characteristics of predominantly domestic sewage, cited by von Sperling (2014).

Table 5.2 presents the comparison between the range of the raw domestic sewage in developing countries and influent concentrations of all constituents analysed.

Table 5.2: Comparison between the typical range of raw domestic sewage and the median of the influent concentration of influent wastewater found in this research.

Parameter	Range*	Median value determined
BOD_5	250 – 400	175
COD	450 – 800	603
TSS	200 – 450	203
NH_4^+-N	20 – 35	38
pH	6.7 – 8.0	7.4

*von Sperling (2014)
Concentration (mg L^{-1}, except pH)

Comparing the range of raw domestic sewage and values determined of the influent concentrations, it is seen that all parameters were in the range of influent concentrations of the raw domestic sewage cited by von Sperling (2014), except for BOD_5. However, BOD_5 influent concentration is close to the lower limit of BOD_5 range. In spite of considerable variability observed in the influent concentration, the values determined indicate typical values of influent concentrations of raw domestic sewage in developing countries. The proximity of values of the influent concentrations to the lower limit of the range suggest a more diluted raw domestic sewage.

5.1.2 Concentrations of the effluent from the planted unit

In Phase 1, only two units were in operation, planted and unplanted units and the system was loaded at the HLR of 0.22 $m^3.m^{-2}\ d^{-1}$ and 0.45 $m^3.m^{-2}\ d^{-1}$ per bed. Table 5.3 presents the physical-chemical characteristics of the effluent from the planted unit during the period of monitoring.

Table 5.3: Physical chemical characteristics of the effluent from the planted unit.

Parameters	n	Mean	Median	Minimum	Maximum	Coefficient of variation
pH	24	7.1	7.1	6.5	7.6	0.05
Temperature ºC	28	23	24	20	28	0.07
Oxygen (mg L^{-1})	30	3.53	3.67	0.35	5.82	0.38
TSS (mg L^{-1})	28	61	59	15	160	0.55
NH_4^+-N (mg L^{-1})	23	16	17	3	31	0.41
TKN (mg L^{-1})	27	20	19	4	34	0.33
BOD (mg L^{-1})	15	46	40	14	103	0.53
COD (mg L^{-1})	28	245	198	60	841	0.65

n: number of samples; **TSS:** total suspended solids; **$NH4^+$-N:** Ammonium; **TKN:** total Kjeldahl nitrogen; **BOD:** biochemical oxygen demand; **COD:** chemical oxygen demand

By observing the data presented in Table 5.3, it can be seen that the effluent from the planted unit was characterized by a lower variability than influent concentrations. On the other hand, it was observed that the quality of the effluent from planted unit improved compared raw domestic sewage, which indicated the influence of physical and biological process in planted wetland unit related with raw domestic sewage treatment (Brix, 1997). In addition, it was observed that almost all constituents analysed met the regional discharge standard (Minas Gerais State, Brazil) except for COD concentration. According to DN 01/2008 COPAM/CERH were stated the following discharge standard: 60 $mg.L^{-1}$ for BOD, 180 $mg.L^{-1}$ for COD, 100 $mg.L^{-1}$ for TSS and 20 $mg.L^{-1}$ for Ammonium. Surprisingly, the average COD concentration of the effluent was higher than expected and a great variability was observed, a fact that led to more research. Molle *et al.* (2005), investigating the first stage treatment of VFCW commented about variations of COD removal in the system. According to Molle *et al.* (2005), for low hydraulics loads, a greater variation in COD removal can be observed. Furthermore, heterogeneity in distribution of the liquid can lead to some deficiencies in COD removal due to flow short-circuiting and COD removal is sensitive to infiltration rate (Molle *et al.*, 2005).

5.1.3 Concentrations of the effluent from the unplanted unit

Such as with the planted unit, the data analysis showed that the system performed well in the unplanted unit. The results demonstrated the reduction of the effluent concentrations and satisfactory removal efficiency of organic matter in the unplanted bed.

Table 5.4 presents physical-chemical characteristics of the effluent from the unplanted unit during the period of monitoring.

Table 5.4: Physical chemical characteristics of the effluent from the unplanted unit.

Parameters	N	Mean	Median	Minimum	Maximum	Coefficient of variation
pH	17	7.3	7.4	6.6	7.8	0.05
Temperature ºC	22	23	23	21	27	0.08
Oxygen ($mg\ L^{-1}$)	23	4.09	4.21	0.22	6.08	0.35
TSS ($mg\ L^{-1}$)	26	73	62	17	204	0.63
NH_4^+-N ($mg\ L^{-1}$)	21	18	19	8	25	0.27
TKN ($mg\ L^{-1}$)	18	22	24	7	30	0.31
BOD ($mg\ L^{-1}$)	15	45	35	13	100	0.59
COD ($mg\ L^{-1}$)	22	194	156	76	487	0.59

n: number of samples; **TSS:** total suspended solids; **NH_4^+-N:** Ammonium; **TKN:** total Kjeldahl nitrogen; **BOD:** biochemical oxygen demand; **COD:** chemical oxygen demand

The analysis showed that the effluent coming from the unplanted unit was also characterized by a low variability and almost all constituents met the regional discharge standard, except for COD (Minas Gerais State, Brazil). Thereby, an effluent with good quality was obtained. Comparing the planted and unplanted units in terms of effluent concentrations, were not found statistical difference ($p > 0.05$) between them. In contrast with the results obtained by Lana, (2013) at the same VFCW, the effluent concentrations obtained in the unplanted unit were higher, indicating a lower removal efficiency of the unplanted unit when compared with the results obtained in this study. This fact can be associated by different hydraulic loading rate applied in both operational strategies and/or to the presence of plants.

5.1.4 Pollutants removal efficiency

In general, the system operating only with two units showed a good performance in terms of organic matter removal. The results concerning removal efficiencies in both units are summarized in Table 5.5.

Table 5.5**:** Median removal efficiencies in the planted and unplanted units.

Parameter	**Removal efficiency (%)**	
	Planted	Unplanted
BOD	84	78
COD	59	69
TSS	72	65
TKN	55	45
NH_4^+-N	55	47

From the analysis it was observed that the system was satisfactory in terms of organic matter removal. The median biochemical oxygen demand (BOD) removal efficiency of 84% in the planted unit and 78% in the unplanted can be considered satisfactory. However, the median removal efficiency obtained was lower than that obtained by Molle *et al.* (2005), except for BOD. This fact can be explained by operational strategy adopted associated with a hydraulic loading rate (flow divided over only two units), the period of feed and rest and the characteristics of the influent.

5.1.5 Experiments with the reduction of surface area and number of units

The first stage treatment of VFCW performed well with the reduction of surface area and number of units. In the first phase of operation, the system operated with only two units in parallel (only 2/3 of the total area taken by the traditional first stage of the French system). Thus, the total HLR applied was of 0.22 $m^3.m^{-2}\ d^{-1}$. In order to evaluate two different operational strategies the results obtained during Phase 1 were compared to those obtained when the system was operating with three units in parallel (traditional first stage of the French system), in a total HLR of 0.15 $m^3.m^{-2}\ d^{-1}$. Lana (2013) carried out this operational mode, in the context of her Master´s degree research. Thereby, two different experiments were considered. The first one conducted by Lana (2013) and the second experiment conducted in the context of this research. The results are presented and discussed in average concentrations and in average removal efficiencies. The results of the two experiments are given in Table 5.6

Table 5.6: Average concentrations and removal efficiencies of the wastewater constituents measured during both experiments.

Parameter	**Influent**		**Effluent**			
Concentration ($mg.L^{-1}$)			Experiment 1 (3 beds) HLR (total): 0.15 $m^3.m^{-2}.d^{-1}$ HLR (bed): 0.45 $m^3.m^{-2}.d^{-1}$		Experiment 2 (2 beds) HLR (total): 0.22 $m^3.m^{-2}.d^{-1}$ HLR (bed): 0.45 $m^3.m^{-2}.d^{-1}$	
	Exp1	Exp2	Planted unit	Unplanted unit	Planted unit	Unplanted unit
BOD	279	229	36	38	46	45
COD	465	561	71	70	245	194
TSS	293	218	34	39	61	73
TP	3.9	-	2.1	2.0	-	-
TKN	31	40	13.9	15.0	20	22
NH_4^+-N	26	34	10.1	10.8	16	18
NO_3^--N	0.1	-	12.6	8.2	-	-
NO_2^--N	0.01	0.01	0.2	0.1	0.2	0.2
Removal efficiency (%)			**Planted unit**	**Unplanted unit**	**Planted unit**	**Unplanted unit**
BOD			82	80	74	76
COD			81	81	59	61
TSS			85	78	67	60
TP			21	33	-	-
TKN			56	47	49	42
NH_4^+-N			59	52	51	44

By observing Table 5.6 the results obtained in the two experiments showed good performance of the system in terms of organic matter and other pollutants removal. The data analysis showed that, even with area reduced and higher hydraulic loads, the system showed strength and resistance. Seemingly, the system showed good performance in both experiments. It is important to reinforce that the system worked well when it was operating with three units (Lana, 2013). Undoubtedly, the results obtained between two experiments show the influence of HLR in the operation and performance of the VFCW. Molle *et al.* (2005) discussed the role of HLR in the raw domestic sewage treatment using VFCW. Nevertheless, the author emphasized that it is difficult to find any consensus about hydraulic limits in the literature. However, it is a consensus that dependent of VFCW design for lower and higher hydraulic loading it can affect the performance of the system concerning organic matter removal and pollutant removal. On the other hand, it was seen by Molle *et al.*, (2005) that high HLR reduce the removal efficiencies of organic matter, total suspended solids and nitrogen in vertical flow constructed wetlands. In this case, it was seen that with higher hydraulic loading rates the system reduced its performance.

The characterisation of domestic sewage can be expressed as mass loading rate (g m^{-2} d^{-1}). This parameter expresses the influent mass (g) of the pollutant applied per unit area (m^2) per unit time (d). The mass loading rate applied in the VFCW can potentially influence its performance in terms of pollutant removal. Hoffmann and Platzer (2010) commented that the amount of BOD discharged by individuals is characterised by its variations due to the differences in diet as well as socio-economic differences. The mass loading rate of the raw domestic sewage applied in the working unit and in the whole system in both experiments is shown in Table 5.7.

Table 5.7: Mass loading rate of the domestic sewage applied in the working unit and in the whole system in both experiments.

Parameter	Average influent concentration (mg L^{-1})		Mass loading rate in the working unit (g m^{-2} d^{-1})		Mass loading rate in the whole system (g m^{-2} d^{-1})	
	Exp1	**Exp2**	**Exp1**	**Exp2**	**Exp1 (3 units)**	**Exp2 (2 units)**
BOD	279	229	125	105	42	51
COD	465	561	208	251	69	125
TSS	293	218	131	97	44	49
TKN	31	40	14	18	7	9

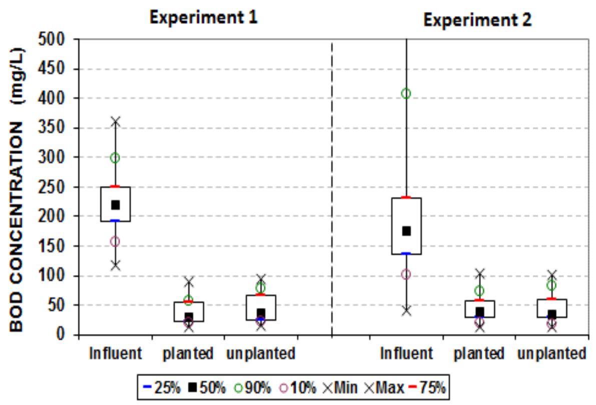
Table 5.7 shows that the mass loading rate obtained during experiment 2 was lower than experiment 1 due to the small area investigated in experiment 2. However, the mass loading rates obtained in both experiments were lower when compared to literature (Molle *et al.*, 2005). This fact can be associated with the characteristics of the raw domestic sewage investigated in both experiments. Previously, it was seen that the raw sewage investigated here was more diluted when compared with the characteristics of the raw sewage found in the literature.

Table 5.8 presents the p-values of the Mann–Whitney U-test comparing the median effluent concentrations and removal efficiencies in both experiments, and a comparison between the planted and unplanted units. The results show differences of the influent concentrations obtained between both experiments, therefore were compared the median effluent concentrations and the median removal efficiencies in both experiments. Figure 5.1 shows box-and-whisker plots of the concentrations of the parameters analysed in both experiments, allowing visualization of central tendency and dispersion.

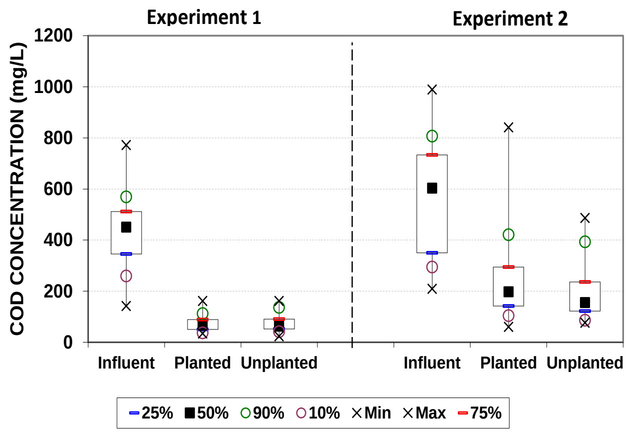
Table 5.8: *p*-values of the Mann-Whitney U-test comparing effluent concentrations and removal efficiencies in both experiments and planted and unplanted units.

Constituent	Experiment 1 x Experiment 2		Planted x Unplanted (effluent concentrations)		Planted x Unplanted (removal efficiencies)	
	Effluent concentrations	**Removal efficiencies**	**Experiment 1**	**Experiment 2**	**Experiment 1**	**Experiment 2**
BOD	0.3013	0.1615	0.3298	0.7875	0.2674	0.6334
COD	0.0000 (*)	0.0000 (*)	0.8878	0.1682	0.4504	0.4344
TSS	0.0001 (*)	0.0001 (*)	0.1341	0.4010	0.1344	0.2989
TKN	0.0005 (*)	0.2720	0.0317 (*)	0.1944	0.0232 (*)	0.0953
NH_4^+-N	0.0007 (*)	0.1700	0.2953	0.3628	0.1637	0.0761

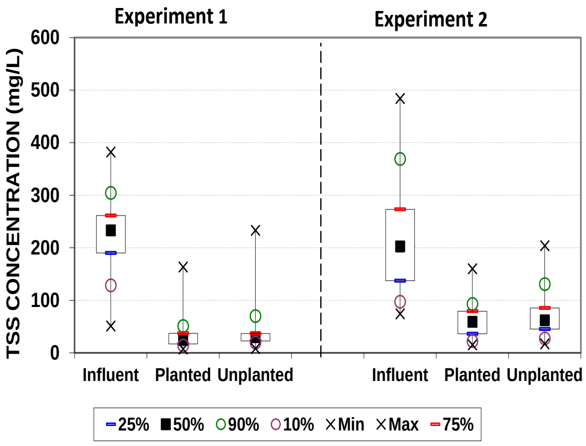
(*) *p*-values ≤ 0.05: sample medians are significantly different



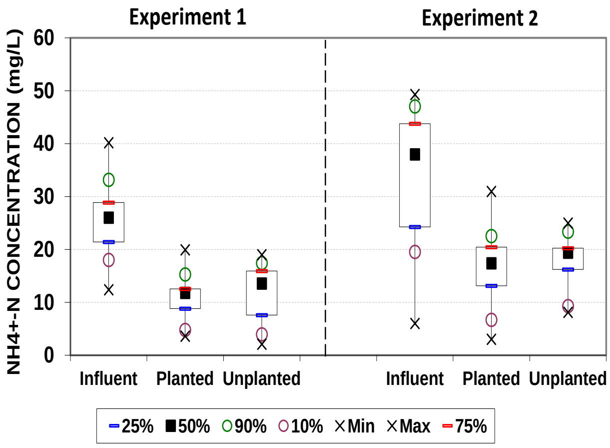
a) BOD



b) COD



c) TSS



d) NH_4^+-N

Figure 5.1: Box-and-whisker plots of the concentrations of the parameters analysed covering both experiments in the planted and unplanted units, a) BOD, b) COD, c) TSS and d) NH_4^+-N.

The results showed that in experiment 1, when the system operated with three beds, the mean removal efficiencies of COD, TSS and TKN in the planted unit were 81%, 85% and 56%, respectively. These results are very similar to those found by Molle *et al.* (2005), based on a comprehensive survey of the first stage (also with three beds) of the French system treating raw domestic sewage in France: mean values of 79%, 86% and 58% respectively.

In experiment 2, with a decrease of the total available surface area, the effluent concentrations increased and removal efficiencies decreased. However, if one considers that only a single stage was being used to treat raw sewage with only 2/3 of the area of the typical first-stage vertical wetland, the mean BOD removal of 74% in the planted unit and 76% in the unplanted unit can be considered satisfactory. In the planted unit, which represents a real wetland, 88%

of the BOD samples met the regional discharge standard (Minas Gerais state, Brazil, 60 mg L^{-1}).

In experiment 2, not only the influent concentrations of some constituents were higher, but also their variability (as shown in Figure 5.1). However, the system showed to be able to absorb well the variations, a fact that is also endorsed by Molle *et al.* (2005). The poorer performance for some constituents (COD and TSS) during experiment 2 is probably associated not only with the characteristics of the influent, but also with the higher loading applied on the system, which is expected to influence organic matter removal and nitrification capacity (Brix *et al.,* 2005).

However, in terms of nitrification, there was a satisfactory ammonium removal in both phases (mean values of ammonium in the planted unit of 59% and 51% in experiments 1 and 2, respectively), highlighting one of the advantages of the French system, compared with many wastewater treatment systems which are not capable of removing ammonium. In spite of the higher organic loading, ammonium removal was still present in experiment 2. Total N removal could not be investigated in both phases due to the lack of laboratory results of nitrate in Experiment 2. Phosphorus removal was not substantial.

In terms of statistical analysis between experiments 1 and 2, it can be seen from Table 5.8 that effluent concentration values from most constituents were significantly different ($p \leq 0.05$). However, as shown in the box-and-whisker plots, the influent concentration in experiment 2 was also higher. This led to the fact that the removal efficiencies in both phases were not significantly different, with the exception of COD and TSS.

When comparing the planted and unplanted units in terms of effluent concentrations and removal efficiencies, it can be said that, in general, there were no significant differences between them (as shown in Table 5.8, with the single exception of TKN in experiment 1). These findings are similar to those obtained by Ciria *et al.* (2005) and Torrens *et al.* (2009).

The plant Tifton 85 showed very good adaptability to all the different hydraulic loads and feeding regimens tested. The plant grew satisfactorily and covered the whole bed surface, all year round. Similar results were observed by Matos *et al. (*2008), who observed good adaptability and biomass productivity with Tifton 85, applied in a constructed wetland treating dairy effluents. There was a concern here that plant growth could be impaired in the second experiment due to the long rest period of seven days without sewage or water supply, but this was not the case, even during the dry months without rainfall. In experiment 1, the average value of Tifton 85 biomass production observed was 2.9 kg of dry matter/m^2.year (29 ton dry matter/ha.year). Tifton 85 was responsible for removing 3.1% of the total N mass applied to the planted unit, indicating that plant uptake accounted for only a small fraction of the total N removal (Lana, 2013).

During experiment 2, clogging on parts of the surface of the unplanted unit were observed and no sludge removal was required during the monitoring period.

The concentrations and removal efficiencies obtained at experiment 1 complied with Brazilian discharge standards for all parameters evaluated. The three beds showed removal efficiencies of COD, TSS and TKN in the planted unit of 81%, 85% and 56%, respectively. These results are very similar to those found by Molle *et al.* (2005), based on a comprehensive survey of the first stage (also with three beds) of the French system treating raw domestic sewage in France: 79%, 86% and 58% respectively.

There was no statistical difference between planted and unplanted beds in terms of organic matter (BOD and COD) removal ($p > 0.05$). These findings are similar to those obtained by Ciria *et al.* (2005) and Torrens *et al*. (2009). Once there is no statistical difference between planted and unplanted beds, the main analysis of the results were focused only in planted beds. In experiment 2 the planted unit achieved different results in terms of BOD and COD: good mean BOD removal of 74% but worse than experiment 1 and poor mean COD removal of only 59% (worse than in experiment 1), a fact that still deserves further investigation. It was observed that 88% of BOD samples in the planted unit met the Brazilian discharge standard (60 mg L^{-1}).

5.2 Planted and unplanted units - VFCW treating raw domestic sewage

5.2.1 Concentrations and efficiencies removal

As mentioned in the previous sections for the Phase 2 of operation the unplanted unit received Tifton 85, becoming also planted. In phase 2, all conditions of operation were unchanged. Thus, the system operated with two planted units with the same HLR. In Phase 2 of operation, the first stage treatment showed good performance in terms of organic matter and other pollutant removal. In this phase of operation, it was evident the influence of plants in the

VFCW treatment. In order to compare the results obtained in both phases, Table 5.9 summarizes the main results obtained in both phases.

Table 5.9**:** Descriptive statistics of concentrations of the influent and effluent and removal efficiencies of physical chemical parameters monitored during both phases.

			Phase 1			Phase 2		
	Parameter	**Statistic descriptive**	**Influent**	**Planted PU**	**Unplanted CU**	**Influent**	**Planted PU1**	**Planted PU**
Concentration (mg L^{-1})	BOD_5	n	30	15	15	42	18	24
		Median	175	40	35	254	64	57
		Average	229	46	45	269	72	63
		c.v	0.78	0.53	0.59	0.36	0.56	0.52
	COD	n	50	28	22	26	11	15
		Median	603	198	156	505	117	150
		Average	561	245	194	506	119	154
		c.v	0.37	0.65	0.59	0.32	0.31	0.57
	TSS	n	45	27	18	25	10	15
		Median	203	59	62	149	34	39
		Average	218	61	73	210	43	53
		c.v	0.49	0.55	0.63	0.57	0.49	0.82
	NH_4^+-N	n	44	23	21	39	17	22
		Median	38	17	19	26	14	13
		Average	34	16	18	28	14	14
		c.v	0.35	0.41	0.27	0.24	0.31	0.58
	TKN	n	45	27	18	36	18	18
		Median	43	19	24	28	15	14
		Average	40	20	22	29	14	15
		c.v	0.30	0.33	0.31	0.27	0.32	0.44
Removal efficiency (%) (Average)	BOD			74	76		74	74
	COD			59	61		76	67
	NH_4^+ -N			51	44		45	52
	TSS			68	60		76	72
	TKN			49	42		49	48

PU- Planted unit; CU – Control unit; PU1- Planted unit 1; PU – Planted unit; C.V- Coefficient of variation

In general, very good performance of the system in both phases of operation was observed. The system improved its performance when it was working with two planted units (Phase 2). On the other hand, the results obtained in Phase 2 indicated that the influent concentrations of all parameters analysed were in the range of the raw domestic sewage in developing countries (von Sperling, 2014). Although no statistically significant differences between the planted unit and the unplanted unit in terms of effluent concentrations and removal efficiencies were observed, the removal efficiencies of almost all parameters increased in the planted unit, giving an indication of the role of the plants in pollutants removal. In both phases, the average of removal efficiencies of parameters as COD, TSS and TKN were similar to those found by Molle *et al.* (2005). Regarding evaluation of the first treatment stage of VFCW about hundreds of the systems were installed in France (average of removal efficiencies of 79% of CDO; 86% of TSS and 54% of TKN).

Table 5.10 presents *p*-values of the Many-Whitney U-test comparing median removal efficiencies within and between both Phases.

Table 5.10: *p*-values of the Mann-Whitney U-test comparing removal efficiencies in both experiments and planted and unplanted units.

Parameter	**Phase 1** **PU X CU**	**Phase 2** **PU1 X PU**	**Phase 1 x Phase 2** **CU X PU**
BOD	0.6333	0.6176	0.9880
COD	0.4343	0.1690	0.5566
TSS	0.2989	0.5417	0.1295
NH_4^+-N	0.0760	0.3079	0.0845
TKN	0.0952	0.9369	0.2820

(*) *p*-values $\leq$ 0.05: sample medians are significantly different
PU- Planted Unit; CU – Control Unit; UP1- Planted Unit 1

Based on Mann-Whitney test at a 5% significant level, no statistical significant differences (p > 0.05) of all parameters analysed between both phases and within the phases were observed. This means that the first treatment stage of VFCW performed well in both phases in terms of organic matter and pollutants removal. However, it is important to emphasize the importance of plants in the treatment system, mainly in Phase 2, once in this phase the system increased its capacity in terms of COD, NH_4^+-N, TSS and TKN removals.

5.2.2 Organic matter removal

The results presented in Table 5.9 showed positive performance of the system concerning raw domestic sewage treatment. Considering only two units of first stage treatment of the VFCW, it can be seen that the system showed very good performance compared to other processes such as septic tanks mainly used in developing countries. It is important to reinforce that VFCW received raw domestic sewage without primary treatment. Figure 5.2 presents the BOD and COD removal efficiencies during Phases 1 and 2.

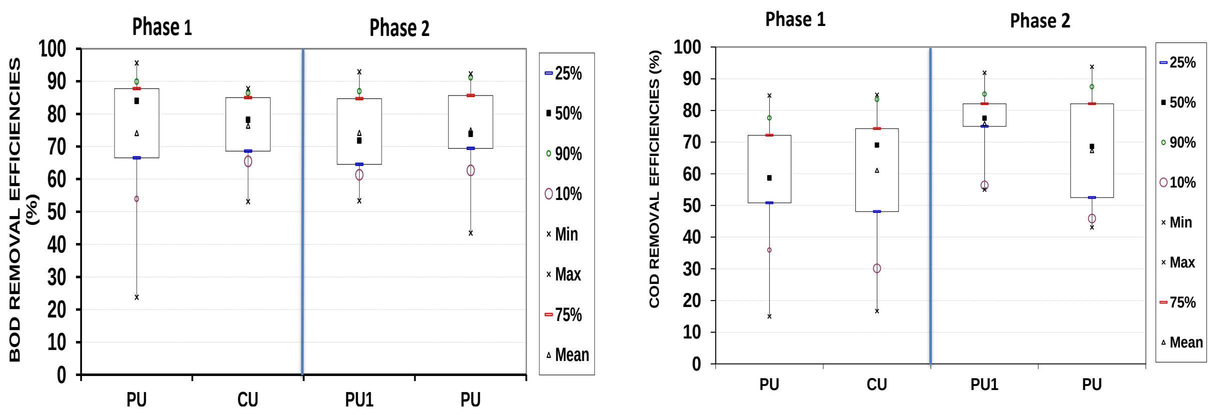


Figure 5.2**:** Box Whisker of BOD and COD removal efficiencies in Phase 1 and Phase 2.

As previously discussed there were no significant differences between both phases regarding BOD and COD removal efficiency found. The percentage removal efficiency of BOD was almost the same in both phases. As indicated in Figure 5.2, average removal efficiency of COD in Phase 2 was higher than in Phase 1. It can be explained by mechanisms involved related to the COD removal (filtration capacity). It was reinforced in the foregoing discussion that the COD removal had shown varied results according to different contexts (Molle *et al.* 2005).

5.2.3 Evaluation of compliance to discharge standards for BOD and COD

Figure 5.3 presents the concentrations distribution and efficiencies distribution obtained for BOD in both phases. The graphics demonstrated very good performance of the system in terms of organic matter removal.

Considering regional discharge standard according to DN COPAM/CERH 01/2008 (Minas Gerais State, Brazil), it was observed that in Phase 1, in planted unit 80 % of the BOD samples met the regional discharge standard (60 $mg.L^{-1}$), whereas in the control unit only 75% of the BOD samples met the regional standard. The analysis showed that in Phase 2, in both planted units, only 55% of the BOD samples met the regional discharge. As stated before, the same performance of the wetland concerning BOD removal in both phases were observed.

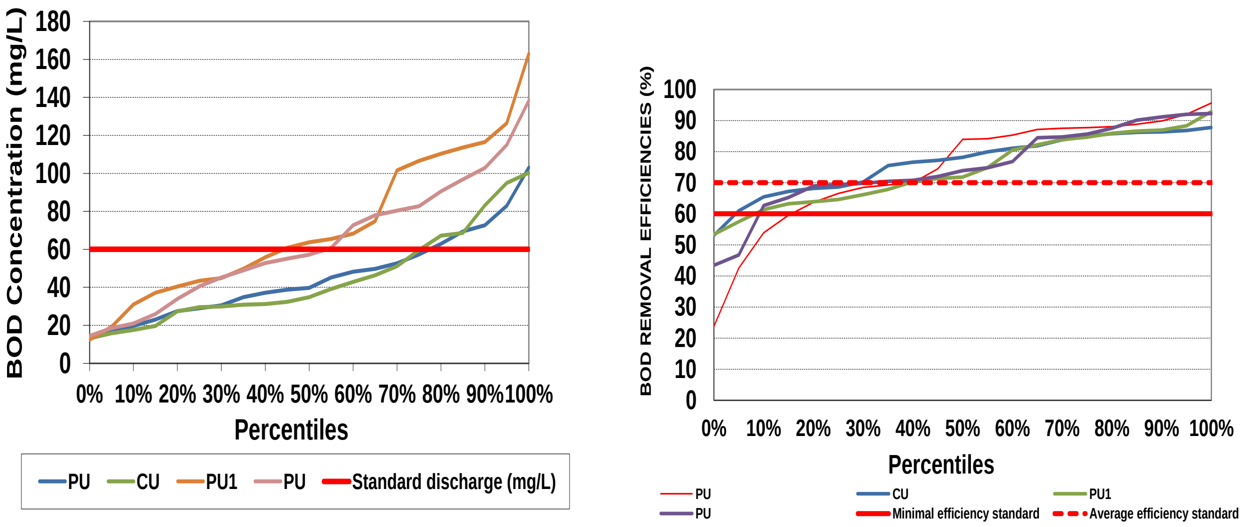


Figure 5.3: BOD concentrations and efficiencies distribution in both phases.

In addition, efficiencies distribution were evaluated to what extent the facility can provide the service in the Minas Gerais region following regional legislation. Thus, it was observed that in Phase 1, in the control unit about 95% of the BOD samples met efficiency standard while in the planted unit, 85% of the samples met the efficiency standard stated by the regional legislation. For BOD removal efficiency, the minimal efficiency standard stated by the regional legislation was of 60% and the average removal efficiency standard was of 70%. In Phase 2, in the two planted units 90% of BOD samples met the efficiency standard stated by the legislation. Figure 5.4 presents the concentrations distribution and efficiencies distribution obtained for COD removal in both phases.

The system also showed very good capacity concerning COD removal. For COD removal the discharge standard of Minas Gerais state, stated by the legislation was of 180 mg L^{-1} and the minimal removal efficiency was of 55% whereas the average removal efficiency was of 65%. The analysis from Figure 5.4 demonstrated that during Phase 1, in the planted unit only 40% of the COD samples met the regional standard discharge, while in the control unit about 65% of the COD samples met the regional discharge standard. In Phase 2, in the planted unit 65% of the COD samples met the regional discharge standard, whereas in the planted unit 1, which had been previously the control unit in Phase 1, about 100% of the COD samples met the regional discharge standard. The results revealed that the system was able of COD removal but Phase 2 showed a higher capacity of COD removal than Phase 1.

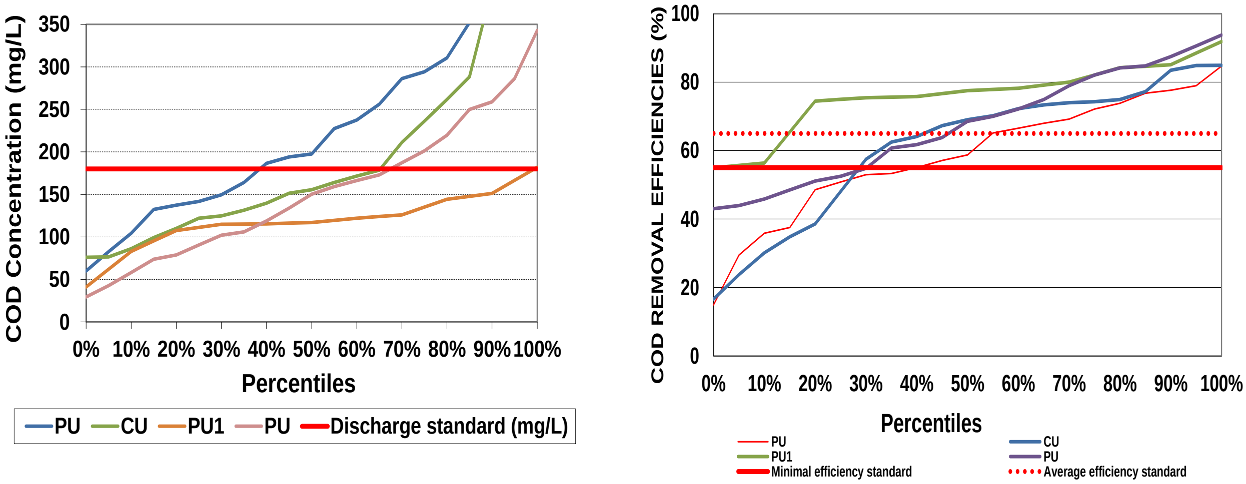


Figure 5.4: Box Whisker of COD concentration and efficiencies distribution in both phases.

Concerning the minimal removal efficiency of COD, the analysis showed that in Phase 1, in the planted unit, 60% of the COD samples met the regional minimal efficiency while in the control unit 70% of the COD samples met the regional minimal efficiency. In terms of treatment efficiency, Phase 2 showed higher values than Phase 1. About 90% of the COD samples met regional minimal efficiency in phase 2.

The removal efficiencies of the constituents were higher in the planted beds, indicating the influence of plants in relation to the treatment. This is according to Ciria *et al.* (2005), Tanner (2001) and Brix (1997). These authors reported that the presence of the macrophytes stabilize the surface of the beds, provide good conditions for physical filtration and prevent vertical flow systems from clogging (Brix, 1997; Tanner, 2001). The system was able to perform partial ammonium removal, indicating the presence of an oxidative environment. On the other hand, a higher percentage of ammonium removal efficiency in the planted units than in the unplanted units of VFCW were observed.

5.2.4 Total suspended solids

Many authors like Ciria *et al.* (2005) and Koottatep *et al.* (2004) have demonstrated that the removal of TSS is almost entirely due to physical process, filtration capacity rather than the biological process associated with the microbial community or with the plants. Thereby, the TSS removal is associated with the operational and environmental conditions of the system. Table 5.8 showed very good TSS removal efficiency, mainly in Phase 2, when the system was working with two planted units. Although statistical significant differences were not observed, the system showed a better performance in the planted units than in the unplanted unit. Similar results were found by Lana (2013) at the same treatment system. Figure 5.5 presents TSS removal efficiencies and the concentration distribution in both phases.

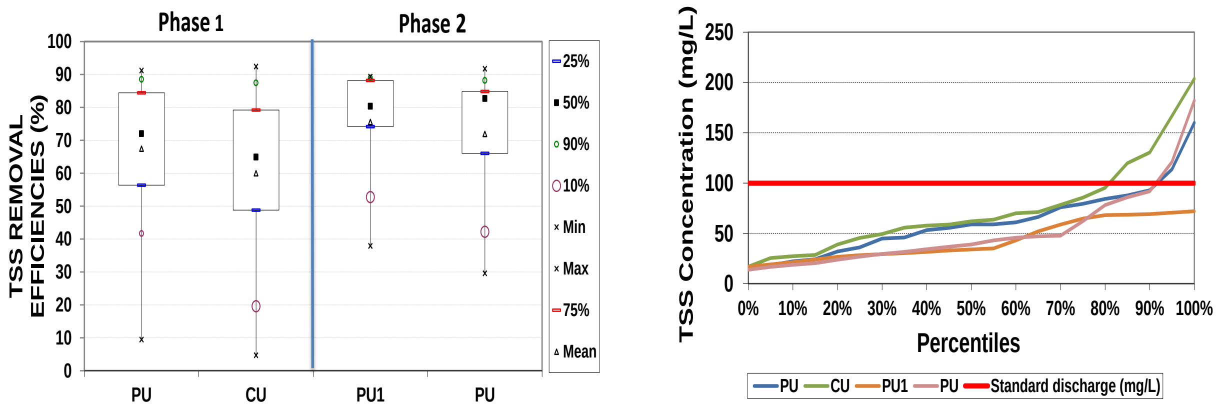


Figure 5.5: Box Whisker of TSS removal efficiency and TSS concentration distribution.

Given the TSS removal process discussed previously, it was observed in Figure 5.5 great variability of the TSS removal between planted and unplanted unit in Phase 1 and between two planted units in Phase 2. However, the system showed satisfactory performance in terms of TSS removal. The average removal efficiency of 76% in the planted unit during Phase 2 was similar to that found by Molle *et al.* (2005), equivalent to 86%.

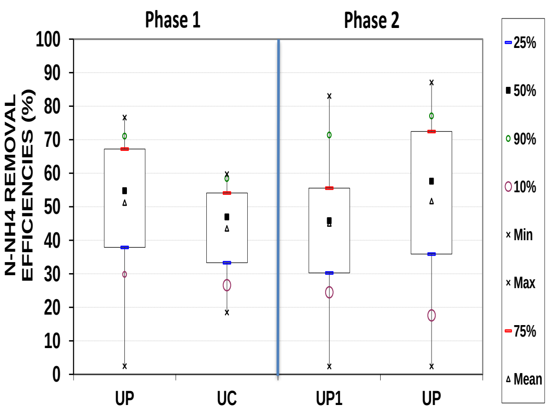
The analysis in Figure 5.5 indicated that in Phase 1, around 95% of the TSS samples met the discharge standard, while in Phase 2 about 100% of the TSS samples met the discharge standard of Minas Gerais State, Brazil (100 $mg.L^{-1}$).

When the system operated with two planted units only, it increased its performance in terms of pollutants removal. The mean removal efficiencies achieved were of 74% for BOD, 76% for COD, 76% for TSS, 49% for TKN. These results were very similar to those found by Molle *et al.* (2005).

5.2.5 NH_4^+-N and TKN removal

The results obtained for NH_4^+-N and TKN removal efficiencies are presented in Table 5.9. Based on the results obtained, it was inferred that the system showed good capacity for nitrification. Figure 5.6 presents $NH4^+$-N and TKN removal efficiencies during Phase 1 and

Phase 2. Due to problems related with the ionic chromatography equipment were analysed only two forms of the nitrogen, $NH4^+$-N and TKN respectively.



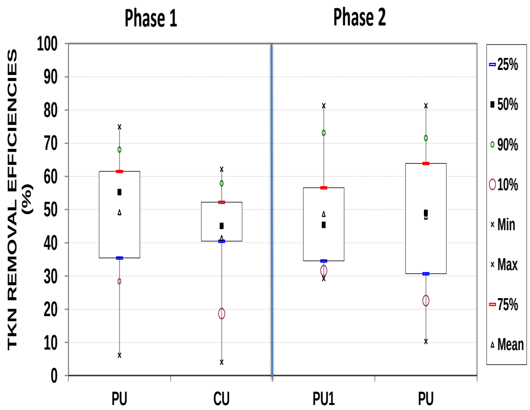


Figure 5.6: Box Whisker of NH_4^+- N and TKN removal efficiencies in both phases.

The data analysis demonstrated that the system was able to achieve nitrification. The removal efficiencies of NH_4^+-N and TKN were higher in the planted units compared to the control unit in Phase 1. The TKN removal efficiency obtained in both phases were similar to that obtained by Molle *et al.* (2005) equivalent to 54%. The results of NH_4^+-N and TKN removal reinforced the theory that the plants in VFCW can influence the environment by providing oxygen for the nitrification process (Cota, 2011; Kadlec & Wallace, 2008; Brix, 1997).

5.2.6 Evaluation of compliance with discharge target for NH_4^+-N

Different results found mainly in the ammonia removal can be explained by oxygen availability for nitrification in both units (Molle *et al.* 2005).

Figure 5.7 presents NH_4^+- N concentrations distribution obtained in both phases. The graphics demonstrated very good performance of the system regarding NH_4^+-N removal.

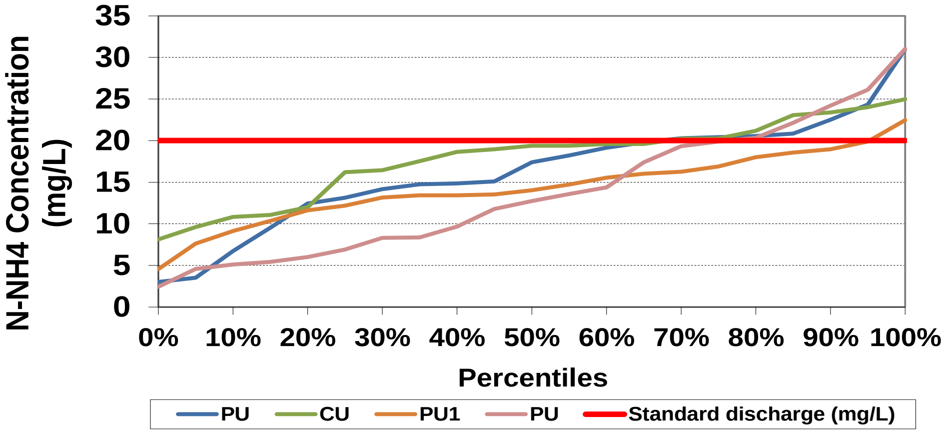


Figure 5.7: $NH4^{+}$- N concentration distribution in both phases.

In Figure 5.7 it was observed that in Phase 1 only 65% of the NH_4^{+}-N concentration met the discharge target of 20 $mg.L^{-1}$, while in Phase 2, about 95% of the NH_4^{+}-N concentration met discharge target. This result demonstrates the importance of vegetation in the treatment process.

5.3 Hydraulic behaviour and solids mass balance of raw domestic sewage

During Phase 2, measurements were taken in order to evaluate the hydraulic behaviour of the system and its performance in terms of solids mass reduction over time in the effluent. The hydraulic evaluation and TSS mass balance of raw domestic sewage were conducted during the period of November 2014 up to August 2015. Below are presented the results obtained during the evaluation of mass balance in the system treating raw domestic sewage.

5.3.1 Hydrographs of effluent flow

Authors like Kadlec and Wallace (2008), Kayser and Kunst (2005) revealed that in the intermittent pulse feed (PF) systems, normally there is a rising outflow for a brief period, followed by a declining outflow. The authors agree that the pronounced increase and decrease of effluent curves show a good and fast dewatering of the filter, which indicates an effective oxygen transfer through convection and diffusion in the unit bed. The curves represented in Figure 5.8 illustrate the variation of the effluent outflow over time conducted during four days. On 14/11/2014 and 08/04/2015 during the test, the VFCW unit was planted. On 01/04/2015 and 15/04/2015, the test was done when the unit bed was unplanted. For comparison purposes are presented and discussed the variations of effluent outflow in the planted and unplanted units. Each curve indicates the variation of effluent outflow over time corresponding to a batch loading of 0.540 m^3.

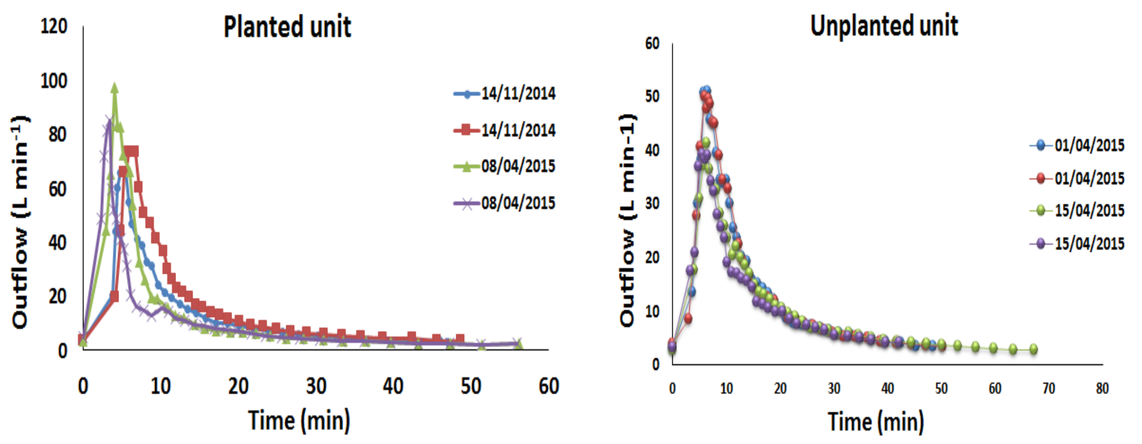


Figure 5.8: Variation of effluent outflow over time in the planted and unplanted units.

The variation of effluent outflow over time in the planted unit was subsequently measured for ten (10) batches loading as illustrated in Figure 5.9.

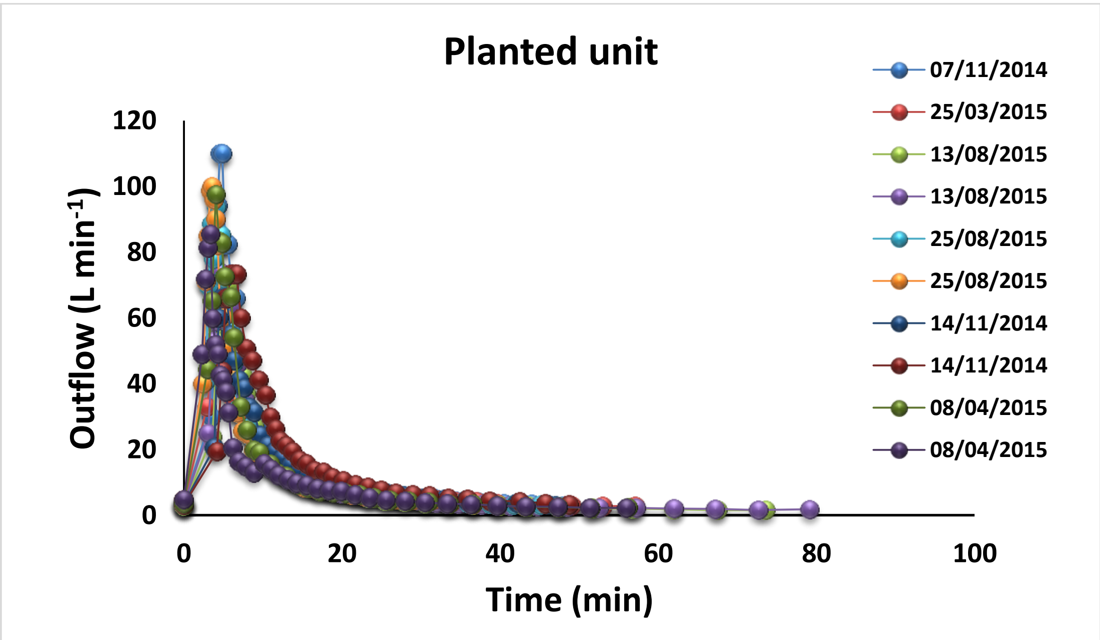


Figure 5.9: Variation of effluent outflow over time in the planted.

As observed in Figure 5.8 and Figure 5.9, the curves of the outflow increase up to the peak and then are followed by a slow reduction. The tests in the planted unit showed that after feeding, in five minutes, the effluent outflow increases and it achieves the peak which varied from 1.10 L s^{-1} up to 1.63 L s^{-1}. Lana (2013) found similar results evaluating the hydraulic behaviour at the same VFCW. In the unplanted unit a similar pattern or profile of the curve exhibiting a pronounced peak was observed. At the first six minutes after feeding, the maximum effluent outflow achieved was of 0.85 L s^{-1}. The data analysis showed higher peaks of the effluent outflow in the planted unit and lower peaks of the effluent outflow in the unplanted unit. Cota (2011) investigating the same system found similar results as those found here. Molle *et al.* (2005) reported that when applying batches on full-scale experiments a decrease of the Infiltration Rate (IR) during the operation with variation of saturation level in the filters can be observed. The higher peak of the effluent outflow observed in the planted unit than in the unplanted unit can be associated with the increase of the IR due to the presence of plants, with moisture saturation and preferential water pathways after feeding (Molle *et al,*. 2005; Cota, 2011).

According to Molle *et al.* (2006), after the pulse loading on full-scale experiments decrease the IR during the operation with moisture saturation in the filters. Therefore, preferential pathways of water into the reed bed can explain the differences observed in the curves of effluent outflow over time. Another factor is correlated with the water saturation level in the pores and the water distribution on the top of the surface. In planted and unplanted units apparently there is even hydraulic behaviour represented by the same profile of the curves. This indicates good and fast dewatering of the filter in the units evaluated.

5.3.2 Hydrographs of cumulative volume profile over time

Cumulative volume over time was also measured in the planted and unplanted units. This parameter expresses the variations of volume over time after pulse loading and total volume drained at the outlet. Figure 5.10 shows cumulative volume profile over time in the planted and unplanted units.

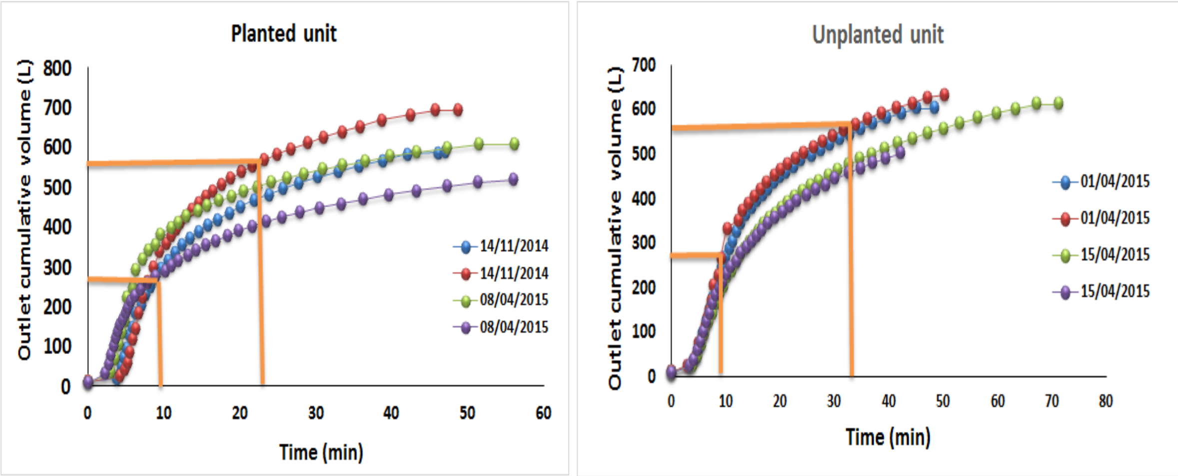


Figure 5.10: Cumulative volume profile over time in the planted and unplanted units.

By observing Figure 5.10 and Figure 5.11 it can be seen that the cumulative volume in the outlet increase up to maximum value which remains constant until the end of the test in the planted unit and unplanted units. The analysis showed that in the planted unit, 50% of volume applied was achieved at the sixth and ninth minute and 100% at the twenty third minute after pulse loading. Contrary the planted unit, the test showed that in the unplanted unit 50% of volume applied was achieved at the tenth and twelfth minute after pulse loading and 100% at the twenty third minute. Again, there is higher IR in the planted unit than in the unplanted unit and the preferential pathways of water into the reed bed can explain the differences of cumulative profile over time.

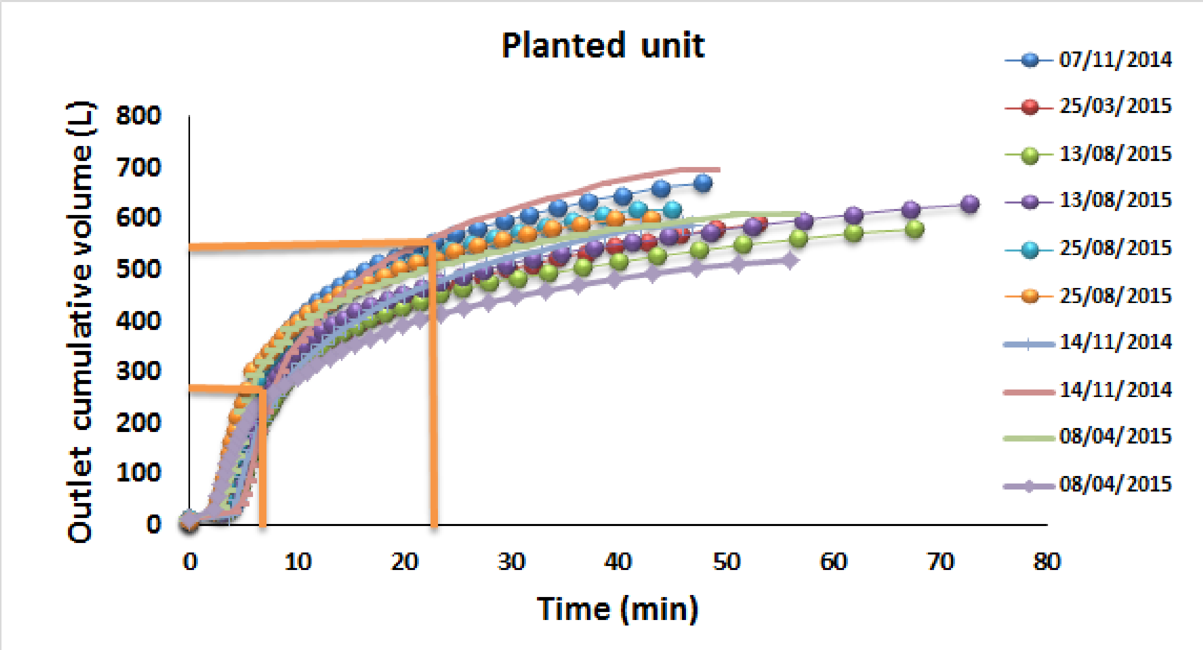


Figure 5.11: Cumulative volume profile over time in the planted unit.

5.3.3 Effluent TSS concentration profiles over time

Figure 5.12 and Figure 5.13 present the variation of the effluent TSS concentration over time. The TSS concentration profile was determined along the curves of the effluent outflow previously discussed. The planted and unplanted units were compared in terms of effluent TSS concentration over time.

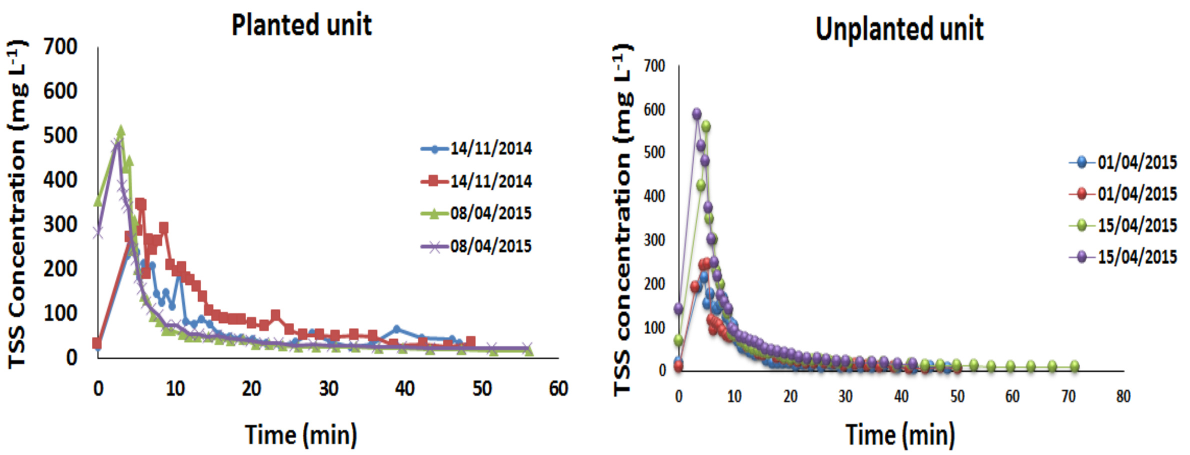


Figure 5.12: Variation of TSS concentration over time in the planted unit and unplanted unit.

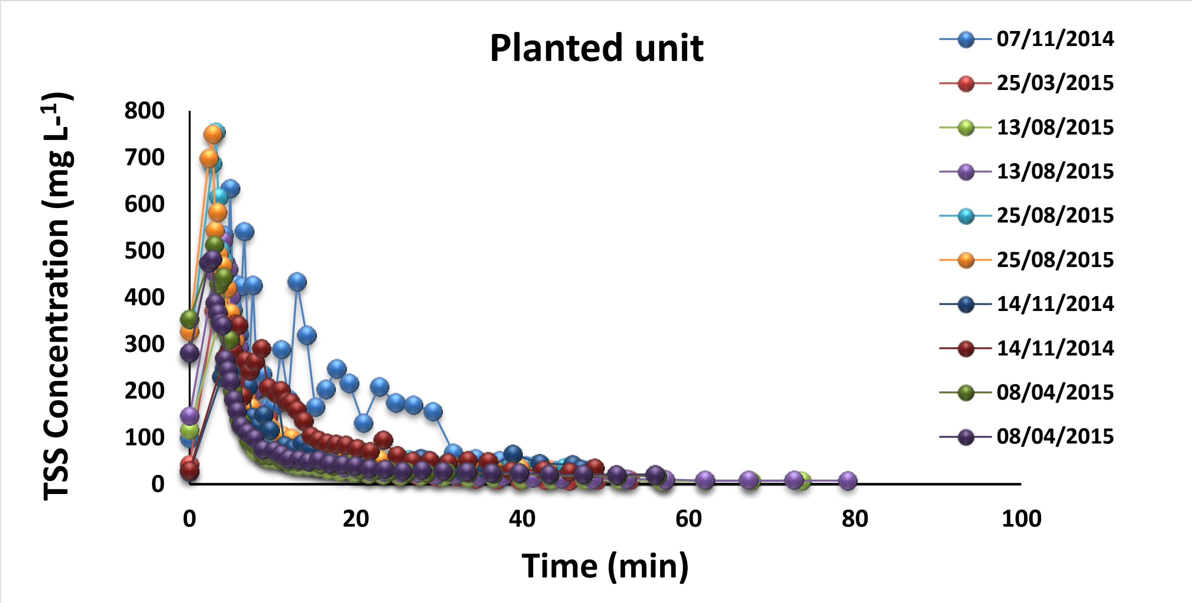


Figure 5.13: Variation of TSS concentration over time in the planted unit.

Analysing Figure 5.12 and Figure 5.13, the data analysis showed that, after pulse loading the TSS concentration increases and decreases slowly over time as expected following the same behaviour as the outflow. These variations of TSS concentrations were similar to the effluent outflow. However, it can be seen from all graphics that the curve of TSS concentration and the effluent outflow had the same pattern or profile. On the other hand, the analysis showed that higher TSS concentration is associated with higher outflow and lower TSS concentration is associated with lower outflow.

Such as the curve of the effluent outflow, similar behaviour was observed between the curves of TSS concentration. Nevertheless, this type of fitting has demonstrated good performance of the system concerning raw domestic sewage treatment. In the planted unit it was observed that after five minutes of feeding the peak of TSS concentration was achieved, ranging from 286 mg L^{-1} to 513 mg L^{-1}. On the other hand, in the unplanted unit a pronounced peak of TSS concentration in the effluent was observed ranging from 245 mg L^{-1} up to 588 mg L^{-1}. The analysis showed correlation between the performances of the system and Hydraulic Retention Time (HRT). In other words, the quality of the effluent obtained after pulse loading improved with time in the planted and unplanted unit. The efficiency of the system in terms of solids removal is higher when the HRT increases. The TSS concentration curves showed the same profile as outflow curves, which indicates the same hydraulic behaviour of the pulse loading and the same efficiency in terms of TSS removal.

5.3.4 Effluent solids load profiles over time

In the previous analysis, similar pattern of the curves between the effluent outflow and TSS concentration over time was observed. In this context, it becomes important to evaluate the Solids Load (SL) in the outlet over time (= Outflow x TSS Concentration). The SL expresses the total solids mass of the effluent over time. Figure 5.14 and Figure 5.15 show the variation of the SL in the effluent over time in the planted unit and unplanted units.

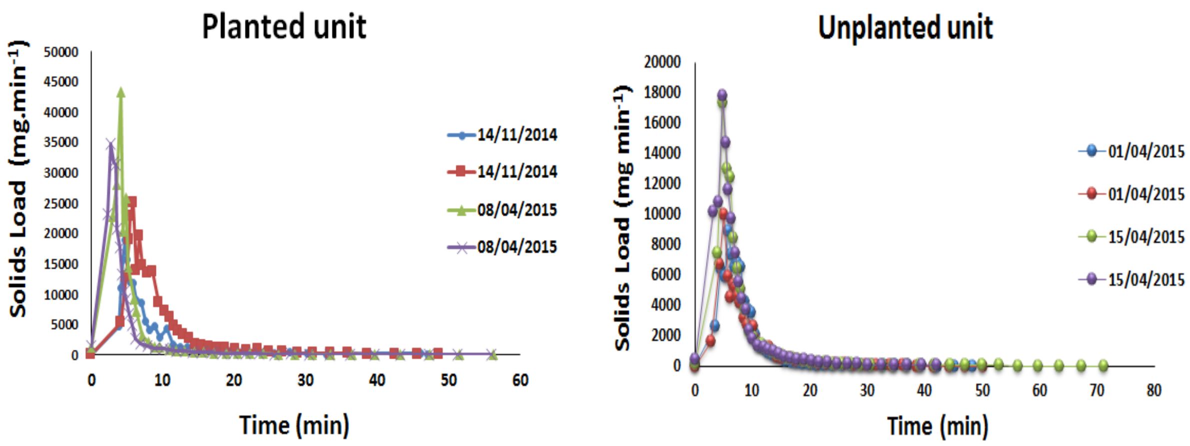


Figure 5.14: Variation of solids load over time in the planted and unplanted unit.

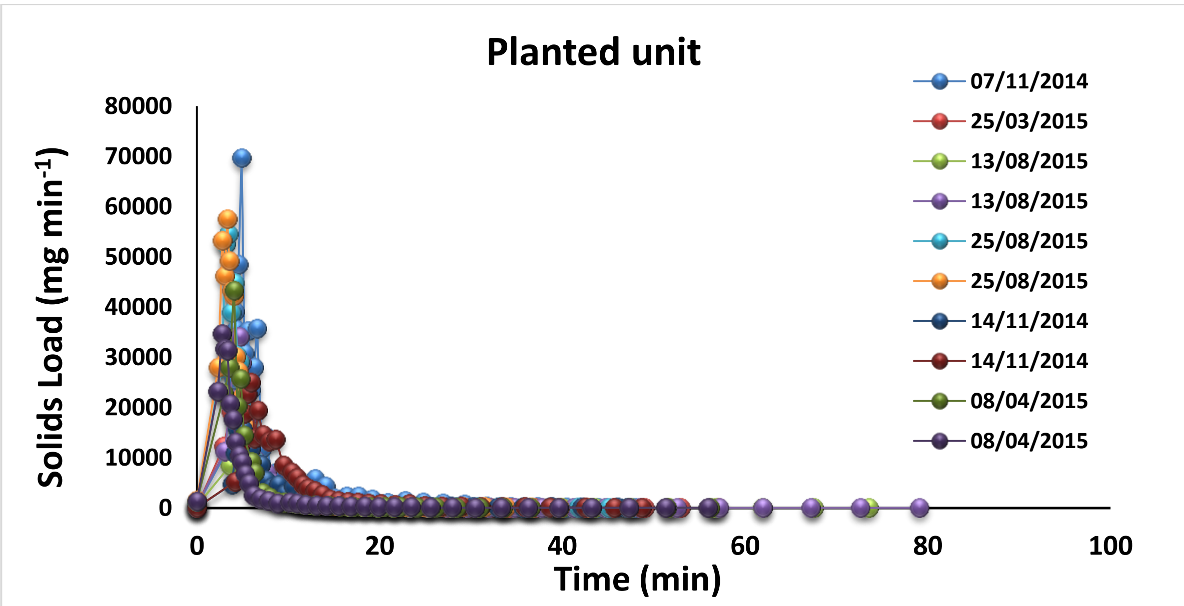


Figure 5.15: Variation of solids load over time in the planted.

Analysing Figure 5.14 and Figure 5.15 the same pattern can also be seen of the curves between outflow of the effluent, TSS concentration and SL over time. Besides, it was possible to observe substantial similarity between the curves of all parameters evaluated. The same profile between the curves of outflow, TSS concentration and solids loading rate demonstrated suitable operation and the same hydraulic behaviour between batch loading in the system. On the other hand, the SL increase when the TSS are high, especially in the first minutes.

5.3.5 Cumulative influent and effluent solids mass profiles over time

The percentage of the TSS mass applied in the effluent expresses a relation between the cumulative mass at the effluent over time and the TS mass of the influent. This parameter allows the determination of the mass removal efficiency in the system. Figure 5.16 and Figure 5.17 show the percentage of the mass applied in the effluent over time.

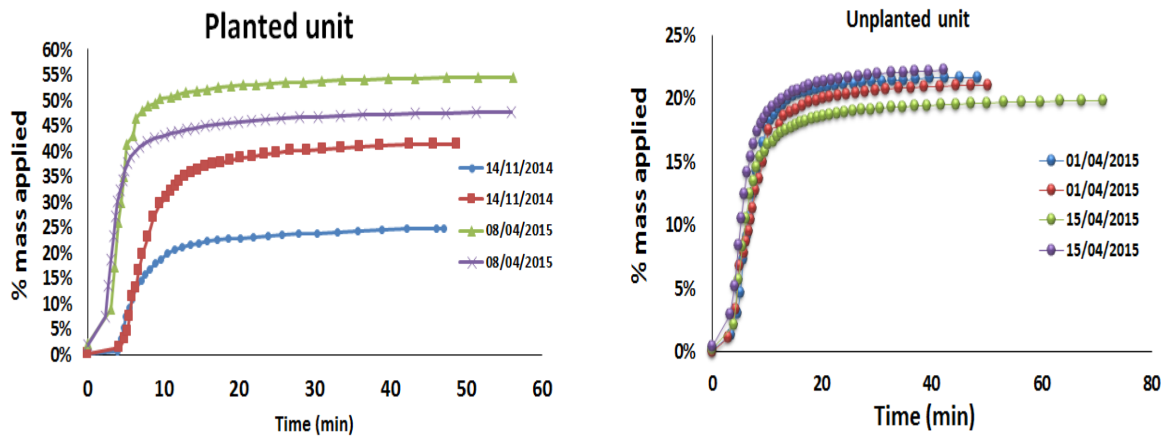


Figure 5.16: Percentage of mass applied that leaves in the effluent in the planted and unplanted units.

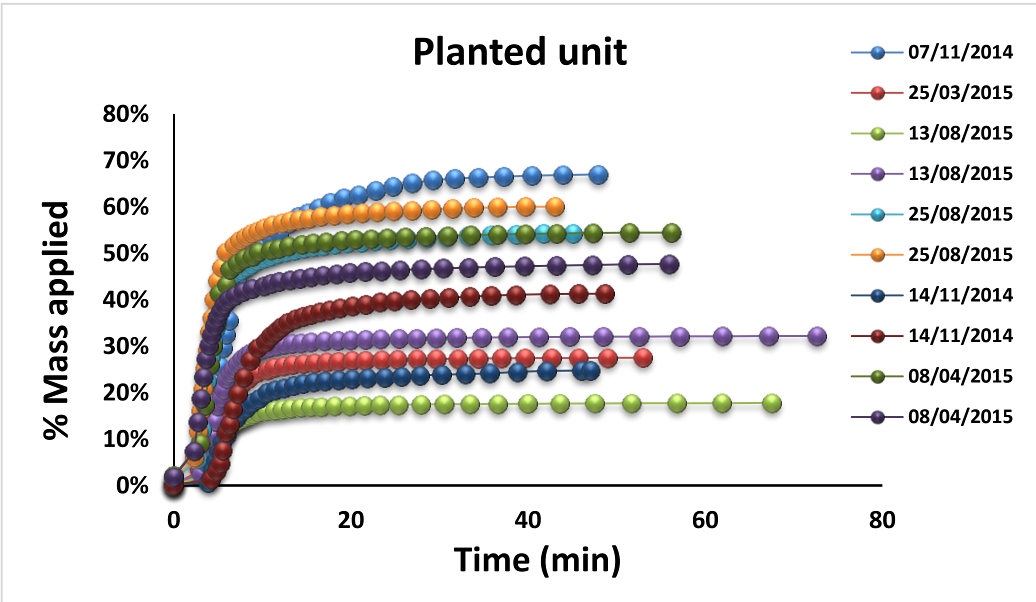


Figure 5.17: Percentage of mass applied that leaves in the effluent in the planted unit.

By observing Figure 5.16 and Figure 5.17 the results show that the percentage of total solids mass applied increase up to maximum value which remains constant until the end of the test. In the unplanted unit, on 01/04/2015 the solids mass applied determined in the effluent was of 21%, this means that the solids mass removal efficiency was of 79%. In the planted unit, on 14/11/2014 for the second pulse loading, it was obtained 41% of solids mass applied in the effluent indicating the total solids mass removal efficiency of around 59%. The curves showed similar behaviour and a decrease of total solids mass removal over time in the system. The high solids mass was removed during the first minutes after pulse loading. On 14/11/2014 in the planted unit the evaluation indicated that about 95% of the total solids mass in the effluent was removed in 23 minutes after the batch feeding while for the test conducted on 01/04/2015 and 15/04/2015, in unplanted unit around 95% of the total solids mass was removed in 17 minutes after feeding.

The total solids mass applied for each batch feeding (= Volume (540 L) x Influent TSS concentration) is presented in Table 5.11. Nevertheless, the results obtained proved that there are dynamic changes related with hydraulic behaviour in the system along the time. Freire *et al.* (2010) reported that there are different water fluxes in the content of the VFCW as result of different porosity areas that the flow rates across the system.

Table 5.11 presents a summary of the main parameters analysed to determine the variation of the effluent outflow and the percentage of mass removal in the system.

Table 5.11: Summary of parameters evaluated for mass balance determination.

VFCW	Planted unit		Unplanted unit		Planted unit		Planted unit	
Parameter	**Batch 1**	**Batch 2**	**Batch 1**	**Batch 2**	**Batch 1**	**Batch 2**	**Batch 1**	**Batch 2**
Measurement duration (min)	48	51	51	52	59	59	74	45
TSS influent ($mg.L^{-1}$)	550	550	410	410	337	337	619	619
TSS maximum effluent ($mg.L^{-1}$)	286	344	213	245	513	483	560	588
TSS minimum effluent ($mg.L^{-1}$)	26	26	7	7	18	21	9	15
Maximum outflow ($L.min^{-1}$)	66	73	51	50	97	85	41	39
Minimum outflow ($L.min^{-1}$)	3	3	3,2	3,4	2	2,3	2,7	3,3
Maximum SL ($mg.min^{-1}$)	18,876	25,007	8,942	9,973	43,317	34,706	17,373	17,822
Minimum SL ($mg.min^{-1}$)	92,00	94,00	24,00	24,00	40,00	48,00	25,00	62,00
Volume applied (L)	540	540	540	540	540	540	540	540
Effluent cumulative volume (L)	588	703	604	634	610	521	614	503
Total solids mass applied (kg TS)	0,297	0,297	0,221	0,221	0,181	0,181	0,334	0,334
Effluent cumulative mass (kg TS)	0,074	0,123	0,047	0,046	0,099	0,086	0,066	0,074
(%) Mass removal efficiency	75	59	78	79	46	52	80	78

In general, the analysis indicated that the system showed good performance in terms of total solids mass removal. However, the system showed higher TS mass removal efficiency in unplanted unit than planted units. The total solids mass removal efficiencies determined were of 75% and 59% in the planted unit, 78% and 79% in the unplanted unit, 46% and 52% in the planted unit 80% and 78% in the planted unit.

5.4 VFCW treating septic tank sludge

5.4.1 Hydraulic loading rate applied during the OS1, OS2 and OS3

Concerning ST sludge treatment, three operational strategies were investigated. In the first operational strategy, the system operated without any retention of the percolate. Weekly all volume transported by septic tank truck was discharged onto the sludge unit without retention period. The ST sludge was loaded at different volumes, ranging from 3.5 to 16 m^3. No control of the Hydraulic Loading Rate (HLR) was possible due to the variation of sludge origin and tank volume in the trucks. Thus, the system was fed with ST sludge without any control of HLR. This resulted in variation of HLR obtained during the monitoring period.

On the other hand, the experiment with retention period of seven days of the percolate conducted by Koottatep *et al.* (2004), the sludge was loaded with a constant volume of 8 m^3 $week^{-1}$. This allowed the control of HLR and SLR during all period of the experiment. Differently from the operational strategy 1, for the operational strategy 2 the system was fed with strict control of HLR. The sludge from septic tank was loaded at the volume ranging from 2.0 to 8.0 m^3 weekly. This strategy allowed retention of the percolate 10 cm below the surface of the bed during seven days. Higher volumes could not be accepted, because the liquid would cover the surface of the bed. Similar, the operational strategy 2, for operational strategy 3 the system was fed with control of HLR. The septic sludge trucks arrived at the facility and discharged onto the unit 1 where the percolate was retained during seven days. After seven days of retention, the percolate was sent to post-treatment unit for retention during seven days completing a cycle of fourteen days. Figure 5.18 shows the variation of HLR obtained during three operational conditions.

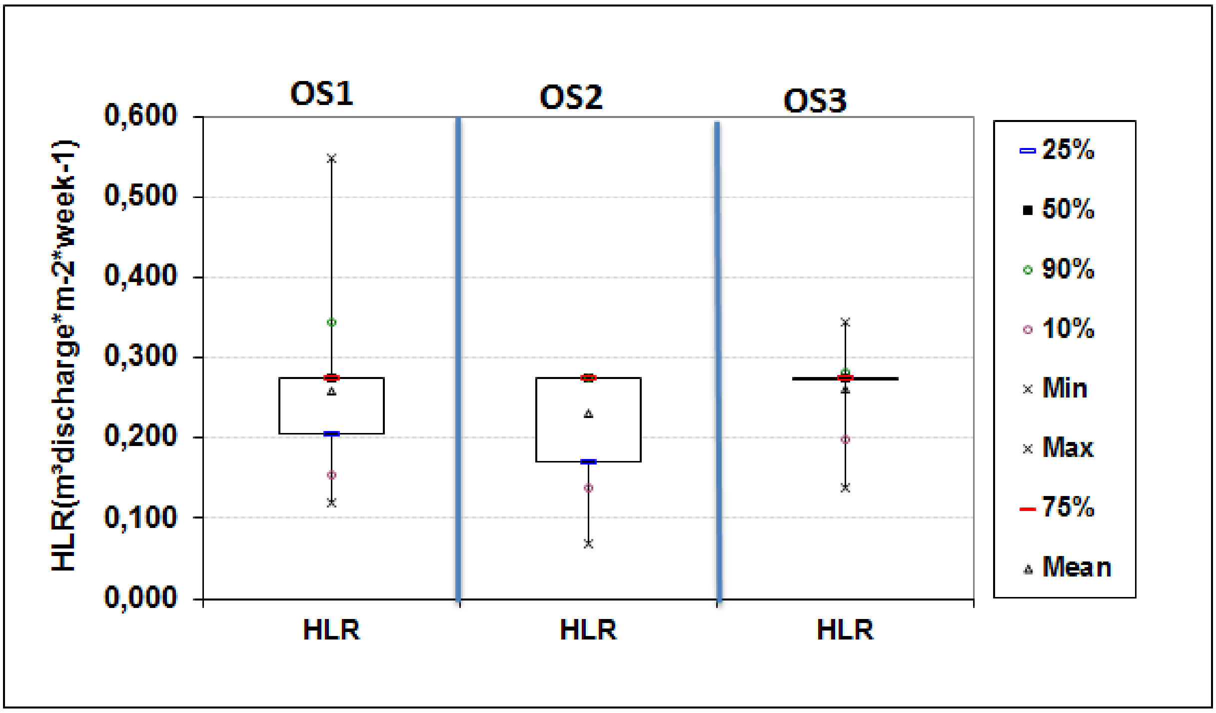


Figure 5.18: Variation of HLR during monitoring period.

In OS1 the minimum value of HLR determined was of 0.120 m³ discharge m^{-2}.week^{-1} and the maximum value found was of 0.549 m³discharge m^{-2}.week^{-1}. The mean of HLR determined was of 0.259 m³discharge m^{-2}.week^{-1} and the median obtained was of 0.275 m³discharge m^{-2}.week^{-1}. The analysis showed expressive variability of HLR applied when the system was operating without retention time. The variation of HLR applied, during the OS1 did not affect the performance of the system. Thus, the system showed itself to work well the under variations of HLR during the monitoring period.

For OS2 the analysis showed less variability of HLR when the system was operating with a retention period of seven days comparing with OS1. In OS2, the HLR ranged from 0.069 to 0.275 m³ discharge m^{-2}.week^{-1}. The mean of HLR determined was of 0.231 m³discharge m^{-2}.week^{-1} and the median obtained was of 0.275 m³discharge m^{-2}.week^{-1}. Besides, previous operations conditions, in OS3 the HLR ranged from 0.137 m³discharge m^{-2}.week^{-1} to 0.343 m³discharge m^{-2}.week^{-1}. The median obtained was of 0.275 m³discharge m^{-2}.week^{-1}. Therefore, less variability of HLR was observed in OS2 and OS3 compared with OS1. The less variability of HLR in OS2 and OS3 is the result of strict control of HLR applied when the system was operating with retention period. Authors like Maeseneer & Peruzzi *et al.* (2008) suggested a hydraulic loading of 1 – 1.5 m³.m^{-2} year^{-1}. However, the system showed itself to work well under the variations of HLR in all operational strategies.

5.4.2 Solids loading rate applied during OS1, OS2 and OS3

Figure 5.19 shows the Solids Loading Rate (SLR) applied for three operational strategies , when the system was operating without a retention period (OS1) and with a retention period (OS2 and OS3). In all cases the VFCW treating sludge from septic tank operated without any control of SLR due to the varied origin and characteristics of ST sludge received. The TS concentrations were determined in the laboratory after sludge application took place. In fact, great variability of SLR during the OS1 and OS2 were observed. For the OS3, the less variability of SLR can be attributed to the limited number of samples analysed during this operation as can be seen in Figure 5.19.

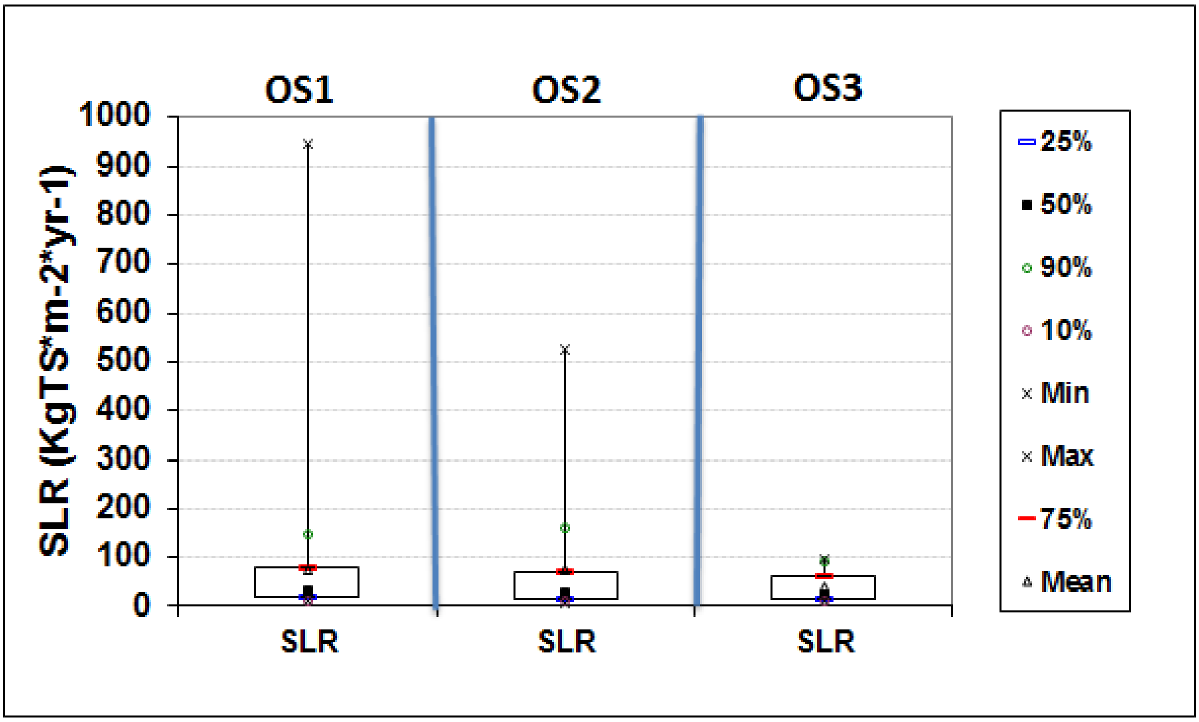


Figure 5.19: Variation of solids loading rate applied for three operational strategies.

In OS1, the minimum value of SLR obtained was of 11 kg TS.m^{-2}.year^{-1} and the maximum value was of 944 kg TS.m^{-2}.year^{-1}. In tropical regions, Koottatep *et al.* (2008) recommended a SLR of 250 kg TS.m^{-2}.year^{-1} as optimum treatment performance with frequency of application of once a week. Treating FS from septic tank and pit latrines at pilot scale, in Cameroon, Kengne *et al.* (2014) recommended a SLR of 200 kg TS.m^{-2}.year^{-1} with frequency of application of once a week. The mean of SLR found in OS1 was of 73 kg TS.m^{-2}.year^{-1} and the median obtained was of 32 kg TS m^{-2}.year^{-1}. Considering the tropical regions and compared with literature, the SLR obtained was lower. This fact can be explained by the lack of control of the total solids concentration influent in the system and great variability of ST sludge analysed. In OS2, the SLR was also characterized by great variability, as can be seen in Figure 5.19. The SLR ranged from 8.0 to 523 kg TS $m^{-2}.year^{-1}$ and the mean of SLR obtained was of 77 kg TS.$m^{-2}.year^{-1}$ and a median of 28 kg TS.$m^{-2}.year^{-1}$. On the other hand, in OS3 the SLR was characterized by less variability, ranged from 9.0 kg TS.$m^{-2}.year^{-1}$ to 95 kg TS.$m^{-2}.year^{-1}$.The mean of SLR obtained was of 40 kg TS.$m^{-2}.year^{-1}$ and a median of 25 kg TS.$m^{-2}.year^{-1}$. Table 5.12 shows the Mann-Whitney comparing the median of SLR applied between three operational strategies.

Table 5.12: *p-values* of the Mann-Whitney U-test comparing the median of SLR applied between three operational strategies.

	Operational strategies SLR (kg TS.$m^{-2}.year^{-1}$)		
Comparison	OS1-OS2	OS1-OS3	OS2-OS3
p-values	0.8926	0.4782	0.6921

(*) *p*-values ≤ 0.05: sample medians are significantly different

In general, the analysis did not show significant difference ($p > 0.05$), between the median of SLR applied in all operational strategies. This means that, all operational strategies were implemented under the same conditions in terms of SLR applied. The results discussed in the next section are concerning characterization of raw ST sludge, percolate from sludge unit and percolate after post-treatment (final effluent).

5.4.3 Analysis of the performance in OS1

5.4.3.1 Characterization of raw septic tank sludge during OS1

The physical-chemical characteristics of the sludge from septic tank are reported in Table 5.13. As mentioned previously, the sludge from septic tank was collected from individual households in different localities in Belo Horizonte using trucks for transportation. Once the system reproduced real conditions, all the sludge that arrived at the plant was discharged in the bed.

Table 5.13: Physical-chemical characteristics of the raw sludge (influent) applied in the sludge unit.

Parameters	n	Mean	Median	Minimum	Maximum	Coefficient of variation
pH	34	7.1	7.3	5.4	10.2	0.14
Temperature ºC	36	27	27	21	33	0.09
BOD (mg L^{-1})	32	2814	1245	143	17720	1.55
COD (mg L^{-1})	45	5774	2970	174	31400	1.24
TS (mg L^{-1})	46	5939	2597	583	61391	1.67
TVS (mg L^{-1})	45	4086	1579	160	58495	2.24
NH_4^+-N (mg L^{-1})	38	87	84	8	201	0.54
TKN (mg L^{-1})	35	119	93	21	564	0.83

n: number of samples; **TS:** total solids; **TVS:** total volatile solids; **NH_4^+-N:** Ammonium; **TKN:** total Kjeldahl nitrogen; **BOD:** biochemical oxygen demand; **COD:** chemical oxygen demand

The quality of raw sludge analysed in Belo Horizonte was characterized by great variability. From the results presented in Table 5.13 it is observed a high coefficient of variation mainly for TS, TVS, NH_4^+-N, TKN, BOD and COD concentrations except for pH and temperature. Similar results were obtained in Yaoundé, Cameroon and in Dakar, Senegal by Kengne *et al.* (2011) and Sonko (2014) respectively. In addition, it is important to emphasize that the great variability of raw sludge was also found by Koottatep *et al*. (2008) in tropical regions. Sludge from septic tank in tropical regions normally is characterized by great variability due to the different sources. The median of influent and effluent concentrations were used as a measure of central tendency due to the large variation of the results obtained. Figure 5.20 shows different types of raw sludge analysed in this research.

Figure 5.20: Different types of the raw sludge fed in the sludge unit during OS1.

The low ratio of COD/BOD = 2.05 obtained, indicated that biodegradable fraction is high, which demonstrated good indication for biological treatment (von Sperling, 2014). In other words, this means that the sludge analysed was not sufficiently digested in the septic tank. Similar results were achieved in Bangkok by Koottatep *et al.* (2004). In OS1, it was observed great variability of TS concentration influent (583 – 61391 mg.L^{-1}). This variation is associated with the different origins of the sludge. On the other hand, the range of TS concentration found here are similar of TS concentration reported in literature. The mean concentrations of raw sludge obtained were of 5774 mg.L^{-1} for COD, 2814 mg.L^{-1} for BOD, 119 mg.L^{-1} for TKN, 87 mg.L^{-1} for NH_4^+-N, 4086 mg.L^{-1} for TVS and 5939 mg.L^{-1} for TS.

Authors have conducted many investigations for FS characterization in the world, mainly in developing countries. Nevertheless, different results have been revealed. In Burkina Faso, Ouagadougou, Bassan *et al*. (2013) conducted a research for FS characterization during dry and rainy seasons. The results observed indicated the mean concentrations of TS (11820 mg.L^{-1}), TSS (9014 mg.L^{-1}), COD (10725 mg.L^{-1}), BOD (1902 mg.L^{-1}), NH_4^+-N (1230 mg.L^{-1}). Research done by Prosab (2009) also showed greater variability of sludge from septic tank in Brazil. The mean concentrations measured were of TS (12880 mg.L^{-1}), TVS (3518 mg.L^{-1}), DBO (2434 mg L^{-1}), COD (6895 mg.L^{-1}), TKN (120 mg.L^{-1}), NH_4^+-N (89 mg.L^{-1}). The characteristics of raw FS found here were very similar to those reported by Meneses *et al* (2009).

Nevertheless, comparing the quality of raw sludge observed obtained here and the quality of sludge observed by the authors, could be detected great variability. This variability is in accordance to origin, TS concentration, biochemical stabilization and the type of treatment system.

The analysis in this study indicated that the FS is "low strength" (Kootattep *et al*. 2005). The FS analysed is characterized by low concentration and is more stabilized compared to public toilet sludge characterized by the authors. On the other hand, this kind of sludge is biochemically stable (low concentration of BOD) and exhibits low NH_4^+-N concentration. According to Strauss (2008), ammonia concentrations in FS range from ≥ 300 mg.L^{-1} in septage to ≤ 5000 mg L^{-1} in high-strength.

5.4.3.2 Characterization of the effluent (percolate) from the sludge unit

The results of the percolate concentrations from the sludge unit are reported in Table 5.14. The analysis show great variability in the percolate for almost all constituents, similar to the raw sludge characterized previously. However, it was observed that the concentrations of the percolate from the sludge unit were lower compared to the raw sludge discharged onto the sludge unit, indicating the role of VFCW in pollutants removal.

Table 5.14: Physical chemical characteristics of the percolate (Effluent 1).

Parameters	n	Mean	Median	Minimum	Maximum	Coefficient of variation
pH	34	6.7	6.8	5.4	7.6	0.09
Temperature ºC	36	27	27	22	32	0.09
Oxygen (mg L^{-1})	36	4.19	4.60	0.18	6.49	0.38
BOD (mg L^{-1})	32	524	335	56	1738	0.84
COD (mg L^{-1})	45	1001	601	72	8460	1.30
TS (mg L^{-1})	45	1505	1206	549	4823	0.60
TVS (mg L^{-1})	45	740	590	186	3337	0.73
NH_4^+-N (mg L^{-1})	38	37	33	5	110	0.71
TKN (mg L^{-1})	35	40	34	4	122	0.69

n: number of samples; **TS:** total solids; **TVS:** total volatile solids; **NH_4^+-N**: Ammonium; **TKN:** total Kjeldahl nitrogen; **BOD:** biochemical oxygen demand; **COD:** chemical oxygen demand

The percolate showed pH values ranging from 5.0 to 8.0, the average temperature was of 27 ºC and the average oxygen concentration was of 4.19 mg.L^{-1}. These findings were similar to those obtained by Jong and Tang (2014). The analysis showed higher variation for all constituents except for pH, temperature and oxygen. The high oxygen concentration of the percolate showed the capacity of the VFCW in oxygen transfer into the filter.

Obviously, the presence of plants in the system play an important role in terms of pollutants removal. The quality of percolate from sludge unit was much better than raw sludge applied in the sludge unit, thus showing the importance of the VFCW concerning the ST sludge treatment. The mean of the percolate from sludge unit concentrations obtained were of 524 $mg.L^{-1}$ for BOD, 1001 $mg.L^{-1}$ for COD, 1505 $mg.L^{-1}$ for TS, 740 $mg.L^{-1}$ for TVS , 37 $mg.L^{-1}$ for NH_4^+-N, and 40 $mg.L^{-1}$ for TKN.

5.4.3.3 Characterization of the effluent after post-treatment of the percolate

The percolate from the sludge unit was post-treated in the post-treatment unit using a pump for sending it from sludge unit to percolate unit. The physical-chemical characteristics of the final effluent obtained after post-treatment are shown in the Table 5.15.

Table 5.15: Physical-chemical characteristics of the effluent after post-treatment (Effluent 2)

Parameters	n	Mean	Median	Minimum	Maximum	Coefficient of variation
pH	30	6.5	6.7	5.4	7.5	0.08
Temperature ºC	33	26	26	21	31	0.09
Oxygen ($mg.L^{-1}$)	32	4.72	5.06	0.18	7.08	0.36
BOD ($mg.L^{-1}$)	25	416	337	51	1107	0.72
COD ($mg.L^{-1}$)	28	716	581	111	2162	0.74
TS ($mg.L^{-1}$)	34	1493	1327	596	2952	0.43
TVS ($mg.L^{-1}$)	34	795	698	210	1803	0.50
$NH4^+$-N ($mg.L^{-1}$)	25	29	22	6	87	0.69
TKN ($mg.L^{-1}$)	26	33	28	7	82	0.65

n: number of samples; **TS:** total solids; **TVS:** total volatile solids; **$NH4^+$-N:** Ammonium; **TKN:** total Kjeldahl nitrogen; **BOD:** biochemical oxygen demand; **COD:** chemical oxygen demand

The analysis show a slight contribution of the post-treatment in the planted units in terms of pollutants removal. The medians of the percolate concentrations and the medians of the effluent after post-treatment were very similar for all parameters evaluated, indicating a slight contribution of the post-treatment unit. This result was expected since the post treatment unit was fed during the rest period of sewage application (without liquid onto the post-treatment unit) and the low efficiency could be attributed to the high instantaneous load applied in the unit (concentrated in a single application). The mean of the final effluent concentrations obtained were of 416 $mg.L^{-1}$ for BOD, 716 $mg.L^{-1}$ for COD, 1493 $mg.L^{-1}$ for TS , 795 for TVS, 29 $mg.L^{-1}$ for NH_4^+-N and 33 $mg.L^{-1}$ for TKN, indicating diluted sludge compared to raw sludge applied in the sludge unit. However, it was detected that all parameters analysed were in the range of the typical raw domestic wastewater characteristics according to von

Sperling (2014). Thus, analysis show that the final effluent obtained from the treatment process is similar to raw domestic sewage.

5.4.3.4 Analysis of the removal efficiencies in OS1

In general, experiment functioning without any retention of the percolate in the sludge unit and post-treatment of the percolate in another unit showed a good performance in terms of pollutants removal. The global medians removal efficiencies achieved were of 73% for BOD, 81% for COD, 42% for TS, 55% for TVS, 66% for $NH4^{+}$-N, 67% for TKN and 25% of *E. coli*. However, these removal efficiencies were lower than reported in literature. The high volume applied in the system and short retention period applied can be the cause of low efficiencies since many authors like Koottatep *et al.,* (2008) have implemented a retention period obtaining excellent removal efficiencies. So it was decided to change the OS1 giving a retention period in OS2 and OS3. More details about the performance in OS2 are discussed in the next sections.

5.4.4 Analysis of the performance in OS2

5.4.4.1 Characterization of raw septic tank sludge, percolate from sludge unit and post-treatment effluent during OS2

Experiment with a retention period of seven days (OS2) was also carried out. The sludge from septic tank was applied in the sludge unit in a loading cycle of one day per week, with seven days for percolation and drying. In this section, results concerning experiment of retention period of seven days are presented, when the percolate was retained 10 cm below the filter for percolation and drying. The physical-chemical characteristics of raw sludge, percolate and the percolate after post-treatment were presented in Table 5.16.

Table 5.16: Physical-chemical characteristic of the raw sludge, percolate from sludge unit and post-treated effluent.

Type of Sludge	Parameters	n	Mean	Median	Minimum	Maximum	Coefficient of variation
Raw sludge (influent)	pH	24	7	8	6	8	0.09
	Temperature ºC	23	25	24	21	32	0.12
	Oxygen ($mg.L^{-1}$)	24	1.27	0.89	0.28	4.42	0.93
	BOD ($mg.L^{-1}$)	23	2223	1220	272	16000	1.48
	COD ($mg.L^{-1}$)	26	8130	3734	561	58574	0.92
	TS ($mg.L^{-1}$)	20	5361	2493	759	34022	1.45
	TVS ($mg.L^{-1}$)	22	3425	1209	283	30363	1,90
	$NH4^{+}$-N ($mg.L^{-1}$)	22	90	79	14	204	0.48
	TKN ($mg.L^{-1}$	15	107	96	24	363	0.72
Percolate from sludge unit (Effluent 1)	pH	27	7	7	7	8	0.04
	Temperature ºC	26	25	25	22	32	0.09
	Oxygen ($mg.L^{-1}$)	27	3.40	3.80	0.66	5.88	0.41
	BOD ($mg.L^{-1}$)	23	76	33	9	470	1.41
	COD ($mg.L^{-1}$)	24	488	352	115	2122	0.91
	TS ($mg.L^{-1}$)	20	863	831	385	1639	0.36
	TVS ($mg.L^{-1}$)	22	238	198	81	600	0.55
	$NH4^{+}$-N ($mg.L^{-1}$	22	32	35	2	74	0.63
	TKN ($mg.L^{-1}$)	15	35	30	7	85	0.41
Final effluent (percolate after post-treatment-Effluent 2)	pH	27	7	7	6	8	0.04
	Temperature ºC	26	25	24	22	30	0.09
	Oxygen ($mg.L^{-1}$)	27	5.63	6.01	2.82	7.10	0.22
	BOD ($mg.L^{-1}$)	23	55	40	9	203	0.95
	COD ($mg.L^{-1}$)	24	440	360	161	1511	0.38
	TS ($mg.L^{-1}$)	19	930	920	456	1705	0.32
	TVS ($mg.L^{-1}$)	20	338	307	150	559	0.44
	$NH4^{+}$-N ($mg.L^{-1}$)	20	22	20	2	58	0.69
	TKN ($mg.L^{-1}$)	15	23	19	5	53	0.18

n: number of samples; **TSS:** total suspended solids; **TVS:** total volatile solids; **$NH4^{+}$ -N:** Ammonium; **TKN:** total Kjeldahl nitrogen; **BOD:** biochemical oxygen demand; **COD:** chemical oxygen demand
Percolate was retained 10 cm below layer in CW; Retention period = 7 days
The percolate was sent to post-treatment unit

In OS2 the COD/BOD ratio obtained of the influent was of 3.65, which indicates slow biodegradable organic matter compared with OS1, however the result obtained demonstrated good indication for biological treatment. As observed in Table 5.16 in general the raw sludge discharged onto the sludge unit was characterized by high variability, except for pH and temperature. As discussed in the previous section, this variability of the influent is a result of different sources and characteristics of the sludge. The results of OS2 showed lower concentrations of the percolate and post-treated effluent as expected.

5.4.4.2 NO_2 and NO_3

The nitrite and nitrate concentrations were also evaluated when the system was operated with a retention period of seven days. As expected the NO_2 concentrations obtained were low for raw sludge, percolate and post-treated effluent. The average of NO_2 concentrations obtained were of 0.38 $mg.L^{-1}$ for raw sludge, 0.42 $mg.L^{-1}$ for percolate and 0.62 $mg.L^{-1}$ for post-treated effluent respectively. von Sperling (2014) reinforced that normally in the raw domestic sewage the NO_2 concentration is low.

Thus, higher NO_2 concentration of the percolate was observed compared to raw ST sludge indicating some level of nitrification. Regarding NO_3 concentration, some level of nitrification in the system was also observed. The NO_3 concentrations obtained were of 0.77 $mg.L^{-1}$ for raw sludge, 5.0 $mg.L^{-1}$ for percolate and 26 $mg.L^{-1}$ for post-treated effluent respectively. The results obtained indicate that there were aerobic conditions prevailing in the filter which supported nitrification (Koottatep *et al.*, 2008). The oxygen transfer in the system occurs by pulse loading, ventilation system, plant, pump that sends the percolate from sludge unit to post-treatment unit and the environment.

5.4.4.3 Escherichia coli

Treating septic tank sludge, the vertical flow constructed wetland working with retention period showed satisfactory performance in terms of *Escherichia coli* removal. The exploratory analysis of *Escherichia coli* concentration and removal efficiencies were presented in Table 5.17.

Table 5.17: Exploratory analysis of *Escherichia coli* concentrations and removal efficiencies during operational strategy 2.

Parameter	Descriptive elements	Concentration (MPN/100mL)			Removal efficiency (%)		
		Influent	Sludge unit	Post-treatment unit	Sludge unit	Post-treatment unit	Global
E. coli	n	5	5	5	5	5	5
	Mean	2.44E+06	6.71E+03	3.80E+04	96.7	-478.7	69.7
	Median	1.46E+06	3.85E+03	6.30E+03	99.7	-690.6	99.6
	Minimum	1.00E+05	1.00E+02	9.60E+02	84.8	-875.7	-48.3
	Maximum	6.13E+06	1.52E+04	1.48E+05	100.0	40.6	100.0
	Coefficient of variation	0.97	0.90	1.65	0.07	-0.96	0.95

The analysis showed that the sludge unit exhibited high *E. coli* removal efficiencies. For the raw ST sludge an median *E. coli* concentration of 1.46E+06 MPN/100 ml was achieved. The percolate from sludge unit achieved an median of 3.85E+03 MPN/100 ml correspondent a 99.7% removal efficiency (2.3 $\log_{10}$ units). Thus, the OS2 was characterized by high *E. coli* removal efficiency. The post-treatment unit gave a negative *E. coli* removal efficiency. The presence of the solids in the final effluent after post-treatment may be related with the higher concentrations of *E .coli*. The high concentration of the solids in the final effluent is attributed to the rest period given to the post-treatment unit before receiving the percolate from sludge unit, and to the instantaneous form of application of the percolate (high instantaneous load).

Previous experiments without retention period of the liquid in wetland showed lower *E. coli* efficiencies removals: Calderon-Vallejo (2015) monitored the *E. coli* concentration along the OS1, the sludge unit gave a negative median removal efficiency, even the concentrations at the effluent were much higher. On the other hand, Sonko *et al.* (2015) obtained an average *E. coli* reduced, only 1.25 $\log_{10}$ units in terms of removal efficiency. In OS1, the post-treatment unit gave a negative median removal efficiency, where the median concentration increased in more than 0.2 $\log_{10}$ values. This occurred due to the solids dragging issue, already explained in previous sections. This indicate that the solids present in the effluents are related with higher concentration of *E. coli*. However, the retention period applied in the system during OS2 improved the *E. coli* removal efficiency due to the presence of physical processes as filtration, sedimentation and adsorption.

5.4.4.4 Analysis of the removal efficiencies in OS2

During OS2 with retention of the percolate in the sludge unit during seven days, followed by sending the percolate for post-treatment unit, the analysis showed excellent performance in terms of pollutants removals. The global medians removal efficiencies were of 97% for BOD, 90% for COD, 73% for TS, 73% for TVS, 75% for $NH4^+$-N and 78% for TKN. Thus, the analysis showed an increase of global removal efficiencies as a result of retention period of seven days applied in the sludge unit.

5.4.5 Analysis of the performance in OS3

5.4.5.1 Characterization of raw septic tank sludge, percolate from sludge unit and post-treatment effluent during OS3

As mentioned in the methodology chapter, the data of operational strategy 3 was obtained by Ávila Lopez (2016) as part of his master's degree. Contrary to operational strategy 2, in operational strategy 3 the sludge from septic tank was applied in the sludge unit in a loading cycle of one day per week, with seven days for percolation and drying. On day seven, the outlet of sludge unit was opened and the percolate was transferred from sludge unit to percolate unit for a further seven days for percolation and drying totalling fourteen days. The physical-chemical characteristics of raw sludge, percolate and post-treated effluent were presented in Table 5.18.

Table 5.18: Physical-chemical characteristic of raw sludge, percolate from sludge unit (after retention of 7 days) and effluent from post-treatment unit (after retention of 7 days).

	Parameters	n	Mean	Median	Minimum	Maximum	Coefficient of variation
Raw sludge (influent)	pH	10	7	7	5	8	0.2
	BOD ($mg.L^{-1}$)	9	906	483	333	2947	1.03
	COD ($mg.L^{-1}$)	10	5200	4211	1001	15183	0.82
	TS ($mg.L^{-1}$)	9	3050	1719	953	6203	0.74
	TVS ($mg.L^{-1}$)	10	1740	942	283	4928	1.00
	$NH4^{+}$-N ($mg.L^{-1}$)	9	97	94	22	186	0.51
	TKN ($mg.L^{-1}$)	10	168	128	61	380	0.59
	E. coli	10	4.86E+07	1.67E+07	1.75E+06	2.91E+08	1.80
Percolate from sludge unit (Effluent 1)	pH	10	7	7	6	7	0.00
	BOD ($mg.L^{-1}$)	9	250	55	26	1036	1.58
	COD ($mg.L^{-1}$)	10	919	751	299	2119	0.71
	TS ($mg.L^{-1}$)	9	961	971	410	1511	0.35
	TVS ($mg.L^{-1}$)	10	389	422	58	839	0.58
	$NH4^{+}$-N ($mg.L^{-1}$	9	40	33	10	91	0.68
	TKN ($mg.L^{-1}$)	10	42	38	14	92	0.61
	E. coli	10	1.52E+04	1.33E+04	1.00E+03	3.14E+04	0.70
Final effluent (from post-treatment unit- Effluent 2)	pH	9	7	7	7	7.5	0.00
	BOD ($mg.L^{-1}$)	8	116	48	39	396	1.15
	COD ($mg.L^{-1}$)	9	304	278	65	541	0.61
	TS ($mg.L^{-1}$)	8	606	665	326	735	0.23
	TVS ($mg.L^{-1}$)	9	186	170	128	260	0.26
	$NH4^{+}$-N ($mg.L^{-1}$)	8	25	20	7	57	0.72
	TKN ($mg.L^{-1}$)	9	24	20	10	51	0.63
	E. coli	9	4.03E+03	1.10E+03	1.00E+03	1.35E+04	1.14

n: number of samples; **TSS:** total suspended solids; **TVS:** total volatile solids; **$NH4^{+}$ -N:** Ammonium; **TKN:** total Kjeldahl nitrogen; **BOD:** biochemical oxygen demand; **COD:** chemical oxygen demand
Percolate was retained 10 cm below layer in CW; Retention period = 14 days

In general, by observing Table 5.18 through coefficient of variation, expressive variability of the influent was seen except for pH and similar result was found in operational strategy 2. As observed the raw sludge discharged onto the sludge unit was characterized by high variability except for pH. It was discussed in the previous section that this variability of the influent is the result of different sources and characteristics of the sludge. The results of operational strategy 3 showed lower concentrations of the effluent percolate and post-treated effluent compared with the influent discharged onto the sludge unit. In general, effluent with excellent quality was observed such as during operational strategy 2.

In OS3 the COD/BOD ratio of the influent obtained was of 5.7 which means major presence of inert materials. This high ratio of COD/BOD obtained can be associated with few data analysed in this operational mode compared with SO2. The results based on COD/BOD ration obtained indicated that the influent is not feasible to apply natural treatment. Based on samples analysed the inert fraction was very high; so physical-chemical treatment is suitable than biological treatment. On the other hand, it was observed that a high fraction of organic matter was removed in the sludge unit.

5.4.5.2 Escherichia coli

E. coli was also monitored during operational strategy 3. The results of *E. coli* analysis obtained during operational strategy 3 are presented in Table 5.19.

Table 5.19: Exploratory analysis of *Escherichia coli* concentrations and removal efficiencies during operational strategy 3.

Parameter	Descriptive elements	Concentration (NMP/100mL)			Removal efficiency (%)		
		Influent	Sludge unit	Post-treatment unit	Sludge unit	Post-treatment unit	Global
E. coli	N	10	10	9	10	9	9
	Mean	4.86E+07	1.52E+04	4.03E+03	99.8	71.9	99.93
	Median	1.67E+07	1.33E+04	1.10E+03	99.94	86.5	99.98
	Minimum	1.75E+06	1.00E+03	1.00E+03	99.2	-3.4	99.6
	Maximum	2.91E+08	3.14E+04	1.35E+04	99.99	96.6	99.9997
	Coefficient of variation	1.80	0.70	1.14	0.002	0.45	0.001

The *E. coli* removal efficiency found in the sludge unit during OS3 was similar as in OS2. A median removal efficiency of 99.94% (equivalent to 3 log removal units) was achieved. However, the post-treatment unit displayed also a positive removal efficiency of 86.5%. This was something predicted due to the prolonged retention period, which in the first unit gave very good results. So, the global median removal efficiency reached a little more than 4 log removal units. It was evident that the long retention period enhanced the *E. coli*. removal efficiency.

5.4.5.3 Analysis of the removal efficiencies in OS3

For OS3 where the percolate was retained during seven days in the sludge unit and further retention of the percolate another seven days in the post-treatment unit the results show excellent performance of the system in terms of pollutant removals. The prolonged retention period enhanced the system making it more robust especially for Total Suspended Solids, Nitrogen and *E. coli* removal efficiencies. The global medians removal efficiencies achieved during the OS3 were of 90% for BOD, 94% for COD, 68% for TS, 80% for TVS, 68% for $NH4^{+}$-N, 87% for TKN and 99.98% for *E. coli*.

As commented in the previous section, the experiments followed three different operational strategies. Thus, as a form to compare the performance of the system operating in these conditions, results related to the three operational strategies were presented and discussed. Statistical analysis using Manny-Whitney U-test were also performed to compare medians in terms of removal efficiencies between three operational strategies.

5.4.6 Organic matter removal in OS1, OS2 and OS3 (BOD and COD)

Figure 5.21 shows the BOD concentrations and removal efficiencies in three operational strategies. Figure 5.22 shows the COD concentrations and removal efficiencies in three operational strategies.

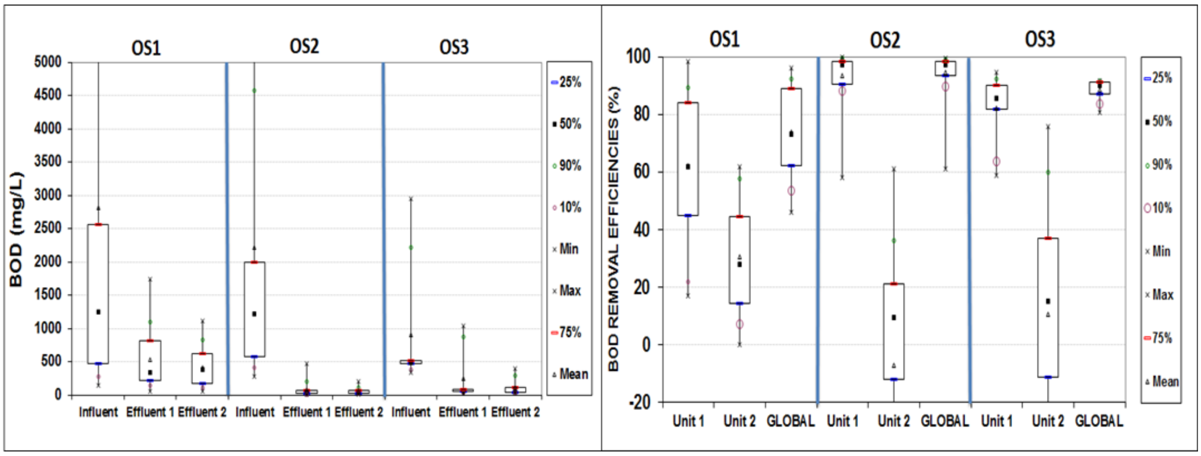


Figure 5.21: Box Whisker graphics of BOD concentrations and removal efficiencies for each experiment.

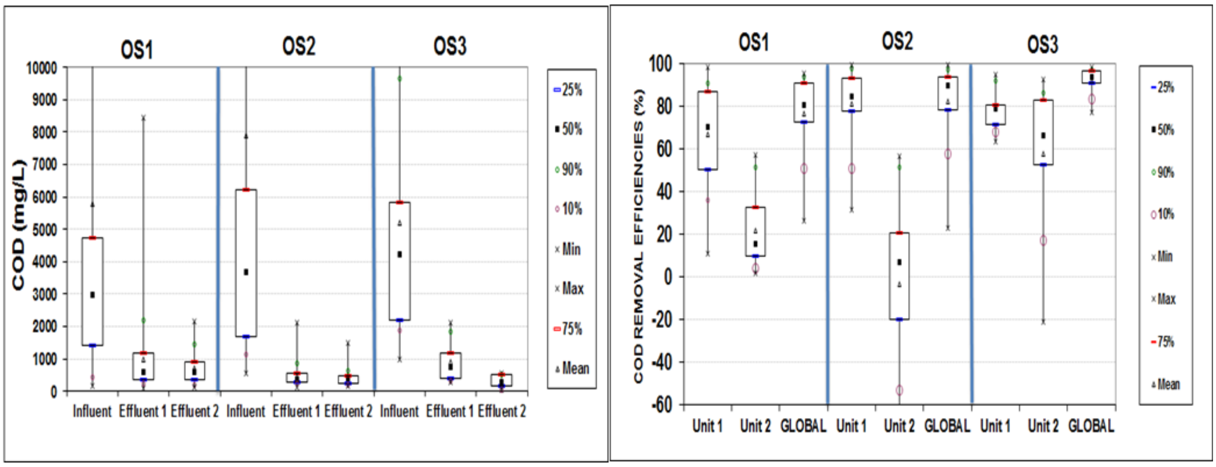


Figure 5.22: Box Whisker graphics of COD concentration and removal efficiencies for each experiment.

Analysing Figure 5.21 and Figure 5.22, it can be observed that the BOD and COD influent concentrations were characterized by great variability for all operational strategies due to the different characteristics and origin of the raw ST sludge. In OS1 the minimum BOD concentration of the raw ST sludge found was of 143 mg.L^{-1} and the maximum BOD concentration determined was of 17720 mg.L^{-1} (143 – 17720 mg.L^{-1}) while for COD concentration the minimum value found was of 174 mg.L^{-1} and maximum of 31400 mg.L^{-1} (174 – 31400 mg.L^{-1}). In OS2 the minimum BOD concentration of the ST sludge found was of 272 mg.L^{-1} and the maximum BOD concentration determined was of 16000 mg.L^{-1} (272-16000 mg.L^{-1}) while the COD concentration ranged from 561 to 58574 mg.L^{-1}. For OS3 the minimum BOD concentration of the ST sludge achieved was of 333 and the maximum BOD concentration achieved was of 2947 mg.L^{-1} (333 – 2947 mg.L^{-1}) whereas the COD concentration ranged from 1001 to 15183 mg L^{-1}). In Bangkok, Thailand Koottatep *et al.* (2008) reported a range of 700 – 25000 mg.L^{-1} for BOD concentration from septic tank sludge. On the other hand, in Ouagadougou, Burkina Faso, for COD concentration, Bassan *et al.* (2013) pointed out an average of 7605 mg.L^{-1} and an average of 12437 mg.L^{-1} of Faecal Sludge (FS) from septic tank and pit latrines respectively. Koné and Strauss (2004) in Bangkok reported a range of 600-5500 mg.L^{-1} for BOD concentration from Faecal Sludge and a range of 1200-76000 mg L^{-1} for COD concentration from Faecal Sludge.

The range of BOD and COD concentrations of ST sludge determined by various authors were similar of those found here. Given different regional conditions in terms of FS management, system of sanitation, authors have found great variability concerning FS characteristics. Koné and Strauss (2004) reinforced that the FS characteristics vary widely within and between cities, based on the types of on-site sanitation in use. Due to the variability of the septic sludge characteristics, the median determined is also presented and analysed.

In OS1, the medians of BOD concentrations obtained were of 1245 mg.L^{-1}; 335 mg.L^{-1} and 377 mg.L^{-1} for raw ST sludge, percolate from sludge unit and effluent from post-treatment unit respectively. The medians of COD concentrations achieved were of 2970 mg.L^{-1}, 601 mg.L^{-1}, 581 mg.L^{-1} for raw ST sludge, percolate from sludge unit and effluent from post-treatment unit respectively. The medians of BOD and COD concentrations of the percolate from sludge unit and effluent from post-treatment unit are in the range of characteristics of the raw sewage in developing countries (von Sperling, 2014). Experiment during OS1 resulted in BOD removal efficiencies of about 62%, 28% and 73% for sludge unit, post-treatment unit and global efficiency respectively whereas the medians of COD removal efficiencies were of 71%, 16% and 81% for sludge unit, post-treatment unit and global efficiency respectively.

From the results obtained during OS1 in terms of BOD and COD concentrations and removal efficiencies can be inferred that the system allows biodegradation of organic matter. Furthermore, most of the organic matter is removed in the sludge unit, since in this unit outstanding results concerning BOD and COD removals were seen. Good results in terms of BOD and COD removal efficiencies can be attributed to the aerobic process due to oxygen introduction over each batch, oxygen transfer by diffusion and convection in the system and filtration process.

In OS2 with a retention period of seven days, the medians of BOD concentration were of 1220 mg.L^{-1} for raw ST sludge, 33 mg.L^{-1} percolate from sludge unit and 40 mg.L^{-1} for effluent from post-treatment unit while the medians of COD concentrations were of 3687 mg L^{-1}, 359 mg L^{-1} and 383 mg L^{-1} for raw ST sludge, percolate from sludge unit and effluent from post-treatment unit respectively. However, the analysis show that the BOD and COD concentrations for percolate from sludge unit and effluent from post-treatment unit obtained during OS2 were lower than those found during OS1, indicating better quality of the effluent when applying a retention of the percolate in the sludge unit during seven days. The BOD median removal efficiencies achieved were of 97% for sludge unit, 9% for post-treatment unit and 97% for global removal efficiency. On the other hand, regarding COD removal efficiencies, an median of 85% for sludge unit, 7% for post-treatment unit and 90% for global removal efficiency were achieved. Evidently, with a retention period of the percolate in the sludge unit during seven days, positive results of BOD and COD removal was observed. This means that the retention of the percolate enhanced organic matter biodegradation in the system. Retention of the percolate proved to be necessary to increase organic matter removal. Such as in OS1, in SO2 almost all organic matter removal took place in the sludge unit. So, the post-treatment unit was not effective for organic matter removal since it was not given prolonged retention of the percolate in both cases. In OS2, it was seen that, the contribution of post-treatment unit in terms of BOD removal efficiency decreased, because almost all biodegradation process took place in the sludge unit due to the retention period implemented. However, the retention of the percolate for seven days in the sludge unit was found as the best alternative comparing to the first operational strategy. With impounding of the percolate, Koottatep *et al.* (2004) found organic matter removal efficiency ranging from 88% to 98% applying the same retention period. These findings were very similar to those found here. The better performance found by Koottatep (2004) could be attributed to the higher and controlled SLR (250 kgTS.m^{-2}·$year^{-1}$). During OS2, considering the BOD median concentration in the percolate from sludge unit and effluent from post-treatment unit the analysis showed that some samples met discharge standard of 60 mg L^{-1} stated in Minas Gerais, Brazil.

Visibly, the operation with a retention period of seven days performed better than that of the first experiment without a retention period. Nevertheless, the system showed very good performance in terms of organic matter removal in OS2 than OS1 as result of retention period.

In OS3, with a retention of the percolate during seven days in the sludge unit and retention of the effluent coming from the sludge unit in the post-treatment unit during seven days, the system improved its condition in terms of organic matter removal. The BOD median concentration achieved were of 483 $mg.L^{-1}$ for raw ST sludge, 55 $mg.L^{-1}$ for percolate from sludge unit and 48 $mg.L^{-1}$ for effluent from post-treatment unit. The COD median concentration achieved were of 4211 $mg.L^{-1}$ for raw ST sludge, 751 $mg.L^{-1}$ for percolate from sludge unit and 278 $mg.L^{-1}$ for effluent from post-treatment unit. Differently from OS2, in OS3 the retention period was extended from seven days to fourteen days. Such as in OS2, in OS3 good quality of the effluent was obtained. The median of BOD removal efficiencies were of 85% for sludge unit, 15% for post-treatment unit and 90% for global removal efficiency whereas the medians of COD removal efficiencies achieved were of 80% for sludge unit, 66% for post-treatment unit and 94% for global removal efficiency.

In OS3, the BOD_5 removal efficiency in sludge was again very good and stable and it was observed that the contribution of post-treatment unit improved when compared in OS2. The results showed small decrease of COD removal efficiency in OS3. This decrease can be attributed to the influent characteristics. However, the prolonged retention period applied in OS3 gave positive results in the post-treatment unit concerning COD and BOD removals. The retention period in the post treatment unit helped to sediment the dragged particles and enhanced the COD removal efficiency up to 94% (Ávila Lopez, 2016). Similarly, in previous operational strategies, in OS3 it was proven that most biodegradable fraction was removed during the first retention period. Table 5.20 shows Mann Whitney U-test comparing the median of BOD and COD removal efficiencies achieved in the sludge unit, post-treatment unit and global removal efficiency between three operational strategies.

Table 5.20: *p-values* of the Mann Whitney U-test comparing median of BOD and COD removal efficiencies in the sludge unit, post-treatment unit and global removal efficiency between three operational strategies.

	BOD		
Comparison	**OS1-OS2**	**OS1-OS3**	**OS2-OS3**
Sludge unit	0.0000 (*)	0.0179 (*)	0.0018 (*)
Post-treatment unit	0.0021 (*)	0.3069	0.5725
Global	0.0000 (*)	0.0559	0.0009 (*)
	COD		
Comparison	**OS1-OS2**	**OS1-OS3**	**OS2-OS3**
Sludge unit	0.0053 (*)	0.1655	0.1492
Post-treatment unit	0.0390 (*)	0.0023 (*)	0.0009 (*)
Global	0.0987	0.0026 (*)	0.1011

(*) *p*-values $\leq$ 0.05: sample medians are significantly different

From statistical analysis, the BOD removal efficiency in the sludge unit was higher in OS2 and OS3 compared to OS1, probably because of the high biodegradation activity observed in OS2 and OS3 as a result of impounding or retention of the percolate. In OS2, the analysis showed higher BOD removal efficiency in the sludge unit than OS3 probably because the characteristics and origin of ST sludge, another cause can be the fewer number of samples analysed during OS3. The BOD removal efficiency in the post-treatment unit was higher in OS1 compared to OS2. It appears that most of biodegradation fraction decomposed during the retention of the percolate in the sludge unit. On the other hand, the analysis did not show significant differences ($p > 0.05$) related to the BOD median removal efficiencies in the post-treatment unit between OS1-OS3 and between OS2-OS3. This fact, can be attributed to the characteristics and origin of the ST sludge; limited number of the samples investigated during OS3. Even with equivalent medians of BOD removal efficiencies between OS1 and OS3, lower BOD removal efficiency was observed in the post-treatment unit during OS3 since most of biodegradation activity took place in the sludge unit as a result of retention of the percolate. Therefore, the BOD removal efficiency in the post treatment unit increased during OS3 compared to OS2 due to retention period given to this unit. Statistical analysis confirmed the higher global BOD removal efficiency achieved in OS2 than in OS1 because of the retention of the percolate.

Regarding COD removal efficiency, the statistical analysis confirmed the higher removal efficiency in the sludge unit during OS2 than OS1. With this result, it is assumed that the sludge unit improved the sedimentation and filtration of particular organic matter having the longest period of seven days. The results did not show any significant differences concerning COD removal efficiencies in the sludge unit between OS2-OS3. It means that the sludge unit operated under the same conditions of retention of the percolate during seven days. On the other hand, the analysis confirmed higher COD removal efficiency in the post-treatment unit during OS1 compared to OS2. It can be inferred that most of sedimentation and filtration process took place in the sludge unit having the longest period of seven days and the accumulated dewatered which contribute to better filterability and increased microbial reactions (Koottatep *et al.,* 2004). Given the retention of the percolate in the post-treatment during OS3, the COD removal efficiency increased significantly and enhanced the global COD removal efficiency up to 94%. The percolate retention during seven days did not have any significant effects on the COD removal efficiency, but the retention of the percolate in the post-treatment unit during seven days brought significant COD removal.

From statistical analysis, it was concluded that the longest period of retention during seven days in the sludge unit and seven days in the post-treatment unit enhanced organic matter removal by improving the biodegradation activities, sedimentation and filtration process. The retention period of fourteen days (seven days in the sludge unit and seven days in the post-treatment unit) was found as the best alternative for organic matter, total solids, nitrogen and *E. coli* removal. However, the selection of the best and appropriate operational mode for ST sludge treatment depends on the quality of the effluent required.

5.4.7 Total solids and Total volatile solids removal in OS1, OS2 and OS3

Figure 5.23 illustrates the TS concentration and removal efficiencies in all operational strategies. Figure 5.24 illustrates the TVS concentrations and removal efficiencies in all operational strategies.

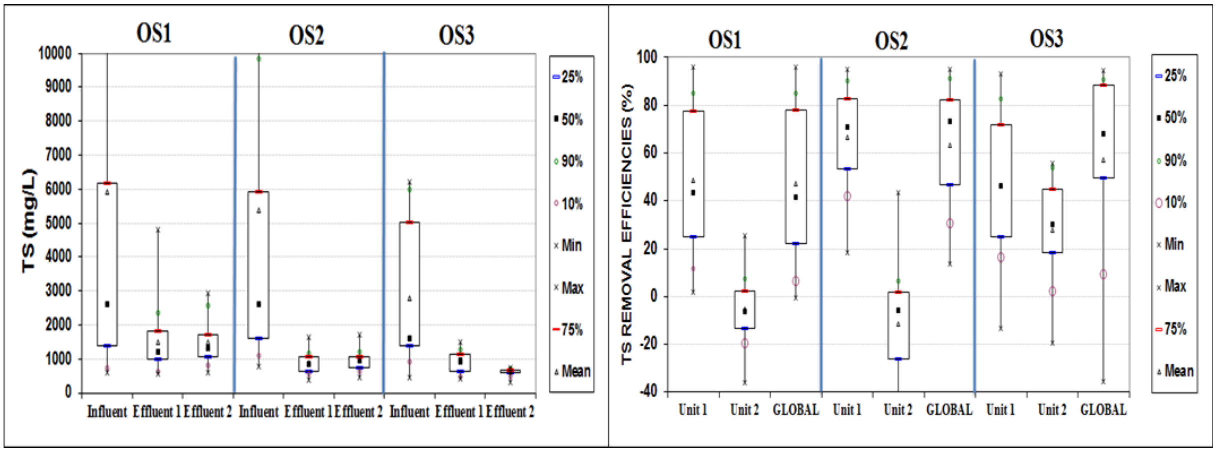


Figure 5.23: Box Whisker graphics of TS concentration and removal efficiencies for each experiment.

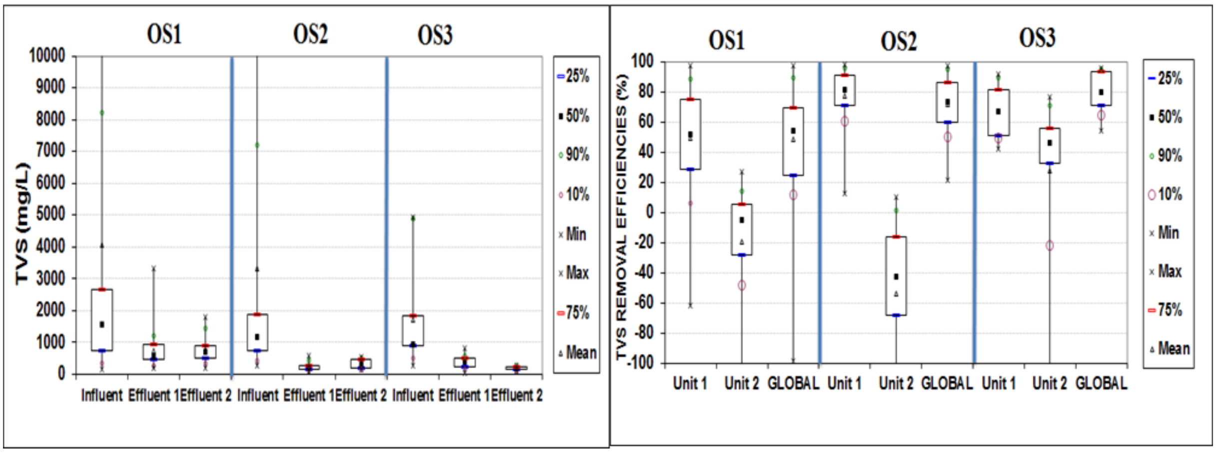


Figure 5.24: Box Whisker graphics of TVS concentration and removal efficiencies for each experiment.

The analysis showed great variability of the TS influent and TVS influent in all operational strategies investigated. It was commented that this variability is due to different origins and characteristics of the ST sludge and the OS3 was implemented with limited number of samples. In OS1, the TS concentration of raw ST sludge ranged from 583 $mg.L^{-1}$ to 61391 $mg.L^{-1}$. In OS1 the median of TS concentration determined was of 2596 $mg.L^{-1}$ for influent, 1206 $mg.L^{-1}$ for percolate from sludge unit and 1327 $mg.L^{-1}$ for effluent from post-treatment unit. In Thailand, Koné and Strauss (2014) reported a TS concentration of ST sludge ranging from 2200 $mg.L^{-1}$to 67200 $mg.L^{-1}$. Thus, the TS concentration of the influent found here during OS1 are similar with that reported in literature. Concerning TVS concentrations, an median of 1579 $mg.L^{-1}$, 590 $mg.L^{-1}$ and 698 $mg.L^{-1}$ for raw sludge were obtained, percolate from sludge unit and effluent from post-treatment unit respectively. For OS2 the median of TS concentrations determined were of 2616 $mg.L^{-1}$, 836 $mg.L^{-1}$ and 950 $mg.L^{-1}$ for raw sludge, percolate from sludge unit and effluent from post-treatment unit. Seemingly, the median of the raw ST sludge concentrations obtained during OS2 were similar with that found in OS1. The median of TVS concentrations obtained during OS2 were of 1188 $mg.L^{-1}$ for raw ST sludge, 202 $mg.L^{-1}$ for percolate from sludge unit and 322 $mg.L^{-1}$ for effluent from post-treatment unit. From TS and TVS concentrations it can be concluded that there were effluent with good quality mainly in the sludge unit because of the retention period of the percolate.

In OS3, TS concentrations obtained were of 1613 $mg.L^{-1}$, 953 $mg.L^{-1}$, and 665 $mg.L^{-1}$, for raw ST sludge, percolate from sludge unit and effluent from post-treatment unit respectively. In terms of TVS, concentrations of 942 $mg.L^{-1}$ for raw ST sludge, 422 $mg.L^{-1}$ for percolate from sludge unit and 183 $mg.L^{-1}$ were achieved for effluent from post-treatment unit. Similar to OS2, good quality of the effluent during OS3 was obtained.

In terms of TS and TVS removal efficiencies, it's clear that the system did not show satisfactory results as expected in all operational strategies, except that of OS2. During OS1, the TS removal efficiencies achieved were of 44% for sludge unit, -6% for post-treatment unit and global TS removal efficiency of 42%. The lack of the same hydraulic distribution in the post-treatment unit resulted in a negative impact since this unit received the percolate during the resting period. Thus, the post-treatment unit was not effective for TS removal. The TVS median of removal efficiencies were of 52% for post-treating unit, -5% for post-treatment unit and global TVS removal efficiency of 55%. Similar to the TS removal efficiency, the TVS removal efficiency was negative in the post-treatment unit indicating the lack of an even hydraulic distribution.

In OS2, the TS removal efficiency in sludge unit was of 71%, -6% in the post-treatment unit and global TS removal efficiency of 73% . The results showed that with a retention period of seven days applied in the sludge unit, the TS removal efficiency increased. In experiments with retention time of six days Koottatep *et al.* (2008) found TS removal efficiency ranging from 36% to 50%. Moreover, the results showed good TVS removal efficiency during OS2; 82% for sludge unit, -42% for post-treatment unit and global removal efficiency of 73%. The post-treatment unit decreased TVS removal efficiency from 82% up to 73% having a negative effect. It appears that the retention period allowed the dragged solids to sediment increasing TS and TVS removal in the sludge unit. On the other hand the filtration process in the sludge unit improved with the retention period. Experiments with ponding period of six days conducted by Koottatep *et al.* (2004) proved that TS removal depend on filtration capacity of the CW rather than organic biodegradation.

In OS3, the TS removal efficiencies obtained were of 46% for sludge unit, 30% for post-treatment unit and TS global efficiency of 68%. The TVS removal efficiencies obtained were 67% for sludge unit, 46% for post-treatment unit and global removal efficiency of 80%. Differently from OS1 and OS2, in OS3 a positive impact of the post-treatment unit by retention of the percolate during seven days was observed.

Relating to the discharge standard of TS concentration in accordance to Minas Gerais legislation (100 mg.L^{-1}), it was observed that all operational strategies for ST sludge treatment implemented did not meet the discharge standard. However, the analysis showed that the effluent from post-treatment unit were in the range of characteristics of the raw domestic sewage in developing countries (von Sperling, 2014).
Table 5.21 shows Mann Whitney U-test comparing the median of TS and TVS removal efficiencies achieved in the sludge unit, post-treatment unit and global removal efficiency between three operational strategies.

Table 5.21: *p-values* of the Mann Whitney U-test comparing median of TS and TVS removal efficiencies in the sludge unit, post-treatment unit and global removal efficiency between three operational strategies.

		TS	
Comparison	**OS1-OS2**	**OS1-OS3**	**OS2-OS3**
Sludge unit	0.0147 (*)	0.8962	0.1230
Post-treatment unit	0.4406	0.0010 (*)	0.0021 (*)
Global	0.0600	0.2434	1.0000
		TVS	
Comparison	**OS1-OS2**	**OS1-OS3**	**OS2-OS3**
Sludge unit	0.0003 (*)	0.1587	0.0959
Post-treatment unit	0.0012 (*)	0.0011 (*)	0.0014 (*)
Global	0.0098 (*)	0.0086 (*)	0.5101

(*) *p*-values ≤ 0.05: sample medians are significantly different

Based on the results presented in Table 5.21, the TS removal efficiency in the sludge unit was higher in OS2 than OS1. The analysis confirmed that the retention of the percolate during seven days had significant effects on the TS removal efficiencies and the retention period allowed the dragged solids to sediment, improving the TS removal efficiency in the wetland. Similar to TS, the TVS removal efficiency in the sludge was higher in OS2 compared to OS1. Thus, the analysis confirmed better filterability in the sludge unit caused by accumulated dewatered ST sludge. Better conditions of the filtration and sedimentation during the retention of the percolate in the sludge unit. Most of TS and TVS are removed in the sludge unit.

Moreover, the analysis showed that the retention period of the percolate in the post-treatment unit has significant effects on the TS and TVS removal efficiencies. The retention of the percolate in the post-treatment unit increased the TS and TVS removal efficiencies.

In order to determine the sludge stabilization the TVS/TS ratio was determined. This ratio is reduced when organic matter removal occurs in the treatment system. The ratio of TVS/TS determined was of 0.68. This means that sludge is well digested in the septic tanks. Andrade (2015) for her research under the same VFCW obtained a TVS/TS ratio equivalent to 0.54. In Burkina Faso, Bassan *et al.* (2013) found a TVS/TS equivalent to 0.53 and 0.61. In spite of the values of TVS/TS, indicating digested sludge, there is a significant fraction of organic matter to be digested. Figure 5.25 shows the level of the ST sludge stabilization with time.

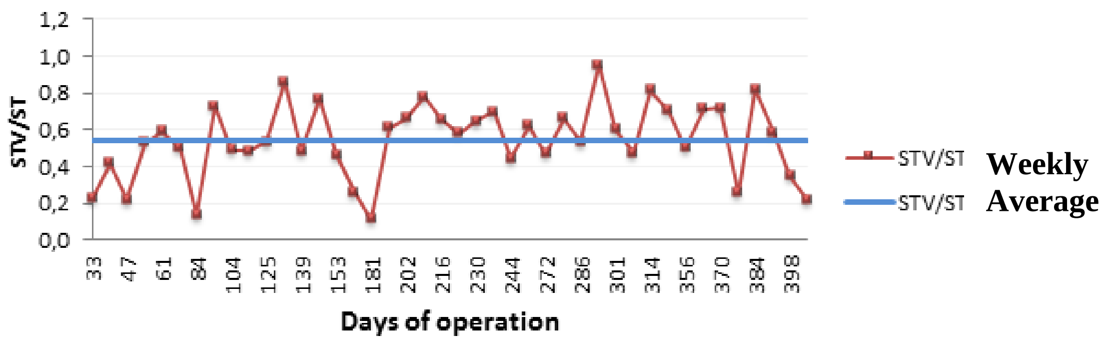


Figure 5.25: Ratio TVS/TS of the raw sludge over time.
Source: Andrade (2015)

5.4.8 NH_4^+-N and TKN removals in OS1, OS2 and OS3

The literature report that normally ST sludge exhibit high concentrations of nitrogen than raw domestic sewage (Strauss, 2004). Figure 5.26 shows NH_4^+-N concentration and removal efficiency in all operational strategies. Figure 5.27 shows TKN concentration and removal efficiency in all operational strategies.

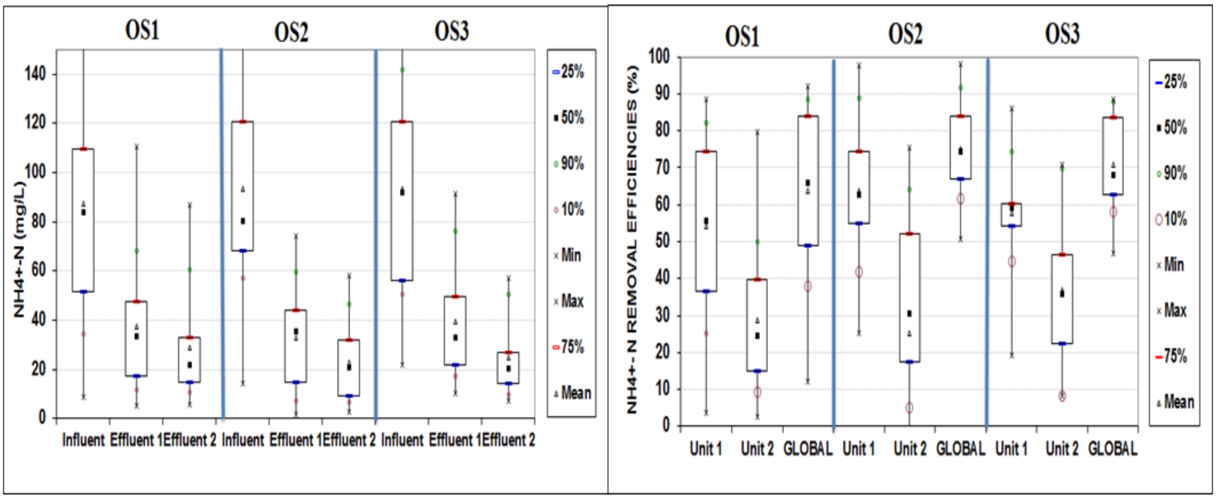


Figure 5.26: Box Whisker graphics of NH_4^+-N concentration and removal efficiency for each experiment.

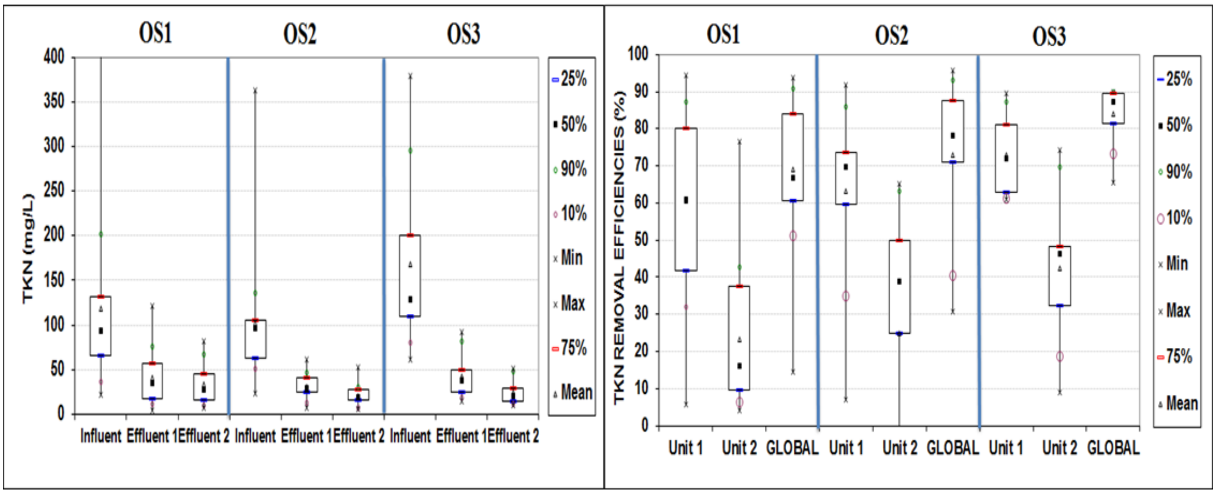


Figure 5.27: Box Whisker graphics of TKN concentration and removal efficiency for each experiment.

In all operational strategies, the analysis showed that the NH_4^+-N concentration in raw ST sludge was characterized by expressive variability as expected. In OS1, the NH_4^+-N influent concentration ranging from 8 mg.L^{-1} to 201 mg.L^{-1}, the median NH_4^+-N concentration in the raw ST sludge, percolate from sludge unit and effluent from post-treatment unit were 84 mg.L^{-1}, 33 mg.L^{-1} and 22 mg.L^{-1} respectively. The median NH_4^+-N concentration in the raw ST sludge obtained was lower compared to literature, probably because of the origin and the characteristics of the ST sludge investigated. Depending on the type of the raw sludge; the literature has reported different NH_4^+-N and TKN concentrations. In Thailand, for NH_4^+-N concentration in the septic tank sludge, Koottatep *et al.* (2004) found a range of 120-1200 mg L^{-1} and an average of 415 mg.L^{-1}. In Brazil, Meneses *et al.* (2001) found an average of 89 mg

L^{-1}. As can be seen the average NH_4^+-N concentration determined by Meneses *et al.* (2001) is similar to that found in present research. With respect to TKN concentrations obtained during OS1, the median TKN concentration in the raw ST sludge, percolate from sludge unit and effluent from post-treatment unit were of 93 mg.L^{-1}, 34 mg.L^{-1}, and 28 mg.L^{-1} respectively. It was found that the influent TKN concentrations ranging from 21 mg.L^{-1} to 564 mg.L^{-1}.

In OS1, the NH_4^+-N removal efficiencies obtained were of 56% for sludge unit, 24% for post-treatment unit and global NH_4^+-N removal efficiency of 66%. The TKN removal efficiencies achieved were of 61% for sludge unit, 16% for post-treatment unit and global TKN removal efficiency of 67%. The highest rate of NH_4^+-N and TKN removal occurred in the sludge unit mainly because of oxygen transfer into the media by convection due to batch loading and diffusion process (Molle *et al.*, 2006).

In OS2, the raw ST sludge NH_4^+-N concentration ranging from 14 mg.L^{-1} to 204 mg.L^{-1} was found. The NH_4^+-N concentrations obtained were of 80 mg L^{-1} in the raw ST sludge, 35 mg.L^{-1} in the percolate from sludge unit and 21 mg.L^{-1} in the effluent from post-treatment unit. Visibly, the same results achieved during the OS1 in terms of NH_4^+-N concentrations were obtained. On the other hand, the raw ST sludge TKN concentration achieved ranged from 24 mg L^{-1} to 363 mg L^{-1}. The TKN concentrations obtained during OS2 were of 96 mg L^{-1} in the raw ST sludge, 30 mg L^{-1} in the percolate from sludge unit and 18 mg L^{-1} in the effluent from post-treatment unit. The NH_4^+-N removal efficiencies achieved were of 63% for sludge unit, 31% for post-treatment unit and global NH_4^+-N removal efficiency of 75%. The TKN removal efficiencies were of 70% for sludge unit, 39% for post-treatment unit and global TKN removal efficiency of 78%. Visibly, the retention of the percolate in the sludge has significant effects on the NH_4^+-N and TKN removal efficiencies. Also Koottatep *et al.*, (2004) proved that having the longest ponding period of six days achieved the highest NH_4^+-N and TKN removal.

Regarding OS3, it was found the raw ST sludge NH_4^+-N concentration ranging from 22 mg.L^{-1} to 186 mg L^{-1}. In the raw ST sludge a median of NH_4^+-N concentration of 92 mg L^{-1}, 33 mg L^{-1} in the percolate from sludge unit and 20 mg.L^{-1} was determined for effluent from post-treatment unit. The TKN concentrations achieved were 128 mg.L^{-1}, 38 mg.L^{-1} and 20 mg.L^{-1} in the raw ST sludge, percolate from sludge unit and effluent from post-treatment unit respectively. The NH_4^+-N removal efficiencies obtained around 59% for sludge unit, 36% for post-treatment unit and global NH_4^+-N removal efficiency of 68%. The results showed that the NH_4^+-N removal in the post-treatment unit increased during OS3 due to retention of the percolate applied. In relation to TKN removal, a median of 72%, 46% and 87% for sludge unit, post-treatment unit and global removal efficiency respectively were obtained.

Analysing the data obtained in the three operational strategies, it was observed that the system showed good performance in terms of NH_4^+-N and TKN removals. Table 5.22 shows Mann Whitney U-test comparing the median of NH_4^+-N and TKN removal efficiencies achieved in the sludge unit, post-treatment unit and global removal efficiency between three operational strategies.

Table 5.22: *p-values* of the Mann Whitney U-test comparing median of NH_4^+-N and TKN removal efficiencies in the sludge unit, post-treatment unit and global removal efficiency between three operational strategies.

		NH_4^+-N	
Comparison	**OS1-OS2**	**OS1-OS3**	**OS2-OS3**
Sludge unit	0.1505	0.7442	0.4506
Post-treatment unit	0.5346	0.5091	0.5744
Global	0.0969	0.5091	0.3930
		TKN	
Comparison	**OS1-OS2**	**OS1-OS3**	**OS2-OS3**
Sludge unit	0.5609	0.1426	0.4150
Post-treatment unit	0.0223 (*)	0.0184 (*)	0.5541
Global	0.3172	0.0251 (*)	0.2575

(*) *p*-values ≤ 0.05: sample medians are significantly different

From statistical analysis, the retention period did not have significant effects on the NH_4^+-N removal efficiency. On the other hand, the analysis showed significant effects on the TKN removal effects in the post-treatment unit. The TKN removal efficiency increased when the retention of the percolate during seven days in the sludge unit and the retention of the effluent from the sludge unit in the post-treatment unit during seven days were implemented. Such as the organic matter, the results showed that most of the NH_4^+-N and TKN are removed in the sludge unit. During the retention period, probably the CW had enough time for plant uptake increasing the TKN removal. On the other hand, the retention of the percolate allowed an oxidative environment in the system.

Koné and Strauss (2004) reinforced that in the CW treating ST sludge, the nitrogen removal is due mainly to the accumulation of organic-N in the dewatered sludge. Moreover, the authors pointed out that the NH_4^+-N and TKN removal are due to ammonia volatilization, plant uptake, nitrification and denitrification.

Based on the results obtained and according to the FS classification proposed by Koottatep *et al.* (2004) it was seen that the FS analysed in this research is classified as Type B (low-strength) characterized by low concentration, usually stored for several years and more stabilized than Type A.

5.4.9 Oxygen, pH and temperature in OS1, OS2 and OS3

Figure 5.28 shows the oxygen concentrations in the raw ST sludge, percolate from sludge unit and effluent from post-treatment unit obtained during the OS1 and OS2. Figure 5.29 presents the variation of pH in the influent, percolate from sludge unit and effluent from post-treatment unit during the three operational strategies, on the other hand it is illustrating the variation of temperature obtained during the OS1 and OS2. It is important to mention that the oxygen and temperature were not analysed during the OS3.

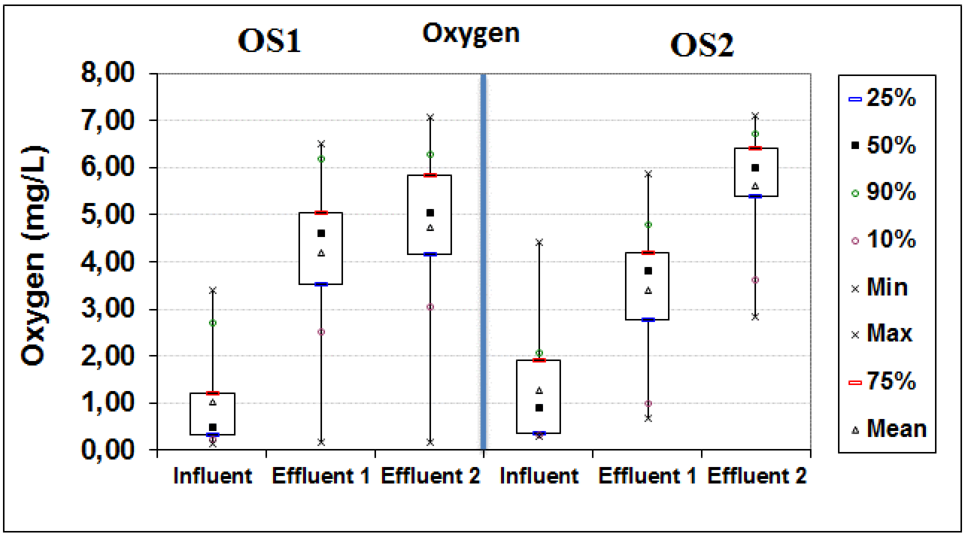


Figure 5.28: Box-plot of oxygen concentration during the operational strategy 1 and operational strategy 2.

As expected, in both cases higher oxygen concentrations in the percolate from sludge unit and effluent from post-treatment unit as compared with the oxygen concentration in the raw ST sludge were observed, indicating better conditions or an environment than can influence the oxygen transfer. For the experiment without retention period, it was not found statistical difference at 5% significant level, between percolate from sludge unit and effluent from post-treatment unit in terms of oxygen concentration. In OS1, the median DO concentrations obtained were of 0.5 $mg.L^{-1}$ for raw ST sludge, 4.6 $mg.L^{-1}$ for percolate from sludge unit and 5.1 $mg.L^{-1}$ for effluent from post-treatment unit respectively. Thus, from dissolved oxygen concentrations obtained in the percolate and effluent from post-treatment unit, it is evident that an oxidative environment in the CW. The results demonstrated effects of the oxygen transfer in the CW enhancing nitrification process (Koottatep *et al.,* 2008).

For experiment with retention period of seven days, statistical difference at 5% significant level were found, between percolate from sludge unit and effluent from post-treatment unit in terms of oxygen concentration. In this case, the oxygen concentration in the effluent from sludge unit was higher than percolate from post-treatment unit. The DO concentrations obtained were 0.9 $mg.L^{-1}$ for raw sludge, 3.8 $mg.L^{-1}$ for percolate from sludge unit and 6.0 $mg.L^{-1}$ for effluent from post-treatment unit respectively. Such as in OS1, the system showed an oxidative environment and revealed the beneficial effects of VFCW mode, the presence of plants and ventilation system (Koottatep *et al.,* 2008).

Regarding pH and temperature, some variations in all cases were noted. In OS1, the pH of the raw sludge varied from 7.13 to 6.68 in the percolate from post-treatment unit and 6.52 in the effluent from post-treatment unit. These variations were significantly different at 5% level and this means alkalinity consumption as result of nitrification (Lana, 2013). Nevertheless, no statistical difference of pH between percolate from sludge unit and effluent from post-treatment unit were found.

Regarding OS2 the same type of variation occurred, wherein the pH of the influent varied from 7.81 to 7.24 in the percolate from sludge unit and 6.92 in the effluent from post-treatment unit. In this case, no statistical difference between influent and percolate from sludge unit were found. The temperature also showed some variations in both experiments as can be seen in Figure 5.29. Thus, it was observed little variation of the temperature tending to its reduction.

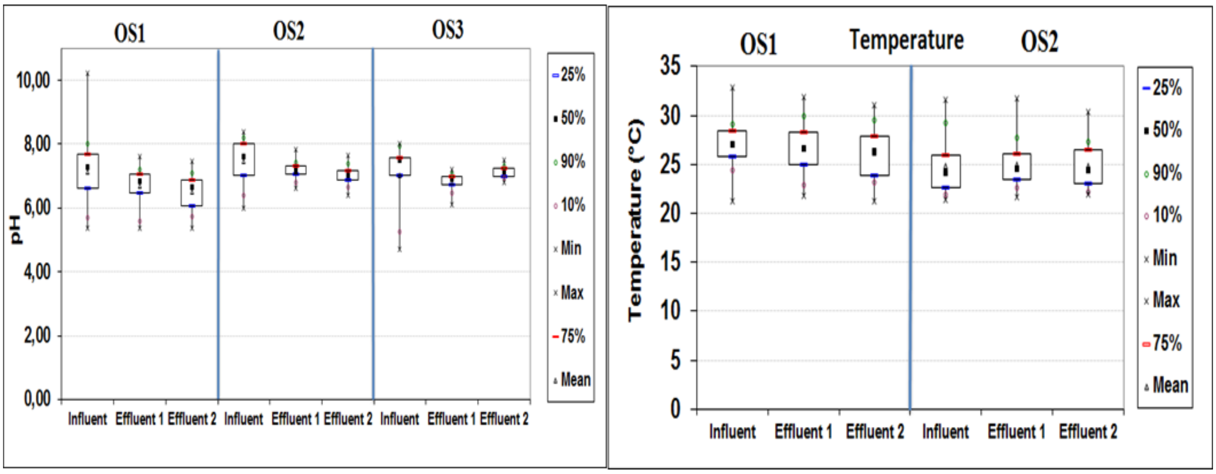


Figure 5.29: Box Whisker graphics of pH (in the left) illustrating the three operational strategies and Temperature (in the right) illustrating the operational strategy 1 and operational strategy 2.

The analysis showed that the pH and Temperature met discharge standards in accordance with legislation of Minas Gerais through of COPAM/CERH-MG nº 01/2008. pH was set between 6.0 and 9.0 and temperature less than 40 ºC. However, the pH of the influent in first operational strategy, was characterized by some variations and a pH higher than 9.0 was observed. Thus, some samples in influent did not meet discharge standard. For temperature, all samples evaluated met discharge standard.

5.4.10 Stabilization and mineralization of top sludge

The stabilization and mineralization of sludge in the sludge unit were successfully observed. At the same experiment, Andrade (2015) determined the level of sludge accumulation during the monitoring period. Figure 5.30 shows the level of sludge accumulation over monitoring period.

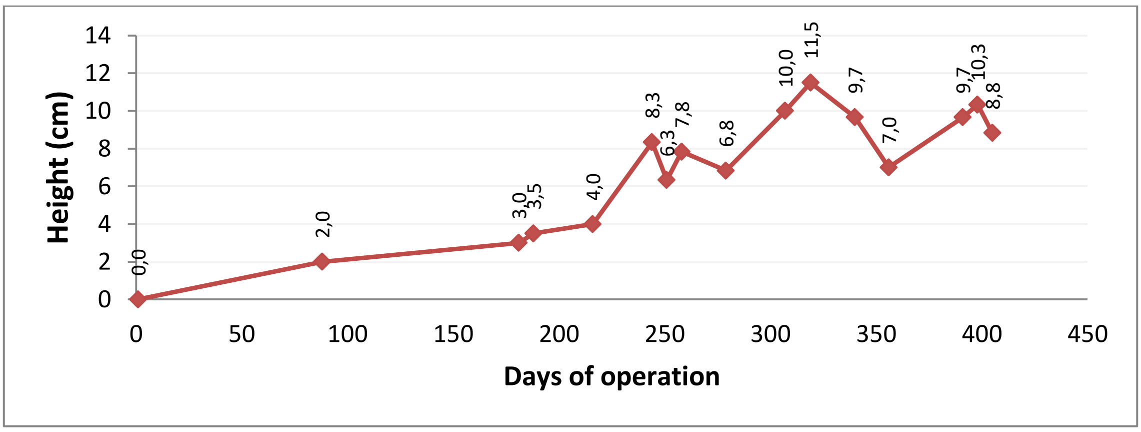


Figure 5.30: Sludge height accumulation during the monitoring period.

The system demonstrated successful stabilization and mineralization of organic matter on top of the surface. From the results, it was seen that from the accumulated sludge in the system around 55% belonged to total solids and 45% was part of wet matter. The height of the sludge on top of the surface was also measured and accumulated sludge of 8.5 cm was found at the beginning of the sludge unit, 10cm at the middle and 8.0 cm at the end of the sludge unit.

Parameters normally used to estimate the evolution of organic matter in soil were determined to determine which process or organic matter for mineralization and/or humification. The ratio of TVS/TS were also calculated in soil. Andrade (2015) found a ratio of TVS/TS equivalent to 0.60. This ratio indicated that the stabilization and mineralization occurred well in the sludge unit, but there was a significant fraction of organic matter to be stabilized.

5.4.11 Plants monitoring

The plants were observed along the experimental period to determine the biomass production, health, height and its capacity for nutrient uptake. In general terms *Cynodon dactylon Pers* showed excellent adaptability under different conditions (higher and less overloads), fast growth, more robustness. The plant achieved about 1 m of the height during forty five days of its growth. The average value of *plant* biomass production observed was 41.2 ton dry matter.ha^{-1}.$year^{-1}$. The density of this *Cynodon dactylon Pers (*Tifton-85) is extremely high, and an average of 1200 units per m^2 were found in the sludge unit (Ávila Lopez, 2016). The concentration of the nutrients in the plant were also measured in order to determine to what extent the nutrient uptake from septic tank sludge by the plants occur. Thus, nutritional informational of the following nutrients were analysed: Nitrogen (N), Potassium (K), Sodium (Na), Phosphorus (P), Copper (Cu), Zinc (Zn), Cadmium (Cd) and Lead (Pb). Table 5.23 presents the capacity of nutrient uptake obtained by plants.

Table 5.23: Evaluation of nutrient uptake by plants (aerial parte) during the operational strategy 2.

Nutrients	**Nutrient uptake rate $kg.ha^{-1}.year^{-1}$**
N	982
K	1086
Na	30
P	8.4
Cu	1.3
Zn	13.0
Cd	0.2
Pb	0.041

The analysis showed that the presence of plants allows nutrients uptake from ST sludge mainly for N, K, Na, P, Zn. In vertical flow constructed wetlands, plants play a key role in sludge stabilization due to evapotranspiration process, which decreases the residual water content of fresh sludge. In the system nitrogen load of 12792 kg N $ha^{-1}.year^{-1}$ was applied, the TKN removal efficiency achieved was of 70% and only 7.7% of nitrogen was removed by plants.

5.5 Hydraulic behaviour and solids mass balance in septic tank sludge unit

Such as the evaluation of the hydraulic behaviour and solids mass balance in raw domestic sewage, during OS1 of ST sludge treatment tests were conducted in order to evaluate the hydraulic behaviour of the system and its performance in terms of percentage of the total solids mass reduction efficiency. The hydraulic evaluation and total solids mass balance of ST sludge were conducted in two days; the first test was done on 01/10/2014 and the second one took place on 08/10/2014. On 01/10/2014, 8 m^3 of ST sludge was discharged onto the sludge unit resulting a total solids mass of 15.4 kg TS according to the TSS measured in the ST sludge. On 08/10/2014, the sludge unit was loaded with a volume of 12 m^3 of ST sludge having a total solids mass of 115.8 kg TS.

5.5.1 Hydrographs of effluent flow

In both cases of two volumes discharged onto the sludge unit it was seen that there was a rising outflow at the first moment followed by a declining outflow over time. Figure 5.31 shows the variation of effluent outflow profile over time.

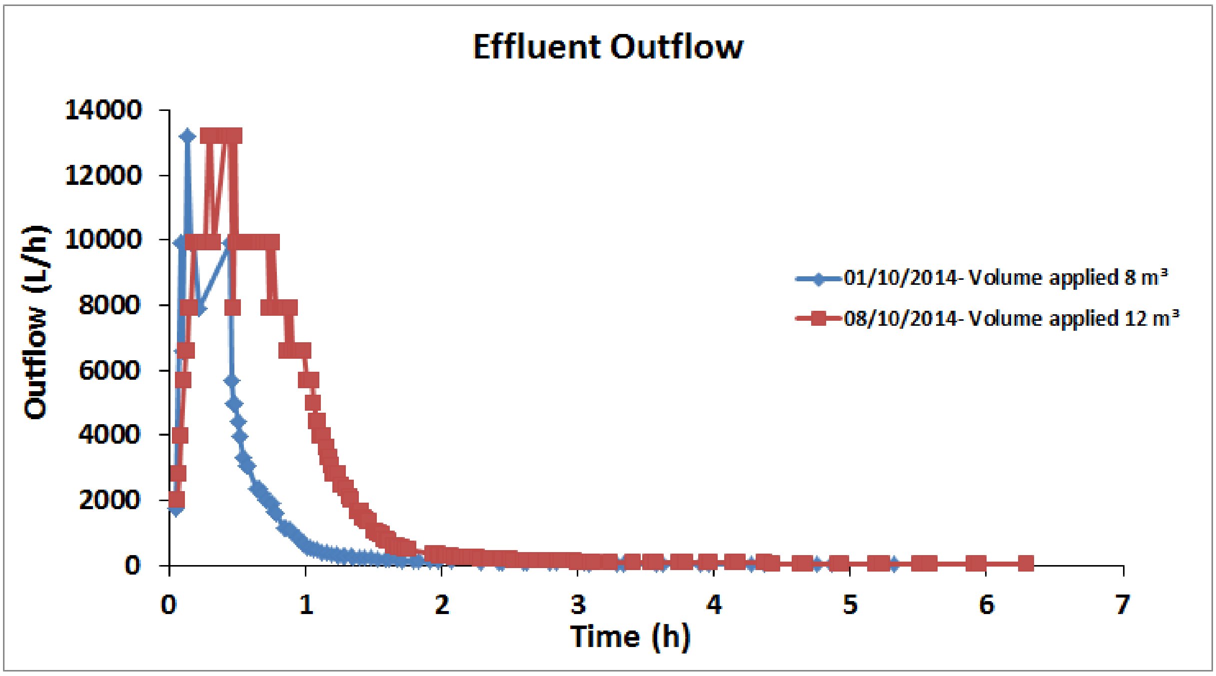


Figure 5.31: Variation of outflow profile over time.

As commented before, it was seen that after feeding, the effluent outflow increased and decreased slowly. Nevertheless, a similar pattern in both curves exhibiting pronounced outflow peak of 13000 L h^{-1} was observed. After batch loading of 8 m^3 of the ST sludge, a pronounced outflow peak at the first 8.4 minutes was achieved and for batch loading of 12 m^3 of the ST sludge the peak was achieved at the first 17.4 minutes. In both cases, from the outflow peak the curves started declining up to the end of outflow. The difference of profile of the curves in both tests is due to higher volume of ST discharged on 08/10/2014 than the first test. Molle *et al.* (2005) observed that the high volume of batch of wastewater into VFCW is positive to oxygen diffusion but negative for hydraulic retention time of water and effective outlet peaks of non-oxidized pollutants. However, steep increase and decrease of effluent curves shows a good and fast dewatering of the filter, which indicates an effective oxygen transfer through convection and diffusion in the reed bed (Kayser and Kunst, 2005).

5.5.2 Outlet cumulative volume profile over time

The cumulative volume express the variation of the volume over time and is an indicator of the total volume obtained during the test. Figure 5.32 shows the cumulative volume profile over time obtained during the tests.

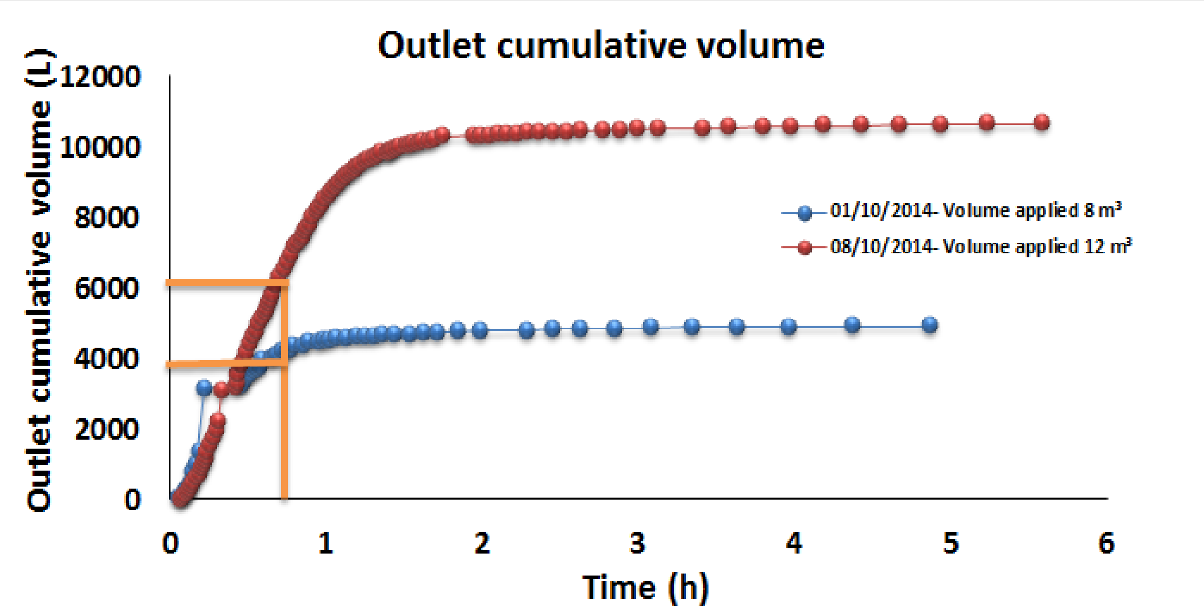


Figure 5.32: Outlet cumulative volume profile over time.

The analysis showed that, for the first case, after the discharge of 8 m³ of the ST sludge, a maximum volume of 4.9 m³ was obtained in 4 hours and 52 minutes and 50% of the volume applied (4 m³) was achieved at the first 37 minutes after batch loading. It is important to emphasize that the effluent outflow continued after the test in both cases. For the second case, a maximum volume of 10.7 m³ was obtained in 6 hours and 50% of the volume applied (6 m³) was achieved at the first 37 minutes after batch loading.

5.5.3 Effluent TSS Concentrations and solids load profiles over time

Figure 5.33 presents the variation of the TSS concentration and the variation of the solids load profile over time is displayed in Figure 5.34. The Profile of the TSS concentration and solids load were determined along the effluent outflow presented in Figure 5.31.

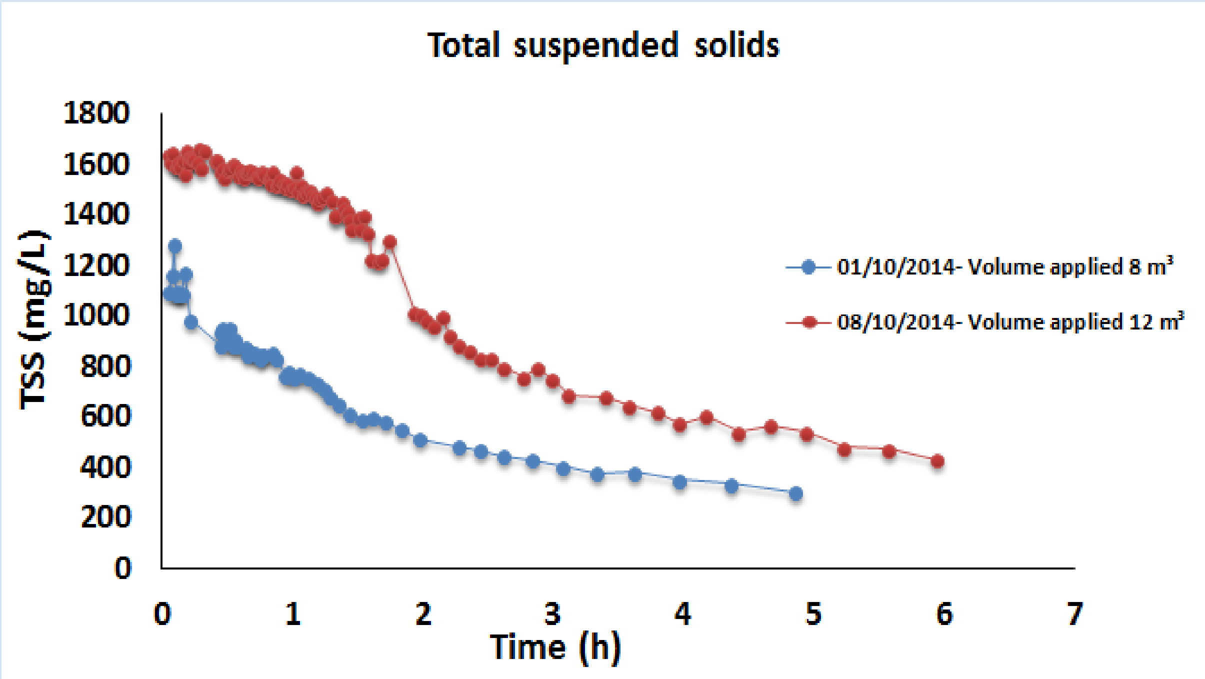


Figure 5.33: Outlet TSS concentration profile over time.

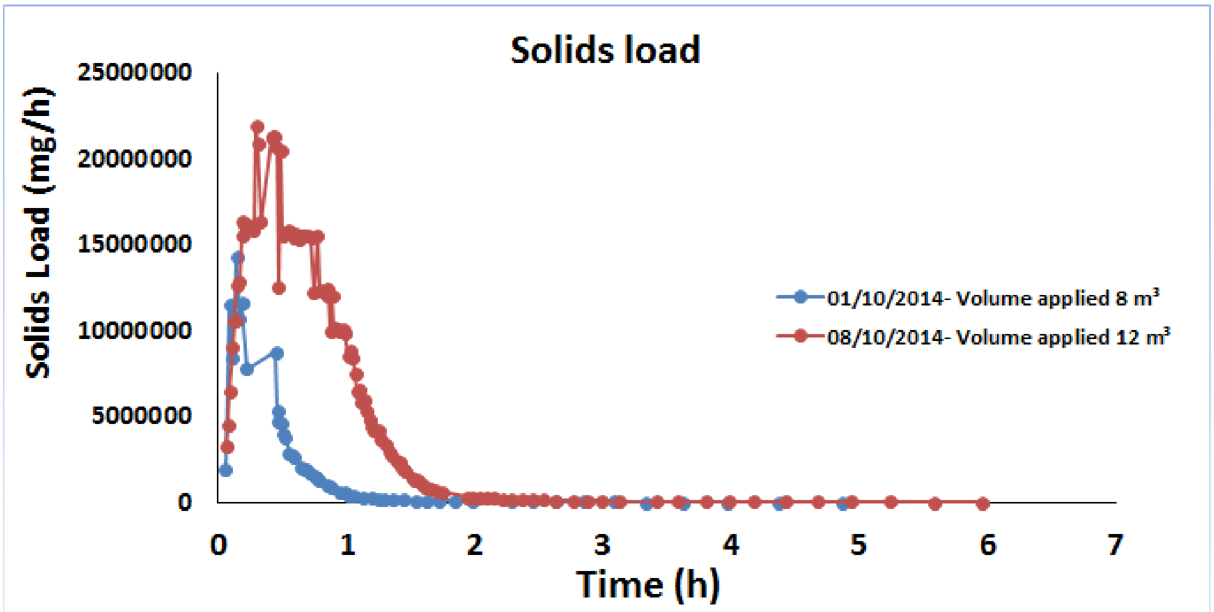


Figure 5.34: Outlet solids load profile over time.

By observing Figure 5.33, it is clear that after feeding, the TSS concentration decreases in both tests. This particularity demonstrated that the quality of the effluent has improved with time. In the first test, it was observed that after feeding the TSS concentration reduced from 1093 $mg.L^{-1}$ up to 299 $mg.L^{-1}$ during 4.8 hours. For the second test, the TSS concentration in the effluent reduced from 1637 $mg.L^{-1}$ up to 430 $mg.L^{-1}$ during 5.9 hours. The reduction of the TSS concentration in the effluent over time is related with HRT. When HRT is high, the quality of the effluent also improve. Comparing the effluent outflow and TSS concentration in the effluent, the analysis shows higher TSS concentrations associated with higher effluent outflow.

As commented before, the SL expresses the variation of the effluent outflow and TSS concentration over time. Such as effluent outflow curves, the test of solids load over time showed some variations, after feeding rising solids load up to peak or maximum value followed by its declining was registered. The profile curve of SL obtained was similar to the curve of outflow.

After batch loading of 8 m^3 of the ST sludge the minimum value of SL obtained was of 2 mg s^{-1} in 4.8 hours. The maximum value was of 3986 mg s^{-1} achieved in 8 minutes. This means that the maximum value of SL was achieved with maximum value of effluent outflow. After batch loading of 12 m^3 of the ST sludge, the minimum value of SL determined was of 3.78 mg s^{-1} in 5.9 hours. The maximum value of SL obtained was of 6087 mg s^{-1} in 17 minutes. The SL depends on effluent outflow and TSS concentration. The SL is high when the effluent outflow increases or TSS concentration increases.

5.5.4 Outlet cumulative mass and TSS mass applied

Figure 5.35 shows the curves of outlet cumulative mass profile over time and Figure 5.36 shows the percentage of total solids mass applied over time.

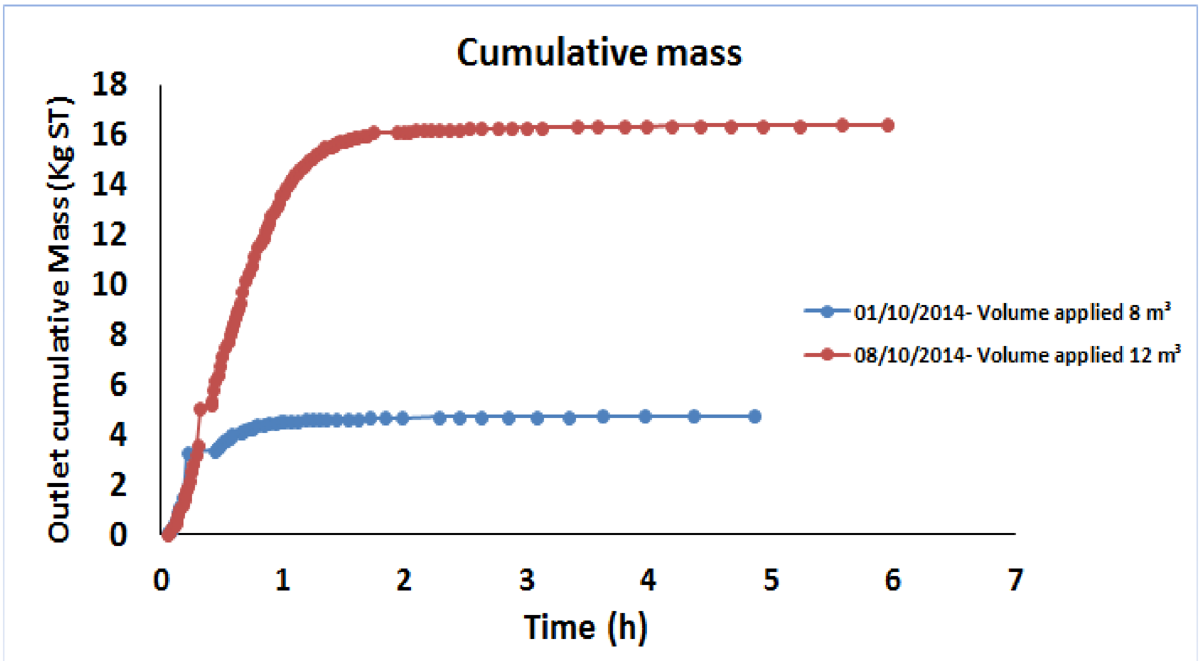


Figure 5.35: Curves of outlet cumulative mass profile over time.

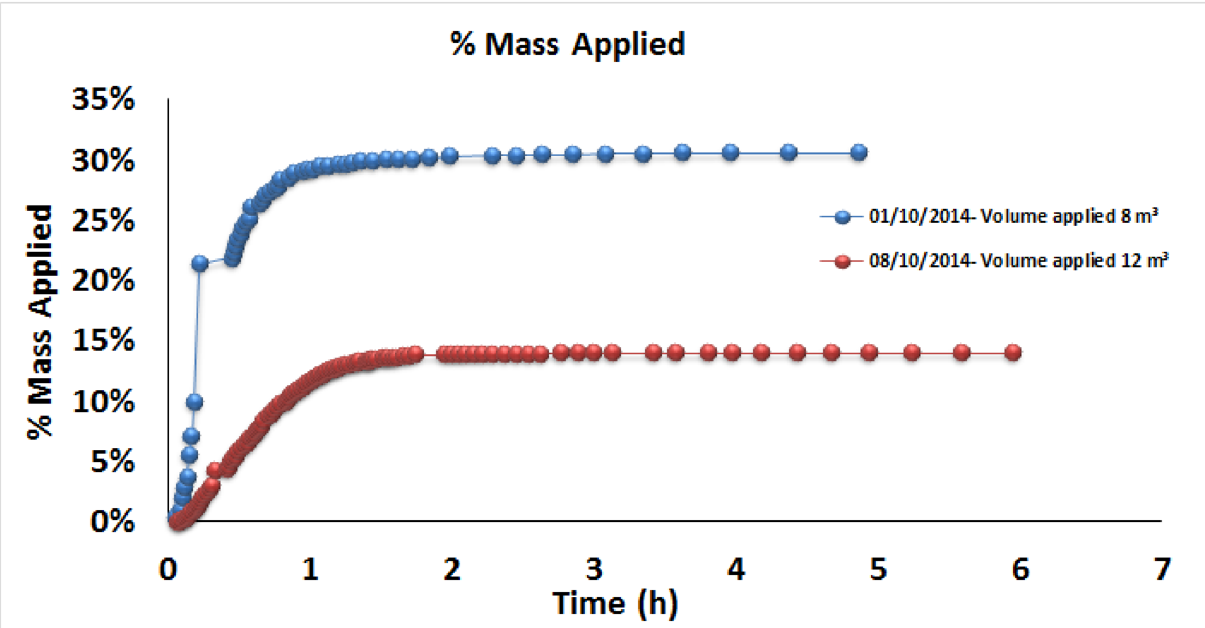


Figure 5.36: Curves percentage of mass profile over time.

Such as the test with raw domestic sewage, the test concerning ST sludge demonstrated that the outlet cumulative mass and the percentage of total solids mass applied increase up to maximum value, which remains constant. In the first test from inlet the total solids mass applied in the system was of 15.4 kg TS. From outlet the total solids mass accumulated in the effluent was 4.7 kg TS equivalent to the percentage of solids mass recovery of the 30.69%. Thus, for the first test the percentage of mass reduction efficiency obtained was of 69%.

Therefore, on 01/10/2014, the test carried out demonstrated that 69% of total solids mass were removed in an hour.

For the second test from inlet, a total solids mass of 115.9 kg TS was applied in the system. From outlet the cumulative total solids mass obtained was of 16.37 kg TS equivalent to the percentage of solids mass recovery of 14.13%, thereby, for the second test the percentage of mass reduction efficiency determined was of 85.9%. So, on 08/10/2014, the test carried out demonstrated that about 85.9% of total solids mass were removed in an hour and half.

The data analysis, demonstrated satisfactory performance of VFCW in terms of solids mass reduction over time and through the results achieved, the tests showed that the methodology adopted to evaluate mass balance was successfully suitable. It is a straightforward method, practical and appropriate in the experimental site.

6 CONCLUSIONS

Regarding VFCW treating **raw domestic sewage** the following aspects can be concluded:

- ✓ The system was able to remove satisfactorily organic matter and suspended solids, when it worked according to the French recommendations for the first stage (two alternating beds in parallel) leading to median removal efficiencies around 84% for BOD_5, 76% for COD, 85% for TSS and 63% for NH_4^+-N and 55% for TKN.
- ✓ When comparing the planted and unplanted units in terms of effluent concentrations and removal efficiencies, there were no significant differences between them, with the single exception of TKN when the system was working with total hydraulic loading rate of 0.15 $m^3\ m^{-2}\ d^{-1}$. In this case, the TKN was higher in the planted unit than the unplanted unit. However, the presence of the plants *Cynodon dactylon pers* prevented vertical flow system from clogging, provided good conditions for physical filtration and stabilized the surface of the unit.
- ✓ Comparing two different hydraulic loading rate of 0.22 $m^3.m^{-2}.d^{-1}$ and 0.15 $m^3.m^{-2}.d^{-1}$ there were no significant difference in the removal efficiencies of BOD, TKN and NH_4^+-N in the planted unit, for both operational modes. The removal efficiencies of COD and TSS decreased with an increase in HLR except for BOD_5, NH_4^+-N and TKN. The total hydraulic loading rate of 0.15 $m^3.m^{-2}.d^{-1}$ working with three units in parallel was found as the best alternative and allows satisfactory operation of the vertical system for organic matter, total suspended solids and total Kjeldahl nitrogen removal.
- ✓ From the overall results, it can be concluded that the utilization of only the first stage of the French/Cemagref systems shows a large potential whenever simple systems are required for the treatment of raw domestic sewage in developing and warm-climate regions.

Regarding VFCW treating **septic tank sludge** the following aspects can be concluded:

- ✓ In terms of septic tank sludge treatment using VFCW, the operational strategy worked without any retention of the percolate in the sludge unit followed by post-treatment of the percolate in another unit performed well obtaining the global median removal efficiencies of around 73% for BOD, 81% for COD, 42% for TS, 55% for TVS, 66% for NH_4^+-N, 67% for TKN and 25% for *E. coli.*
- ✓ The operational strategy with retention of the percolate in the sludge unit for seven days followed by sending the percolate to post-treatment unit, the system increased its performance obtaining the global median removal efficiencies around 97% for BOD, 90% for COD, 73% for TS, 73% for TVS, 75% for $NH4^+$-N and 78% for TKN and 99.94 for *E. coli.*
- ✓ Concerning the operational strategy where the percolate was retained during seven days in the sludge unit and further retention of the percolate for another seven days in the post-treatment unit, the system increased its performance obtaining the global median removal efficiencies of 90% for BOD, 94% for COD, 68% for TS, 80% for TVS, 68% for NH_4^+-N, 87% for TKN and 99.98% (equivalent to more than 3 log units removal) for *E. coli*.
- ✓ In terms of septic tank sludge treatment, the retention period of fourteen days (seven days in the sludge unit and seven days in the post-treatment unit) was found as the best alternative for organic matter, total solids, total Kjeldahl nitrogen and *Escherichia coli* removal.
- ✓ The median hydraulic loading rate of 0.275 m^3discharge m^{-2}.week^{-1} and the median solids loading rate of 32 kg TS m^{-2}.year^{-1} allow satisfactory operation of vertical flow constructed wetland treating septic tank sludge.
- ✓ The system resisted variations when different types of septic tank sludge, different hydraulic loading and solids loading rates were received.
- ✓ During all operational strategies no clogging of the bed surface occurred.
- ✓ The dewatering of accumulated sludge occurred well, producing a top sludge with 55% of dry solids and 45% of wet matter. Concerning sludge stabilization and mineralization a TVS/TS equivalent to 60% was found. This means that the stabilization and mineralization occurred well in the vertical flow constructed wetland, but there were significant fraction of the organic matter to be stabilized.

- ✓ The plant Tifton-85 (*Cynodon dactylon Pers*) adapted well in the system contributing for nutrients uptake when different hydraulic and solids loading rates were applied.
- ✓ There are indications that the final effluent from the system operating with percolate retention in the sludge unit and in the post-treatment unit is good and probably acceptable for the requirements of many developing countries.
- ✓ Therefore, depending on the quality required for the percolated effluent, different operational strategies can be adopted.
- ✓ The system is a very attractive alternative for developing countries when compliance with very stringent discharge standards are not required.

Validation of hypothesis

Hypothesis 1 was accepted - The first stage of the French system of vertical flow constructed wetland can be adapted for the treatment of sludge from septic tanks and the treatment of domestic sewage in warm regions**.**

Hypothesis 2 was accepted - The first stage of French system of vertical flow constructed wetland allows different loading regimens for treatment of raw domestic sewage and septic tank sludge in warm regions.

Hypothesis 3 was rejected - There are differences in terms of removal efficiencies between planted and unplanted units of vertical flow constructed wetlands treating domestic sewage.

Hypothesis 4 was rejected – The performance of vertical flow constructed wetlands treating sludge from septic tanks is influenced by the applied hydraulic and solids loading rates.

Hypothesis 5 was accepted - Vertical flow constructed wetlands for the treatment of sludge from septic tanks allows dewatering, stabilization and mineralization of the sludge in the top deposit layer of the bed.

7 RECOMMENDATIONS

- Concerning VFCW treating raw domestic sewage, a longer-term study is required in order to state more confidently about the possible differences in performance and the full adequacy of the system with only two units in the first stage of the French system.
- Considering only two units of first stage treatment of the VFCW treating raw domestic sewage and septic tank sludge, the system showed strength and robustness, and it thus can be recommended for small communities. However, post-treatment of the percolate is required.
- Studies of hydraulic evaluation in terms of flux and oxygen transfer in order to determine hydraulic changes and nitrification/denitrification processes are required.

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