Decentralised sanitation to fill the gap in urban wastewater treatment within the eThekwini Municipality: a focus on tertiary treatment in vertical down-flow constructed wetlands

by

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As the candidate's Supervisors we agree to the submission of this thesis.

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Water is life, Sanitation is dignity.

This thesis is dedicated to the late **Professor Christopher Andrew Buckley** (5 July 1949 – 27 May 2021) who was at the forefront of innovative sanitation research globally. His drive and passion to provide water and sanitation services to the unserved is a motivation to all in the sector.

Chris, you are a legend and I miss you.

PREFACE

The research contained in this thesis was completed by the candidate while based at the Water, Sanitation & Hygiene Research and Development (WASH R&D) Centre in the Discipline of Chemical Engineering, School of Engineering of the College of Agriculture, Engineering and Science, University of KwaZulu-Natal, Howard College Campus, South Africa. The research was financially supported by the South African Water Research Commission (WRC), eThekwini Water and Sanitation (EWS) unit and the Bremen Overseas Research and Development Association (BORDA).

The contents of this work have not been submitted in any form to another university and, except where the work of others is acknowledged in the text, the results reported are due to investigations by the candidate.

Copyright permission was granted from Professor Elizabeth Tilley for use of some illustrations from the *Compendium of sanitation systems and technologies*.

The referencing style used in this thesis is *Water SA*, the accredited scientific journal of the WRC.

DECLARATION 1: PLAGIARISM

I, Preyan Arumugam, declare that:

(i) the research reported in this thesis, except where otherwise indicated or acknowledged, is my original work;

(ii) this thesis has not been submitted in full or in part for any degree or examination to any other university;

(iii) this thesis does not contain other persons' data, pictures, graphs or other information, unless specifically acknowledged as being sourced from other persons;

(iv) this thesis does not contain other persons' writing, unless specifically acknowledged as being sourced from other researchers. Where other written sources have been quoted, then:

   a. their words have been re-written but the general information attributed to them has been referenced;

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(v) where I have used material for which publications followed, I have indicated in detail my role in the work;

(vi) this thesis is primarily a collection of material, prepared by myself, published as journal articles or presented as a poster and oral presentations at conferences. In some cases, additional material has been included;

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Signed: Preyan Arumugam

PhD Candidate

DECLARATION 2: SCIENTIFIC OUTPUTS

All scientific papers, reports and conference presentations emanating from this research are listed here. My role in each output is described. * Indicates corresponding author / presenter.

JOURNAL ARTICLES

The portion of the data from this research was documented in one peer-reviewed research article. The purpose of this paper was to conduct a performance evaluation of the entire decentralised wastewater treatment system (DEWATS), including the hybrid constructed wetland (CW) system after long term operation. I conducted the field work, interpreted the data, and wrote the manuscript which was edited by the co-authors.

<u>Arumugam P</u>*, Zuma L, Mercer S, Govender L, Pocock J, Brouckaert CJ and Gounden T 2022. The potential of decentralised wastewater treatment in urban and rural sanitation in South Africa: lessons learnt from a demonstration-scale DEWATS within the eThekwini Municipality. Water SA (accepted for review)

REPORTS

Some of the research results presented in this thesis, including the design limitations and subsequent upgrades to the demonstration-scale CWs prior to the performance evaluations emanate from a project funded by the Water Research Commission, project K5/2579 entitled:

Performance assessment of DEWATS constructed wetlands.

The research was first conceptualised by the late Prof. Buckley (as my main supervisor from July 2015 to December 2020) which was then amended by me as the project progressed. I undertook all of the research activities, interpreted the data, and wrote up all of the deliverable reports.

<u>Arumugam P</u>* and Buckley C (2020) Performance evaluation of constructed wetlands in a BORDA-designed decentralised wastewater treatment system. WRC Report No. TT 812/20. ISBN 978-0-6392-0126-9, Pretoria, South Africa.

CONFERENCE PRESENTATIONS

In each of the conference presentations, I conceptualised the experimental plan together with Prof. Buckley, conducted the analyses, interpreted the data, and presented the findings.

Presentation 5 was an invitation from the WRC for their virtual conference on Women in Water and Science under the session “Emerging Scientists/Young Professionals in Sanitation”.

1. Arumugam P*, Buckley CA, Gill L, Rodda N and Trois C (2016) Performance assessment of a full-scale vertical down-flow constructed wetland in a decentralised wastewater treatment system for instrumentation and modification. International Conference on Natural and Constructed Wetlands, 21–22 June 2016, Galway, Ireland. Poster. (International)
2. Arumugam P* and Buckley CA (2017) Post treatment of anaerobically treated domestic wastewater in a vertical down-flow constructed wetland: experiences from first pilot DEWATS in South Africa. 7th College Postgraduate Research and Innovation Day, 26 October 2017, UKZN Westville Campus, Durban. Oral. (Local)
3. Arumugam P*, Mwenje R and Buckley CA (2017) Performance assessment of a demonstration vertical down-flow constructed wetland purifying anaerobically treated domestic wastewater. International Young Water Professionals Conference, 10–13 December 2017, Cape Town, South Africa. Oral. (International)
4. Arumugam P* and Buckley CA (2019) The future of decentralised wastewater treatment in South Africa. 6th South African Young Water Professionals Conference, 20-23 October 2019, Durban, South Africa. Oral. (Local)
5. Arumugam P* (2021) DEWATS – from pilot to implementation in non-sewered areas. WRC Conference on Woman and Science and the Impact of COVID 19, 18-19 March 2021. Virtual. Oral. (Local)

21 July 2022

Signed: Preyan Arumugam

Date

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I would like to acknowledge my mentor and supervisor, the late Prof. Chris Buckley. You accepted me when I had almost no hope in completion of my PhD degree. Thank you for always giving me a chance. Your assistance towards my development as a research scientist, in a sector I was completely new to, will always be valued. I am also so grateful for your love of family and allowing me to be a first-time mum to my twin boys who decided to arrive right in the middle of my research. I wish you were still here; your advice and guidance will be remembered always.

My supervisors, Dr. Jon Pocock and Dr. Chris J Brouckaert, your support towards the write-up and completion of the thesis will be eternally appreciated. You both have been my lifesavers towards the end.

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I would not have been able to conduct this work had it not been for the additional funding and site access provided by the eThekwini Water and Sanitation unit (EWS): Ms. Lungi Zuma and Mr. Teddy Gounden, and expert advice and further funding from the BORDA team: Mr. Lloyd Govender, Mrs. Carley Truyens, Mr. Andreas Schmidt, and Mr. Uwe Klaus.

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My closest friends, Mr. Travis Kunnen and Dr. Cassandra Naidoo, thank you for your time and valuable critique when reviewing my thesis each time I pestered you outside of your respective working hours. I am also grateful to Dr. Edwina Mercer for graciously reviewing the final draft.

Lastly, I would like to acknowledge my mentors in Life Sciences: Prof. Yogis Naidoo for nudging me towards the engineering sector, and Dr. Nicola Rodda for guiding me towards issues in sanitation. I will not look back.

ABSTRACT

South Africa's bulk sanitation infrastructure is failing, and there is an urgent need to look at other appropriate sanitation solutions. Moreover, there is no data on the proportion of population with access to safely managed sanitation services, an indicator for the United Nation's Sustainable Development Goal (SDG) 6.2.1a. In a safely managed sanitation service, the user is provided with an improved facility, not shared with other households, and the excreta is safely disposed *in situ* or transported and treated off-site.

In the city of eThekwini, informal settlements spring up faster than services can be delivered, severely impacting on public health, the environment, and the social well-being of these communities. The eThekwini Municipality sees the benefits of decentralised sanitation solutions for *in situ* informal settlement housing upgrades, but the selected system needs to produce fully compliant effluent with the Department of Water and Sanitation's (DWS) Revised General Authorisation (GA) limits for safe discharge to a water resource. Since 2010, a modular-designed demonstration-scale decentralised wastewater treatment system (DEWATS) for raw domestic wastewater from 84 households has been in operation in eThekwini. The DEWATS operates with no electricity or chemicals for treatment, but was designed according to European best practice, and not according to the community served (such as influent characterisation and hydraulic loading). This study evaluated the applicability of vertical down-flow constructed wetlands (VFCWs) as the tertiary treatment module in DEWATS in four design configurations, to determine an appropriate design that can be applied for the formal housing upgrades where safe discharge of the final effluent is required.

These designs, all receiving anaerobically treated domestic wastewater from the demonstration-scale DEWATS and operating in the field, were:

1. A single-stage demonstration-scale VFCW (design 1) compared to its hybrid configuration with a horizontal flow CW (HFCW) (design 2).
2. VFCWs with extended filter depths (1 m) consisting of 2-3 mm coarse sand media (at pilot-scale) (design 3).
3. Two-stage VFCWs (at pilot-scale, operating under field conditions) (design 4):
    a. First stage: 0.5 m filter depth consisting of 2-3 mm coarse sand media.
    b. Second stage: 0.6 m filter depth with 0.5-2 mm fine to coarse sand media.

Neither design was able to produce fully compliant effluent for safe discharge to a water resource. Depth had no impact on the treatment efficiency of the pilot-scale single-stage VFCWs; although the design with a two-stage VFCW, adapted from the Austrian design, did achieve higher total nitrogen removal compared to single-stage VFCWs with/without extended filter depths. Overall, design 2 with the demonstration-scale hybrid CW design (VFCW-HFCW) produced the highest quality effluent. However, nitrate-N removal was limited in the HFCW due to low residence times, mixed aggregate media, high dissolved oxygen (DO) concentrations and lack of available carbon as an energy source for denitrification. A plant-based carbon source from dried plant material of the invasive Giant reed, *Arundo donax* L., was used to augment the carbon availability for denitrifying bacteria within the HFCW. However, it is surmised that the DO concentration above 0.5 mg L^{-1} limited NO_3-N removal.

It is recommended that the DEWATS design with the hybrid CW system be redesigned according to the raw wastewater characterisation and media gradation within both CWs to ensure sufficient residence times, natural aeration in the VFCW, limited diffusion of oxygen into the HFCW, and increased availability of biodegradable chemical oxygen demand carbon for denitrification. Moreover, if the upgraded households are installed with urine diversion flushing toilets, then the nutrient load to the DEWATS will be reduced, potentially resulting in fully compliant effluent. Consequently, DEWATS will then be considered a safely managed sanitation service, allowing South Africa to track their progress against SDG 6.2.1a.

Keywords: decentralised sanitation; DEWATS; vertical down-flow constructed wetlands; safely managed sanitation services; SDG 6.2.

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ABBREVIATIONS

ABR	Anaerobic Baffled Reactor
AF	Anaerobic Filter
bCOD	biodegradable Chemical Oxygen Demand
BORDA	Bremen Overseas Research and Development Association
COD	Chemical Oxygen Demand (total)
CW	Constructed Wetland
DWS	Department of Water and Sanitation
EWS	eThekwini Water and Sanitation
GA	General Authorisations
HFCW	Horizontal Flow Constructed Wetland
HRT	Hydraulic Retention Time
HLR	Hydraulic Loading Rate or q
JMP	Joint Monitoring Programme
KZN	KwaZulu-Natal
NM	Newlands Mashu research site
OM	Organic Matter
O&M	Operation and Maintenance
PE	Person Equivalent
SDG	Sustainable Development Goals
SF	Surface Flow
SFD	Shit Flow Diagram
SSF	Subsurface Flow
TN	Total Nitrogen
TSS	Total Suspended Solids
UDFT	Urine Diversion Flushing Toilet
UN	United Nations
VFCW	Vertical Down-Flow Constructed Wetland
WES	Whole Effluent Sampler
WUL	Water Use Licence
WWTW	Wastewater Treatment Works

UNITS OF MEASUREMENT

Metre (m)	distance or height
Square metre (m^2)	area
Grams per square metre per day ($g\ m^{-2}\ d^{-1}$)	loading rate
Litre (L)	volumes < 1000 L
Cubic metre (m^3)	volumes ≥ 1000 L
Milligrams per litre [$mg\ L^{-1}$]	concentration
Meter per day ($m\ d^{-1}$)	hydraulic loading rate
Millimetre per day ($mm\ d^{-1}$)	daily rainfall
Cubic metre per day ($m^3\ d^{-1}$)	daily flow

CHAPTER 1. INTRODUCTION

"Sanitation is more important than political freedom"

Mahatma Gandhi (1869-1948)

Sanitation is not a privilege but a human right. Yet, in 2022 not all persons have gained access to safely managed sanitation services.

1.1 CURRENT GLOBAL SANITATION STATISTICS

The Joint Monitoring Programme (JMP) is mandated to provide global indicators of the progress in achieving the United Nations (UN) Sustainable Development Goals (SDGs). Goal 6.2.1a specifies the "*Proportion of population using safely managed sanitation services*" (WHO and UNICEF, 2021). According to the JMP, safely managed sanitation services are defined as the "*use of improved facilities that are not shared with other households and where excreta are safely disposed of in situ or removed and treated off-site*" (WHO and UNICEF, 2021). These improved sanitation facilities include (1) a flushing toilet (traditional/pour flush) that is connected to a sewer, septic tank, or pit latrines; (2) ventilated/non-ventilated pit latrines with slabs; and (3) composting toilets (WHO and UNICEF, 2018; 2021). Between 2015-2020, the overall proportion of the global population who had access to safely managed sanitation services increased by seven percentage points to 54% (WHO and UNICEF, 2021). The increase is mainly attributed to the adoption of on-site or non-sewered sanitation facilities, which has sparked a new generation of toilets and treatment systems that do not need to be connected to a centralised sewer system (WHO and UNICEF, 2021).

However, efforts to reduce the global population without access to safely managed sanitation services are insufficient (WHO and UNICEF, 2021). In the latest SDG Goal 6 progress report, 3.6 billion people still lack access to safely managed sanitation services. Furthermore, 494 million practice open defaecation, 92% of which reside in rural areas (WHO and UNICEF, 2021). We are in a crisis, given the fact that in addition to this, 44% of domestic wastewater is left untreated (WHO and UNICEF, 2021). Based on the 2020 statistics, the proportion of the global population who will have access to safely managed sanitation services is projected to reach 67% by 2030 – nowhere near the target of universal access.

1.2 PROGRESS IN SUB-SAHARAN AFRICA

The proportion of the population in sub-Saharan Africa who have access to at least basic sanitation services (i.e., an improved sanitation facility not shared by other households but the excreta is not treated) is 33%, increasing by 10 percentage points between 2000 and 2020 (WHO and UNICEF, 2021). However, during this period, the population growth was 73%. Data from 2015 to 2020 indicate an increase of two percentage points in the proportion of the population who have access to safely managed sanitation. At present, this is estimated to reach 24% by 2030, which is the lowest progress compared to the rest of the world. In addition, 68% of the overall population still lacked access to at least basic sanitation services in 2020, 18% of which practiced open defecation (WHO and UNICEF, 2021). Rural populations form the majority who lack any sort of sanitation facility. Moreover, only 8% of the wastewater generated in this region is treated, but this statistic was based on data from only five countries in this region (WHO and UNICEF, 2021). The impact of unsafe and poor sanitation services directly leads to polluted environments, poor public health through the spread of infectious diseases, and thus, economic losses due to increased health costs (WHO, 2017; DWS, 2018).

1.3 SANITATION IN SOUTH AFRICA

According to STATS SA (2020), improved sanitation in South Africa is defined as either (1) a flush toilet, connected to a sewer or septic tank, or (2) a ventilated pit latrine. South Africa is on track to ending open defecation, with less than 1% of the population excreting in the open environment instead of in a toilet facility in 2020. Furthermore, in 2020, 83.2% of the population had access to improved sanitation, increasing by 22.2 percentage points over 18 years (STATS SA, 2020).

Surprisingly, there is no data on the proportion of the population who have access to safely managed sanitation services (WHO and UNICEF, 2021). The National Water and Sanitation Master Plan state that 56% of the approximately 1150 centralised municipal wastewater treatment works (WWTWs) are characterised as being in "*poor or critical condition and in need of urgent rehabilitation and skilled operators*" (DWS, 2018). Moreover, most of these centralised WWTWs are operating above their hydraulic capacity (Vosloo et al., 2019).

The Green Drop Certification is an incentive-driven programme initiated by the Department of Water and Sanitation (DWS). It is aimed at evaluating all WWTWs in South Africa to inform regulations and aid Water Services Institutions in identifying areas of improvement to ensure the proper functionality of the infrastructure. In the latest report, only 23 out of the 995 WWTWs evaluated achieved green drop status (i.e., a score of ≥ 90% based on various performance indicators, one of them being effluent quality). For effluent quality compliance, all microbiological, chemical, and physical parameters must be compliant 90% of the time (i.e., 90% of the total sample set must be ≤ the General Authorisation (GA) limit for that particular parameter) (DWS, 2022). Municipal plants with high-risk status constitute 70.1%. The most apparent risks observed were at the treatment level. Some older WWTWs operate above their hydraulic design capacity due to population growth, urbanisation, and economic growth. Other factors were effluent non-compliance and improper monitoring (DWS, 2022). In other cases where infrastructure was upgraded, these systems were not yet commissioned or operational. Furthermore, in some systems where infrastructure is being upgraded, raw wastewater is allowed to divert directly into a water resource (DWS, 2022). Untreated wastewater entering a water resource (river or natural wetland) has negative consequences in a country already impacted by water insecurity and poor management of its water supply. Already, 50% of natural wetlands have been lost (potentially due to degradation), and 33% of the remaining ones are noted to be in poor ecological condition (DWS, 2018). Another challenge is electrical cable theft, resulting in WWTWs facing a complete shut down for long periods (DWS, 2022). Therefore, interventions are required to manage and treat wastewater, particularly domestic wastewater.

Adding to the burden is the remaining 17% of the population, the majority of whom reside in informal settlements, and do not have access to improved sanitation. This presents an even greater challenge as informal settlements are becoming more numerous and increasingly dense within the urban and peri-urban edge (eThekwini Municipality, 2021). Informal settlements in cities echo the legacies of apartheid. The Group Areas Act (Act No. 41 of 1950) enforced segregation amongst the different racial groups by geographical location which impacted service delivery. During the apartheid regime, access to basic services (such as clean water, safe sanitation, electricity, and refuse removal) was minimal for non-whites. Thus, poverty amongst these population groups increased, while negatively impacting their social well-being (Sutherland et al., 2014).

Despite the democratic Government aiming to redress inequalities and undo the social injustices brought on by the apartheid by 2030 (National Planning Commission, 2011), service delivery remains unequal (eThekwini Municipality, 2021). Thus, informal settlements propagate as a direct outcome of the influx of migrants from their rural homesteads into the cities, in search of employment and better livelihoods, in conjunction with population growth. Municipalities are therefore unable to cope with the increased service demand with the limited funding sources available, creating multiple service delivery backlogs and poor living conditions in these communities (STATS SA, 2020; eThekwini Municipality, 2021).

In eThekwini, one of South Africa's largest metropolitan areas serving over 3.4 million citizens, sanitation services are divided according to three service levels (eThekwini Municipality, 2015; 2021):

1. Waterborne sanitation (via flushing toilets) is connected to the sewer where the landscape permits sewer networks and reticulation systems.
2. Shared facilities in the form of community ablution blocks (CABs), as a temporary sanitation provision for informal settlements within the waterborne edge, that are connected to existing sewer lines, or pits for non-sewered areas.
3. Dry sanitation (in the form of urine diversion dehydrating toilets) for rural communities, where extending sewer networks is impractical, due to the exorbitant costs associated with it (Gounden et al., 2006).

The National Sanitation Policy (DWS, 2016) sought to address the sanitation challenges and gaps in service delivery in South Africa, and make recommendations based on sustainability, rather than historical planning. Such planning involved waterborne sanitation with centralised WWTWs for urban populations and dry sanitation for rural areas (Eales, 2010). Leading a transformative approach, the policy dictates appropriate technology choices based on resource availability (such as water availability for flushing systems) (DWS, 2016).

South Africa is a drought-prone country, with extreme weather events aggravated by climate change (DWS, 2018). Therefore, waterborne sanitation for urban areas, already faced with exponential population growth, is unsustainable. Traditional water-intensive toilets, in primarily urban households, use approximately 6-10 L of potable water for flushing (DWS, 2017). Considering that the average daily water consumption per capita was 235 L in 2014 (Hedden and Cilliers, 2014), increasing slightly to 237 L in 2021 (Ngobeni and Breitenbach, 2021), it is not practical to continue with waterborne sanitation. Moreover, South Africa's daily water use per capita is much higher than the global average of 173 L per capita (Hedden and Cilliers, 2014; Ngobeni and Breitenbach, 2021), alluding to the wasteful use of potable water.

Yet, the aspiration for flushing toilets remains high amongst the indigent populations residing within these informal settlements. This is compounded by the fact that dry sanitation is not user acceptable, often referred to as the poor man's solution, and complaints of clogging and bad odours are widespread (Roma et al., 2013). Therefore, in line with redressing service delivery inequalities, municipalities are sometimes pressured to provide flushing toilets for informal settlement upgrades. However, even if fitted with water-efficient toilets (such as low flush technologies), connections to the centralised sewer network may not be possible due to cost of added sewer reticulation systems and/or ageing, and possibly degraded, existing infrastructure, being unable to accommodate the increased loading. Jung et al. (2018) found that sewer reticulation systems in centralised WWTWs can equate to 84% of the total capital costs. Moreover, according to the 2022 Green Drop report, the eThekwini Municipality's WWTWs achieved a score of 76%, reducing by 14 percentage points compared to 2013 (DWS, 2022). The declining state of WWTWs further supports the application of more appropriate and sustainable sanitation solutions.

Delays with sanitation provisions directly impact housing delivery, especially for informal settlements. Previously, housing developments were dependent on the availability of land. A new directive favours *in situ* formal housing upgrades of informal settlements instead of identifying new plots and moving communities (eThekwini Municipality, 2021). In 2019, funding from the Department of Human Settlements has permitted 560 informal communities within the eThekwini Municipality to be upgraded[*]. However, until appropriate sanitation measures are put in place (from the user interface to treatment and then safe disposal/discharge or reuse), these housing upgrades are delayed, leading to civil unrest. Thus, the Municipality is forced to look at alternative, innovative, and affordable sanitation options for waterborne sanitation in the South African context.

More recently, the catastrophic flooding events in eThekwini during April and May 2022 resulted in bulk sanitation infrastructure being severely damaged[†], subsequently leading to raw wastewater flowing directly to water resources. Despite the consequences of contaminated water and the spread of infectious diseases, many informal settlements are situated where land is available, generally near water resources such as along riverbanks and flood plains (Williams et al., 2018). The risks associated with these communities occupying these areas are access to the contaminated water and riverbanks collapsing during extreme flooding events. This places

[**] https://www.iol.co.za/mercury/news/city-reveals-plan-to-upgrade-informal-settlements-35513239 (Date accessed 17 November 2019)

[†] https://issuu.com/glen.t/docs/wasa_may_june_2022 (Date accessed 13 June 2022)

these communities at greater risk with higher vulnerabilities. Other services that were disrupted were bulk water and electricity supply. Therefore, the eThekwini Municipality sees the need to look at building resilient sanitation systems for their informal settlement upgrades. Such resilient systems would consist of smaller-scale systems operating without electricity or chemicals, and would potentially have the following advantages:

1. Will alleviate the burden on current centralised WWTWs.
2. Will reduce the need for large sewer reticulation networks.
3. Will allow Municipalities to track their wastewater flows.
4. Will build resilience against a changing climate.
5. Will reduce the capital, operation, and maintenance costs associated with supplying sanitation services.

1.4 THE DECENTRALISED APPROACH TO SANITATION

The use of decentralised sanitation technologies has gained traction over the last few decades. Unlike centralised WWTWs with advanced treatment technologies which require high energy and chemical inputs, decentralised wastewater treatment systems (DEWATS) use biological and physical processes via simple engineered technologies in a modular design. The gravitational flow to and within these systems allow them to operate without pump stations and extensive sewer reticulation systems, making them a lower-cost option where waterborne sanitation is needed, especially in fast-growing cities. Moreover, maintenance requirements are minimal, avoiding the need for advanced skilled operators. These benefits are well known and have been reported by others (Singh et al., 2009; Kerstens et al., 2012; Reynaud, 2014). In 16 developing countries globally, DEWATS have been designed and implemented by the non-governmental German organisation, Bremen Overseas Research and Development Association (BORDA) (Sasse, 1998; Gutterer et al., 2009), where the capital costs of building new or upgrading centralised WWTWs were too high to be deemed practical. However, in Africa, of the 228 BORDA-designed DEWATS installed and in operation, only one is in South Africa (BORDA, 2017), despite the National Sanitation Policy encouraging the use of DEWATS (DWS, 2016). The slow adoption of DEWATS is a challenge in South Africa.

The demonstration-scale[‡] DEWATS was constructed in partnership between BORDA and the eThekwini Municipality. The purpose of this DEWATS was to test the feasibility of decentralised sanitation for communities in eThekwini (on the east coast of South Africa) where the undulating landscape provides the head required for gravitational flow to and through the DEWATS. In the South African context, DEWATS must not be characterised as package plants. Package plants are prefabricated and then installed on-site, requiring electricity for flow (to power pumps) and chemicals for disinfection (Niekerk et al., 2009). Moreover, DEWATS is not classified as a non-sewered sanitation system as defined by SANS 30500 (SANS, 2019), since a sewer network is still required to convey the raw wastewater to the DEWATS, although not as extensive as a centralised WWTP.

1.5 PROBLEM STATEMENT

The eThekwini Municipality sees the benefits of applying BORDA-designed modular DEWATS as a sanitation solution for *in situ* informal settlement upgrades within the city. However, it is unclear if the current design of the demonstration-scale DEWATS (settler, anaerobic baffled reactor, anaerobic filter, vertical down-flow constructed wetland, horizontal flow constructed wetland) will achieve fully compliant effluent within the Revised GA limits for safe discharge to a water resource (DWS, 2013). Even if the current design does achieve safe discharge quality effluent, space limitation in informal settlements demand the entire DEWATS design configuration to require less surface area. Arumugam and Buckley (2020) implemented minor design modifications to the tertiary treatment modules of the demonstration-scale DEWATS (the hybrid subsurface flow constructed wetland system) to augment effluent treatment efficiency.

The treatment processes in the primary and secondary modules in the demonstration-scale DEWATS produce ammonium-N rich effluent that is above the GA limit for safe discharge to a water resource (Pillay et al., 2013). Intermittently loaded vertical down-flow constructed wetlands (VFCWs) have a lower surface area demand and have higher oxygen transfer capacity for biological nitrification (oxidation of ammonium-N to nitrate-N) compared to other CWs (Kadlec and Wallace, 2009; Stefanakis et al., 2014; Vymazal, 2022). Moreover, it aligns with

[‡] Demonstration-scale refers to a system tested at full-scale capacity to test operational stability and identify system errors under field conditions.

the simple design principles of DEWATS by not requiring electricity for operation or chemicals for treatment. However, there is a stipulated GA limit for oxidised nitrogen species (such as nitrate-N) (DWS, 2013). Reduction of nitrate-N (denitrification) occurs in very low DO concentrations and in the presence of endogenous or exogenous carbon (Kadlec and Knight, 1996; Kadlec and Wallace, 2009). In VFCWs, aerobic conditions exist due to the diffusion of oxygen via the open pore spaces as the wastewater permeates through the filter media (Kadlec and Wallace, 2009) which may limit the removal of nitrate-N (NO_3-N).

1.6 RESEARCH QUESTION

Therefore, there is an urgent need to investigate different design configurations of intermittently loaded VFCWs in the South African context to formulate a DEWATS design for informal settlement upgrades where the final effluent quality meets the regulatory requirements for safe and continuous discharge to a water resource. Hence, this research in this thesis is based on the following research question:

What configuration of vertical down-flow constructed wetlands can be applied in the design of decentralised wastewater treatment systems that would result in safe discharge quality effluent as per the South African General Authorisations for treated domestic effluent?

1.7 AIM AND OBJECTIVES

The aim of this study was to evaluate the performance of vertical down-flow constructed wetlands (VFCWs) as a tertiary treatment module in the DEWATS receiving anaerobically treated domestic wastewater, in different design configurations. These configurations formulated four DEWATS designs consisting of a settler, anaerobic baffled reactor, anaerobic filter and:

1. A single-stage VFCW (design 1).
2. A hybrid configuration comprising a single-stage VFCW with subsequent treatment in a horizontal flow CW (HFCW) (design 2).
3. Single-stage VFCWs with extended filter depths (design 3).
4. Two-stage VFCWs (design 4).

The specific research objectives were to:

1. Monitor the overall performance of the demonstration-scale VFCW against its hybrid configuration with the HFCW under the continuous operation of the DEWATS and compare the effluent quality of both with the Revised GA limits for safe discharge into a water resource.
2. Determine the feasibility of improving nitrate-N removal by the addition of dried plant material of *Arundo donax* L. as a plant-based carbon source in the CW.
3. Design and construct pilot-scale[§] VFCWs with different configurations (single and two-stage) and operation configurations (specifically; media gradation, depth, and hydraulic loading rate) to determine the total nitrogen removal capacity of each.

1.8 SCOPE AND LIMITATIONS

The demonstration-scale DEWATS in eThekwini is a field application of a sanitation system, tested under real-life conditions. The climate in eThekwini is warm sub-tropical, with average minimum and maximum temperatures at 7.9°C and 30.5°C, respectively. However, the BORDA design guidelines are based on European best practices (Sasse, 1998; Gutterer et al., 2009), and not on the influent wastewater characterisation of South African communities. Moreover, due to the diurnal flow rate to the DEWATS, the organic loading was uncontrolled.

An attempt was made in this study to measure flow and concentration data of the raw wastewater entering the demonstration-scale DEWATS, using a whole effluent sampler (WES), designed by the WASH R&D Centre at the University of KwaZulu-Natal. The function of the WES is to operate as a flow proportional composite sampler (Appendix A). However, commissioning was affected by the high scum build-up in the diversion manhole which disrupted operation of the macerating pump inside the WES, inhibiting full operation of the system (refer to Appendix A). The high scum was a result of an absence of an inlet screen or grit chamber upstream of the DEWATS. Although screens and grit chambers are standard design for treatment systems, the DEWATS was originally designed without a screen at the inlet to reduce the daily maintenance needs of the system (Pillay et al., 2013). Therefore, characterisation of the raw wastewater was not measured for this community.

[§] Pilot-scale refers to a new design tested, in this case, under field conditions, but at a relatively smaller capacity than demonstration-scale and full-scale systems.

In terms of the DEWATS design for informal settlement upgrades within the eThekwini Municipality, the tertiary treatment modules need to be simple and practical, use readily available media and vegetation types, operate without any electrical or chemical demands, and require basic skills for operation and maintenance personnel (potentially even members of the community being served, if adequate training is facilitated). This will then provide Municipalities, including those with low revenue, and especially those within the province of KwaZulu-Natal, where the landscape is generally undulating, with appropriate and simple sanitation solutions.

In addition, it is presumed that all households (HH) that are connected to the demonstration-scale DEWATS in eThekwini have water-intensive toilets (> 6 L of water for a single flush). Combined with the average daily water usage per capita in South Africa (Hedden and Cilliers, 2014), the DEWATS may receive diluted wastewater compared to an informal settlement. Crous et al. (2013) undertook a water demand study of four community ablution blocks (CABs) at the Frasers Informal settlement, north of eThekwini. It was found that the average water demand was 82 L HH^{-1} d^{-1} at a household capacity of 2.2 persons per HH. The demonstration-scale DEWATS was designed based on the assumption of 5 persons per HH. Moreover, if the upgraded HH in the informal settlement is provided with low flush pedestals, then it is unclear how this will impact the transport and conveyance of the raw wastewater to the DEWATS, and through the treatment modules due to insufficient wastewater, since the flow is gravitated. Moreover, with less water in the raw wastewater, the concentration of pollutants may be higher.

The analysis of the effluent quality was limited to only the microbiological (*Escherichia coli*) and the minimum chemical parameters for discharge up to 2000 m^3 d^{-1} as listed in Table 2.2 of the Government Gazette No. 665 (DWS, 2013). As such, this thesis does not include an assessment of heavy metals or free chlorine removal since the raw wastewater constitutes only domestic wastewater, and disinfection via chlorination is not included in the treatment process. Moreover, micropollutant (MP) removal, including pharmaceuticals, or the microbial community profiles in the CWs was beyond the scope of this thesis.

Lastly, due to the COVID-19 Lockdown restrictions, most of the performance evaluations presented in this thesis had short time frames. Moreover, during this time, the Respirometer, used to determine the biodegradable fraction of the total chemical oxygen demand, had malfunctioned and could not be repaired timeously during the Lockdown restrictions. Therefore, during the field application of dried plant material as a plant-based carbon source to augment denitrification in the demonstration-scale CW, the biodegradable fraction was not determined.

1.9 TIMELINE OF RESEARCH ACTIVITIES

An overview of all research activities and civil works undertaken in this thesis as well as that documented by Arumugam and Buckley (2020) are illustrated in Figure 1-1.

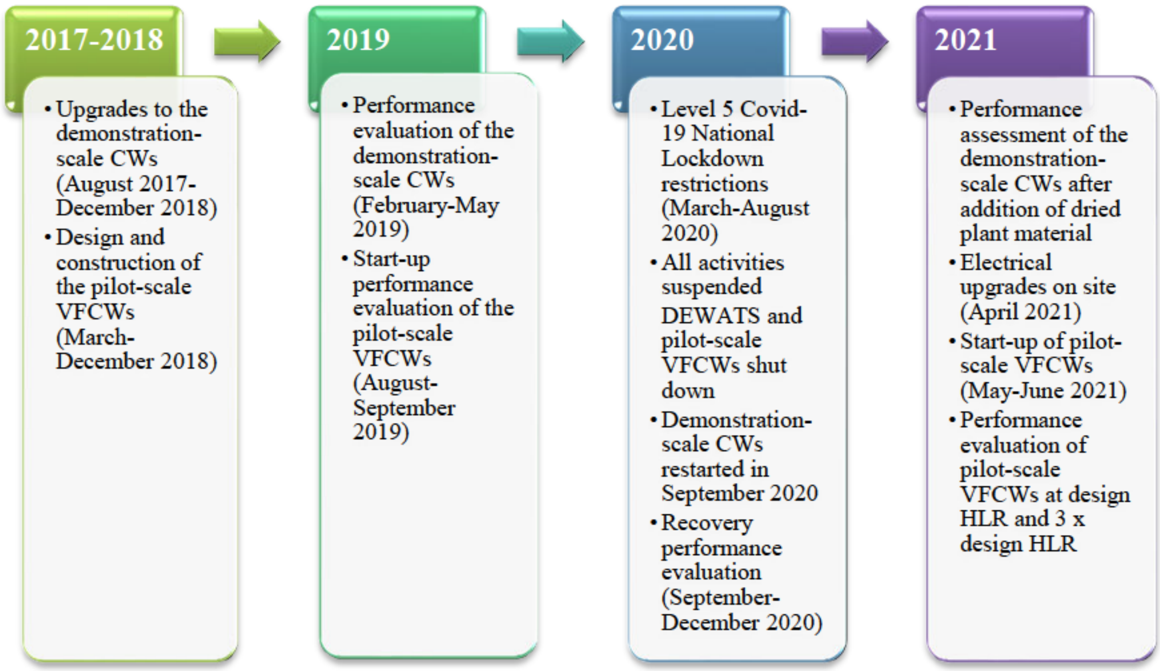


Figure 1-1: Timeline of research activities

1.10 RESEARCH APPROACH

Due to the need for field testing of sanitation technologies to determine appropriateness, technology readiness and speed up adoption, this research was based at the Newlands Mashu Research site which houses the demonstration-scale DEWATS. In so doing, the pilot-scale VFCWs could also be fed with anaerobically treated effluent like the demonstration-scale CWs but, potentially not of the same quality due to the flow distribution within the primary and secondary treatment modules.

After the design modifications to the demonstration-scale CWs implemented and documented by Arumugam and Buckley (2020) and the design, construction, instrumentation of the pilot-scale VFCWs, the overall study was divided into four research phases.

The first phase was the comparative performance evaluation of design 1 and design 2 under continuous operation. During Phase 1, the following hypothesis was tested: *The effluent quality of the demonstration-scale VFCW (design 1) will achieve fully compliant effluent for safe discharge into a water resource.*

In Phase 2, the start-up performance of the pilot-scale VFCWs operating under continuous operation at design flow was monitored (i.e., no variability in the hydraulic loading per day although the organic loading was uncontrolled).

After interruptions in operation, the demonstration-scale CWs were evaluated to determine recovery time after long-term (approximately 5 months = 162 days) shut-down. These results were documented with the evaluation of the demonstration-scale CWs in Phase 1.

In Phase 3, after resumption of continuous operation, dried plant material of the invasive Giant reed was used to augment carbon availability for denitrification in the demonstration-scale CWs. The application of the plant-based carbon source was limited to the HFCW in order to prevent any disturbances to the media in both CWs. During this phase, the following hypothesis was tested: *The addition of dried plant material at the inlet of the demonstration-scale HFCW will meet the COD:N demand for denitrification, resulting in the hybrid CW system (design 2) achieving fully compliant effluent within the GA limits for safe discharge into a water resource.*

The last phase of the research focussed on the operation and performance of the pilot-scale VFCWs. The remaining effluent after secondary treatment was used as the feed for the pilot-scale VFCWs so as to not interfere with the operation of the demonstration-scale CWs. Two hypotheses were tested during this phase: (1) *The single-stage VFCW with an extended filter depth (design 3) will achieve the same, if not better effluent quality than a two-stage VFCW (design 4)* and, (2) *Increasing the hydraulic loading rate will increase carbon availability from the incoming wastewater and thus, improve nitrate-N removal across the pilot-scale VFCWs (single-stage and two-stage systems).*

1.11 STRUCTURE OF THE THESIS

This thesis is divided into seven chapters.

Chapter 1 provides the background information, problem statement, research aims and objectives, scope and limitations of the study, and the timeline of all research activities.

In *Chapter 2*, a general literature review is presented on the classification of CWs, the design and operational parameters affecting CW treatment efficiency and a brief description of all pollutant transformation processes occurring within CWs. The review presents the General Authorisations (GA) for safe discharge of treated domestic effluent into a water resource and the minimum monitoring requirements for effluent quality assessment. It also details the application of CWs in South Africa, including the hybrid CW system in the demonstration-scale DEWATS, and lists all the design upgrades to the CWs. Lastly, the chapter highlights the knowledge gaps in the application of CWs in the DEWATS design for informal settlement upgrades.

Chapters 3-5 are formatted as separate studies (Figure 1-2) with each chapter focussing on one research objective as outlined in Section 1.6 and tests specific hypotheses related to the research objective of that study.

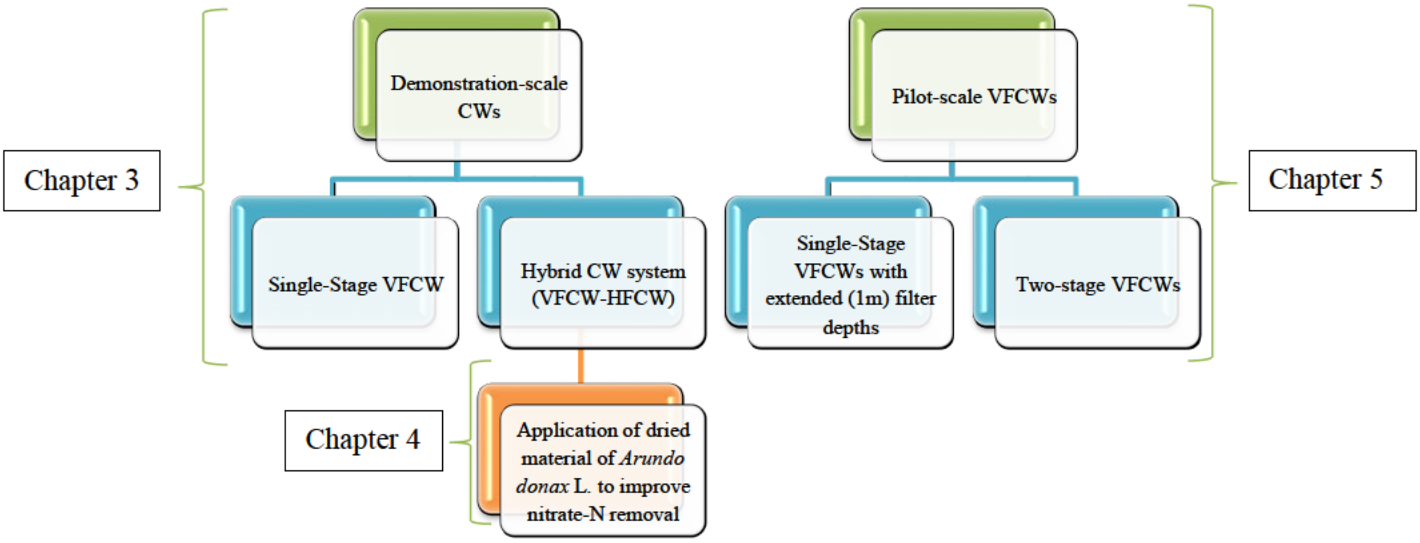


Figure 1-2: Overview of research studies

In *Chapter 3*, the performance of the demonstration-scale VFCW (design 1) was compared to its hybrid configuration with an HFCW (design 2) under continuous operation following design upgrades to improve performance. The performance indicators were determined by evaluating the final effluent from each CW against the GA limits for safe discharge into a water resource. Thereafter, the recovery performance of the better performing design was assessed after long-term shut-down (162 days). *Chapter 4* focussed on the field application of dried plant material of the invasive Giant reed, *Arundo donax* L., as a plant-based carbon source to improve the nitrate-N removal capacity in the hybrid CW system. *Chapter 5* concentrated on the design of the pilot-scale VFCWs, where the impact of extended filter depths and high hydraulic loadings were evaluated in single- and two-stage VFCWs. While emphasis was placed on the total nitrogen removal capacity across each configuration, the effluent from each CW was compared to the GA limits.

The composite discussion of the thesis research findings including the recommended DEWATS design for *in situ* informal settlement upgrades to formal housing, is presented in *Chapter 6*.

Lastly, the conclusions and recommendations for future research are given in *Chapter 7*.

CHAPTER 2. LITERATURE REVIEW

2.1 OVERVIEW OF CONSTRUCTED WETLANDS

Since their first experimental application in the 1950s (Seidel, 1961), constructed wetlands (CWs), have been recognised as efficient, cost-effective, and environmentally sustainable technologies for the treatment of various wastewaters. Some authors refer to them as *treatment wetlands* (Kadlec and Wallace, 2009; Fonder and Headley, 2010; Dotro et al., 2017; Langergraber et al., 2019), as they are designed to mimic wetlands by promoting natural processes facilitated by vegetation, sediment, and microbial assemblages, in a controlled manner (Lee et al., 2009; Vymazal, 2010). With the addition of macrophytes that are tolerant to nutrient and organic overload, CWs are successful in removing organic matter, suspended solids, nutrients, and pathogens by various physical, chemical, and biological processes and, to a lesser extent, the uptake of nutrients by plants (Kadlec and Wallace, 2009). Their ability to buffer hydraulic and organic load fluctuations demonstrate the resilience and reliability of these systems for effective treatment of various wastewaters (Morvannou et al., 2014). Moreover, the presence of plants provides an aesthetic value to CWs (Gutterer et al., 2009; Chang et al., 2012). The benefits are a substantial reduction in capital, operation, and maintenance costs (Potter and Karanthanasis, 2001; Carty et al., 2008; Rai et al., 2013) as compared to centralised wastewater treatment works (WWTWs), which have advanced technologies requiring energy and chemical input. Based on these traits, Dotro et al. (2017) recommend CWs as small decentralised treatment systems.

Vymazal et al. (2021) noted that between 2019 and June 2020, nearly 700 research papers were published on constructed wetlands, highlighting the widespread interest in the added benefits of this technology. Accordingly, there have been extensive reviews on the application of the various types of CWs globally (Vymazal, 2008; Vymazal and Kröpfelová, 2008; Vymazal, 2010; 2011; Vymazal, 2014; Dotro et al., 2017; Vymazal, 2022) and the recent design advancements and practical use treating various wastewaters (Langergraber et al., 2019).

In this review, the general classification of the major types of CWs for anaerobically treated domestic wastewater are briefly discussed which operate via gravitated flow, require only passive aeration (if necessary, via pipes) (i.e., no electricity required), and no chemicals for treatment. The advantages and limitations of each CW are presented including an overview of the design and operational parameters affecting performance and treatment efficiency. In addition, a brief description of the major pollutant transformation processes occurring within CWs is presented. Moreover, the South African regulatory requirements for safe discharge of treated domestic effluent into a water resource are reviewed. Concerning the general application of CWs for domestic wastewater treatment in South Africa, emphasis is placed on field applications, particularly CW applications in decentralised wastewater treatment systems (DEWATS). Lastly, the current gaps are identified with respect to integrating CWs with an appropriate design to achieve fully compliant effluent for safe discharge in South Africa.

2.2 CLASSIFICATION OF CONSTRUCTED WETLANDS

Fonder and Headley (2010) present an in-depth classification of CWs based on hydrology, vegetation, and flow regime, which is summarised in Figure 2-1. Ideally, based on the position of the water, CWs can be divided into two major groups: surface and subsurface flow (Fonder and Headley, 2010). In this section, a brief comparison between the major groups of CWs is discussed with a summary of the main features of each CW presented in Table 2-1.

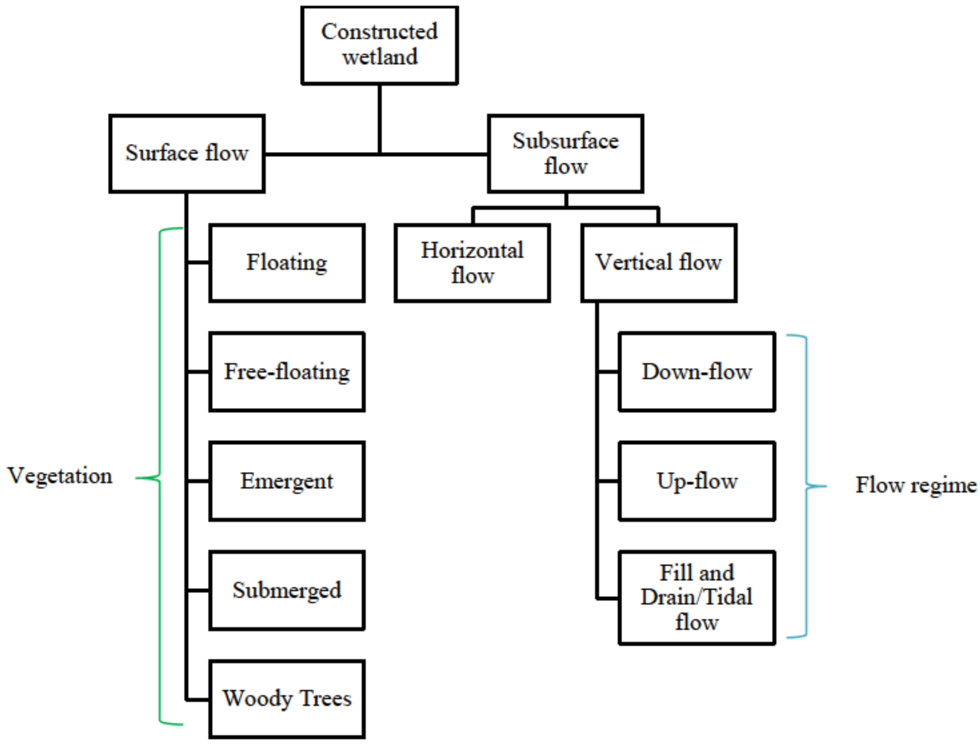


Figure 2-1: Classification of constructed wetlands (adapted from Kadlec and Wallace (2009) and Fonder and Headley (2010))

2.2.1 *Surface flow constructed wetlands*

Developed in the 1970s (Kadlec et al., 1979), surface flow CWs (SFCWs) are engineered to mimic natural wetlands and require a large surface area compared to other CWs (Vymazal, 2014). Sizing is usually dependent on influent characterisation and treatment goal (e.g. nitrogen removal) (Kadlec and Wallace, 2009). These CWs are shallow, a few centimetres up to 1 m in depth (Kadlec et al., 2000), with one or more basins lined with an impermeable layer at the bottom to prevent groundwater contamination (Stefanakis et al., 2014). The wastewater flows over the, generally, soil media from the inlet to the outlet in a more or less horizontal path and is thus, sometimes referred to as "free water surface wetlands" (Kadlec and Wallace, 2009; Vymazal, 2014). The plug-flow conditions in these systems are facilitated by the shallow water depth, the low flow velocity of the incoming wastewater, and the regulated water flow mediated by the presence of plant stalks and litter (Reed et al., 1995). Ideally, the design purpose is to augment treatment processes by allowing the existing microbial communities to contact the wastewater via reactive surfaces (Kadlec and Knight, 1996).

The different types of SFCWs are distinguished by the type of vegetative growth: macrophytes (floating, free-floating, emergent, or submerged) or woody trees (Vymazal and Kröpfelová, 2008; Fonder and Headley, 2010).

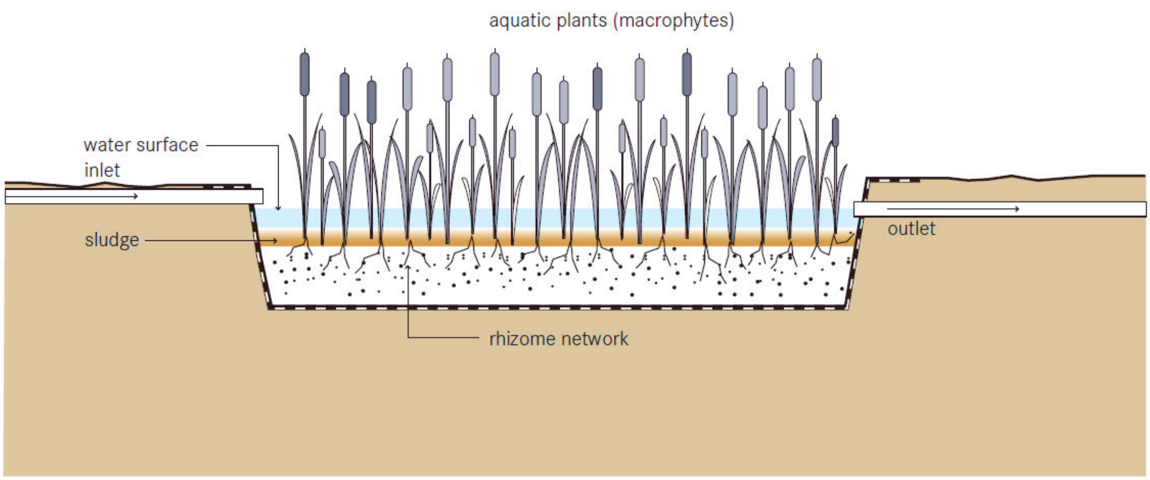


Figure 2-2 Schematic diagram of a surface flow constructed wetland (Tilley, 2014)

The open water creates aerobic zones near the surface (through the atmospheric diffusion of oxygen) while anoxic and anaerobic zones are present near the sediment. During higher loadings, the anoxic zone rises closer to the surface, while the carbon produced from the decay of plant litter provides the necessary conditions for nitrate-N (NO_3-N) removal via denitrification. At the same time, nitrification, an oxygen-dependent process, is limited by the availability of dissolved oxygen (DO) (Kadlec and Knight, 1996). Surface flow CWs are usually applied to treat or polish secondary or tertiary treated effluent (Brix, 1993; Kadlec and Wallace, 2009; Dotro et al., 2017). However, the main disadvantage of open water in SFCWs is the risk of human contact with the wastewater, pest breeding, and other disease-spreading vectors.

2.2.2 Subsurface flow constructed wetlands

Unlike SFCWs, the hydraulic flow in subsurface flow (SSF) CWs is through porous media (sand or gravel) below the surface of the bed. These types of CWs are generally planted with emergent vegetation however, the main distinction is the direction of hydraulic flow (horizontal or vertical) (Kadlec and Wallace, 2009; Fonder and Headley, 2010; Stefanakis et al., 2014; Tilley, 2014; Vymazal, 2022).

2.2.2.1 Horizontal flow constructed wetlands

Since its inception, horizontal flow CWs (HFCW) (Figure 2-3) have been widely applied in the United States of America and large parts of Europe (Vymazal et al., 2006). The main difference between HFCWs and SFCWs is that in the former, the flow regime is below the surface of the porous media (Vymazal et al., 2006; Vymazal, 2011). In terms of person equivalents (PE), the surface area demand for HFCWs are 3-10 m^2/PE (Kadlec and Wallace, 2009). More specifically for pre-treated domestic wastewater, Hoffmann et al. (2011) recommend a surface area of 3 m^2/PE and 8 m^2/PE for warm (average annual temperature > 20° C) and cold climates (average annual temperature < 10° C), respectively.

There is usually a channel at the inlet of the HFCW with large aggregate media (50-200 mm) (Langergraber et al., 2019) to ensure uniform distribution of the wastewater across the entire width of the bed (Akratos and Tsihrintzis, 2007; Vymazal, 2022). Due to the continuous flow (feeding mode) to these CWs, which is predominantly gravitated through a bottom slope of 1-3% (Kadlec and Wallace, 2009), there is a longer contact time between microbial populations and the wastewater for effective treatment of organic matter (OM), total suspended solids (TSS) and pathogens (Vymazal, 2022). Although, there are generally anaerobic zones within HFCWs promoting anaerobic OM degradation and denitrification (Kadlec and Wallace, 2009), aerobic zones do exist near the root zone as a result of leached oxygen into the matrix by the rhizomes of the planted vegetation (Vymazal and Kröpfelová, 2009), which aid in aerobic OM degradation (Vymazal, 2001).

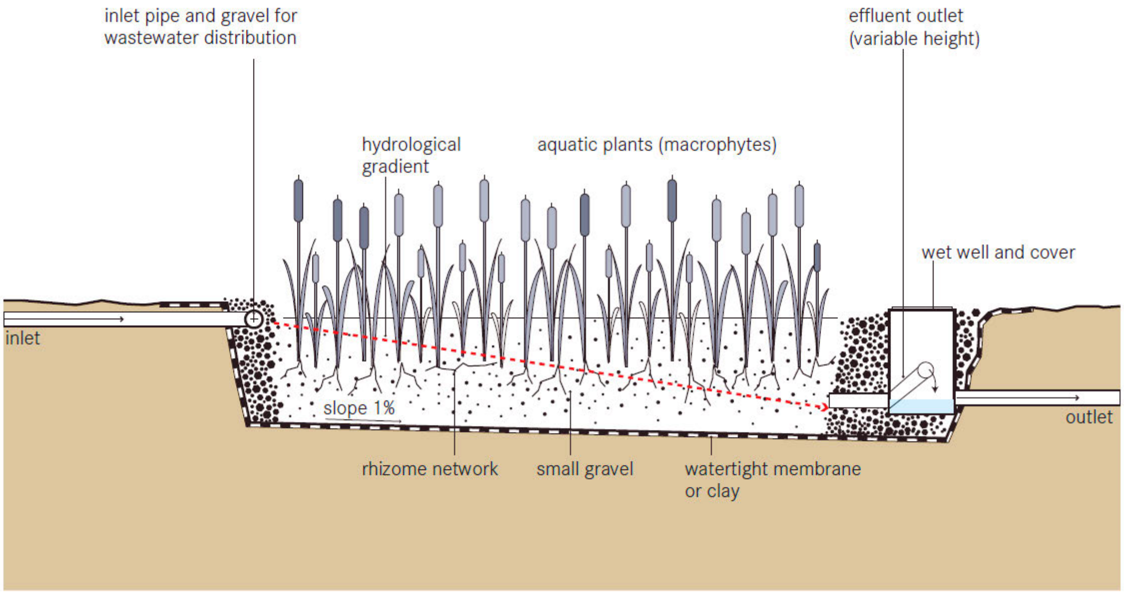


Figure 2-3 Schematic diagram of a horizontal flow constructed wetland (Tilley, 2014)

The first experimental HFCWs consisted of coarse sand media (Seidel, 1966) but with low hydraulic conductivity, these CWs became clogged (Haberl and Perfler, 1991). Gravel was found to be more suitable (Brix and Schierup, 1989), with Langergraber et al. (2019) recommending media size of 8-16 mm based on global practical experiences, and Vymazal (2022) stating the most common aggregate size as 5-20 mm. The working depth in HFCWs ranges from 0.3-0.8 m (Vymazal et al., 2006).

Control of the saturation or wastewater level inside the HFCW by adding swivelling elbows or flexible hoses to the outlet pipe, allows the designer to increase the residence time in the HFCW and improve treatment efficiency (Vymazal et al., 2006; Akratos and Tsihrintzis, 2007; Vymazal, 2022). Some authors refer to this as impounding (Langergraber et al., 2009; Panuvatvanich et al., 2009b; Langergraber et al., 2014). However, careful design must ensure that surface water is not visible and thus, the saturation level should be 0.02-0.15 m below the surface of the media (Vymazal et al., 2006). Other design modifications include step-feeding

(Sundaravadivel and Vigneswaran, 2001; Stefanakis and Tsihrintzis, 2009a; Stefanakis et al., 2011), step-feeding combined with intermittent aeration (Patil and Chakraborty, 2017), and recirculation of the effluent (Brix et al., 2003; Reese, 2005; Stefanakis and Tsihrintzis, 2009b).

2.2.2.2 Vertical flow constructed wetlands

The hydraulic flow in vertical flow CWs (VFCWs) is along vertical axes (Figure 2-4). This type of CW was initially designed as a pre-treatment for HFCWs since the latter exhibited poor oxygenation capacity, which limited nitrification (Seidel, 1965). The design of earlier applications of VFCWs typically consisted of more than one stage (i.e., 2-4 stages), however, was later compacted to a single-stage system in the late 1990s (Cooper, 1999), thus reducing its land area requirement (Vymazal and Kröpfelová, 2008).

Vertical down-flow CWs did not receive much interest in the past, possibly due to the greater operating complexity of the system for intermittent loading (Vymazal, 2010) and hence, performance data of these systems were scarce. However, in the 1990s, Europe recognized the potential for higher removal efficiencies from these systems due to their better oxygenation capacity (Cooper, 2009) compared to the favoured passive, continuously fed HFCWs. Since then, VFCWs have been widely implemented in Europe, usually designed for less than 4000 PE, especially for domestic wastewater treatment from small communities (i.e., decentralised systems) (Sani et al., 2013). Presently, these systems are chosen as a small secondary treatment option for domestic sewage (Cooper, 2009). Since 2005, a growing trend has emerged on the use of VFCWs based on the demand for higher ammonia/ammonium removal from the influent wastewaters (Brix and Arias, 2005; Vymazal, 2008), greater hydraulic loading rates, and lower area requirement compared to HFCWs. Typically, VFCWs require 1-3 m^2/PE (Cooper, 1999; Kadlec and Wallace, 2009). For pre-treated domestic wastewater, Hoffmann et al. (2011) recommend a surface area of 1.2 m^2/PE and 4 m^2/PE for warm (average annual temperature > 20° C) and cold climates (average annual temperature < 10° C), respectively.

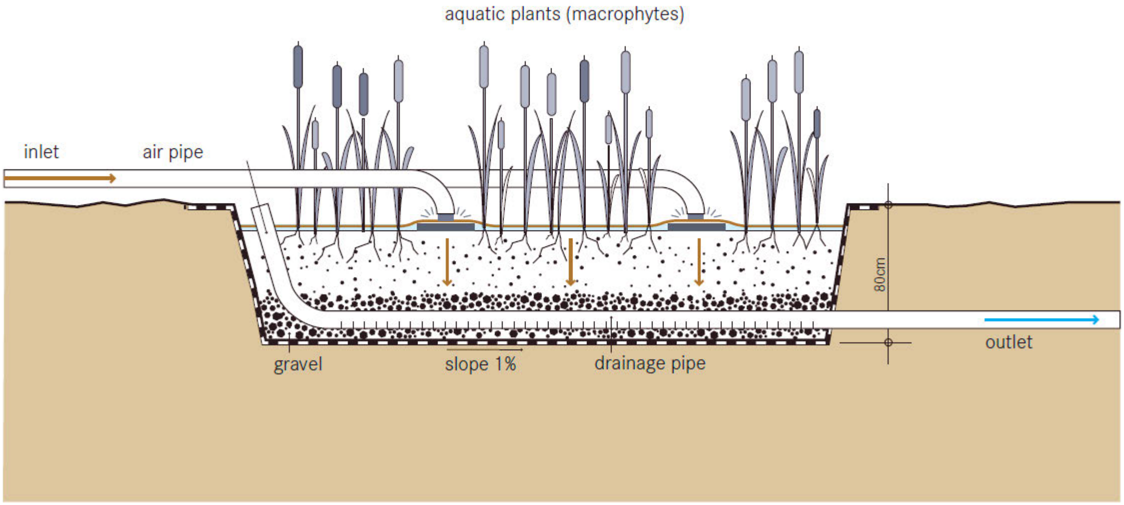


Figure 2-4: Schematic of a vertical down-flow constructed wetland (Tilley, 2014)

In the most common type of VFCW, the intermittently loaded down-flow VFCW, the sand media (0.06-4 mm or 1-4 mm) (Langergraber et al., 2019), with low hydraulic conductivity, results in temporary surface water accumulation after dosing. As the wastewater percolates through the media, atmospheric oxygen diffuses through the open pore spaces creating an aerobic matrix for OM degradation and nitrification (Cooper and Centre, 1996; Cooper, 1999; Kadlec and Wallace, 2009; Stefanakis et al., 2014). Aeration ventilation pipes extended from the surface to the drainage layer allow for passive aeration at the bottom of the bed (Sasse, 1998).

The typical depth of VFCWs is 0.45-1.2 m, although some VFCWs with sand media can be constructed with depths up to 3 m (Gutterer et al., 2009). The bottom slope of 1-2% permits gravitated free drainage however, batch and permanent impounding of the drainage have been used to augment treatment efficiency (Panuvatvanich et al., 2009a; Panuvatvanich et al., 2009b). In addition, greater or extended filter depths affects the redox potential and DO concentrations within these CWs thus, influencing the biological reactions responsible for pollutant removal. Furthermore, depth limits the type of plants to be used in a CW (Chang et al., 2015). Organic matter is degraded in the top 0.2 m of the filter depth as this layer is populated with aerobic microbial biomass due to the high DO concentrations (Stefanakis et al., 2014). In terms of nitrogen removal, CWs with extended filter depths promote anoxic conditions required for denitrification toward the bottom of the CW (Kadlec and Wallace, 2009).

Fonder and Headley (2010) classify the different types of VFCWs which are mainly differentiated by their hydrological profile (down-flow, up-flow, and fill and drain/tidal flow). Stefanakis et al. (2014) review the current types in a single-stage configuration, which are briefly summarised here:

- Vertical down-flow (usually a single-stage VFCW with intermittent loading i.e., temporary surface water accumulation after dosing, drainage then a resting period between doses) (Seidel, 1966).
- Recirculating VFCWs (a portion of the treated effluent is recirculated to the inlet of the system, improving microbial contact time with the wastewater, and thus overall treatment efficiency) (Laber et al., 1997).
- Tidal flow CWs (consisting of more than one parallel beds designed to operate in a continuous fill and drain cycle purposed for high-strength wastewaters including domestic wastewater) (Sun et al., 1999).
- Saturated vertical up-flow CWs (the wastewater enters at the bottom of the bed and thus, moves upward to improve the hydraulic retention time within the bed) (Breen, 1990; 1997).
- Saturated vertical down-flow CWs (designed with a similar objective as saturated vertical up-flow CWs however, the wastewater is applied on the surface of the bed, and the level is maintained a few centimetres below the surface to ensure that the bed remains saturated) (Visesmanee et al., 2008; Dan et al., 2011).
- Integrated VFCWs (a hybrid VFCW where the first stage is a down-flow VFCW, followed by a second stage up-flow VFCW, with a partition wall separating both stages thus, the flow is gravitated to the second stage) (Perfler et al., 1999; Chang et al., 2012; Xie et al., 2012).

There are two common designs of two-stage VFCWs. The first is the French system, designed for raw wastewater, eliminating the need for a settler, but a screen is added before the VFCWs to remove all solid matter and prevent clogging of the CW. The typical surface area requirement is 2-2.5 m^2/PE (Molle et al., 2005). The second two-stage VFCW design has been implemented at full scale in Austria (Langergraber et al., 2014) which was later adopted in

Germany (Nivala et al., 2018). In these systems, coarse sand (2-4 mm) is the filter media of the first stage VFCW, which prevents complete OM degradation, allowing for the effluent to contain available carbon. The effluent from the first stage then flows to the second stage VFCW via a mechanical dosing device, such as a siphon, or a pump. The second stage VFCW with fine to coarse sand (0.06-4 mm) in the filter media is then able to denitrify the effluent due to the lower hydraulic conductivity of the filter media, increasing the residence time within the VFCW, and available carbon from the effluent of the first stage VFCW effluent. The first stage is impounded to the level of the drainage layer (Langergraber et al., 2014). Overall, this two-stage VFCW design demonstrated higher total nitrogen removal compared to single-stage VFCWs (Langergraber et al., 2008; Langergraber et al., 2009; Langergraber et al., 2014).

2.2.2.3 Hybrid constructed wetlands

To achieve higher pollutant removal efficiencies, a hybrid CW design is used to augment the treatment efficiency of the entire CW system by using different types of CWs. The most common configuration is the single-stage VFCW (for biological nitrification) followed by a single HFCW (for subsequent denitrification) (Kadlec and Wallace, 2009). In these CWs there is usually serial design, but the CWs can also operate in parallel (Omondi and Navalia, 2020). Another type of hybrid design consists of secondary treated effluent flowing into a HFCW, which degrades OM (organic matter) and removes total suspended solids (TSS) while also providing some denitrification, followed by nitrification in a single VFCW which further removes OM and TSS. The VFCW effluent is then recirculated to the HFCW to improve the final effluent quality (Cooper, 2001; Vymazal, 2005a). However, recirculation usually requires a pump contributing to the operation costs of the CW system (Dotro et al., 2017). Many other configurations and modifications to the mentioned types of hybrid systems exist (refer to Vymazal and Kröpfelová (2008) and Stefanakis et al. (2014)).

Table 2-1: A summary of the main features of the different constructed wetlands (Kadlec and Wallace, 2009; Dotro et al., 2017; Langergraber et al., 2019)

	Surface Flow	**Subsurface flow**	
		HFCW	**VFCW**
Treatment step	Tertiary	Secondary	Secondary
Size (m^2)/ Population equivalent (PE)	Generally based on land area availability or volume; areal is preferred	3-10 m^2/PE	1.2-5 m^2/PE
Depth (m)	few cm – 1 m	0.3-0.8 m	0.45-1.2 m (up to 3 m)
Vegetation	Macrophytes • Floating • Free-floating • Emergent • Submerged Woody trees	Macrophytes • Emergent	Macrophytes • Emergent
Media	Soil	Gravel: Inlet/Outlet 50-200 mm Filter: 8-16 mm	Sand: 0.06-4 mm or 1-4 mm
Saturation level/Water depth (m)	0.1-0.6 m above the media	0.02-0.15 m below the surface of the media	Only if designed to be saturated; not applicable to down-flow intermittently fed VFCWs

2.3 DESIGN AND OPERATIONAL PARAMETERS

The purpose of this section is to present a general overview of the various design and operational parameters that direct the effective and sustainable functioning of CWs, and are therefore, critical for achieving treatment goals. Hoffmann et al. (2011) describe the main design and operational parameters in terms of sizing SSFCWs for domestic wastewater treatment as the area demand/PE, organic loading, and hydraulic loading. Wu et al. (2015) includes hydraulic retention time (HRT), media, depth, and vegetation. Considering that the flow configuration, feeding mode and depth have already been discussed in the previous section, it will not be repeated here.

2.3.1 Media

Kadlec and Wallace (2009) describe the function of the media in a CW as a substrate for vegetative and microbial growth. In addition, it facilitates the movement of wastewater along the depth of the CW (Kadlec and Wallace, 2009), thereby promoting adsorption (such as phosphorus) and filtration of certain pollutants (Saeed and Sun, 2012). Stefanakis et al. (2014) recommend that the permeability of the selected media must be adequate based on loading rates. Moreover, the media must facilitate the creation of aerobic and anaerobic zones to promote organic degradation and biological nitrification and denitrification. If possible, the selected media must also provide a carbon for denitrifying bacteria to use as an energy source by the decomposition of plant litter or accumulated OM within the media (Stefanakis et al., 2014). Careful emphasis must be placed when selecting media types so as to not reduce the operation and life span of the CW as a result of clogging (Wu et al., 2015). Clogging is common in all CWs, occurring more frequently in VFCWs due to sand media with low hydraulic conductivity (Stefanakis et al., 2014).

Nivala et al. (2013) suggested that media selection should be based on the geographical location of the CW. The most common types of media used in CWs are sand and gravel due to their availability and cost. Saeed and Sun (2012) review the different media types for organic and nitrogen removal in CWs which include zeolite, peat, compost, slag, and shale amongst others.

2.3.2 Vegetation

Vegetation refers to the macrophytic species planted in CWs. Generally, CWs with vegetation perform better than those without vegetation, due to the most reactive zone being in the rhizosphere (Stottmeister et al., 2003; Wu et al., 2015). At the root zone, oxygen is released, promoting oxygen-dependent processes such as nitrification. In addition, the uptake of nutrients, such as nitrate and phosphorus, occurs. The roots also promote an increased contact time between the wastewater and the microbial communities, by reducing the flow velocity and thus, improving overall pollutant removal (Saeed and Sun, 2012; Stefanakis et al., 2014). Despite these advantages, some authors argue that the uptake of nutrients is minimal and may leach back into the CW during senescence (Verhoeven and Meuleman, 1999; Zhang et al., 2009). However, more recently, root zone uptake is considered a major nutrient removal process during low hydraulic and organic loadings in HFCWs (Vymazal, 2020).

Vymazal (2022) reviews most of the common macrophyte species in CWs, such as *Phragmites australis* (the common reed, being widely distributed in many regions of the world, including South Africa), *Typha* species (cattails), *Scirpus* species (bulrushes) and *Cyperus* species. For practicality, the choice of vegetation is based on local availability of the species and tolerance to the design loading (Wu et al., 2015). Moreover the establishment of the species should be rapid after planting to ensure that there is equal distribution over the surface area of the CW (Scholz and Lee, 2005). Usually, more than one species is recommended for all types of CWs (Zurita et al., 2009; Zhang et al., 2010; Zhu et al., 2010; Langergraber et al., 2019).

2.3.3 Hydraulic loading rate

The treatment efficiency of CWs can be influenced by operating conditions such as hydraulic loading rate (HLR) which contributes significantly to the design, operation, pollutant removal, and land area requirements (Chang et al., 2015). The HLR is defined by Equation 2.1.

$$\boldsymbol{q = \frac{Q}{A}} \tag{2.1}$$

Where:

q is the hydraulic loading rate (m d^{-1}) to the CWs, Q is flow rate (m^3 d^{-1}) and A is the surface area of the CW (m^2) (Kadlec and Wallace, 2009).

Due to intermittent loading in VFCWs, the HLR is often described as time averaged flow rate (Kadlec and Wallace, 2009). Generally, CWs with lower HLRs have higher removal efficiencies of the major pollutants such as nutrients and suspended solids (Cui et al., 2010; Weerakoon et al., 2013; Ávila et al., 2014; Xu et al., 2014). This is because the HLR affects the pollutant mass loading rate to CWs, with VFCWs more tolerant to higher organic and nitrogen mass loadings than other CWs (Saeed and Sun, 2012; Stefanakis et al., 2014). Contrary to this, some studies have documented that CWs operating at higher HLRs demonstrated better removal efficiencies of OM and nitrogen (Stefanakis and Tsihrintzis, 2012).

2.3.4 Hydraulic retention time

The hydraulic retention time (HRT) is essentially the residence time of the volume of wastewater within a CW and is inversely proportional to the HLR. Ideally, the longer the residence time, the greater contact time between the wastewater and the microbial populations near the root zone of the CW (Ghosh and Gopal, 2010) and those attached to the media, leading to higher pollutant removal efficiencies (Kadlec and Wallace, 2009; Stefanakis et al., 2014). Lee et al. (2009) found that longer HRTs were required for nitrogen removal via denitrification compared to the HRT required for OM removal. This is due to the slow-growing nitrifying bacteria (Lee et al., 2009). Typical retention times range from 2-6 days (Wu et al., 2015) however, a longer period may reduce biological oxygen demand (BOD) removal efficiencies (Saeed and Sun, 2012). Moreover, longer HRTs may result in clogging and short-circuiting (or preferential flows) in HFCWs, that affects the hydrological pathways of the wastewater within the CW (Vymazal, 2018b).

Tracer tests are often used to measure HRTs in CWs (Kadlec and Wallace, 2009). However, it is difficult to measure the accurate HRT due to the porosity of the media within SSFCWs. Larger particles with larger pore spaces facilitate faster movement of the wastewater through the filter media. For sand and gravel media, the porosity (dimensionless) is 0.3-0.45 (Kadlec and Wallace, 2009).

Langergraber et al. (2019) provide a general overview of the main design and operational parameters affecting performance and treatment in the three main groups of CWs, which is summarised in Table 2-2.

Table 2-2: Main design and operational parameters affecting treatment in the different types of constructed wetlands (CWs) treating domestic wastewater (Langergraber et al., 2019)

CW type	**Surface flow CWs**	**Subsurface flow CWs**	
		Horizontal flow CWs (HFCWs)	**Vertical flow CWs (VFCWs)**
Design and operational parameters affecting performance	Climate	Media type	Media type
	Hydraulic retention time and hydraulic design (length to width ratio)	Distribution of wastewater	Filter depth
	Only one vegetation species – which can allow for a particular insect species to breed	Upstream treatment processes and hydraulic, organic, and total suspended solid loading rates that can cause clogging and surface water accumulation	Loading interval; volume of dose and resting periods
	Large open spaces due to inadequate vegetation cover		Hydraulic and organic loading rates
	Poor selection of vegetation species – overgrowth can alter hydraulic flow patterns		Distribution system on surface (i.e., number of holes in distribution pipes)
	Variations in water depth and disruptions in flow		

2.4 POLLUTANT TRANSFORMATION AND REMOVAL PROCESSES

The major pollutants in domestic wastewater are organic matter (OM), solids, nutrients, and pathogens (Kadlec and Wallace, 2009). This section gives a brief overview of the main removal processes of these pollutants in each type of CW (SFCW, HFCW, and VFCW) with a summary presented in Table 2-3.

2.4.1 Organic matter

Organic matter (OM) is usually broken down into the biodegradable fraction and the non-biodegradable fraction, which together make up the total chemical oxygen demand (COD). Degradation of OM occurs by microbial assemblages attached to the rhizomes of the planted vegetation and the media, in aerobic and anaerobic conditions, respectively (Vymazal, 2005a).

In HFCWs, anaerobic degradation of OM occurs at a slower rate than aerobic degradation. If DO concentrations are low then anaerobic degradation will be the dominant process for OM removal in HFCWs (Cooper and Centre, 1996). Particulate OM is generally removed by filtration and sedimentation (Dotro et al., 2017) while the soluble OM fraction is removed by aerobic and anaerobic microbial degradation (García et al., 2010). In addition, OM accumulation in CWs, either through litter decay, or during high organic loadings (Saeed and Sun, 2012), can provide an endogenous source of carbon for denitrifying bacteria to use as an energy source (Zhang et al., 2016).

2.4.2 Suspended solids

Constructed wetlands, in general, remove total suspended solids (TSS) by settling, sedimentation, filtration and adsorption (Kadlec and Wallace, 2009). In SFCWs, TSS are removed by sedimentation and filtration while in HFCWs, TSS are normally retained by settling near the inlet (Vymazal, 2005a; García et al., 2010). However, according to Vymazal (2018b) if TSS concentration is above 10 g TSS m^{-2} d^{-1} then clogging may occur in the HFCW. Other TSS removal processes in HFCWs are sedimentation and decomposition (Kadlec and Wallace, 2009). Kadlec and Wallace (2009) align TSS removal to three design and operational parameters in VFCWs, particularly the feeding mode, inlet organic and TSS loading, and media. For the intermittent down-flow VFCWs with sand media, TSS is mainly removed in the upper layer of the filter media by filtration and decomposition. The intervals between loadings can be altered to assist in the prevention of clogging (Kadlec and Wallace, 2009).

2.4.3 Nitrogen

The major nitrogen removal processes in CWs are biological nitrification followed by denitrification (Kadlec and Wallace, 2009; Lee et al., 2009; Saeed and Sun, 2012). Nitrification is a two-step microbial transformation process as indicated by Equation 2.2 and 2.3.

Nitrosomonas $$2NH_4^+ + 3O_2 \longrightarrow 2NO_2^- + 2H_2O + 4H^+ \qquad (2.2)$$

Nitrobacter $$2NO_2^- + O_2 \longrightarrow 2NO_3^- \qquad (2.3)$$

During nitritation (first process), ammonium-N (NH_4-N) is oxidized to nitrite-N (NO_2-N) using CO_2 as a carbon source mediated by the autotrophic bacteria of the genus *Nitrosomonas*.

Thereafter, in the second process (nitrification) NO_2-N is oxidised to nitrate-N (NO_3-N) by facultative *Nitrobacter* bacteria. In both reactions energy is produced, which is used by the bacteria for cell synthesis. Dissolved oxygen (DO) concentration within the CW is the rate-limiting factor affecting nitrification rates (Vymazal, 2007; Kadlec and Wallace, 2009; Sun et al., 2012). According to stoichiometry, the oxygen (O_2) demand for complete nitrification is 4.23 mg O_2 per mg NH_4-N (Liu and Wang, 2012). Other parameters affecting nitrification are pH and water temperature. Optimum nitrification rates occur at pH levels of 7.5 to 7.8 and temperatures of 25 to 35°C (Lee et al., 2009). Since nitrification is a slower process compared to OM degradation, the latter may inhibit DO availability for nitrifying bacteria (Lee et al., 2009). In addition to consuming oxygen, nitrification consumes alkalinity at approximately 7.04 g $CaCO_3$ per g N oxidised (Liu and Wang, 2012).

Denitrification is the reduction of NO_3-N to N_2 by heterotrophic bacteria in the absence of oxygen (anoxic conditions) and the presence of carbon (Verhoeven and Meuleman, 1999; Kadlec and Wallace, 2009). This process is depicted in Equation 2.4.

$$2NO_3^- \longrightarrow 2NO_2^- \longrightarrow 2NO \longrightarrow N_2O \longrightarrow N_2 \qquad (2.4)$$

Bertino (2010) found that denitrification occurs in low DO concentrations, but not above 0.3-0.5 mg O_2 L^{-1}. Above this DO concentration, denitrifying bacteria utilise oxygen instead of NO_3-N as the terminal electron acceptor in their respiration (Bertino, 2010). As NO_3-N is reduced to N_2, which is then released to the atmosphere, there is an increase in alkalinity, approximately 3 g $CaCO_3$ per g of NO_3-N reduced (Kadlec and Wallace, 2009). Denitrification rates are also affected by pH and temperature, with Saeed and Sun (2012) noting that optimum rates occur between pH levels of 7-7.5 and temperatures of 20 to 25°C. Moreover, carbon (usually from organic matter) provides the energy to drive the reaction. Kadlec and Wallace (2009) report that the carbon demand for denitrification is 3.02 g OM per NO_3-N. Carbon sources can be endogenous, from accumulated OM in the media from litter decay, and exogenous from the incoming wastewater entering the CW. However, OM degradation is a faster process and may be consumed before being available for the reduction of NO_3-N. Therefore, some authors suggest a COD:N ratio of 5:1 (Baker, 1998; Ingersoll and Baker, 1998) to ensure that there is a sufficient supply and availability of a carbon source. On the other side, vegetation may also indirectly influence denitrification rates as they also effect oxygenation near their root zones (Kadlec and Wallace, 2009). Thus, CW performance is dependent upon overcoming the conflicting dependence between organic and nitrogen removal (Kadlec and Wallace, 2009; Saeed and Sun, 2012; Stefanakis et al., 2014).

Vegetation or plant-based carbon sources have been used to augment denitrification in CWs, particularly SSFCWs (Hang et al., 2016; Fu et al., 2017). However, the risk with carbon supply from plant material is that OM is degraded faster than NO_3-N can be reduced (Li et al., 2014; Ma et al., 2020). Autotropic denitrification has also been investigated using inorganic electron donors, but this requires an electrical circuit (Ma et al., 2020).

Relatively newer compared to the classical nitrification-denitrification nitrogen removal pathway, is the anaerobic ammonium oxidation (anammox) process. During this process, autotropic anammox bacteria oxidise NH_4-N to N_2 gas by employing NO_2-N as the final electron acceptor with carbon dioxide as the carbon source (Saeed and Sun, 2012) as depicted in Equation 2.5.

$$NH_4^+ + 1.32NO_2^- + 0.066HCO_3^- + 0.13H^+ \longrightarrow 1.02N_2 + 0.066CH_2O_{0.5}N_{0.15} + 0.26NO_3^- + 2.03H_2O \tag{2.5}$$

Although, the advantages of anammox is not requiring aeration and external carbon sources are not required and energy consumption is minimal (Saeed and Sun, 2012), the growth rate of anammox bacteria is slow with a doubling time of 10.6 days (Jetten et al., 2001).

2.4.4 Phosphorus

Orthophosphate (PO_4-P) is the common form of phosphorus (P) in domestic wastewater (Vymazal, 2007). The main removal process of P is through plant uptake, and subsequent harvesting, with macrophytes taking up the majority of the required P during their growing season (Vymazal, 1995). Other main removal mechanisms are adsorption and precipitation, whereby the physicochemical properties of the media type influence the retention of P. While fine sands provide a larger surface area for adsorption, it may clog VFCWs (Vymazal, 2007; García et al., 2010). Moreover, lower retention times in VFCWs due to free drainage may limit P removal. In addition, if the adsorption sites in the filter media become saturated, then P adsorption stops (Ballantine and Tanner, 2010) and there exists the possibility of desorption. Overall, CWs do not have high removal efficiencies of P with Verhoeven and Meuleman (1999) mentioning that it remains around 50%.

Table 2-3: Summary of pollutant removal processes and removal efficiencies in constructed wetlands (García et al., 2010; Vymazal, 2010; Stefanakis et al., 2014; Dotro et al., 2017)

		Surface flow	**HFCW**	**Single-stage VFCW***
Organics	Removal processes	Aerobic and anaerobic degradation of soluble OM (by autotrophic bacteria) Microbial consumption (by heterotrophic bacteria) Settling of colloidal particles	Aerobic degradation (concentrated near root zone) Anaerobic degradation (dominant process)	Aerobic degradation near the top layer of filter media (dominant process) Chemical oxidation Settling and physical filtration of particulate OM
	Removal efficiency	> 80%	> 80%	> 90%
Suspended solids	Removal processes	Filtration (through dense vegetation) Sedimentation (by gravity)	Filtration Sedimentation	Filtration Sedimentation Adsorption Decomposition
	Removal efficiency	> 80%	> 80%	> 90%
Nitrogen	Removal processes	Nitrification Denitrification	Nitrification (limited due to low dissolved oxygen concentration) Denitrification (dominant process) Plant uptake	Nitrification (dominant process) Denitrification (limited in down-flow CWs) Plant uptake
	Removal efficiency	30-50%	30-50%	< 20%
Phosphorus	Removal processes	Precipitation (limited due to low wastewater contact time with media) Adsorption Plant uptake	Adsorption Plant uptake	Adsorption Plant uptake Filtration
	Removal efficiency	10-20%	10-20%	10-20%
Pathogens (*E. coli*)	Removal processes	UV degradation / radiation Natural die-off and predation	Sedimentation Filtration	Sedimentation Filtration
	Removal efficiency	1 $\log_{10}$	2 $\log_{10}$	2-4 $\log_{10}$

* Based on sand media (0.06-4 mm) (Dotro et al., 2017)

2.4.5 Pathogens

In CWs performance evaluations, faecal indicator organisms, such as *E. coli*, are the most common pathogens which are monitored. Vymazal (2005b) presents an overview of the *E. coli* removal mechanisms in CWs. The three main removal processes of pathogens include: physical removal (via filtration and sedimentation), cell death (by natural chemical sterilisation, oxidation, or UV radiation) and adsorption (onto OM). Predation and natural die-off represent biological removal (Vymazal, 2005b). Decamp and Warren (2000) found that higher HRT increases *E. coli* removal in HFCWs. Headley et al. (2013) found the highest $\log_{10}$ removal of *E. coli* in aerated HFCWs compared to VFCWs.

2.5 GENERAL AUTHORISATIONS FOR SAFE DISCHARGE IN SOUTH AFRICA

Prior to the new legislative requirements (pre-1994), the South African general standards for domestic wastewater treatment included limits for COD, TSS, and NH_4-N, but not NO_3-N (Batchelor and Loots, 1997). Monitoring of NO_3-N is essential, as any discharge into a water body at high concentrations, leads to eutrophication, which affects aquatic life (Kadlec and Wallace, 2009; Stefanakis et al., 2014).

In accordance with the National Water Act, 1998 (ACT No. 36 of 1998), the Department of Water and Sanitation (then the Department of Water Affairs) revised the General Authorisations (GA) for the discharge of treated domestic effluent into a water resource (DWS, 2013). The parameters required to be measured on a monthly basis by grab sampling are presented in Table 2-4, extracted directly from Government Gazette No. 36820 dated 6 September 2013. Free chlorine concentration is also a requirement for monthly monitoring for discharged domestic effluents up to 2000 m^3 L^{-1}, however considering that no chemicals are added in DEWATS, this measurement is not required (DWS, 2013). *Escherichia coli* are used in the national microbial monitoring programme for surface water contamination in South Africa (Luyt et al., 2012) and generally as an indicator of faecal coliforms.

Table 2-4: Wastewater limit values applicable to discharge of treated domestic wastewater into a water source (up to 2000 m^3 d^{-1}) (DWS, 2013)

Parameter	**General Authorisation (GA)**
COD [mg L^{-1}]	75
TSS [mg L^{-1}]	25
NH_4-N [mg L^{-1}]	6
NO_x-N (NO_2-N + NO_3-N) [mg L^{-1}]	15
PO_4-P [mg L^{-1}]	10
Faecal coliforms (*E. coli*) (CFU 100 mL^{-1})	1000
pH	5.5-9.5
Electric Conductivity (EC) (mS m^{-1})	70 mS/m above intake to a maximum of 150 mS s^{-1}

2.6 APPLICATION OF CONSTRUCTED WETLANDS FOR DOMESTIC WASTEWATER TREATMENT IN SOUTH AFRICA

The earliest research programmes of SFCW application in South Africa were implemented in the early to mid-1980s for nutrient removal from secondary treated domestic wastewater of small communities (Batchelor and Loots, 1997; Wood et al., 1999). In 1985, a single VFCW, 1.5 m in depth, was designed to remove PO_4-P in Mpophomeni, located in the province of KwaZulu-Natal (Wood, 1995). However, early engineering design was based on American and European best practice (Batchelor and Loots, 1997) and thus, many challenges arose regarding treatment efficiency (Wood et al., 1999) since the design basis of CWs usually depend of the influent characterisation and hydraulic loading (Kadlec and Wallace, 2009). Vymazal and Kröpfelová (2008) provide a general overview of CW application in South Africa. Lakay (2013) evaluated HFCWs around the Western Cape ranging from a single household to two farms, one with five households and the other with 25 households and 12 guest houses. In eThekwini, CWs at full or demonstration-scale have been limited to a single hybrid CW system integrated into a decentralised wastewater treatment system (DEWATS) for community sanitation.

2.6.1 DEWATS with constructed wetlands

Operational since 2010, the BORDA-designed DEWATS in eThekwini treats raw wastewater generated from 84 households originally defined as gap housing. Gap housing is provided for individuals who earn above the limit for state-subsidised housing, but below the limit for personal home loans (GCIS, 2017). It must be noted that this community was not chosen due to its socioeconomic status, but because the entire sewer line from these households could be diverted to the DEWATS for research purposes, of which the final effluent then flows back into the main sewer line to ensure safe disposal. Moreover, the DEWATS was constructed on land already owned by the eThekwini Municipality, thus allowing for easy access. This site was named the Newlands Mashu (NM) Research site (Figure 2-5). The raw domestic wastewater (mixture of blackwater and greywater) is primarily treated in a settler while secondary treatment is in an anaerobic baffled reactor (ABR) and anaerobic filter (AF), connected in series (Figure 2-6).

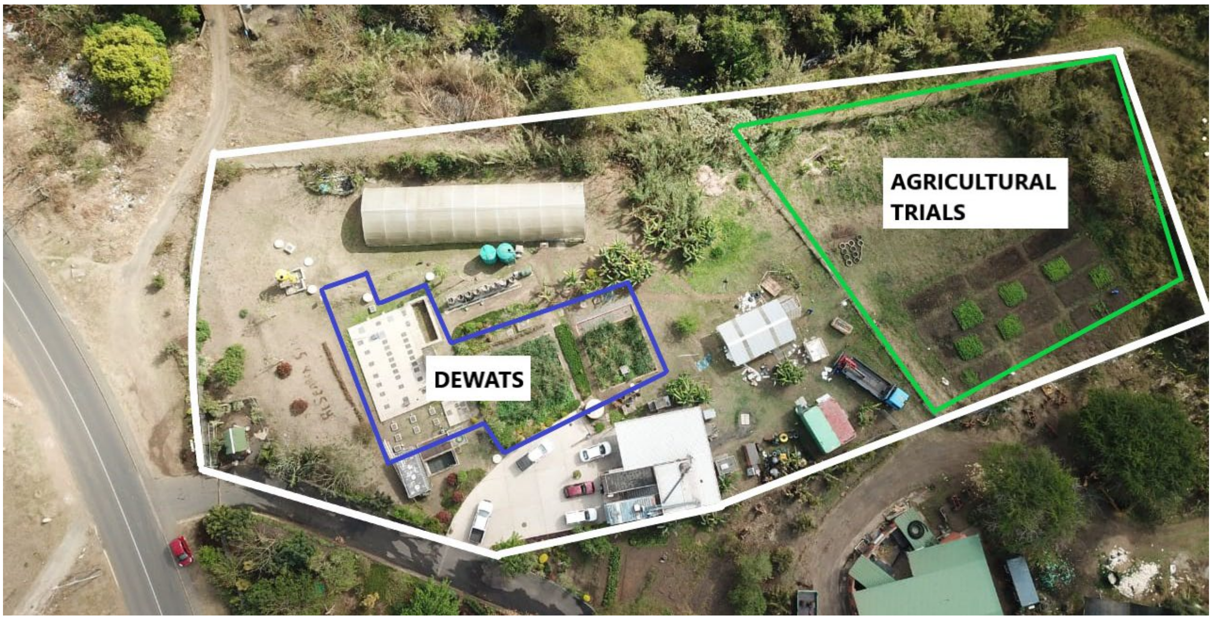


Figure 2-5: Aerial view of the Newlands Mashu Research site (taken September 2019)

The NM research site is delineated by the white boundary line and the DEWATS and agricultural field trials by the blue and green lines, respectively.

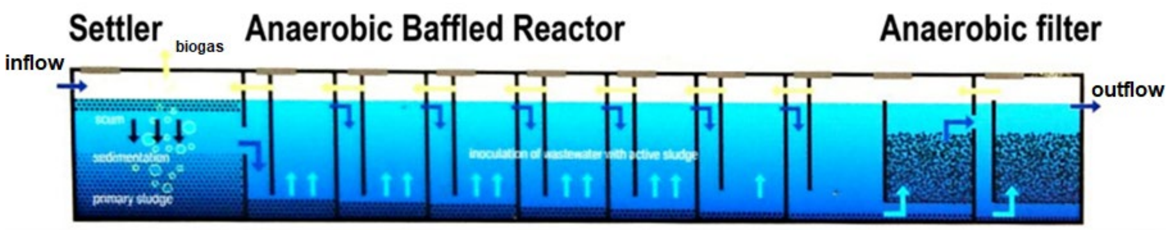


Figure 2-6: Illustrative side view of the primary and secondary treatment modules of the eThekwini DEWATS at the Newlands Mashu Research site (Arumugam and Buckley, 2020)

Originally, the eThekwini Municipality purposed the system to provide the community served with anaerobically treated domestic wastewater for irrigation. Proteins and amino acids from the raw organic waste are broken down in the primary and secondary treatment modules resulting in high ammonium-N concentrations (Gutterer et al., 2009). Reuse of the anaerobically treated domestic wastewater (herein referred to as AF effluent), reduces the demand of potable water for agricultural practices and the results of several field trials have been documented for a variety of crops (Musazura et al., 2015; Odindo et al., 2016). However, a recent study by Musazura and Odindo (2021) highlighted that the high nutrient loading can result in delayed flowering and uneven ripening for maize crops. They also found that pathogen (*E. coli*) contamination was possible with overhead irrigation of leafy vegetables such as cabbage and lettuce. The authors suggested dilution of the anaerobically treated domestic wastewater or further treatment, to reduce the nutrient load to the irrigated crops and deactivate the pathogens that may be present (Musazura and Odindo, 2021).

A portion of the AF effluent is tertiary treated in a hybrid SSFCW system consisting of a single VFCW (with sand media) and a single HFCW (with gravel media), operating in series for biological nitrification/denitrification. (Figure 2-7).

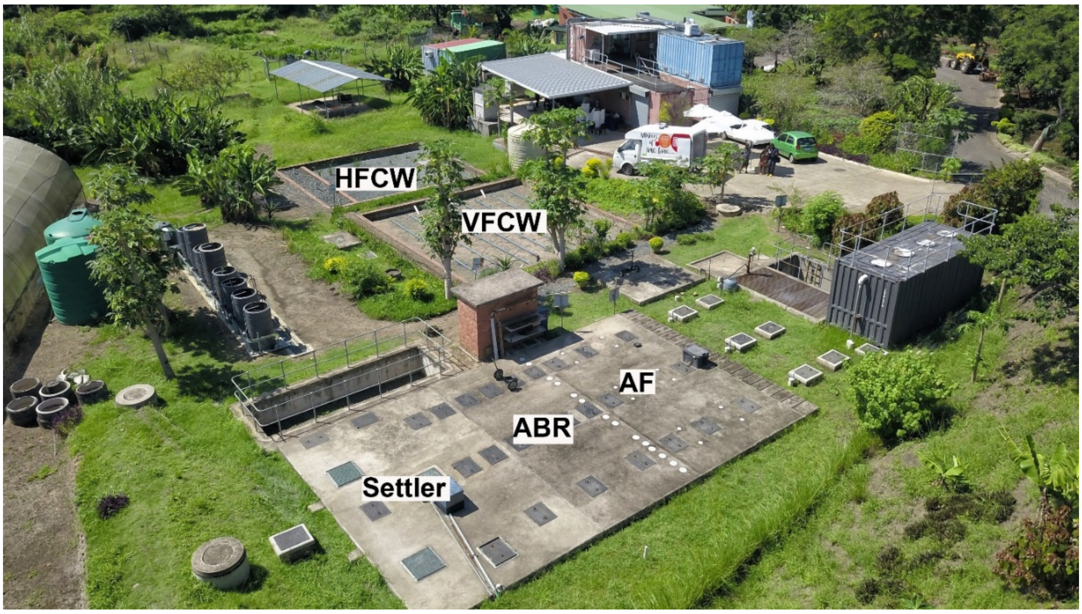


Figure 2-7: Treatment modules of the Newlands Mashu decentralised wastewater treatment system (taken March 2018)

ABR = anaerobic baffled reactor; AF = anaerobic filter; VFCW = vertical down-flow constructed wetland; HFCW = horizontal flow constructed wetlands

In other BORDA-designed DEWATS, VFCWs are rarely included due to the greater operating complexity of the system for intermittent loading and thus, increases the operational and maintenance demands of the DEWATS (Sasse, 1998; Gutterer et al., 2009). When testing the operational limits of the DEWATS, Pillay et al. (2013) increased the hydraulic loadings to the VFCW. High loadings caused the mechanical dosing device (a float siphon), responsible for the dosing regime of the intermittently fed VFCW, to fail thus, resulting in a continuous loading to the VFCW which, in turn, clogged the bed. Based on these observations, the author then suggested that for the South African context, HFCWs should be the only CW integrated into DEWATS design because of the gravel media which has a higher permeability (Pillay et al., 2013) thus, the possibility of clogging is reduced and the risks associated with human contact, especially children, with the wastewater is negated. However, based on the revised GA limits, which warrants removal of ammonium-N and nitrate-N via nitrification and denitrification, respectively, both CWs have been in continuous operation since mid-2014.

In 2016, Arumugam and Buckley (2020) identified numerous design and operational limitations of the demonstration-scale CWs at the NM DEWATS which are listed here:

1. The VFCW was constructed at half of the recommended depth by BORDA to maintain the hydraulic gradient for continuous flow to the HFCW. As a result, composite sampling was not possible in the outlet of the VFCW.
2. Irregular operation of the mechanical dosing device, sometimes resulting in continuous flow to the VFCW.
3. Uneven distribution of the wastewater on the surface of the VFCW, with many dry/dead zones observed after dosing.
4. The VFCW had a top layer of gravel (7-13 mm) on the surface to facilitate hydraulic movement of the incoming wastewater.
5. The vegetation in both CWs consisted of weeds and invasive species.
6. The filter media of the HFCW consisted of loosely packed, irregular-shaped, mixed gravel of 8-20 mm and 25-80 mm aggregates/broken stones in a ratio of 5:1.
7. The outlet pipe maintained the saturation level inside the HFCW at 0.3 m below the surface.

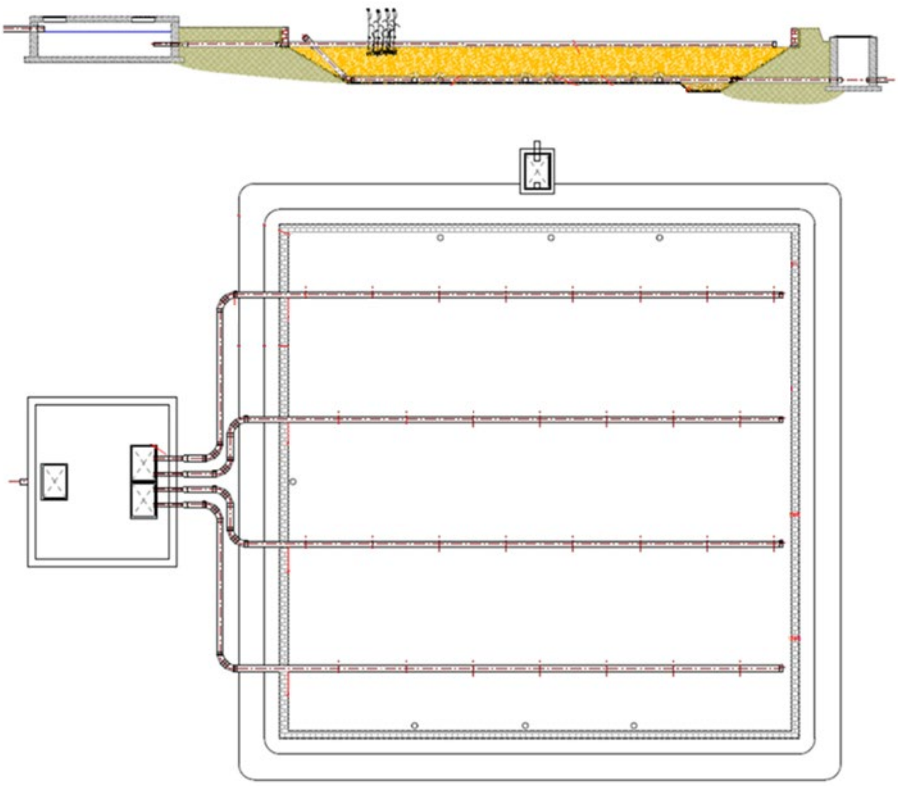
Arumugam and Buckley (2020) concluded that it was not feasible to amend the depth of the VFCW, as (1) this would have altered the flow regime to the HFCW and (2) it was not practical from an engineering point of view, considering that the DEWATS was already an established system. Moreover, it was not practical to amend the current media in either CW. However, to improve the treatment efficiency of the CWs, the following upgrades were done during 2017-2018 (Arumugam and Buckley, 2020).

1. The siphon was repaired to ensure stability in the mechanical operation of the float and thus, equal volumes per dose.
2. The surface gravel on the VFCW was removed and replaced with a 0.05 m top layer of 0.5-2 mm fine to coarse sand. The sand was manually compacted by foot-

stamping the surface to ensure temporary flooding or surface water accumulation after dosing from the siphon.

3. The distribution system on the surface of the VFCW was redesigned from four (110 mm dia.) perforated pipes (Figure 2-8a) to eight (75 mm dia.) pipes with alternating perforations (i.e., not in pairs but evenly distributed along the length of the pipe) (Figure 2-8b), to improve wastewater distribution.
4. The invasive vegetation and weeds were removed and replaced with locally available (± 5 km radius of the site) macrophytes (*Cyperus sexangularis* and *Typha capensis*) to ensure adequate aeration at the root zone and improve overall CW performance.
5. Sampling sumps were installed at the end of each CW to allow for composite sampling of the entire drainage from a single dose of the siphon. Cross connections between the inlet and outlet pipes of each sump allowed for the hybrid CWs to operate under normal flow regimes when sampling did not occur (Figure 2-9).
6. A PVC flexible hose (110 mm dia.) was installed at the outlet pipe of the HFCW to permanently impound the CW, maintaining the wastewater level at 0.15 m below the surface, to improve the hydraulic retention time (Figure 2-10).

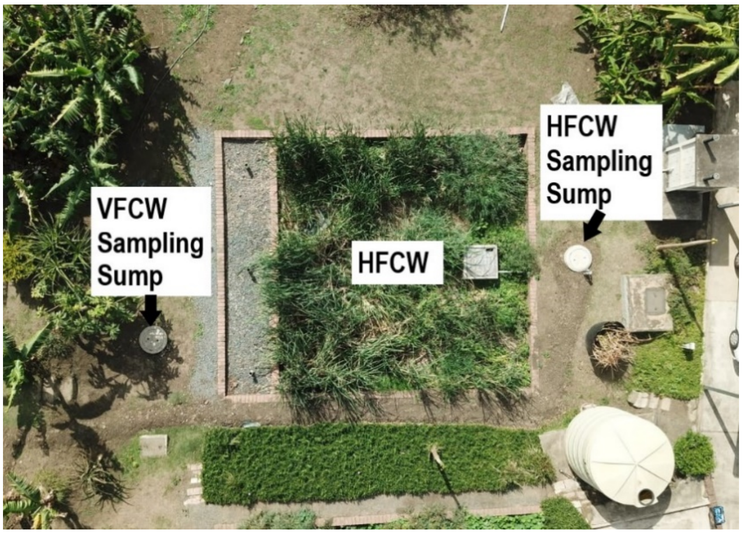
a.



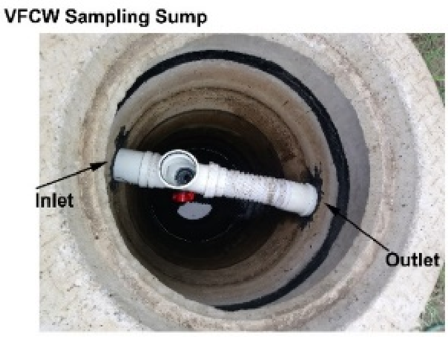
b.

Figure 2-8: a. Illustrated view of the demonstration-scale vertical down-flow constructed wetland before upgrades and b. after modification of the distribution system

a.



b.



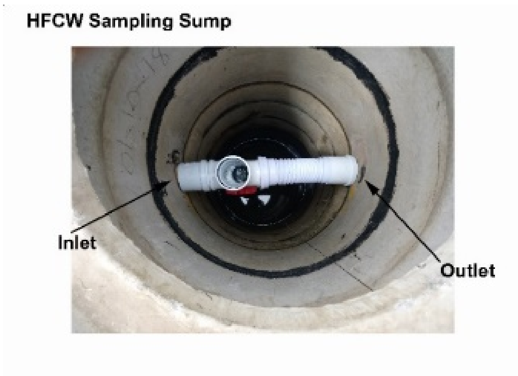


Figure 2-9: a. Positioning of sampling sumps at the outflow line of each constructed wetland for composite sampling of the effluent and b. cross connections between the inlet and outlet pipes of the sump

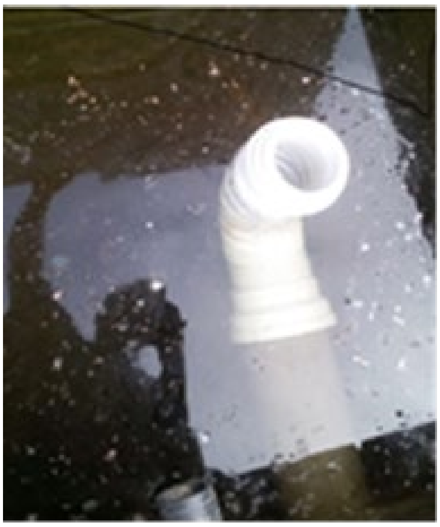


Figure 2-10: Flexible hose connected to the outlet pipe of the horizontal flow constructed wetland to maintain the saturation level at 0.15 m below the surface (permanent impounding)

2.7 GAPS IN CURRENT KNOWLEDGE

There is a considerable wealth of knowledge about the different types of CWs and the different mechanisms to enhance treatment, especially those that require any electrical input. However, as mentioned by Vymazal et al. (2021) and Vymazal (2022), the majority of research and performance data emanate from laboratory-scale experiments under controlled conditions. In fact, Vymazal (2018a) found that only 26% of research papers produced in 2017 stemmed from full-scale CW applications. The author argues that the short duration of laboratory experiments does not allow for systems to fully establish thus, long term performance evaluations are not possible (Vymazal, 2018a). Vymazal (2022) comments that the major challenge in CW research is transitioning from laboratory-based experiments to field applications. Thus, there is a need for performance data from field applications of CWs. While the evaluation of the current demonstration-scale CWs in the eThekwini DEWATS will contribute to performance data of CWs receiving anaerobically treated domestic wastewater, it is imperative that the other configurations be tested in the field as well.

When analysing the requirements for CWs in DEWATS designed for *in situ* informal settlement upgrades in eThekwini, and South Africa in general, the CW system must:

- Operate without any electrical needs (i.e., the flow must be gravitated).

- Have a low surface area demand considering the space limitation in these densely populated informal settlements.
- Be able to nitrify, and then denitrify, the anaerobically treated domestic wastewater effluent after primary and secondary treatment.
- Operate without any chemical input for treatment.

Vertical down-flow CWs meet these criteria, except for the ability to denitrify the AF effluent according to the design of the CW (Table 2-3). Studies have shown that VFCWs have a lower carbon footprint compared to centralised WWTWs (Pan et al., 2011) and a lower surface area requirement compared to HFCWs (Fuchs et al., 2011; Stefanakis et al., 2014). Therefore, an in-depth performance evaluation is required to determine if the upgraded VFCW in the demonstration-scale DEWATS in eThekwini can achieve safe discharge quality effluent when evaluated against the GA limits. However, it is possible that there would be very little or no carbon for the reduction of NO_3-N. This is based on the fact that more than 90% of total chemical oxygen demand (COD) and NH_4-N removal occurs in the upper 0.2 m of the sand media in VFCWs (von Felde and Kunst, 1997). Moreover, 90% of the biodegradable chemical oxygen demand (bCOD) is removed from the primary and secondary treatment modules of the DEWATS (Sasse, 1998). If NO_3-N removal is limited in the VFCW or above the GA limit of 15 mg L^{-1}, then availability of carbon in the HFCW needs to be enhanced. Perhaps the addition of dried plant material in the HFCW would be a passive way of improving denitrification, but this needs to be tested in the field.

Saeed and Sun (2011) evaluated lab-scale simulations of hybrid CW systems (VFCW-HFCW) using gravel, organic wood mulch and a combination of both substrates in each CW. They compared performances between the different VFCWs and found that the VFCW with the organic wood mulch media, demonstrated the highest NH_4-N and TN removal at 99.6% and 97.8%, respectively. Moreover, when comparing the HFCWs, they found that the HFCW with gravel media, performed better overall. They concluded that the organic substrates are preferred in VFCWs and not HFCWs (Saeed and Sun, 2011). Considering that available carbon would be required in the bottom layer of the filter media in the demonstration-scale VFCW at the NM DEWATS, it would not be practical to amend the media gradation to add a carbon source, even a plant-based material. Moreover, if the plant material needs to be added to the HFCW, it needs to be added at the inlet, without disrupting the media in the bed. Any disturbances in the media would result in preferential flow through the bed, resulting in decreased retention times and therefore, creating dead zones within the HFCW. The use of waste (excess) plant biomass from invasive species will add to the ecological management of the species since the supply will exceed its demand.

Torrens et al. (2009) document VFCWs with extended filter depths to increase HRTs and improve performance. The undulating landscape in eThekwini will allow for VFCWs with extended filter depths, but this needs to be tested with the AF effluent to determine if conditions for denitrification (adequate carbon and anoxic conditions) are met in a single-stage VFCW.

Alternatively, two-stage VFCWs have been successfully implemented in Austria for higher TN removal compared to single-stage VFCWs (Langergraber et al., 2008; Langergraber et al., 2009; Langergraber et al., 2014) using simple sand and gravel media. Langergraber et al. (2009) used 2-3.2 mm (coarse sand) in the filter media of the first stage in their two-stage VFCW for the purpose of restricting the complete COD degradation in the first stage. If the same media type is used in a single-stage VFCWs with an extended filter depth, then the denitrification potential increases toward the bottom of the filter depth where the conditions will be potentially anoxic. The alternating aerobic and anoxic zones within a single VFCW will reduce the area demand of the tertiary treatment modules in DEWATS and may improve nitrate-N removal to concentrations which are compliant with the GA limit. The two-stage VFCW, adapted from the Austrian design and based on local availability of sand and gravel media, needs to be evaluated in the South African context for the treatment of anaerobically treated effluent.

2.8 SUMMARY

While VFCWs possess certain advantages over the other types of CWs, such as higher oxygen transfer potential into the filter media and lower area demands, there is a general lack of detailed studies evaluating VFCWs in South Africa as well as studies focussing on design optimisation for improved total nitrogen removal.

Based on the designs of VFCWs currently existing in literature, the following design configurations of VFCWs in DEWATS can be tested in this study:

1. Settler-ABR-AF-VFCW.
2. Settler-ABR-AF-VFCW-HFCW.
3. Settler-ABR-AF-VFCW (1 m filter depth).
4. Settler-ABR-AF-two-stage VFCW.

CHAPTER 3. PERFORMANCE EVALUATION OF TWO VERTICAL DOWN-FLOW CONSTRUCTED WETLAND DESIGN CONFIGURATIONS (SINGLE-STAGE VERSES HYBRID) RECEIVING ANAEROBICALLY TREATED DOMESTIC WASTEWATER

3.1 INTRODUCTION

In South Africa, there is an urgent need to evaluate appropriate sanitation solutions for waterborne sanitation based on the National Sanitation Policy encouraging the use of decentralised systems (DWS, 2016). This comes after centralised wastewater treatment works (WWTWs) continue to degrade and fail (DWS, 2022). Moreover, climate change and cable theft demands resilient systems which do not depend on electricity for operation. However, to be considered appropriate, the final effluent from the selected technology needs to meet the regulatory requirements for safe discharge to a water resource (DWS, 2013). A modular-designed decentralised wastewater treatment system (DEWATS) has been in operation since 2010 in eThekwini (Pillay et al., 2013), one of South Africa's fastest growing cities (eThekwini Municipality, 2021). To date, the final effluent quality has not been evaluated against the Revised General Authorisations (GA) limits for safe discharge as listed in DWS (2013).

In the demonstration-scale DEWATS, two design configurations are possible, differing only in the tertiary treatment module - a single-stage vertical down-flow constructed wetland (VFCW) (design 1) or the single-stage VFCW operating in series with a horizontal flow CW (HFCW) (design 2, referred to as a hybrid CW system). Constructed wetlands (CWs) are common decentralised sanitation technologies (Sani et al., 2013), and appropriate for DEWATS which operate without electricity or chemicals for treatment (Gutterer et al., 2009). It is hypothesised that the design with the single VFCW will achieve fully compliant effluent for safe discharge into a water resource.

In addition to effluent quality, understanding the recovery time of the CWs, and the entire DEWATS in general, is imperative if this sanitation solution is to be applied for *in situ* informal settlement upgrades to formal housing. In informal settlements, circular migratory patterns of residents to their rural homesteads over holiday periods (i.e. where inward migration to cities is only for labour purposes) (Posel and Casale, 2006), will impact on the flow to the DEWATS. However, it is unclear how long the system will take to recover after interruptions in flow.

Therefore, this study evaluated the two design configurations of the DEWATS under continuous operation by comparing the final effluent quality with the GA limits. Moreover, it assessed the recovery time of the better performing design after long-term shut down (i.e., interruption in flow). It was hypothesised that the effluent quality of the demonstration-scale VFCW (design 1) will achieve fully compliant effluent for safe discharge into a water resource.

3.2 MATERIALS AND METHODS

3.2.1 Description of the DEWATS

The demonstration-scale DEWATS is located in Newlands East (29° 46' 25. 648" S, 30° 58' 28. 329" E), north of central eThekwini, on the east coast of Kwa-Zulu Natal. The site is owned by the eThekwini Municipality and named the Newlands Mashu Research site. It was designed according to BORDA guidelines (Sasse, 1998) and treats raw wastewater (black and grey water) from 84 households. Dimensioning of the DEWATS was based on an estimated 5 persons per household (HH), 90 L d^{-1} of wastewater generation per capita including a 10% reserve thus, equating to a design daily flow of 41.6 m^3 d^{-1} (Pillay et al., 2013).

Primary treatment of the raw domestic wastewater consists of sedimentation followed by anaerobic digestion. The influent flows into a two-chamber settler (primary treatment) with a total volume of 31.5 m^3, that later distributes the flow into three parallel anaerobic baffled reactor (ABR) trains. The purpose of the three-train ABR was to test the performance of the ABR under different hydraulic loading rates (Pillay et al., 2013). Each ABR train has a total volume of 22.05 m^3. Trains 1 and 2 are identical, consisting of seven equal sized chambers. Train 3 was constructed with four chambers; the first three double in size compared to a single chamber from Trains 1 and 2, with the final chamber the same size as a single chamber (Pillay et al., 2013; Schoebitz, 2013). A two-chambered anaerobic filter (AF), with a volume of 26.66 m^3 and gravel media at the bottom, treats the ABR effluent. Sequential treatment of the settler effluent in the ABR and AF constitutes secondary treatment of the raw wastewater. A schematic of the design of the primary and secondary treatment modules indicating the pipe distribution is shown in Figure 3-1.

The flow from Train 1 of the ABR is diverted to the hybrid CW system for tertiary treatment.

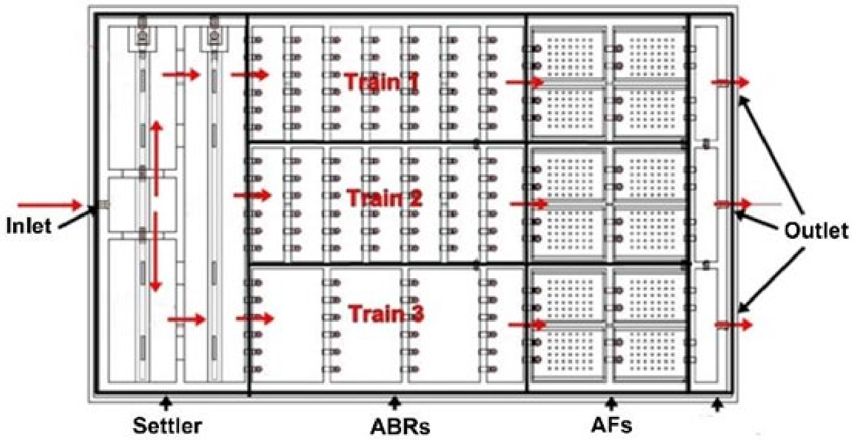


Figure 3-1: Illustrative plan view of the primary and secondary treatment modules of the Newlands Mashu decentralised wastewater treatment system (Schoebitz, 2013)

The red arrows indicate the direction of flow.

3.2.1.1 Design of the demonstration-scale constructed wetlands

A low-capacity float siphon doses the VFCW with AF effluent to allow for intermittent loading as per the design of this type of CW. The average volume is 1.87 m^3 per dose (refer to Arumugam and Buckley (2020) for full design specifications of the float siphon).

The area of the VFCW is 96 m^2 (9.8 m x 9.8 m; l x b) and 0.75 m in depth with a design hydraulic loading rate (HLR) of 0.14 m d^{-1}. The 0.55 m filter layer includes well-graded (Cu = 4.64) unwashed Umgeni River sand (D_{10} = 0.4 mm), over a 0.15 m drainage layer of 4-25 mm coarse gravel (D_{10} = 4.4 mm). On the surface is a top layer (≈ 0.05 m) of washed fine to coarse sand (0.5-2 mm). The media in the top and filter layer are compacted to improve the hydraulic retention time (HRT). The distribution system consists of eight 75 mm (dia.) pipes with alternate perforations (i.e., not in pairs). The average time taken for the drainage of an entire dose volume of 1.87 m^3 within the VFCW is 3:59 (hh:mm), at an average outflow rate of 0.6 L s^{-1} (Arumugam and Buckley, 2020).

The VFCW effluent then flows continuously to the HFCW, which has a surface area of 66 m^2 (8.1 m x 8.15 m; l x b) and depth of 0.9 m. At the inlet, the media consists of 50-80 mm gravel stones. The filter media in the middle of the HFCW consists of irregular-shaped mixed gravel of 8-20 mm and 25-80 mm aggregates/broken stones in a ratio of 5:1. The HFCW is permanently impounded to maintain the saturation level at 0.15 m below the surface of the bed

(Arumugam and Buckley, 2020). The discharge from the HFCW is piped by gravitated flow to the trunk sewer so that none of the treated effluent is released into the immediate environment.

Both CWs were planted with *Cyperus sexangularis* and *Typha capensis* (5-6 plants per m^2). The sampling sumps at the outflow line of each CW has a volume of 440 L.

The hybrid CW system (VFCW-HFCW) has been operating in series from mid-2014. Arumugam and Buckley (2020) modified both CWs during 2017-2018, after which the CW system operated at continuous flow since November 2018. The shut-down period occurred in March 2020 where the flow to the DEWATS was interrupted for 162 days. The CWs were started up again in September 2020.

3.2.2 Performance monitoring

3.2.2.1 Daily flow

Electromagnetic flow meters (SAFMAG) installed in the sump following the AF of each train measured the flow rate per second ($m^3\ s^{-1}$) which was used to calculate the cumulative daily flow ($m^3\ d^{-1}$). This data was logged in 10-minute intervals using an Omniflex Teleterm M3e Data logger and the daily flow was measured from 00h00-23h59.

3.2.2.2 Siphon dose volume and hydraulic loading rate to the VFCW

A pressure transducer (dipperLog Nano b, Heron Instruments Inc.) was suspended in the siphon chamber to measure the volume of AF effluent inside the siphon chamber and confirm the siphon dose volume which was used to calculate the discharge rate. The discharge rate was also measured from 00h00-23h59 each day of the monitoring period, but since the pressure transducer measured the dosing frequency, it was not expected to be equal to the daily flow measured by the data from the SAFMAG flow meter.

3.2.2.3 Sampling

3.2.2.3.1 Performance during continuous operation

In order to compare both DEWATS designs, the effluent from the VFCW (design 1) and the hybrid CW system (after the HFCW representing design 2) were sampled and analysed. Sampling was done over a 16-week period starting in February 2019, after three months of continuous operation. Grab samples of the AF effluent were taken from the siphon chamber on Day 1 and repeated every week at the same time (Figure 3-2).

A Zilmet V180F submersible pump was temporary installed in the respective CW sump to drain the sump once filled. With the addition of the pump, the working volume of each sampling sump was 398 L. Due to the fill and drain process of the VFCW sampling sump by action of the pump, emptying of the sump resulted in a pulsing feed to the HFCW, which is designed to operate under continuous flow. Therefore, the effluent from both CWs could not be sampled concurrently. As a result, both CWs were sampled on alternate weeks so as not disrupt the continuous flow to the HFCW.

Figure 3-2: Overview of the demonstration-scale decentralised wastewater treatment system

1 = sampling point for the AF effluent

2 = sampling point for the VFCW effluent (design 1)

3 = sampling point for the HFCW effluent (design 2)

Solid build up was found in the VFCW sampling sump on Day 2, suspected to be due to groundwater ingress through cracks in the wall of the sump, which was subsequently repaired. As a result, on Day 2, composite samples were taken from the HFCW sampling sump while sampling of the VFCW effluent took place on Day 9 (Figure 3-2). This equated to eight sampling events per CW over the 16-week period.

Based on the average dose volume of the siphon at 1.87 m^3, and the working volume of the sump at 398 L, it was predicted that the entire drainage volume from a single dose will fill up the CW sampling sump 4.7 times. For each fill of the sump, three 1 L samples were taken (with 30 second intervals between each sample). Each of these samples were referred to as "sub-composite samples". The first sub-composite sample from Fill 1 was mixed with the first sub-composite sample from Fill 2, and so on. Likewise, the second and third composite samples from Fill 1 was mixed with the second and third sub-composite samples from Fill 2, respectively and continued until all fills were sampled. Thus, there were three final composite samples for analysis. Considering the diurnal flow rate of the DEWATS, these final composite samples were not regarded as true replicates and were measured as individual samples.

3.2.2.3.2 Operation after resumption of flow (recovery performance)

The recovery performance evaluation was carried out intermittently during a 16-week period during September-December 2020, after resumption of flow to the DEWATS. The shut down was not intentional and occurred during the National COVID-19 Lockdown restrictions. Due to suspension of maintenance activities on site, a blockage upstream interrupted the flow to the DEWATS which inadvertently resulted in a shutdown of the entire system for 162 days.

Single composite samples were taken per sampling event (i.e., only one 1 L sample taken per fill of the CW sampling sump). Lastly, because recovery time was measured, the % of samples achieving compliance was not evaluated.

3.2.2.4 Effluent analysis

3.2.2.4.1 Chemical parameters

A Jenway 3540 pH & Conductivity meter was used to measure pH and electrical conductivity (EC) while a BOECO - O-580 hand-held meter was used to measure concentration of dissolved oxygen (DO). Total suspended solids (TSS) were measured according to Standard Methods (2540 D) (APHA, 2017). Total chemical oxygen demand (COD), ammonium-N (NH_4-N), NOx-N, based on the sum of nitrite-N (NO_2-N) + nitrate-N (NO_3-N), total nitrogen (TN), orthophosphate-P (PO_4-P) and alkalinity (ALKY) concentrations were measured with a Merck NOVA 60 Spectroquant. These samples were prepared and analysed according to the standard operating procedure (SOP) within the test kit supplied by Merck. All samples, except for COD and TN, were filtered (using a Whatman 1.2 µm pore size filter paper) and the filtrate was used for analysis to avoid TSS interference on the spectrophotometer.

The overall biodegradable COD (both readily/soluble and slowly/particulate fractions) from the COD concentration was measured by the Oxygen Uptake Rate (OUR) using a BM-Evo Respirometer (with a Knick – Stratos® Eco LCD display) at the WASH R&D Centre laboratory, University of KwaZulu-Natal. Ideally, the respirometer allows the user to calculate the rate of respiration (by a living organism/s) by measuring its rate of oxygen consumption over time. Three samples, taken at the beginning, middle and toward the end of the monitoring period, were prepared according to the method described by Wentzel et al. (1995) but adapted to use 1 L of sample as per the maximum volume of the respirometer. The sample was added to the continually stirred batch reactor and kept at a temperature of 20.9°C. For each sample, the time taken to reach the endpoint, or when all the readily available bCOD was consumed, was not less than 60 hours. Since the slowly available bCOD can take much longer to be consumed (> 60 hours), the calculated bCOD concentrations are regarded as estimates.

3.2.2.4.2 Pathogen indicator organisms

Faecal coliform indicator bacteria, *Escherichia coli* colony forming units, were measured using the Merck Petrifilm 3M plate using aseptic methods and recorded as colony forming units (CFU) per mL, and then converted to CFU 100 mL^{-1}.

All analyses, except for bCOD, were carried out in the on-site laboratory at the Newlands Mashu Research site where the demonstration-scale DEWATS was constructed.

3.2.2.5 <u>Data analysis</u>

3.2.2.5.1 Loading rate

The organic, nitrogen and pathogen loading to the CWs were calculated using Equation 3.1.

$$\textbf{Loading rate}\ (\mathbf{g\ m^{-2}\ d^{-1}}) = \mathbf{q}\ \times \mathbf{C_i} \qquad (3.1)$$

Where:

q is the hydraulic loading rate (m d^{-1}) to the CWs and Ci is the concentration of the AF effluent (mg L^{-1}).

3.2.2.5.2 Removal efficiency

Percentage removal (removal efficiency) of each pollutant was calculated by Equation 3.2.

$$\textbf{Removal efficiency (\%)} = \left(\frac{C_i - C_o}{C_i}\right) \times 100 \tag{3.2}$$

Where:

C_i is the concentration of the AF effluent (mg L^{-1}) and C_o is the concentration of the CW effluent (mg L^{-1}).

3.2.2.5.3 Mass removal rate

The mass removal rate of organic and nutrient content was calculated by Equation 3.3.

$$\textbf{Mass removal rate } (\mathbf{g}\, \boldsymbol{m^{-2} d^{-1}}) = \mathbf{q}\ (C_i - C_o) \tag{3.3}$$

Where:

q is the hydraulic loading rate (m d^{-1}) to the CWs; C_i is the concentration of the AF effluent (mg L^{-1}) and C_o is the concentration of the CW effluent (mg L^{-1}).

3.2.2.6 <u>Comparison of the CW effluent quality in relation to the General Authorisations</u>

The % of samples achieving compliance was calculated using Equation 3.4.

$$\%\ \textbf{of Samples achieving Compliance} = \frac{\mathbf{n}}{\mathbf{n\ (tot)}}\ \mathbf{x\ 100} \tag{3.4}$$

Where:

n is the number of CW effluent samples that were equal to or below the General Authorisations (discharge) limit for that parameter and n (tot) is the total number of samples analysed.

3.3 RESULTS

The results presented in this section pertain to the effluent quality over continuous operation and during recovery (after the resumption of flow following long term shut down).

3.3.1 Continuous operation

3.3.1.1 Daily flow upstream of the constructed wetlands

The total daily flow into the DEWATS ranged from 4.6-178.6 m^3 d^{-1}. However, for approximately 90% of the monitoring period, the DEWATS operated below the design flow of 41.6 m^3 d^{-1} with an average of 37.2 (± 16) m^3 d^{-1}. The daily flow through Train 1, feeding the CW system, ranged from 1.7-104.4 m^3 d^{-1} with an average flow of 17.7 (± 11) m^3 d^{-1} over the monitoring period (Figure 3-3). This was attributed to the fact that Train 1 received 47.6% of the total flow through the DEWATS. A scheduled desludging event of the settler in the middle of the monitoring period, stabilised the flow through Train 1 and to the CWs below its design of 13.9 m^3 d^{-1} (Figure 3-4). Low flows were attributed to blockages upstream of the DEWATS, while high flows were observed over holiday periods and heavy rainfall events, implying that rainfall impacts on the flow to the DEWATS and may dilute the raw wastewater.

3.3.1.2 Daily siphon dosing rate and hydraulic loading

Based on the data from the pressure transducer suspended inside the siphon chamber, the average time taken for an entire dose to be discharged was 208 seconds. Combined with the average dose volume of 1.87 m^3 into the VFCW, this equated to a dosing rate of 0.53 m^3 min^{-1} per dose while the average daily flow to the VFCW was 15.6 m^3 d^{-1}. On average the siphon discharged eight times per 24-hour cycle, equating to a time averaged flow rate to the VFCW as 0.16 m d^{-1}.

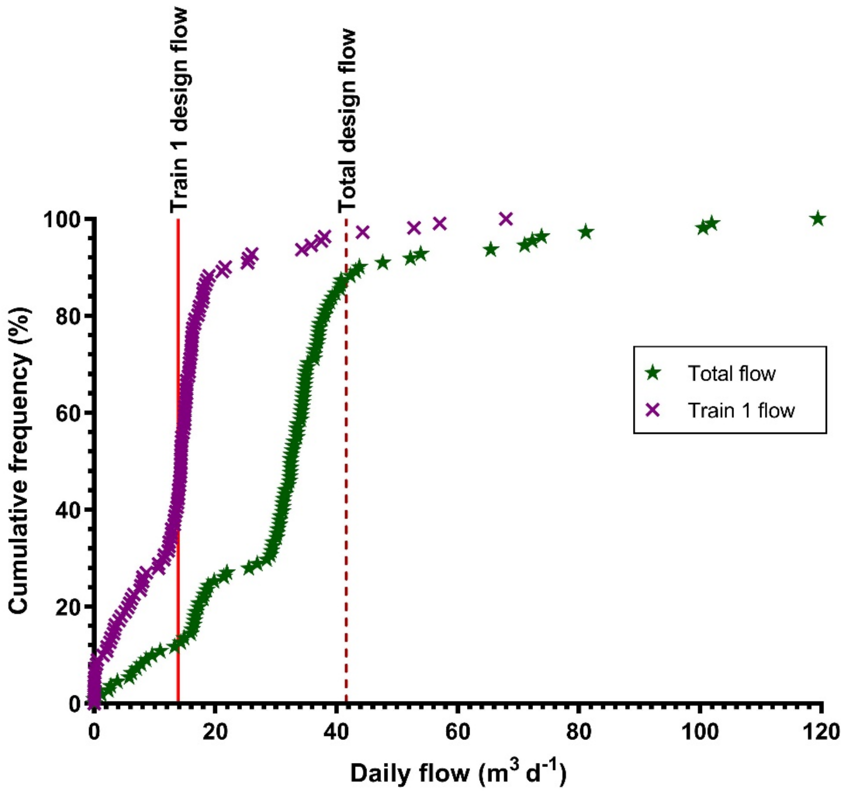


Figure 3-3: Cumulative frequency (%) of the daily flow into the DEWATS (total) and through Train 1 feeding the hybrid constructed wetland system during continuous operation (January-May 2019)

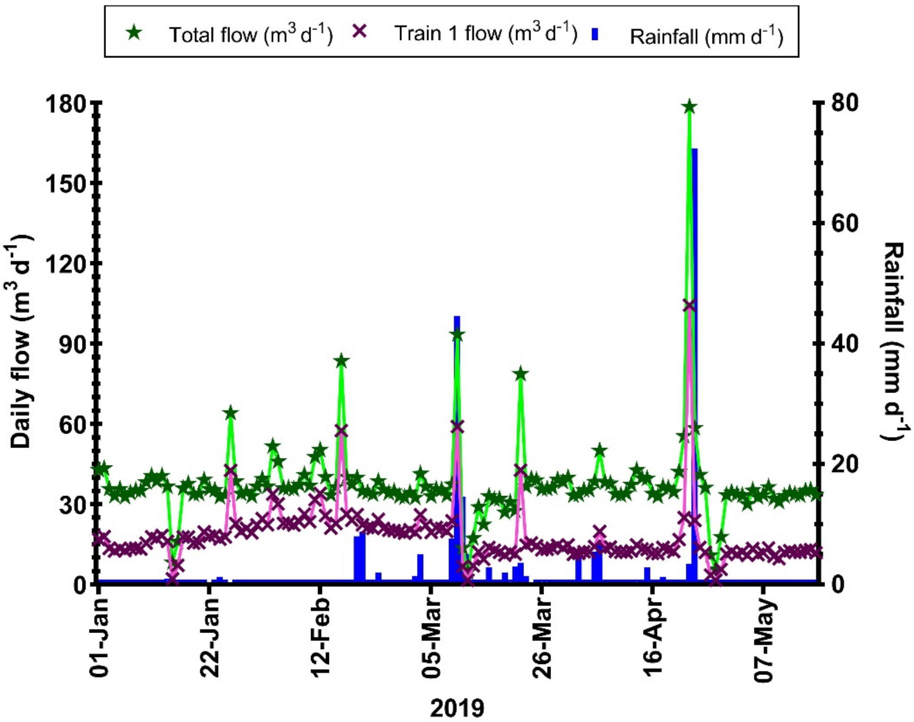


Figure 3-4: Daily flow (m^3 d^{-1}) into the DEWATS (total) and through Train 1 in relation to daily rainfall (mm d^{-1}) during continuous operation (January-May 2019)

3.3.1.3 AF effluent quality (Train 1 - feed)

The quality of the AF effluent from Train 1 of the ABR that feeds the hybrid CW system is presented in Table 3-1. Organic mass loading (COD) into the VFCW during the monitoring period was 57.2 g COD m^{-2} d^{-1}, of which 77% was estimated to be biodegradable (inclusive of the soluble and particulate fractions). The mean COD concentration was 357.8 (± 198.7) mg L^{-1}, while the mean TSS concentration was 49.3 (± 23.1) mg L^{-1}.

Total nitrogen loading was 9.8 g N m^{-2} d^{-1} with a mean concentration of 61 (± 12.9) mg L^{-1}. The major fraction (85%) was ammonium-N with a loading of 8.3 g NH_4-N m^{-2} d^{-1} and mean concentration of 52 (± 12) mg L^{-1}. No NO_X-N was detected in the AF effluent, implying no ingress of any industrial wastewater into the DEWATS, therefore, the remaining 15% of TN is assumed to be organic forms of N.

Orthophosphate-P (PO_4-P) concentration ranged from 2.6-17 mg L^{-1} with a mean of 8.3 (± 3.4) mg L^{-1}. The mean *E. coli* was 3.2 x 10^5 (± 1.8 x 10^5) CFU 100 mL^{-1} over the monitoring period (Table 3-1).

Table 3-1: Quality of the anaerobically treated effluent from Train 1 (January-May 2019).

	n	Range (Min-Max)	Mean ± Std Dev
COD [mg L^{-1}]	40	140-860	357.8 (± 198.7)
bCOD [mg L^{-1}]	3	274	
TSS [mg L^{-1}]	48	18-130	49.3 (± 23.1)
NH_4-N [mg L^{-1}]	48	22-68	52 (± 12)
NO_X-N	Not detected		
NO_2-N [mg L^{-1}]			
NO_3-N [mg L^{-1}]			
TN [mg L^{-1}]	48	26-74	61 (± 12.9)
PO_4-P [mg L^{-1}]	48	2.6-17	8.3 (± 3.4)
E. coli (CFU 100 mL^{-1})	48	3×10^4-8.2×10^5	3.2×10^5 (± 1.8×10^5)
pH	48	7.2-8.3	
EC (mS m^{-1})	48	75.9-105.3	95 (± 7.9)
ALKY [mg $CaCO_3$ L^{-1}]	48	92.3-412	318.5 (± 78)
DO [mg L^{-1}]	Did not measure		

3.3.1.4 VFCW effluent quality (design 1)

The quality of the VFCW effluent is presented in Table 3-2. The organic mass removal was 48.7 g COD m^{-2} d^{-1}, equating to a removal efficiency of 85% after treatment in the VFCW. The number of effluent samples of VFCW that were compliant with the COD GA limit of 75 mg L^{-1} was just over 85%. No bCOD was detected in any of the samples analysed. The removal of TSS was 26.4%, with a mean concentration of 36.3 (± 18.1) mg L^{-1}. Only 20.8% of the VFCW effluent samples were compliant with the GA limit of 25 mg L^{-1} for TSS.

The mean ammonium-N (NH_4-N) concentration was 4.3 (± 4.7) mg L^{-1}, with a removal efficiency of 91.7%, compared to the AF effluent quality. This indicated a mass removal of 7.6 g NH_4-N m^{-2} d^{-1}. Of the total number of samples, 87.5% achieved compliance with the GA limit of 6 mg L^{-1} for this parameter. There were only trace amounts of nitrite-N (NO_2-N) in the VFCW effluent, ranging from 0.2-2.5 mg L^{-1}. Nitrate-N (NO_3-N) was the major fraction of

NO_X-N, and overall TN concentration ranged from 27-69 mg L^{-1}, with a mean of 29.6 (± 13.8) mg L^{-1}. Only 12.5% of the samples analysed achieved compliance for the NO_X-N GA limit of 15 mg L^{-1}. Overall, there was a 19.7% removal efficiency of TN in comparison to the AF effluent with a mass removal of 1.9 g N m^{-2} d^{-1}.

Table 3-2: Effluent quality from the demonstration-scale vertical down-flow constructed wetland during continuous operation (January-May 2019).

	n	Range (Min-Max)	Mean ± Std Dev	Removal efficiency (%) / Log reduction (log_{10})	GA limit	% Samples achieving compliance
COD [mg L^{-1}]	21	24-115	53.5 (± 28.3)	85%	75	85.7%
bCOD [mg L^{-1}]	3	Not detected				
TSS [mg L^{-1}]	24	4-72	**36.3** (± 18.1)	26.4%	25	20.8%
NH_4-N [mg L^{-1}]	24	0-16	4.3 (± 4.7)	91.7%	6	87.5%
NO_X-N	24	4-54.2	**30.6** (± 14.1)		15	12.5%
NO_2-N [mg L^{-1}]	24	0.2-2.5	1.1 (± 0.8)			
NO_3-N [mg L^{-1}]	24	3.8-53.8	29.6 (± 13.8)			
TN [mg L^{-1}]	24	27-69	49 (± 11.5)	19.7%		
PO_4-P [mg L^{-1}]	24	3.5-7.1	5.1 (± 1)	38.6%	10	91.7%
E. coli (CFU 100 mL^{-1})	24	0-3.7 x 10^5	**5.5 x 10^4** (± 9.9 x 10^4)	0.8	1 000	12.5%
pH	24	5.6-7.9			5.5-9.5	100%
EC (mS m^{-1})	24	53.7-81.7	70.8 (± 7.3)		150	100%
ALKY [mg $CaCO_3$ L^{-1}]	24	0-160.5	17.8 (± 37.7)			
DO [mg L^{-1}]	24	1.7-2.9	2.1 (± 0.2)			

* Based on difference from AF effluent quality of Train 1; Red text denotes mean concentrations above the GA limit for discharge; green shaded cells denote the parameter is compliant with the Green Drop requirements (≥ 90%).

The reduction in PO_4-P concentration was 38.6%, with 91.7% of the samples achieving compliance with the GA limit. All samples were 100% compliant for the pH and EC GA limits. The log_{10} reduction of *E. coli* was 0.8 log_{10}, with 12.5% of the samples achieving compliance with the GA limit of 1000 CFU 100 mL^{-1}. The DO concentration in the VFCW effluent was 2.1 (± 0.2) mg L^{-1} (Table 3-2).

3.3.1.5 Hybrid CW effluent quality (design 2)

The HFCW effluent quality representative of full treatment in the hybrid CW system is presented in Table 3-3. Compared to the AF effluent quality, the overall COD removal efficiency in the hybrid CW system was 89.4%, increasing by 4.4 percentage points compared to treatment in the demonstration-scale VFCW alone (design 1). The organic mass removal rate was 51 g COD m^{-2} d^{-1}. As with the VFCW effluent, no bCOD was detected in the HFCW effluent. The mean TSS concentration was within the GA limit of 25 mg L^{-1}, with 75% of the samples achieving compliance. This was an increase of 54.2 percentage points compared to treatment in the VFCW alone. It is suspected that the action of the pump in the sampling sumps resulted in suspended matter from the walls of the sump to interfere with actual TSS concentrations in the drainage from each CW. Therefore, the actual TSS in the CW effluent is suspected to be much lower in the actual samples (Table 3-2 and Table 3-3).

Although all samples achieved compliance with the GA limit for NH_4-N, the overall reduction in TN was 37.9%, increasing by only 18.2 percentage points as compared to the VFCW operating on its own. The overall TN mass removal rate in the hybrid CW system was 3.7 g N m^{-2} d^{-1}. When comparing the NO_X-N concentration in both CW effluents, there was a 12.4% reduction in the HFCW effluent. Similar to the VFCW effluent, the major fraction of the NO_X-N concentration was NO_3-N, with only trace amounts of NO_2-N present. Overall, the concentration of NO_3-N decreased by 9 percentage points compared to the VFCW effluent.

The parameters that achieved 100% compliance within the GA limits were PO_4-P, pH, and EC, while faecal coliforms (*E. coli*) attained 83.3% sample compliance. Mean *E. coli* was 508 (± 854) CFU 100 mL^{-1} indicating a $\log_{10}$ reduction of 2.8 $\log_{10}$, compared to the AF effluent. There was no substantial change in DO concentration in the HFCW effluent compared to the VFCW effluent (Table 3-3).

Table 3-3: Effluent quality from the demonstration-scale horizontal flow constructed wetland during continuous operation (January-May 2019)

	n	Range (Min-Max)	Mean ± Std Dev	Removal efficiency (%) / Log reduction (log_{10})	GA limit	% Samples achieving compliance
COD [mg L^{-1}]	23	4-86	38.1 (± 26.2)	89.4%	75	87%
bCOD [mg L^{-1}]	3	Not detected				
TSS [mg L^{-1}]	24	1-68	20.5 (± 19.8)	58.4%	25	75%
NH_4-N [mg L^{-1}]	24	0-5	1 (± 1.6)	98.1%	6	100%
NO_X-N	24	10.6-41.8	**26.8** (± 10.2)	12.4%†	15	12.5%
NO_2-N [mg L^{-1}]	24	0-0.2	0.04 (± 0.06)	75.7%†		
NO_3-N [mg L^{-1}]	24	10.6-41.8	26.8 (± 10.2)	9%†		
TN [mg L^{-1}]	24	26-52	37.9 (± 9.2)	37.9%		
PO_4-P [mg L^{-1}]	22	1.3-6.4	3.1 (± 1.3)	62.7	10	100%
E. coli (CFU 100 mL^{-1})	24	0-2.8 x 10^3	508 (± 854)	2.8	1 000	83.3%
pH	24	6.4-7.9			5.5-9.5	100%
EC (mS m^{-1})	24	49.3-76.2	65.2 (± 7.1)		150	100%
ALKY [mg $CaCO_3$ L^{-1}]	24	0-87	23.5 (± 20.6)			
DO [mg L^{-1}]	24	1.3-2.5	2 (± 0.3)			

* Based on difference from AF effluent quality of Train 1; † reduction in concentration from the VFCW effluent; Red text denotes mean concentration above the GA limit for discharge; green shaded cells denote that parameter is compliant with the Green Drop requirements (≥ 90%).

3.3.1.6 Cumulative frequency curves of pollutant reduction across the hybrid constructed wetland system

Figure 3-5-3.8 illustrate the cumulative frequency of samples for COD, TSS, NH_4-N, NO_X-N, TN, and PO_4-P concentrations including *E. coli* from the AF effluent to the HFCW effluent in relation to the GA limit for safe discharge. This demonstrates the reduction of pollutants across design 1 and design 2.

The majority of the organic reduction occurred in the VFCW, while the HFCW acted as a polishing CW, although compliance with the COD GA limit was not achieved in the hybrid CW system (Figure 3-5). The HFCW was responsible for the majority of the TSS removal (Figure 3-6).

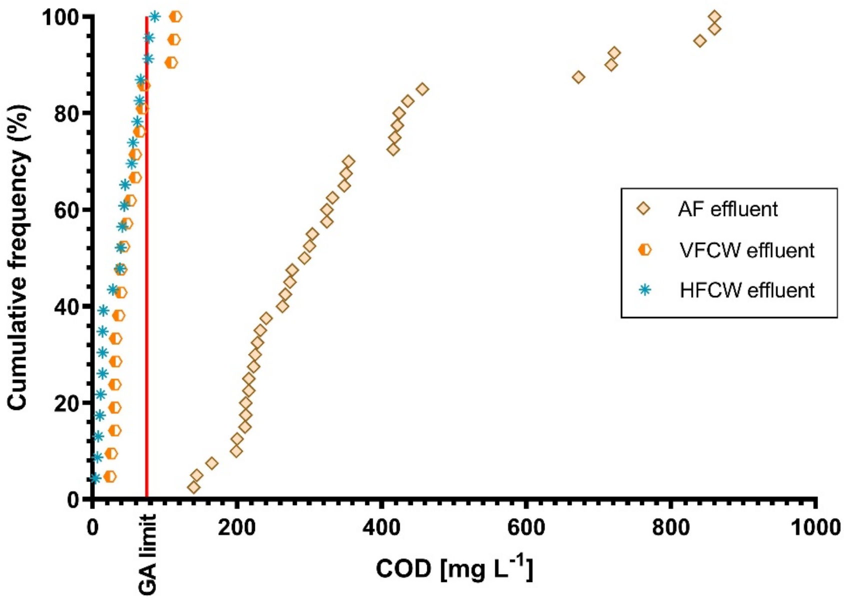


Figure 3-5: Cumulative frequency (%) of total chemical oxygen demand (COD) concentration in the anaerobically treated effluent (AF effluent) in comparison to successive treatment in design 1 (VFCW only) and design 2 (VFCW-HFCW)

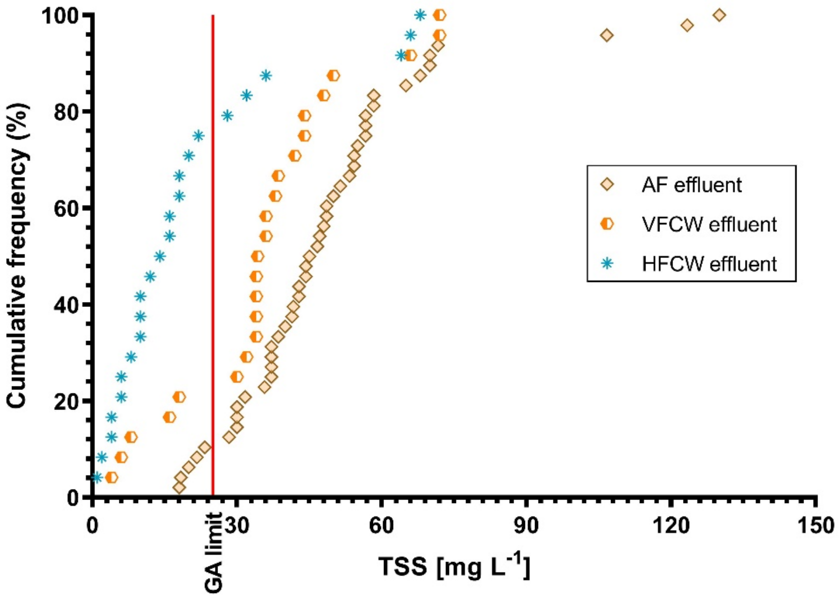


Figure 3-6 Cumulative frequency (%) of total suspended solids (TSS) concentration in the anaerobically treated effluent (AF effluent) in comparison to successive treatment in design 1 (VFCW only) and design 2 (VFCW-HFCW)

Similar to COD degradation, the majority of the NH_4-N concentration is reduced in the VFCW (Figure 3-7a) however, compliance was only achieved after treatment in the hybrid CW system. The same trend was not observed for the NO_X-N species. The dominant fraction (almost 99.9%) was NO_3-N in both CW effluents. Figure 3-7b demonstrates that the majority of the samples in both CW effluents are above the GA limit of 15 mg L^{-1} for NO_X-N, with only a minor reduction of NO_3-N in the HFCW (design 2) compared to the VFCW (design 1). There seems to be an almost equivalent % reduction in TN concentration after treatment in the VFCW and subsequently, in the HFCW (Figure 3-7c).

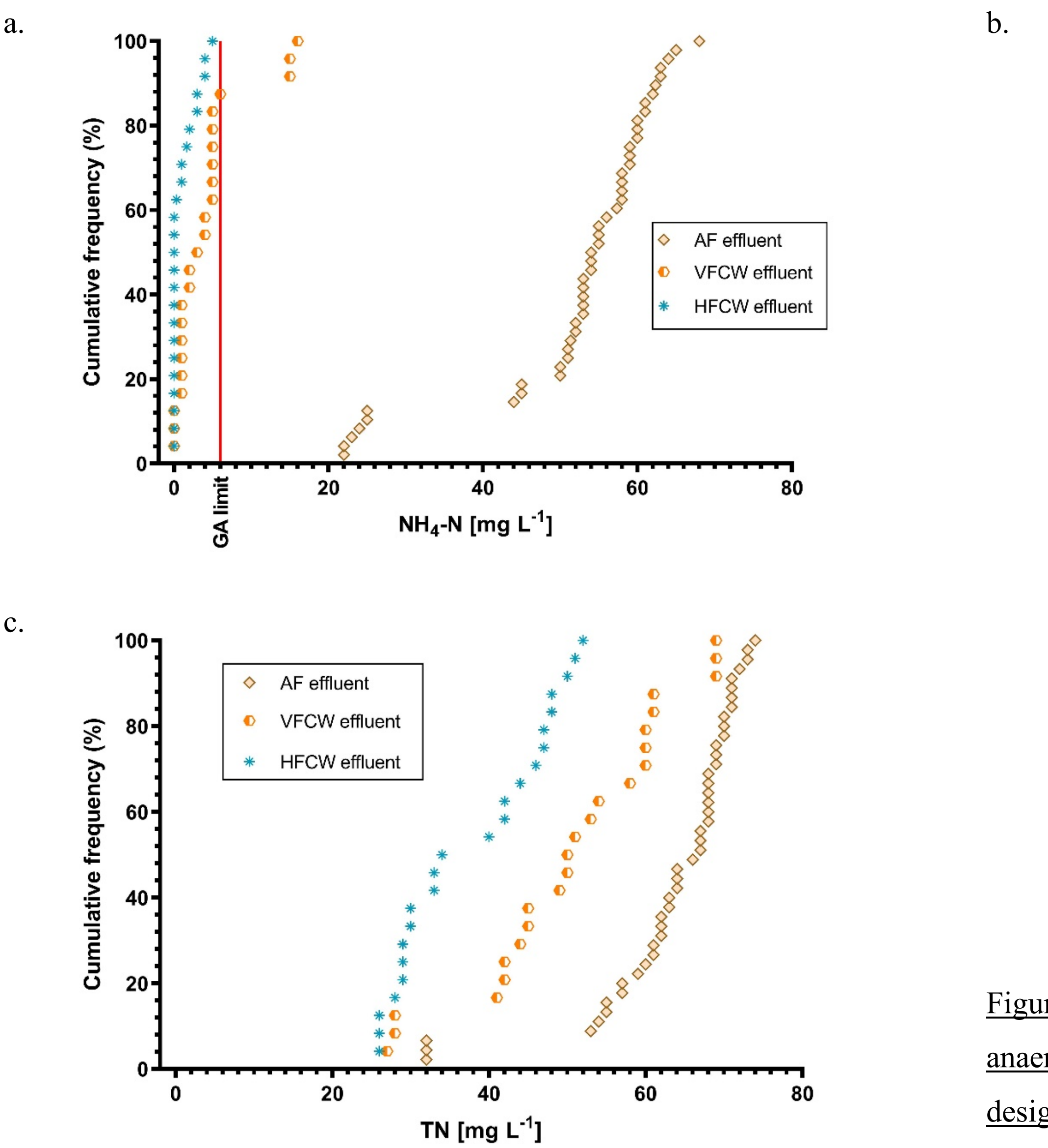


Figure 3-7: Cumulative frequency (%) of nitrogen species in the anaerobically treated effluent in comparison to successive treatment in design 1 (VFCW only) and design 2 (VFCW-HFCW); a. ammonium-N, b: nitrite-N + nitrate-N (NO_X-N) and nitrate-N and c: total N

Overall, orthophosphate-P (PO_4-P) concentration was not a concerning parameter in the AF effluent, and in both CW effluents, with more than 80% of the AF effluent samples compliant with the GA limit of 10 mg L^{-1}.

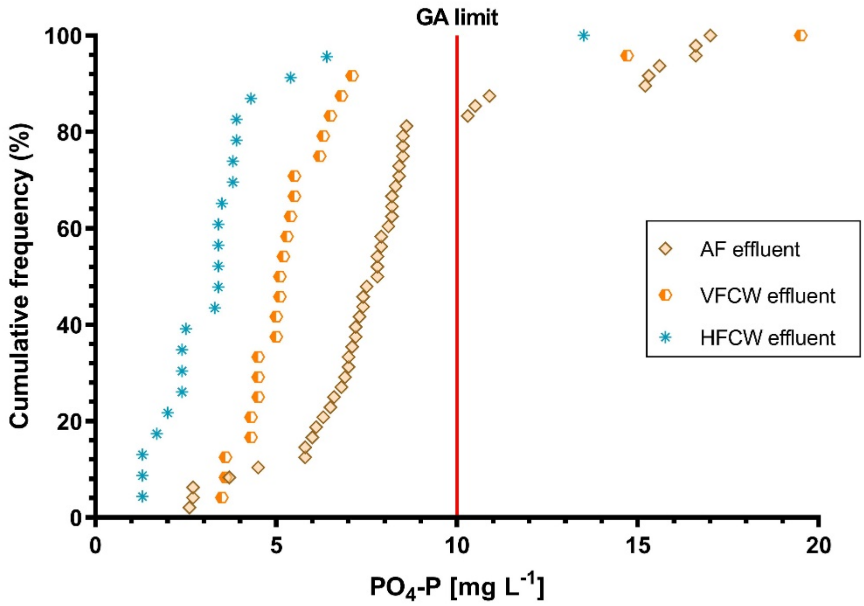


Figure 3-8: Cumulative frequency (%) of orthophosphate-P concentration in the anaerobically treated effluent in comparison to successive treatment in design 1 (VFCW only) and design 2 (VFCW-HFCW)

The notable difference in treatment efficiency between the VFCW operating alone and in series with the HFCW, was pathogen removal. The $\log_{10}$ reduction of *E. coli* was 2 $\log_{10}$ in the HFCW (design 2), compared to treatment in the VFCW alone (design 1) (Figure 3-9).

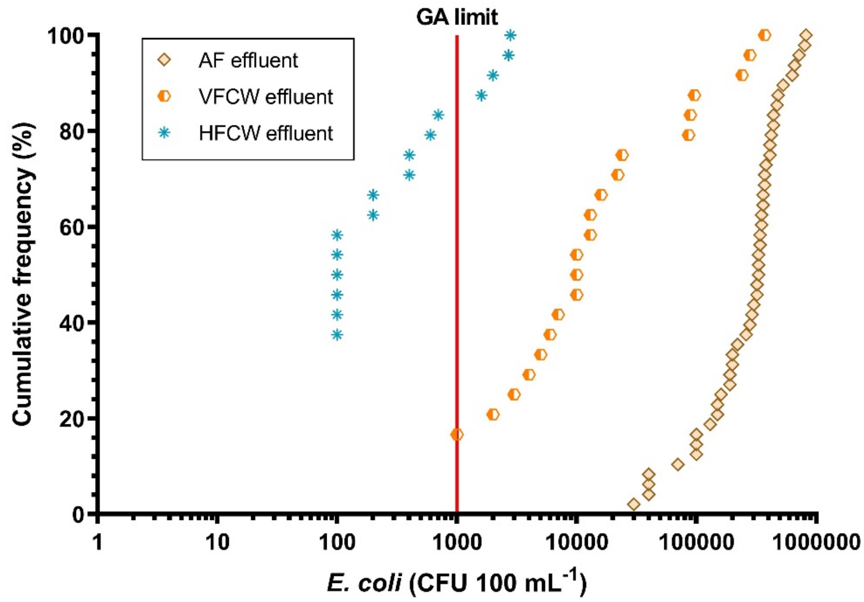


Figure 3-9: Cumulative frequency (%) of *Escherichia coli* in the anaerobically treated effluent, including after successive treatment in design 1 (VFCW only) and design 2 (VFCW-HFCW)

3.3.2 Performance during recovery

After the monitoring of both designs during continuous operation, it was deduced that design 2 (full treatment of the AF effluent in the hybrid CW system) achieved better quality effluent that design 1 (VFCW only) (Figure 3-5-3.8). Therefore, when determining the recovery time of the DEWATS, after 162 days of shut down, only design 2 was evaluated.

Considering that NO_3-N is the major fraction of NO_X-N concentration during continuous operation, NO_2-N was not measured during this performance evaluation. Thus, the GA limit for NO_X-N applies to NO_3-N concentration hereinafter.

To measure the time taken for the hybrid CW system to recover, the number of weeks required for the NH_4-N concentration in the HFCW effluent to be ≤ the GA limit was used as an indicator, considering that the VFCW was responsible for the majority (91.7%) of NH_4-N removal during continuous operation.

3.3.2.1 Daily flow to the CWs

After removal of the scum and solid build-up upstream of the DEWATS, the total daily flow to the DEWATS was 0-119.4 m^3 d^{-1} with an average of 32.6 (± 19.2) m^3 d^{-1} over September-December 2020. Flow through Train 1 ranged from 0-67.9 m^3 d^{-1} with an average of 14.4 (± 11.1) m^3 d^{-1} which equated to 44.3% of the total flow. Overall, the CWs operated above the design HLR for 60% of the monitoring period. Low flows were attributed to further blockages upstream of the DEWATS. High flows were observed during heavy rainfall events (Figure 3-10).

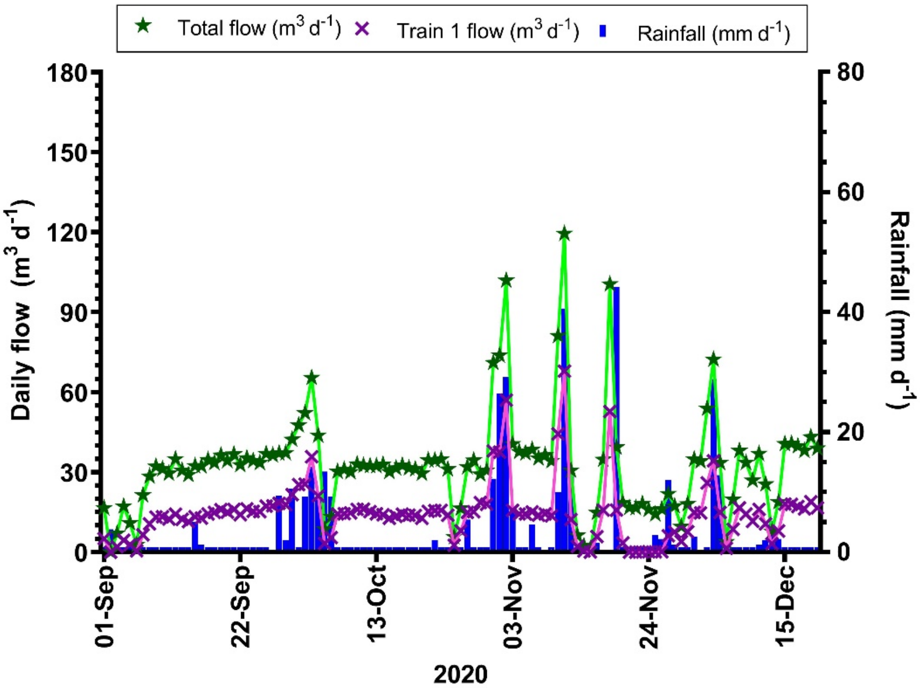


Figure 3-10: Daily flow to the DEWATS (total) and through Train 1 in relation to rainfall after resumption of flow (September-December 2020)

3.3.2.2 Siphon dosing rate and hydraulic loading

During this monitoring period, the average siphon dosing frequency was seven times per day, with no change in the discharge rate or volume per dose when the DEWATS was evaluated under continuous operation in 2019. This indicated no mechanical failures of the siphon. The HLR to the VFCW was slightly lower than the 2019 monitoring campaign at 0.14 m d^{-1}, equal to the design HLR.

3.3.2.3 HFCW effluent quality

The change in pollutant concentration is illustrated in Figures 3-11-3.13. Only in Week 16, was the NH_4-N concentration below the GA limit. However, sampling was interrupted between Weeks 11-15, therefore, almost full nitrification could have been achieved earlier than 16 weeks. However, in terms of NO_3-N, the % of samples achieving compliance increased to 33.3% as a result of not achieving full nitrification in the VFCW.

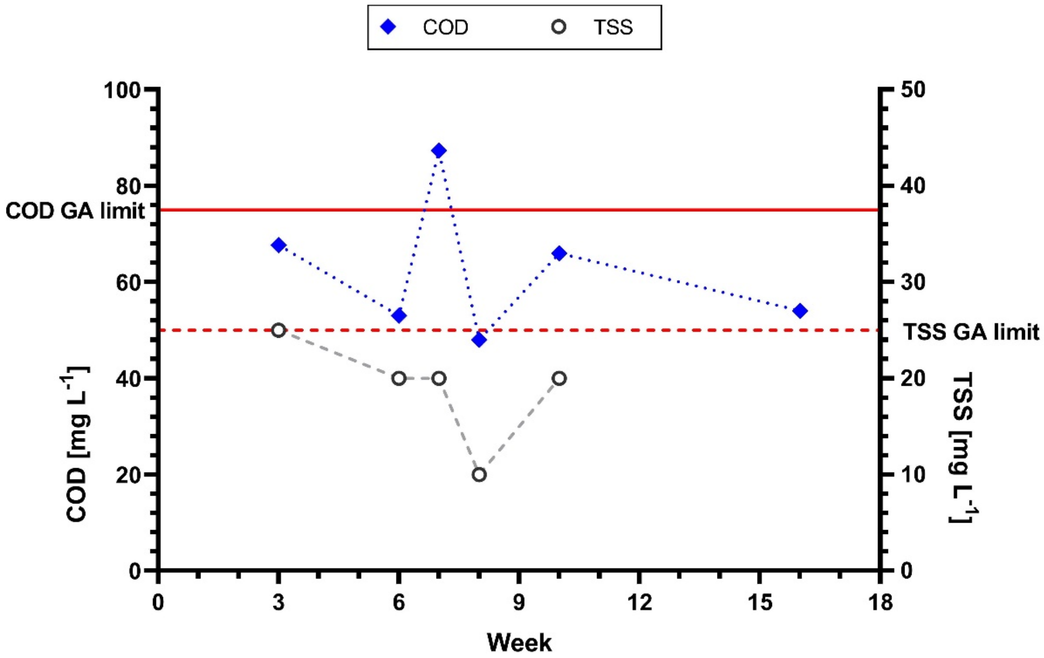


Figure 3-11: Change in total COD and TSS concentration in the horizontal flow constructed wetland effluent (September-December 2020)

Figure 3-12 demonstrates the change in NH_4-N concentration in relation to NO_3-N and TN concentration over the monitoring period. It can be seen that there was a steady decline in NH_4-N concentration between Weeks 3-8, with an increase in Week 10, then below the GA limit in Week 16.

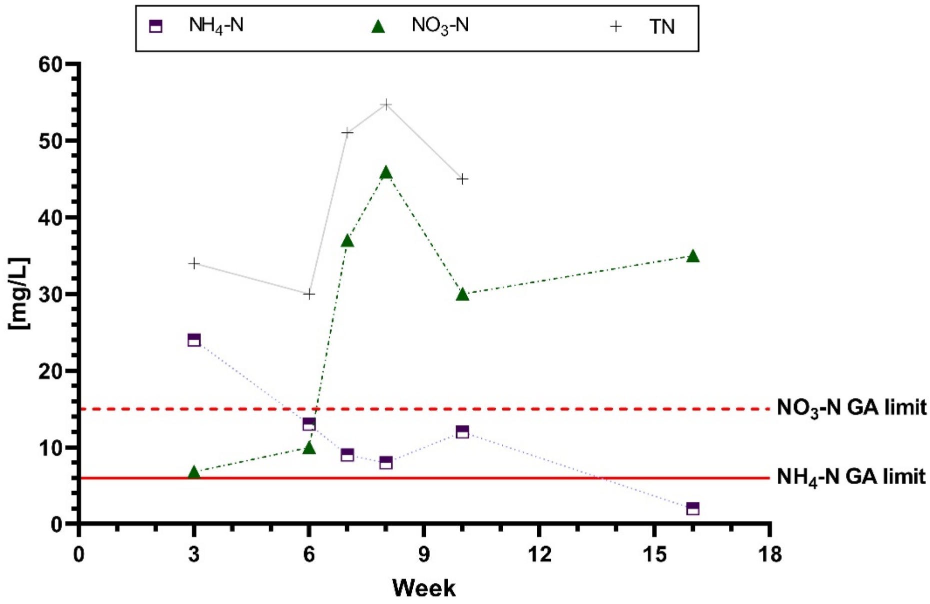


Figure 3-12: Change in ammonium-N (NH_4-N), nitrate-N (NO_3-N) and total nitrogen (TN) concentration in the horizontal flow constructed wetland effluent (September-December 2020)

In Week 3, the *E. coli* bacterial cells counts were too numerous to count (hence, this data point was omitted from the data set), however, by Week 6, it was below the GA limit for faecal coliforms. Fluctuations between Weeks 7-10 were seen during continuous operation. In Week 16, there was an unusually high count, presumably due to shock loading into the DEWATS (Figure 3-13).

The HFCW effluent quality during the recovery period is presented in Appendix B. During the recovery performance, PO_4-P concentration including pH and EC, were consistently below the GA limits for these parameters.

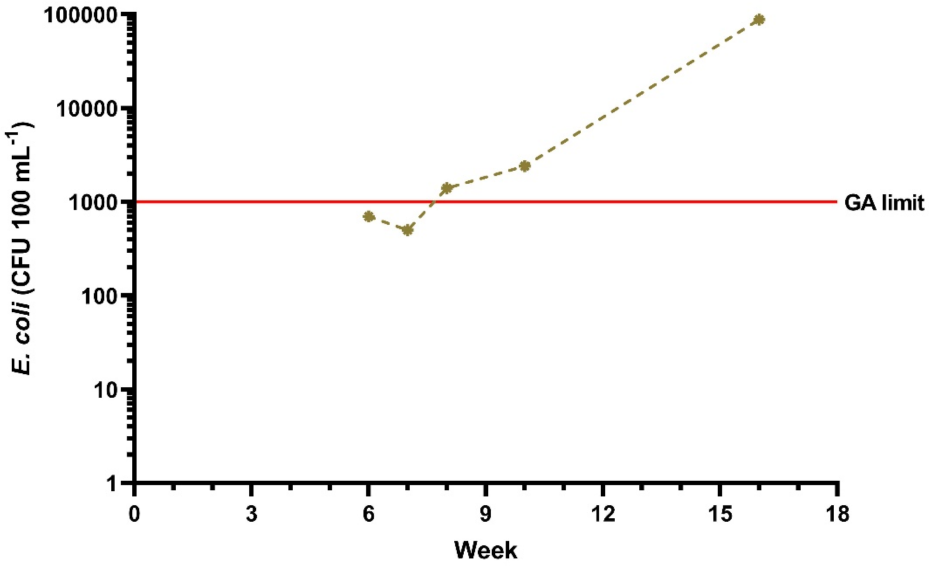


Figure 3-13: Change in *E. coli* colony forming units in the HFCW effluent (September-December 2020)

3.4 DISCUSSION

It is evident that the AF effluent is not safe for discharge to a water resource, as none of the tested pollutant parameters were above 90% compliant with the GA limits (Table 3-1). This data supports the fact that a discharge point after primary and secondary treatment is not possible and further treatment is required, supporting the suggestions from Musazura and Odindo (2021).

3.4.1 Continuous operation

The temporary flooding on the surface of the VFCW after each dose improved the residence time and permitted natural re-aeration through diffusion of oxygen as the wastewater permeated through the pore spaces in the media. However, this resulted in all of the bCOD being degraded (Table 3 2). It is assumed that this is the result of improved aeration of the VFCW since the ammonium-N (NH_4-N) removal efficiency was highest in the VFCW (91.7% after treatment in the VFCW, increasing by 6.4 percentage points after treatment in the HFCW).

Like OM degradation, nitrification is also an oxygen dependant process but occurs at a slower rate (Lee et al., 2009; Saeed and Sun, 2012). During the nitrification process, alkalinity is consumed (Kadlec and Wallace, 2009). The reduction in the mean alkalinity concentration from the AF effluent to the VFCW effluent was 300 mg $CaCO_3$ L^{-1} (Tables 3-1 and 3-2). Considering that 91.7% of the NH_4-N was reduced in the VFCW (Table 3-2), implied that the alkalinity in the AF effluent was sufficient to meet the demand for nitrification in the VFCW. However, alkalinity is produced during denitrification (Kadlec and Wallace, 2009). The change in mean alkalinity concentration from the VFCW effluent to the HFCW effluent was only 5.7 mg $CaCO_3$ L^{-1} (Tables 3-2 and 3-3), indicating poor denitrification. Considering that all of the available bCOD from the AF effluent was degraded in the VFCW, implied that there was no available carbon entering the HFCW. Arumugam and Buckley (2020) noted that the VFCW was constructed at half of its recommended depth as stipulated by BORDA guidelines, to maintain the hydraulic gradient for gravitated flow to the HFCW. Based on the complete degradation of bCOD at the current depth (0.75 m), suggests that even if the VFCW was constructed at its recommended depth of 1.5 m, denitrification would not have been possible in the VFCW due to the lack of available carbon as the energy source (Table 3-2).

Based on the high NO_X-N concentration in the VFCW effluent (30.6 ± 14.1) mg L^{-1}, the VFCW alone cannot achieve fully compliant effluent for safe discharge to the receiving environment. In fact, the VFCW effluent was only compliant with the GA limits for PO_4-P, pH, and EC. Thus, the hypothesis that the effluent quality of the upgraded demonstration-scale VFCW (design 1) will achieve compliance for safe discharge into a water resource, is not supported by the data from this study.

Moreover, the high NO_X-N concentration in the HFCW effluent at 26.8 ± 10.2 mg L^{-1} (with NO_3-N being the most abundant fraction) is surmised to be due to high DO concentrations (2 ± 0.3 mg L^{-1}). At DO concentrations > 0.3-0.5 mg L^{-1}, denitrifiers do not reduce NO_3-N and instead utilise oxygen, rather than nitrate-N, as terminal electron acceptors because of the higher energy generation during the metabolism of organic matter (Bertino, 2010). As a result of poor denitrification rates, overall TN removal in the hybrid CW system (design 2) was 37.9%. Accordingly, the HFCW effluent is also not safe for discharge to a water resource as the mean concentration of nitrate-N is above the GA limit of 15 mg L^{-1} (Table 3-3). Temporary surface water accumulation was occasionally noted on the HFCW during high hydraulic loadings (especially after a dose from the siphon when the peak outflow rates were observed). It is suspected that permanently impounding the HFCW in combination with the mixed, irregular-shaped gravel media with unpredictable pore spaces caused the bed to become over-saturated thus, permitting the transfer of atmospheric oxygen into the bed as the wastewater percolated through the gravel media, similar to the natural aeration process in the VFCW. It is suggested that the saturation level in the HFCW be maintained at 0.3 m below the surface to avoid surface water accumulation during peak flows.

The most notable difference between the VFCW and HFCW effluent quality is the log_{10} reduction in *E. coli*. It is clear that the hybrid CW system is best suited for high (> 80%) faecal coliform removal from the AF effluent as compared to treatment in the VFCW alone (Tables 3-2 and 3-3), further highlighting the non-compliance of the VFCW effluent for safe discharge to the receiving environment.

Pillay et al. (2013) had suggested that for future DEWATS in South Africa, only HFCWs should be selected as the tertiary treatment module due the relatively low maintenance of these CWs, no surface water accumulation and no operational requirements for the feeding regime (i.e., a dosing device). However, this study demonstrated that if required to produce effluent quality that is safe for discharge to water resource, design 2 with the VFCW connected in series to the HFCW is recommended. However, the media within the HFCW needs to be uniform in shape (rounded) and smaller in size to improve the residence time within the CW. Sasse (1998) recommend 6-12 mm or 8-16 mm medium gravel. Moreover, a carbon source needs to be added to the HFCW to ensure carbon availability for denitrification.

3.4.2 Recovery performance

Due to site restrictions, imposed by the lockdown regulations during the monitoring campaign in 2020, it is difficult to estimate the recovery time of the CWs in relation to the effluent quality pre-shut down or interruption in flow. However, based on removal efficiency of NH_4-N concentration over time, the recovery time of the hybrid CW system is estimated at 16 weeks (Figure 3-12). This implies that if design 2 is to be applied in future DEWATS where safe discharge of the final effluent is required, then interruptions in flow or complete shut down will require at least 16 weeks before the effluent can be discharged once flow has been resumed, provided it meets compliance with the GA limits.

3.4.3 Implications on DEWATS design

Considering that the DEWATS is an established system, it is not practical to redesign or change the media in the demonstration-scale CWs to augment overall total nitrogen (TN) removal. Moreover, based on the fact that the VFCW was constructed at half of its recommended depth to maintain the hydraulic gradient for gravitated flow to the HFCW, the only feasible action is to investigate the application of a selected carbon source in the HFCW. However, the plant material would need to be applied at the inlet of the HFCW, so as to not disturb the current media of the established CW. Excavating and improper repacking of the media can cause void spaces which has the potential to be filled with TSS and result in clogging (Matos et al., 2017; Ergaieg et al., 2021). Gersberg et al. (1984) observed plant biomass as an effective carbon source in constructed wetlands to increase denitrification rates, and within the last two decades, studies have employed waste biomass, such as reeds, as a low-cost plant-based carbon source (Ovez, 2006; Hang et al., 2016; Fu et al., 2017). However, the feasibility and sustainability of the carbon availability needs to be investigated under field conditions.

3.5 CONCLUSION

The VFCW effluent quality did not meet the GA limits for safe discharge. Overall, the hybrid CW system (design 2) outperforms the VFCW operating as a single-stage system (design 1), in terms of removal efficiencies of the measured pollutants. However, the major limitation of the hybrid CW achieving compliant effluent against the GA limits for safe discharge is reduction of NO_3-N, limited by DO concentrations greater than 0.5 mg L^{-1} and limited carbon availability, which directly affects TN removal. Plant-based carbon sources offer a cheaper option than chemical-based carbon sources but field application is required.

CHAPTER 4. THE POTENTIAL OF THE INVASIVE *ARUNDO DONAX* L. AS A PLANT-BASED CARBON SOURCE FOR AUGMENTED NITRATE REMOVAL IN A HORIZONTAL FLOW CONSTRUCTED WETLAND RECEIVING NITRIFIED DOMESTIC EFFLUENT

4.1 INTRODUCTION

In constructed wetlands (CWs), especially horizontal flow CWs (HFCWs) receiving secondary treated domestic wastewater, low carbon availability and high dissolved oxygen (DO) concentrations can limit the reduction of nitrate-N (NO_3-N) to N_2 (Ingersoll and Baker, 1998; Saeed and Sun, 2012). Ingersoll and Baker (1998) recommended a COD:N of 5:1 for optimum denitrification rates, while Bertino (2010) noted that at DO concentrations above 0.5 mg L^{-1}, the heterotrophic denitrifying bacteria will use the available DO instead of NO_3-N as the terminal electron acceptor in cellular respiration. Chemical carbon sources, such as methanol, are often used to augment the denitrifying potential of the CW (Kadlec and Wallace, 2009), however, it contributes to the operation costs and thus are not applicable for CWs required to operate with no chemical demands. Plant-based carbon sources are a low-cost alternative to chemical sources (Wen et al., 2010; Fu et al., 2017). However, the carbon from plant materials are readily degraded and thus, becomes an unsustainable source (Ma et al., 2020).

The Giant reed, *Arundo donax* L., is largely considered one of the most burdensome invasive species (Jiménez-Ruiz et al., 2021). In South Africa, the reed was introduced to assist with soil erosion in the 1700s (Canavan et al., 2017) but is now classified as a Category 1b invasive species (DEA, 2016), implying that propagation of any kind is prohibited. Therefore, interventions to manage the species must be employed. However, the advantage of high biomass production is that the biomass can be used for energy production (Mack, 2008; Pilu et al., 2012; Li et al., 2018; Oginni and Singh, 2019), with the potenital to improve soil fertility, and carbon sequestration through the formation of biochar through pyrolysis (Saikia et al., 2015).

According to Hang et al. (2016), to assess the suitability of plant-based carbon sources, denitrification (in this case, the nitrate-N removal efficiency) should be monitored over a 90-day cycle. Fu et al. (2017) employed lab fermentation techniques on dried plant material of *A.*

donax in combination with *Pontederia cordata* and found TN removal of 92.8% in pilot-scale vertical flow CWs (VFCW) treating artificial wastewater. Using air-dried plant material of *A. donax* in the HFCW seems to be a relatively simpler process than pyrolysis, which is the process of forming biochar by heating the plant material at very high temperatures (400-800 °C) (Jeguirim and Trouvé, 2009; Chandler and Resende, 2018).

Arundo donax is referred to as a lignocellulosic biomass since the plant material is composed of lignin, cellulose, and hemicellulose (Jeguirim and Trouvé, 2009; Suárez et al., 2021). Both lignin and cellulose are the two most abundant organic substances, with cellulose hydrolysis occurring much faster than lignin decomposition (Horwath, 2007). In their recent characterisation analysis, Suárez et al. (2021) found higher lignin and cellulose content in the stems/culms and leaves of *A. donax*, compared to the roots.

The purpose of this study was to evaluate the field or *in situ* application of dried plant material of *Arundo donax*, in an established demonstration-scale horizontal flow CW (HFCW) receiving nitrified effluent for improved nitrate-N removal. Denitrification in the HFCW is limited by low COD:N and high oxygen availability (Section 3.4).

It was hypothesised that the HFCW effluent will meet compliance for nitrate-N after the addition of pre-dried *A. donax* plant material at the inlet of the CW through natural anaerobic fermentation. To determine the impact of the application of the selected species, compliance of the HFCW effluent quality was compared to the Department of Water and Sanitation's (DWS) Revised General Authorisation (GA) limits for safe discharge of treated domestic effluent into the receiving environment.

4.2 MATERIALS AND METHODS

4.2.1 Design of the horizontal flow constructed wetland

The HFCW is the last treatment module in a demonstration-scale decentralised wastewater treatment system (DEWATS) in eThekwini (29° 46' 25. 648" S, 30° 58' 28. 329" E), on the east coast of South Africa. The DEWATS treats raw domestic wastewater from 84 households with a design flow of 41.6 m^3 d^{-1}. The DEWATS has three flow trains, of which flow from Train 1 is treated in a hybrid constructed wetland system consisting of a vertical down-flow CW and the HFCW (Arumugam and Buckley, 2020). The surface area of the HFCW is 66 m^2 (8.1 m x 8.15 m; l x b) at a depth of 0.9 m. At the inlet of the HFCW there is 50-80 mm gravel stones. The filter media toward the middle of the HFCW consists of loosely packed, irregular-shaped, mixed gravel of 8-20 mm and 25-80 mm aggregates/broken stones in a ratio of 5:1. The saturation level in the HFCW is maintained at 0.3 m below the surface of the bed by a flexible hose attached to the outlet pipe. The HFCW is planted with *Cyperus sexangularis* and *Typha capensis* (5-6 plants per m^2). The sampling sump at the outflow line of the HFCW has a volume of 440 L.

4.2.2 Lab-scale assessment of COD in plant material

Before field application of the waste biomass, a laboratory-scale fermentation experiment was conducted to determine the total chemical oxygen demand (COD) concentration in the solution and confirm the suitability of the species as a plant-based carbon source. The COD concentration was measured with a Merck NOVA 60 Spectroquant using the COD Merck Spectroquant test kit.

4.2.2.1 Preparation of plant material

Different sections of fresh plant material (top leaves, mid leaves and stems, and bottom stems) from 5 plants (± 4 m in height) were cut into smaller (5-10 cm) pieces and then weighed.

4.2.2.2 Lab-scale fermentation experiment

For the laboratory fermentation experiment, the wet biomass was dried in an oven at 60°C for 7 days until constant dry mass was achieved. The method for anaerobic fermentation of the dried plant material was adapted from Fu et al. (2017) with a few modifications. Four batch experiments were conducted with each batch reactor containing 25 g of dried plant material in different variations (top leaves, mid leaves and stems, bottom stems, and a combination of all) cut into 1-2 cm pieces, added to a 1 L conical flask, and topped with 1 L of tap water (Appendix C). The opening of the flasks were covered with foil, the edges sealed with masking tape, and placed in an oven at 30°C for 7 days to allow for anaerobic fermentation (Appendix D). The COD concentration measured from the upper surface solution of each flask is represented in Table 4-1.

Table 4-1: Total COD in the supernatant of each batch test in the oven

	COD [mg L^{-1}]			
	Batch 1	**Batch 2**	**Batch 3**	**Batch 4**
	Top Leaves	**Mid leaves and stems**	**Bottom stems**	**Combination**
Day 7	3623.3	2510	1440	2325
Day 14	4118.3	2780	1300	2270

Based on the laboratory fermentation experiment, it was confirmed that the dried waste biomass of the local *Arundo donax* L. did provide sufficient organic matter (COD) and thus, would be applicable in the South African context as a plant-based carbon source for denitrification. However, the top leaves appeared to have degraded faster than the mid leaves with the decomposition and release of available carbon being the slowest in the bottom stems. Based on this observation, and to ensure a slow steady release of available carbon for denitrification and minimise the need for frequent plant material addition, the combination batch was selected for the field application.

4.2.3 Field assessment of natural anaerobic fermentation

To understand the concentration of COD that will be available via natural anaerobic fermentation, 25 g of combined dried plant material was placed into a 1 L conical flask topped with nitrified effluent (the same feed as the HFCW) and covered entirely with foil (Appendix E). The flask was left in the field (in shade) for three weeks, undisturbed. The purpose of placing the flask in the shade was to simulate the actual waterlogged conditions under the gravel media of the HFCW where the plant material would be added. A sample of the solution was used to measure the COD concentration weekly.

Using the COD concentration of the solution at Day 14 as a comparison, it was clear that the natural fermentation process in the field was slower than lab-based fermentation in the oven, at 1385 mg COD L^{-1} (Table 4-2) and 2270 mg COD L^{-1} (Table 4-1), respectively. However, it was unclear what mass should be added to the HFCW to ensure sufficient carbon availability for denitrification as well as ensure a slow hydrolysis of the plant tissues to sustain the supply of carbon.

Based on the saturation level within the HFCW at the inlet to ensure that the plant material was inserted in waterlogged conditions, and the area of the quadrant, a mass of 5 kgs was selected.

Table 4-2: Total COD in the supernatant of the batch test in the field

Day	**COD [mg L^{-1}]**
0	0
7	1480
14	1385
21	1305

4.2.4 Experimental design - Field application

4.2.4.1 Preparation of plant material

For the field experiment, whole plant wet biomass was air dried in the growing tunnel on site for 10 days until constant dry mass was recorded. A combination of air-dried stem and leaf material was then cut into 1-2 cm pieces to accelerate the anaerobic fermentation process (Figure 4-1a).

a. b.

Figure 4-1: a. Cut pieces (1-2 cm) of the air-dried plant material of *Arundo donax* and b. excavated quadrant at the inlet of the HFCW where the plant material was added

4.2.4.2 Application of dried plant material to the inlet of the HFCW

Using the middle PVC pipe in the inlet distribution bed as a midpoint, a 2 m x 0.5 m (l x b) quadrant was excavated as shown in Figure 4-1b and Figure 4-2. All of the gravel media in this section was removed until the wastewater level was reached. It was anticipated that the natural fermentation process will occur at a much lower rate than that of the lab-scale fermentation experiment. A total of 5 kgs of cut, air-dried plant material was added such that all of the plant material was submerged in the wastewater. The gravel was replaced and compacted causing minimal disturbance to the HFCW itself, as well as reducing the potential diffusion of DO through the gravel pore spaces. The experiment was conducted between March and June 2021 (Autumn into early Winter) where the air temperatures ranged from 12.5-34°C (based on weather station readings in eThekwini North[**]).

[**] https://data.ethekwinifews.durban/instrument/stations

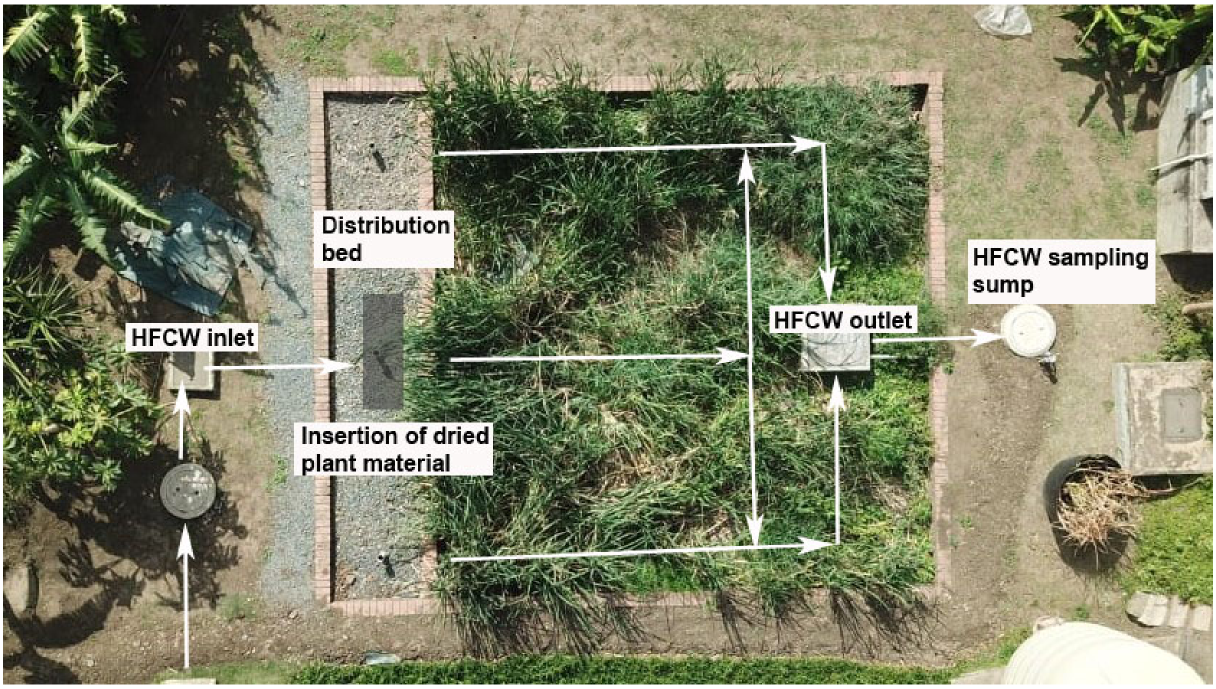


Figure 4-2: Aerial view of the horizontal flow constructed wetland indicating the distribution bed where the dried plant material of *Arundo donax L.* was added

White arrows indicate the direction of flow; the demarcated area in the distribution bed is where the dried plant material was added.

4.2.5 Performance monitoring

4.2.5.1 Flow monitoring

The daily flow into Train 1 was recorded as described in Section 3.2.2.1 to determine the hydraulic loading to the HFCW.

4.2.5.2 Sampling

Grab samples were taken randomly from the VFCW effluent (feed to the HFCW) (n = 5) to determine the organic matter (OM) and nitrate-N loading rate to the HFCW over the monitoring period, without disturbing the continuous flow to the HFCW.

A grab sample from the outflow in the HFCW sampling sump was taken on Day 1 (prior to any media removal/disturbance) for the base COD and nitrate-N (NO_3-N) concentrations.

Composite samples of the HFCW effluent were then taken every seven days thereafter for 13 weeks (92 days since the addition of the dried plant material) (n = 13 excluding the base value on Day 1).

4.2.5.3 Chemical and microbiological analysis

4.2.5.3.1 VFCW effluent

Only COD and nitrate-N concentration was measured in the VFCW effluent.

4.2.5.3.2 HFCW effluent

A Jenway 3540 pH & Conductivity meter was used to measure pH and electrical conductivity (EC). Dissolved oxygen concentration was measured using a BOECO - DO-580 handheld meter. Total suspended solids was measured according to Standard methods (2540 D) (APHA, 2017). Total chemical oxygen demand (COD), ammonium-N (NH_4-N), nitrate-N (NO_3-N), total nitrogen (TN), orthophosphate-P (PO_4-P) and alkalinity (ALKY) concentration were measured with a Merck NOVA 60 Spectroquant using the respective Merck Spectroquant test kits.

All samples, except for COD and TN, were filtered (using a Whatman 1.2 µm pore size filter paper) and the filtrate prepared for analysis to avoid TSS interference on the spectrophotometer.

Faecal coliform indicator bacteria, *Escherichia coli* colony forming units, were measured using the Merck Petrifilm 3M plate and recorded as colony forming units (CFU) 1 mL^{-1}, converted to CFU 100 mL^{-1}.

Biodegradable COD could not be measured due to instrument downtime at the WASH R&D Centre, University of KwaZulu-Natal.

4.2.5.4 Data analysis

4.2.5.4.1 Loading rates

The organic and nitrate-N loading to the HFCWs were calculated using Equation 4.1.

$$\textbf{Loading rate}\ (\mathbf{g\ m^{-2}\ d^{-1}}) = \mathbf{q}\ \times \mathbf{C_i} \tag{4.1}$$

Where:

q is the hydraulic loading rate (m d^{-1}) to the HFCWs and Ci is the concentration of the VFCW effluent (mg L^{-1}).

4.2.5.4.2 Comparison of the HFCW effluent quality in relation to the General Authorisations

The % of samples achieving compliance was measured using Equation 4.2.

$$\textbf{\% of Samples achieving Compliance} = \frac{\mathbf{n}}{\mathbf{n\ (tot)}}\ \mathbf{x\ 100} \qquad (4.2)$$

Where:

n is the number of HFCW effluent samples that were equal to or below the General Authorisations (discharge) limit for that parameter and n (tot) is the total number of samples analysed.

4.2.5.4.3 Statistical analyses

A non-parametric Spearman Rank Correlation test was performed to determine any correlation between COD and NO_3-N concentration in the HFCW effluent after the addition of the plant material at the inlet of the HFCW using GraphPad Prism 9 Software (Version 9.0.0, 2020). Confidence intervals were kept at 95%.

4.3 RESULTS

4.3.1 Daily flow

During March-June 2021, the total flow to the DEWATS ranged from 35.2-79.1 m^3 d^{-1} with an average flow of 42.7 (± 6.5) m^3 d^{-1}, of which 36.2% flowed into Train 1 feeding the hybrid CW system. The daily flow through Train 1 ranged from 12.1-38.1 m^3 d^{-1} with an average flow of 15.4 (± 3.7) m^3 d^{-1}. Based on the surface area of the VFCW being 96 m^2 (Arumugam and Buckley, 2020), the hydraulic loading rate (HLR) equated to the hybrid CW system was 0.16 m d^{-1}. After the VFCW, the HLR to the HFCW over the monitoring period was 0.23 m d^{-1}. Overall, the CW system operated within the design flow of 13.9 m^3 d^{-1} for 28.7% of the monitoring period (Figure 4-3). High flows were observed during heavy rainfall events (Figure 4-4).

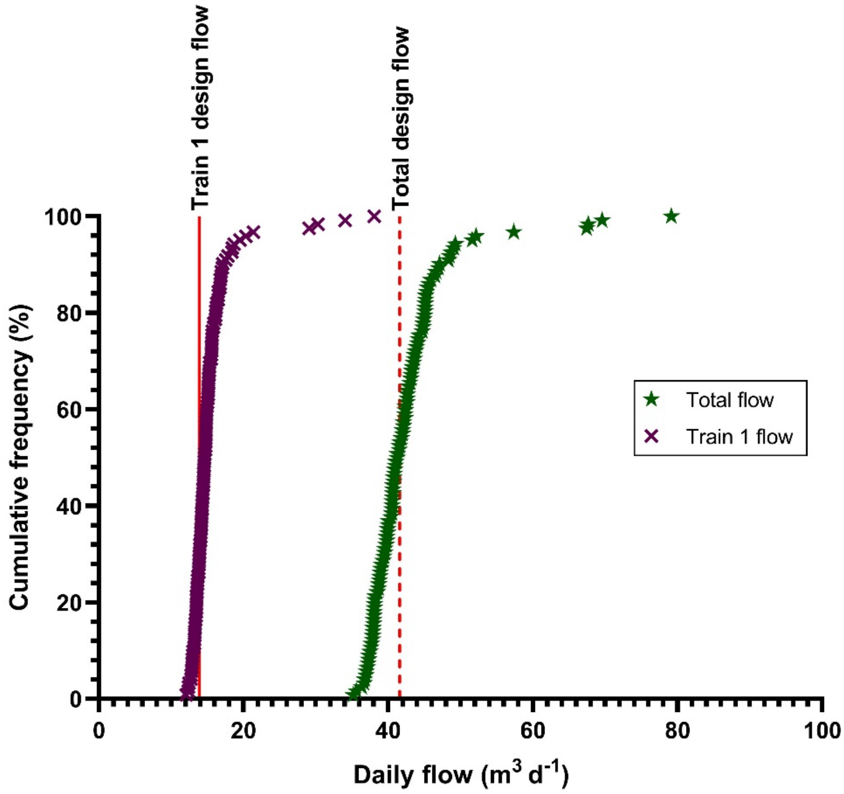


Figure 4-3: Cumulative frequency of the total daily flow to the DEWATS in relation to Train 1 flow

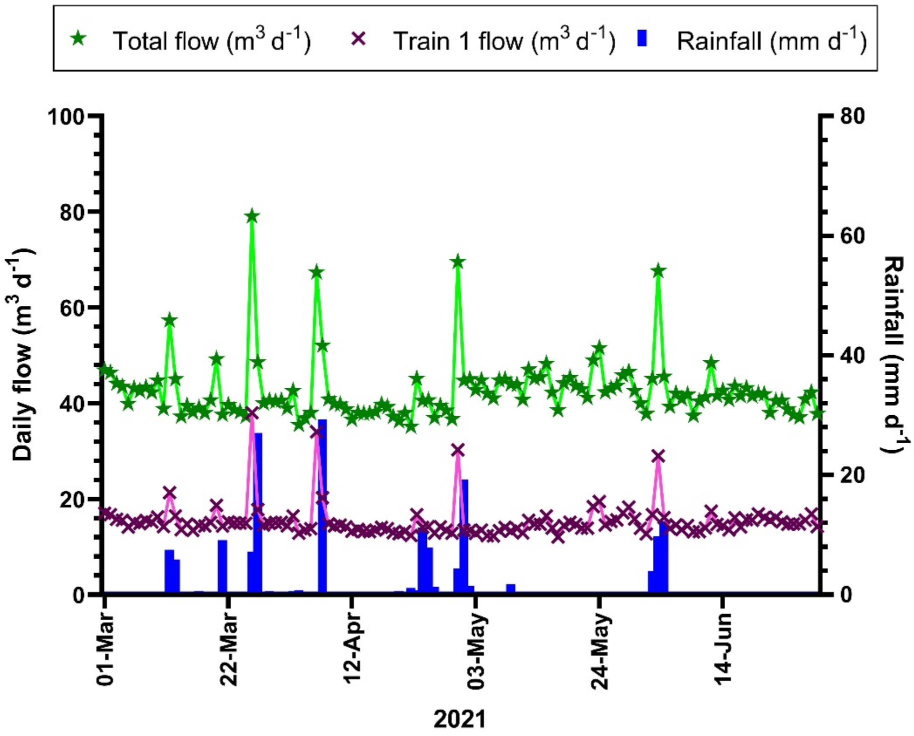


Figure 4-4: Total daily flow to the DEWATS and through Train 1 in relation to rainfall (March-June 2021)

4.3.2 Organic and nitrate-N loading to the HFCW

The COD concentration in the VFCW effluent ranged from 114-189 mg L^{-1}, with a mean concentration of 157.4 (± 30.7) mg L^{-1}. Based on the HLR of 0.23 m d^{-1}, this equated to an organic loading of an HLR of 36.7 g COD m^2 d^{-1}. The nitrate-N concentration ranged from 32.2-40 mg L^{-1} with a mean concentration of 37.4 (± 4.4) mg L^{-1}. Therefore, the nitrate-N loading to the HFCW was 8.6 g NO_3-N m^{-2} d^{-1}.

4.3.3 HFCW effluent quality

The HFCW effluent quality is presented in Table 4-3. The baseline (Day 1) COD concentration in the HFCW effluent was 72 mg L^{-1}, increasing by 860% on Day 29 (Week 4) (Figure 4-5a). There was a steady decline in COD concentration between Weeks 5-7, reaching 215 mg L^{-1} in Week 7 and then increasing to 432 mg L^{-1} in Week 10. In Week 12, the COD concentration was 218 mg L^{-1}. None of the HFCW effluent samples were compliant with the COD GA limit of 75 mg L^{-1}, but were 100% compliant for TSS and PO_4-P concentration, including pH and EC.

The NH_4-N concentration ranged from 1-16 mg L^{-1} during the monitoring period, with 76.9% of the samples achieving compliance with the GA limit of 6 mg L^{-1}. The baseline NO_3-N concentration was 34 mg L^{-1} (Figure 4-6). During the 13-week sampling period, there was no improvement in NO_3-N removal, and only a minor decline in NO_3-N concentration was noted after the addition of *A. donax.* The NO_3-N concentration ranged from 4.8-44.4 mg L^{-1} with a mean of 33.3 (± 10.4) mg L^{-1}. Only 7.7% of the samples (equating to a single sample) achieved compliance with the GA limit of 15 mg L^{-1}. High DO concentrations were measured in the HFCW effluent at 1.2 (± 1.4) mg L^{-1}.

Moreover, *E. coli* counts were high in the HFCW effluent, with only 16.7% of the samples achieving compliance with the GA limit of 1000 CFU 100 mL^{-1}.

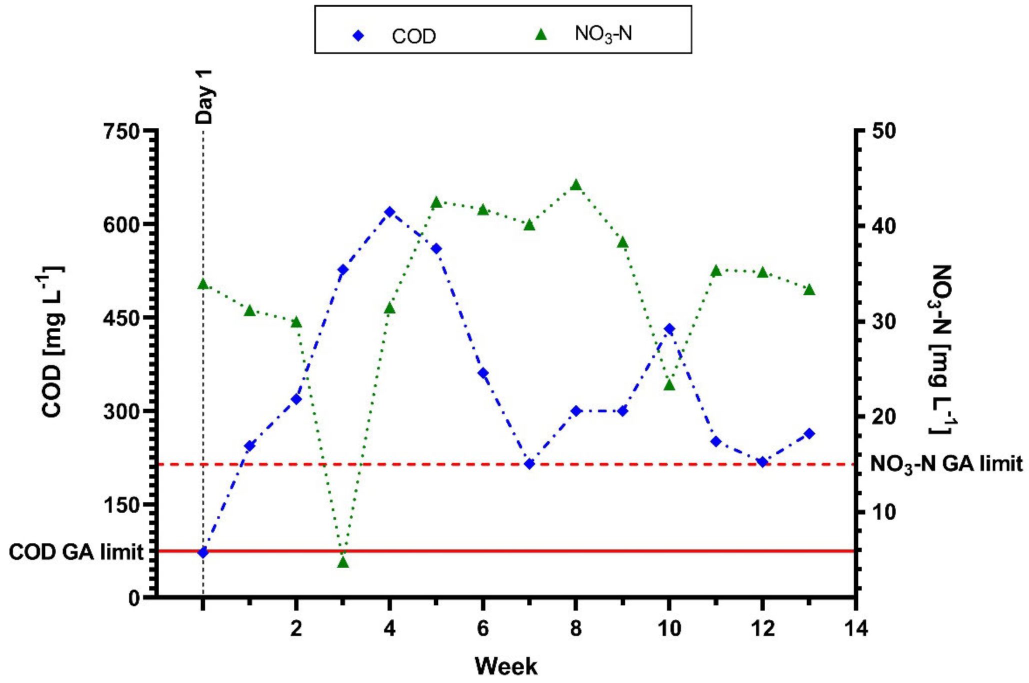
Table 4-3: Effluent quality from the horizontal flow constructed wetland after the addition of dried plant material of *Arundo donax* L. (March-June 2021)

	n	Range (Min-Max)	Mean ± Std Dev	GA limit	% Of samples achieving compliance
COD [mg L^{-1}]	13	214.7-620	354.8 (± 137.1)	75	0%
TSS [mg L^{-1}]	13	0-25	11.5 (± 8.3)	25	100%
NH_4-N [mg L^{-1}]	13	1-16	5.4 (± 4.5)	6	76.9%
NO_3-N [mg L^{-1}]	13	4.8-44.4	**33.3** (± 10.4)	15	7.7%
TN [mg L^{-1}]	13	16-47	38.5 (± 9.2)		
PO_4-P [mg L^{-1}]	13	3.3-6	4.4 (± 0.8)	10	100%
E. coli (CFU 100 mL^{-1})	13	0-1.2 x 10^4	**3.6 x 10^3** (± 3.3 x 10^4)	1000	16.7%
pH	13	6.8-7.1		5.5-9.5	100%
EC (mS m^{-1})	13	52.9-74.6	67.4 (± 6.9)	150	100%
ALKY [mg $CaCO_3$ L^{-1}]	9	25-65	42.8 (± 17.3)		
DO [mg L^{-1}]	4	0.3-3.2	1.2 (± 1.4)		

4.3.3.1 Correlation between COD and NO_3-N concentration

Figure 4-5b illustrates NO_3-N concentration as a function of COD concentration over the monitoring period. The Spearman Correlation Coefficient (r = -0.1816; p = 0.5498) indicating only a minor inverse correlation.

a.



b.

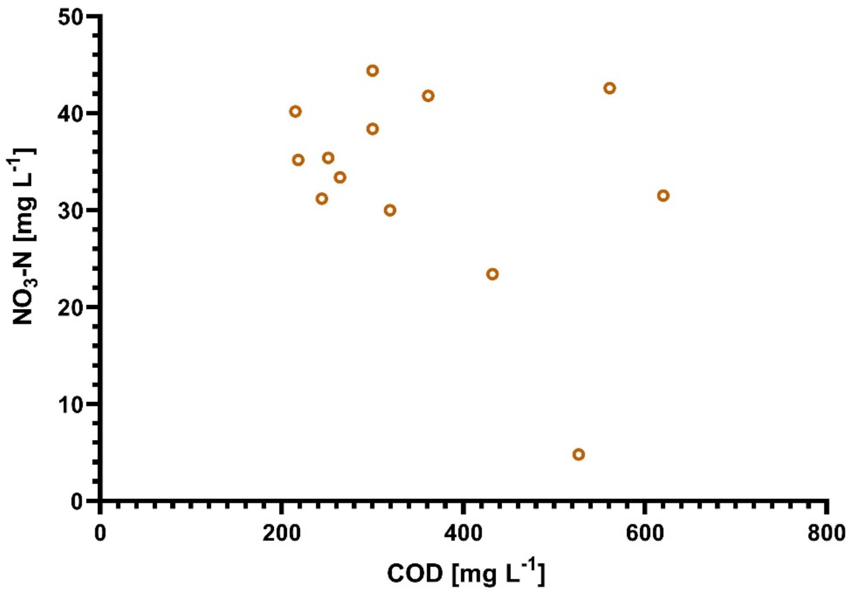


Figure 4-5: a. Change in total COD (COD) and nitrate-N (NO_3-N) in the horizontal flow constructed wetland (HFCW) effluent after addition of the dried plant material and b. NO_3-N as a function of COD concentration

4.3.3.2 Impact on total nitrogen removal

Figure 4-6 shows the change in ammonium-N, nitrate-N and total N concentration in the HFCW effluent over the monitoring period. It is evident that the major fraction of TN was NO_3-N. Only one out of 13 samples (Week 3) was compliant with the GA limit for NO_3-N. This was not due to inadequate nitrification, as the NH_4-N and TN concentrations were also low. It is possible that this was due to the diurnal flow rate and hydraulic loading to the DEWATS.

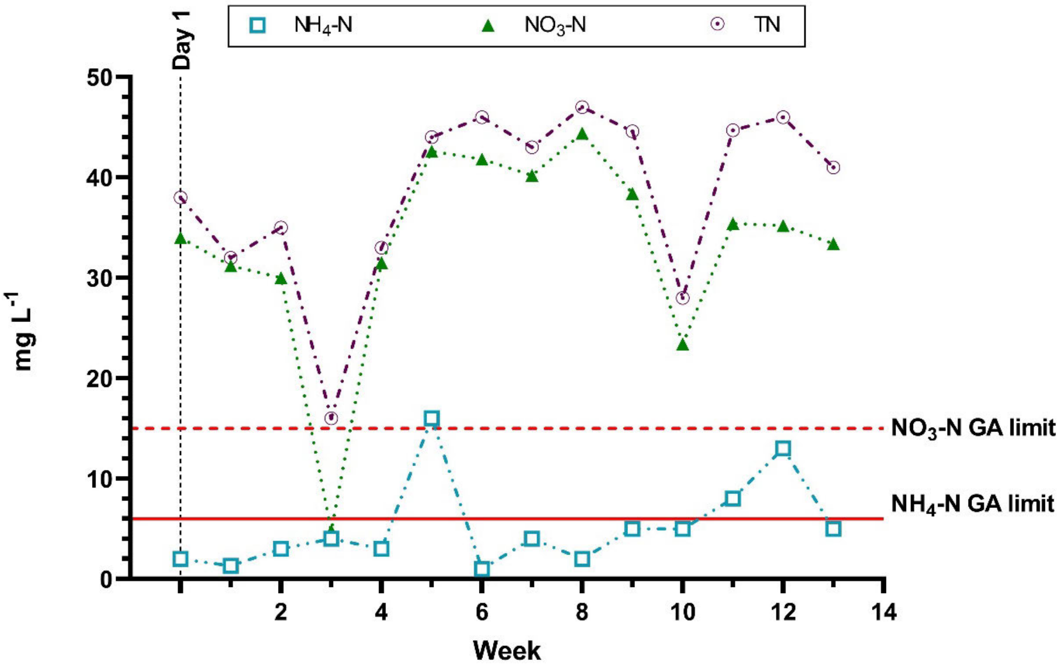


Figure 4-6: Change in ammonium-N, nitrate-N and total nitrogen concentrations after the addition of the dried plant material of *A. donax* at the inlet of the horizontal flow constructed wetland

4.4 DISCUSSION

The relatively immediate increase in COD concentration in the HFCW (within the first week after the application of the air-dried plant material of *Arundo donax* L. at the inlet the HFCW) indicated that natural anaerobic fermentation is possible and offers a simple technique to augment the carbon source in a HFCW under field conditions. The environmental conditions of the inlet combined with the plant material being added in waterlogged conditions promoted the conditions necessary for anaerobic fermentation, although at a much slower rate than being anaerobically fermented in the oven.

However, there was no substantial impact on the NO_3-N concentration (Figure 4-5a and b). The single sample which was compliant with the GA limit for NO_3-N was possibly due to the diurnal flow through the DEWATS and the raw wastewater quality rather than the increase in carbon availability (Figure 4-5a).

The rate limiting factor affecting removal of NO_3-N is assumed to be the high DO concentrations in the HFCW effluent (above 0.5 mg L^{-1}). While no surface water accumulation was observed on the HFCW during peak outflow rates, it is supposed that the high DO concentration could be due to the loosely packed, irregular-shaped, mixed gravel in the HFCW filter media. Sasse (1998) comments that mixed grain sizes in filter media results in unpredictable pore sizes and pore spaces. Therefore, it is probable that there are many small pore spaces within certain areas of the filter media, reducing the hydraulic conductivity of the gravel, thus resulting in temporary surface water accumulation. However, this assumption cannot be supported empirically without disturbing the media.

Due to the high availability of DO and observing the fluctuations in COD concentration after reaching its peak concentration in Week 4, it is most probable that facultative denitrifiers used the oxygen, rather than NO_3-N as the terminal electron acceptor in their respiration. This is further supported by the low alkalinity concentration in the HFCW effluent at 42.8 mg $CaCO_3$ L^{-1} (Table 4-3), below 100 mg $CaCO_3$ L^{-1} which Li and Irvin (2007) associates with insufficient denitrification. An investigation into the microbial community profiles of the HFCW may prove beneficial in understanding the microbiology within the HFCW and provide insight as to which are the dominant species.

In terms of overall compliance, the HFCW effluent achieved compliance for TSS, PO_4-P, pH, and EC. The number of samples achieving compliance for NH_4-N was 76.9%, while the lowest removal efficiency was observed for NO_3-N at 7.7%. Another poor performance indicator was the $\log_{10}$ reduction of *E. coli* where only 16.7% of the samples achieved compliance with GA limit of 1000 CFU 100 mL^{-1} (Table 4-3).

These results do not provide any data to support the hypothesis that the HFCW effluent will meet compliance for nitrate-N after the addition of pre-dried *A. donax* plant material at the inlet of the HFCW through natural anaerobic fermentation. Overall, the hybrid CW system, in its current design is not able to produce compliant effluent quality for safe discharge into a water resource and other CW designs need to be explored, particularly the two-stage VFCW.

Langergraber et al. (2014) evaluated a full-scale two-stage VFCW constructed for a hotel with a design loading of 2.5 m^3 d^{-1}. The average NO_3-N concentration in the effluent was 15.2 ± 6.7 mg L^{-1} during continuous operation (i.e., during the weekly operation of the hotel) and during weekend event loading (peak loads) the concentration was 15.9 ± 10 mg L^{-1}. However, each stage of the VFCW had only a 0.5 m filter layer and a 0.2 m drainage layer with a 0.1 m gravel layer on the surface. The top gravel layer may have inhibited the retention time within each stage of the two-stage VFCW, thus reducing the contact time of the wastewater with the microbial assemblages within the beds. Langergraber (2017) proposed an intermediate layer between the filter and drainage layers to maintain filter stability and prevent fine particles from entering the drainage layer below. This intermediate layer may aid in increasing hydraulic retention times within each stage thereby reducing the NO_3-N concentration in the final effluent. Moreover, two-stage down-flow VFCWs have not been employed for a community-based DEWATS and needs to be evaluated in the South African context.

4.5 CONCLUSION

Natural anaerobic fermentation of *Arundo donax* L., by adding the air-dried plant material at the inlet of the HFCW under the gravel media, provided a good source of available carbon, increasing the COD concentration in the final effluent by 30% after the first week. However, due to high DO concentrations within the HFCW (> 0.5 mg L^{-1}), nitrate-N removal was inhibited, and instead reduction of the available COD occurred as seen by its gradual reduction in the HFCW effluent over the monitoring period. The large aggregate media in the HFCW (50-80 mm) is a design limitation as it reduced the residence time of the wastewater within the HFCW. As a result, the HFCW was not able to produce compliant effluent for safe discharge into a water resource, as defined by South African General Authorisations limits for domestic wastewater.

CHAPTER 5. AN ASSESSMENT OF DIFFERENT CONFIGURATIONS OF PILOT-SCALE VERTICAL DOWN-FLOW CONSTRUCTED WETLANDS RECEIVING ANAEROBICALLY TREATED DOMESTIC WASTEWATER: A FOCUS ON NITROGEN REMOVAL

5.1 INTRODUCTION

Vertical flow constructed wetlands (VFCWs) have been historically favoured for the removal of ammonium (via biological nitrification), due to their greater oxygenation capacity compared to other CWs (Cooper, 2009; Vymazal, 2022). The most common type of VFCW is the intermittently loaded, down-flow VFCW with free drainage and sand media (Vymazal, 2022). However, good aeration limits denitrification and thus, total nitrogen removal is often poor in VFCWs (Kadlec and Wallace, 2009; Stefanakis et al., 2014). Denitrification, which is the reduction of nitrate-N to N_2, requires available carbon and the absence of oxygen (Kadlec and Wallace, 2009). In VFCWs, aerobic microbial biomass degrade the organic matter in the top 0.2 m of the media due to the high DO concentrations, reducing the available carbon for denitrification in the potential anoxic zones toward the bottom of the bed (Stefanakis et al., 2014).

On the east coast of South Africa, the landscape is very undulating, allowing for VFCWs with extended depths. Typically, VFCWs can be designed with depths up to 1.2 m, although some VFCWs with sand media can be constructed with depths up to 3 m (Gutterer et al., 2009). Langergraber et al. (2009) and Langergraber et al. (2014) used 2-3.2 mm coarse sand in the first stage of a two-stage VFCW for domestic wastewater treatment at an organic loading of 80 g COD m^{-2} d^{-1}, to limit complete OM degradation. Therefore, there would be available carbon in the effluent to the second stage VFCW with fine sand to fine gravel (0.06-4 mm) for denitrification (Langergraber et al., 2009; Langergraber et al., 2014). Due to the topography in eThekwini, single-stage VFCWs with extended filter layers may achieve the similar effluent quality to two-stage VFCWs which require a greater land surface area.

Both VFCW configurations (single stage with extended filter layers and the two-stage VFCW) need to be investigated in the South African context. If VFCWs are to be applied in decentralised solutions for domestic wastewater, then they need to operate without electricity or chemical input. However the final effluent quality must meet the regulatory requirements for safe discharge (DWS, 2013). Moreover, if the hydraulic loading to the single stage VFCW with an extended filter layer is increased, then this will increase the total organic loading to the system. It was predicted that the increase in filter depth will result in compaction of the media toward the bottom of the VFCW.

This study therefore tests the following hypotheses:

1. The single-stage VFCW with an extended filter depth will achieve the same, if not better effluent quality than a two-stage VFCW.
2. Increasing the hydraulic loading rate will increase carbon availability in the incoming wastewater and thus, improve nitrate-N removal across the pilot-scale VFCWs (single-stage and two-stage systems).

5.2 MATERIALS AND METHODS

5.2.1 Site location

The pilot-scale VFCWs were constructed at a demonstration-scale decentralised wastewater treatment system (DEWATS) in eThekwini treating raw domestic wastewater. The DEWATS design is modular, consisting of a settler, anaerobic baffled reactor (ABR), and an anaerobic filter (AF) for primary and secondary treatment (Pillay et al., 2013). The anaerobically treated wastewater is high in ammonium (70-90 mg L^{-1}) (Arumugam and Buckley, 2020). The flow through the DEWATS is divided into three Trains, of which, flow from Train 1 is treated in a hybrid subsurface flow CW system consisting of a single-stage VFCW operating in series with a horizontal flow CW.

5.2.2 Design and construction

5.2.2.1 Design

The main design parameters used in the pilot-scale VFCWs were media type, depth, and hydraulic loading rate based on recommendations from literature with minor adaptions.

5.2.2.2 Civil works

The above-ground pilot-scale VFCWs were constructed from 750 mm (dia.) concrete manhole rings, 0.25 m in height. The single-stage VFCWs are 1.5 m and 2 m in height with a working depth of 1 m and 1.5 m, respectively. The free board (0.5 m) at the top ensured no spillage of the AF effluent during each dosing. Each stage of the two-stage VFCWs is 1.5 m in height with a working depth of 1 m (Figure 5-2). Construction of the columns was completed by the end of December 2017.

For the two-stage VFCW, stainless steel (Type 304) drainage sumps with a diameter of 0.3 m and depth of 1.3 m were retrofitted into the ground adjacent to the outflow pipe of the first stage VFCW. The capacity of each sump is 92 L. The outflow pipe was extended so that the drainage from a specific two-stage VFCW could flow directly into the corresponding sump (Appendix F).

5.2.2.3 Media and depth

The media gradation of the pilot-scale VFCWs was adapted from personal communication with Prof. Langergraber[††], based on the successful full-scale two-stage VFCW application in Austria (Langergraber et al. 2014) and locally available sieved media in eThekwini. The filter layer of the pilot-scale VFCWs consisted of 2-3 mm coarse sand with a drainage layer of 16-32 mm gravel. The purpose of this gradation was to prevent complete organic degradation in the filter layer and the possibility of clogging. Langergraber (2017) suggested that an intermediate layer of 4-8 mm gravel would maintain filter stability by inhibiting the washing out of any fine particles in the above filter layer into the drainage layer. Therefore, linings or geotextiles within each column was not needed. In the second stage of the two-stage VFCW, fine to coarse sand (0.5-2 mm) was used in the filter media to improve the hydraulic retention time. The media gradation and depth of each column is given in Table 5-1. When the media was layered, it was compacted so as not cause large void spaces between the grains.

Since the filter depth of the first stage of the Austrian two-stage VFCW was 0.5 m, a filter depth of 1 m was used in this study to determine the effect of an extended filter depth on the nitrogen removal capacity of the VFCW. None of the VFCWs were impounded due to increased depth of the filter and drainage layers compared to Langergraber et al. (2014).

[††] BOKU University, Vienna

5.2.2.4 Feed

The VFCWs were loaded with AF effluent from Trains 2 and 3 of the ABR. The combined AF effluent from both streams were collected in a wet membrane chamber. A Zilmet V180F submersible pump pumped the AF effluent to a 1000 L feeding tank which housed the feed to the pilot-scale VFCWs. The drainage from each VFCW was collected in a channel which was piped to the DEWATS HFCW sampling sump by gravitated flow. Thus, the demonstration-scale CWs and pilot-scale VFCWs could not be monitored at the same time. The final effluent then entered the trunk sewer line (Figure 5-1). No effluent was discharged into the receiving environment.

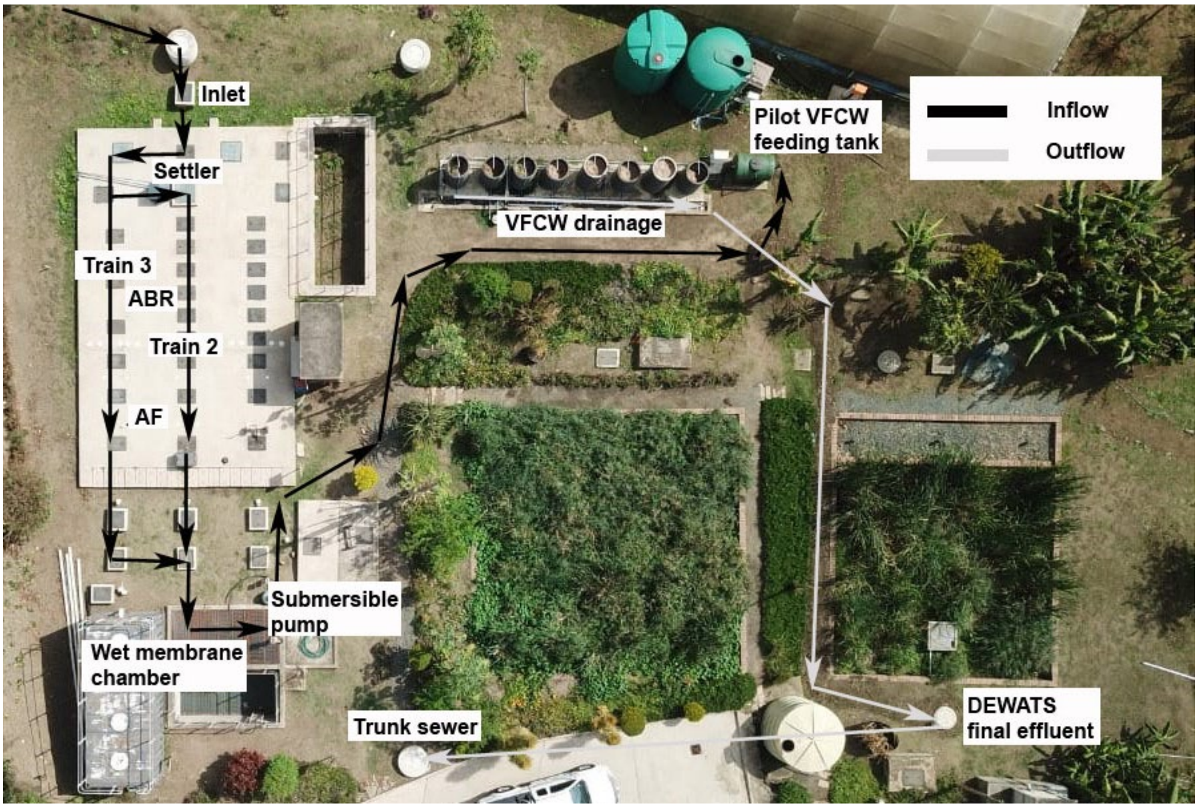


Figure 5-1: Flow patterns from the inlet of the DEWATS to the pilot vertical down-flow constructed wetlands (black arrows) and the outflow to the trunk sewer (grey arrows)

Table 5-1: Media gradation in each pilot-scale vertical down-flow constructed wetlands

	Depth (m) and grain size of media (mm)		
	Filter layer	**Intermediate layer**	**Drainage layer**
Single-stage VFCWs			
CW 1 and 3*	0.5 m (2-3 mm)	0.2 m (4-8 mm)	0.3 m (16-30 mm)
CW 2 and 4	1 m (2-3 mm)		
Two-stage VFCWs			
CW 5a and 6a	0.5 m (2-3 mm)	0.2 m (4-8 mm)	0.3 m (16-30 mm)
CW 5b and 6b	0.6 m (0.5-2 mm)	0.1 m (4-8 mm)	

* The single-stage VFCW with 0.5 m filter depth was used to determine if depth had any effect of the nitrogen removal capacity of the VFCW as well as serve as a representation of the first stage VFCW in the two-stage design.

5.2.2.5 Instrumentation and hydraulic loading

Since the pilot-scale VFCWs were above ground with no gradient for gravitated flow, the feeding mechanism for the pilot-scale VFCWs was designed to be automated using a programmable logic controller (PLC) (National Instruments).

The feed tank was instrumented with a capacitive level sensor (KQ6007) calibrated to switch on the submersible pump in the wet membrane chamber and fill the feed tank after each feeding. Thus, for every feeding cycle, the feed tank would be refilled and a constant head was maintained for each cycle.

Langergraber[‡‡] (pers. comm.) recommended that when using coarse sand media (2-3 mm), a hydraulic loading rate of 0.19 m d^{-1} must be applied to reduce the occurrence of clogging. Therefore, the design HLR (q) for each VFCW design was 0.19 m d^{-1}. A single Rain bird solenoid valve was connected to the inlet pipe of each VFCW to ensure equal volume per dose as controlled by the PLC. During each feeding cycle, a single dose of 16.5 L of AF effluent from the feed tank was pumped to each single stage, and first stage of the two-stage VFCW in series (i.e., CW1 dosed first followed by CW2 and so on) using a pressurized pump (Grundfos

‡‡ BOKU University, Austria

JPC 3-PT). Based on the dose volume, each VFCW (besides the second stage of the VFCW) was dosed every 4:40 (hh:mm) in a 24-hour period (i.e., five doses per day).

For the two-stage VFCWs, an HQB fountain pump with an attached Viyilant® (B-01) float switch was inserted in the sump collecting the drainage from each first stage VFCW. Based on the height of the pump, each sump had a working depth of 0.7 m. Once the drainage volume reached the desired water level, the pump, activated by the float switch, would pump out a volume of 16.5 L to the second stage of the two-stage VFCW. The instrumentation was installed and commissioned by the end of July 2019.

A process flow diagram of the pilot-scale VFCWs is given in Figure 5-3. The 10 m^3 JoJo tank houses the feed for agricultural trials on site (i.e., the use of AF effluent for irrigation). Once the feed tank for the pilot-scale VFCWs is filled after each dosing cycle, the AF effluent from Trains 2 and 3 then fills up the JoJo tank if empty.

a.

b.

Figure 5-2: Pilot-scale vertical down-flow constructed wetlands a. Annotated side-view; b. front view

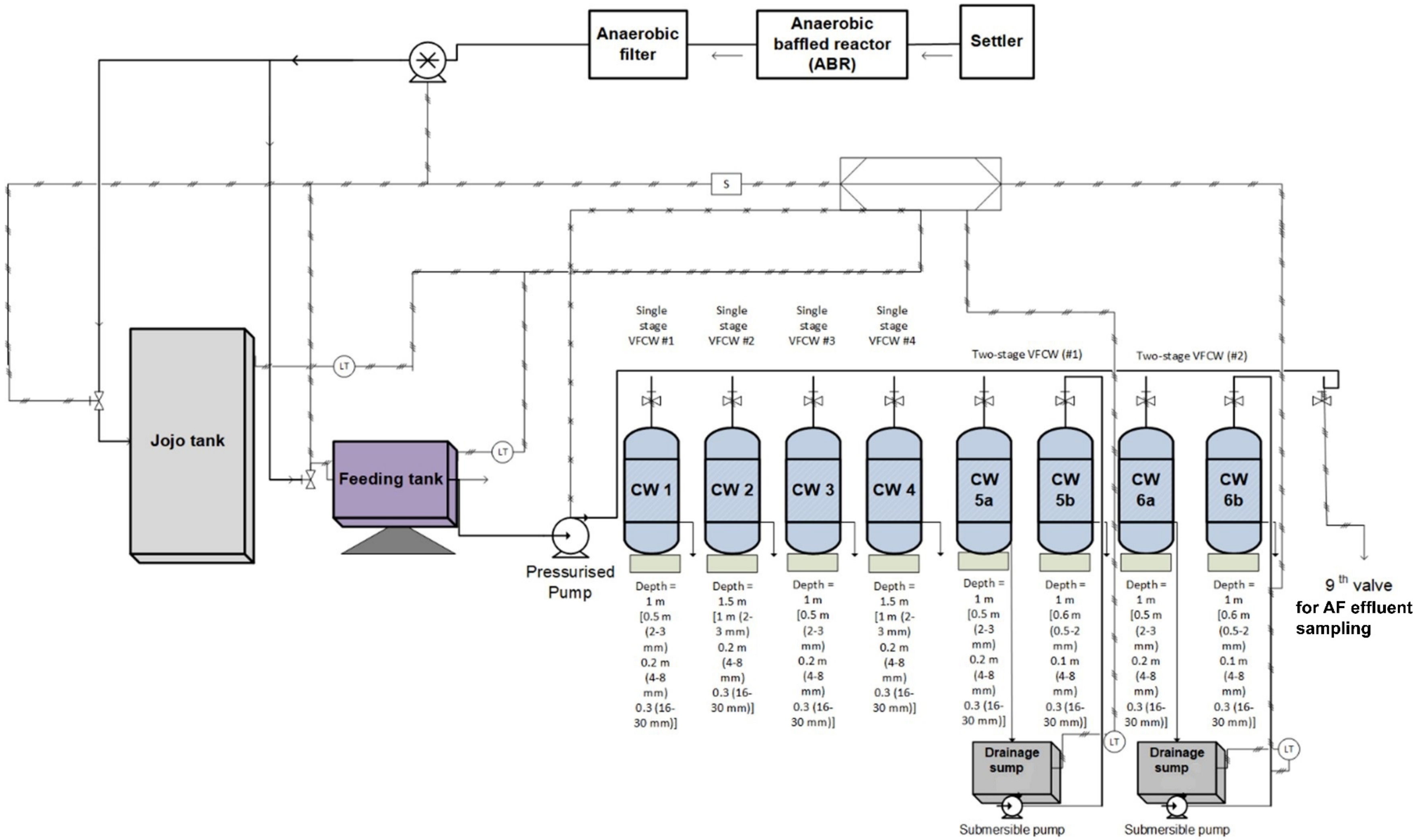


Figure 5-3: Process flow diagram of the pilot-scale vertical down-flow constructed wetlands receiving anaerobically treated domestic effluent.

5.2.3 Start-up

The columns were left unplanted, to reduce the start-up time, and fed with the combined AF effluent from Trains 2 and 3 for eight weeks, undisturbed. The start-up performance of each configuration was assessed from September to October 2019.

5.2.3.1 Flow monitoring

The electromagnetic flow meters (SAFMAG) installed in the sump following the AF of Trains 2 and 3 measured the flow rate per second which was used to calculate the daily flow. This data was logged in 10-minute intervals using an Omniflex Teleterm M3e Data logger. It must be noted that the flow through Train 2 and 3 was monitored only to understand the AF effluent quality and does not relate to the HLR of the pilot-scale VFCWs.

5.2.3.2 Sampling

In week 9, a 1 L composite sample from the drainage of a single dose was collected from CWs 1-4, 5b and 6b for analysis. The quality of the effluent from CW 5a and 6a was assumed to be the same as CW 1 and 3 due to the identical depth, HLR and media gradation. Thus, no sampling occurred for CW 5a and 6a. Sampling was done weekly until Week 15 (end of October 2019).

5.2.3.3 Chemical and microbiological analysis

A Jenway 3540 pH & Conductivity meter was used to measure pH and electrical conductivity (EC). Total COD (COD), ammonium-N (NH_4-N), nitrate-N (NO_3-N) and total nitrogen (TN) and orthophosphate-P (PO_4-P) was measured with a Merck NOVA 60 Spectroquant. The sum of the soluble and particulate fractions of the biodegradable COD (bCOD) from the COD concentration was measured by the Oxygen Uptake Rate (OUR) using a BM-Evo Respirometer as described in Section 3.2.2.4.1. Only during the start-up performance was the bCOD measured after which the instrument was down due to repairs.

All determinants measured with the Spectroquant were prepared and analysed according to the standard operating procedure (SOP) supplied with the test kit by Merck. Total suspended solids were measured according to Standard methods (2540 D) (APHA 2017). All samples, except for COD and TN, were filtered (using a Whatman 1.2 µm pore size filter paper) and the filtrate was prepared for analysis to avoid TSS interference on the spectrophotometer.

Indicator bacteria, *E. coli*, were measured using the Merck Petrifilm 3M plate and recorded as colony forming units (CFU) per 1 mL converted to CFU 100 mL^{-1}.

All analyses, except for the bCOD, were carried out in the on-site laboratory.

5.2.4 Shut down and resumption of flow

After the start-up performance assessment, the pilot-scale VFCWs were left unfed for the duration of 2020. This was due to the National COVID-19 lockdown restrictions and the inaccessibility to the site (March-August 2020). Once access resumed, the VFCWs required re-water proofing which was completed during the latter part of 2020. The VFCWs were each then planted with four one-year old plants of *Typha capensis* gathered within a 10 km radius of the site. During April 2021, electrical upgrades were undertaken on site due to damaged cabling and upgrades to the distribution boards after which the instrumentation was reactivated and the VFCWs were pulse fed as per the design q (0.19 m d^{-1}) for eight weeks, undisturbed.

5.2.5 Experimental operation

After 8 weeks of continuous operation at the design HLR (q) of 0.19 m d^{-1}, the HLR was amended for CW 3, 4 and 6 as per Table 5-2 to assess the impact of higher hydraulic loading on nitrate-N removal and overall performance. An HLR of 0.57 m d^{-1} ($3q$) was applied which equated to three consecutive doses of 16.5 L for each loading interval.

Table 5-2: Hydraulic loading of the pilot-scale VFCWs (May-July 2021)

HLR (m d^{-1})	**Single-stage VFCWs**	**Single-stage VFCWs with extended filter depth**	**Two-stage VFCWs**
0.19 (*q*)	CW 1	CW 2	CW 5a
0.57 (3*q*)	CW 3	CW 4	CW 6a

5.2.6 Performance monitoring

During this phase of the research, flow and effluent monitoring were performed as per the start-up phase described in Sections 5.2.2.2-5.2.2.3, except for bCOD, which could not be measured[§§].

5.2.6.1 Loading rate

The organic and nitrogen loading rate to each VFCW was calculated by Equation 5.1.

$$\textbf{Loading rate}\ (\mathbf{g.m^{-2}.d^{-1}}) = \mathbf{q}\ \times \mathbf{C_i} \tag{5.1}$$

Where:

q is the hydraulic loading rate (m d^{-1}) to the CWs and Ci is the concentration of the AF effluent (mg L^{-1}).

5.2.6.2 Removal efficiency

Percentage removal (removal efficiency) of each pollutant was measured by Equation 5.2.

$$\textbf{Removal efficiency (\%)} = \left(\frac{\mathbf{Ci - Co}}{\mathbf{Ci}}\right) \times \mathbf{100} \tag{5.2}$$

Where:

[§§] The bCOD fraction could not measured due to down-time of the instrument and inability to service it as a result of the COVID-19 pandemic.

Ci is the concentration of the AF effluent (mg L^{-1}) and Co is the concentration of the effluent from each respective VFCW (mg L^{-1}).

5.2.6.3 Comparison of the VFCW effluent quality in relation to the General Authorisations

The % of VFCW effluent samples achieving compliance was measured using Equation 5.3.

$$\textbf{\% of Samples achieving Compliance} = \frac{\mathbf{n}}{\mathbf{n\ (tot)}}\ \mathbf{x\ 100} \qquad (5.3)$$

Where:

n is the number of VFCW effluent samples that were equal to or below the GA limit for that parameter and n (tot) is the total number of samples analysed.

5.3 RESULTS

5.3.1 Start-up performance

5.3.1.1 Daily flow through Trains 2 and 3

During the start-up performance, the total flow to the DEWATS ranged from 6.4 to 99.5 $m^3\ d^{-1}$ with an average of 32.1 (± 9) $m^3\ d^{-1}$. The flow diverted into Train 2 and 3 was 6.6% and 52.7%, respectively. The combined flow from Trains 2 and 3 entering the wet membrane chamber ranged from 4.9-42.2 $m^3\ d^{-1}$ with an average of 19 (± 3.9) $m^3\ d^{-1}$. Peaks in flow were observed during heavy rainfall events (Figure 5-4).

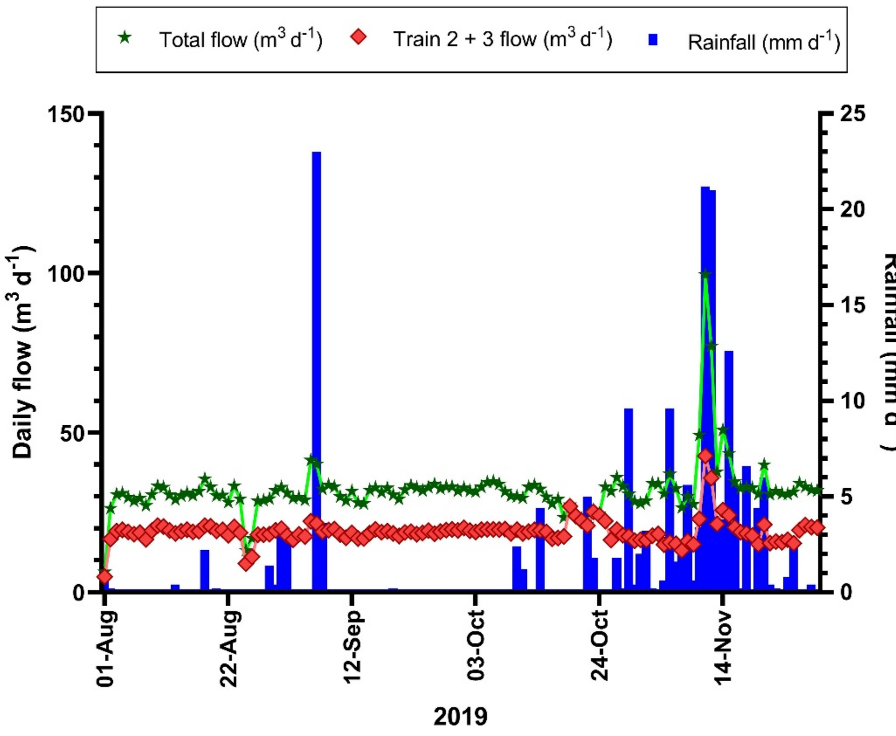


Figure 5-4: Total daily flow (m^3 d^{-1}) to the DEWATS and through Trains 2 + 3 (combined flow) feeding the pilot VFCWs in relation to rainfall (mm d^{-1}) (August-November 2019)

5.3.1.2 AF effluent quality

The combined AF effluent quality is presented in Table 5-3. The mean COD concentration was 238.6 (± 38.5) mg L^{-1}, which equated to a mass loading of 45.3 g COD m^{-2} d^{-1}. The biodegradable fraction (inclusive of the soluble and particulate fractions) was estimated to be 88.8%. The mean TSS concentration was 26.7 (± 14.2) mg L^{-1}.

The major TN fraction was NH_4-N at 86.4%. The mean concentration of NH_4-N was 64.7 (± 39) mg L^{-1}. As expected, NO_3-N was not detected in the AF effluent. Interestingly, no *E. coli* were detected in the AF effluent, potentially due to die-off as a result of storage in the feed tank.

5.3.1.3 CW effluent quality

The final effluent quality from each pilot-scale VFCW is presented in Table 5-3. Sampling was not possible on a weekly basis for each VFCW due to gravel media entering the outlet pipe which resulted in backflow of effluent. Thus, the number of samples analysed for each VFCW differed across the start-up period.

Overall, the average removal efficiency of COD in the single-stage VFCW (0.5 m filter depth), single-stage VFCW (1 m filter depth) and the two-stage VFCW was 17.1%, 17.1% and 56.3% respectively. Overall, the two-stage VFCWs indicated the higher removal efficiencies for TSS, NH_4-N (Figure 5-5), TN (Figure 5-7) and PO_4-P removal. Figure 5-6 shows that NO_3-N is highest in the two-stage VFCWs indicating almost full nitrification.

It was difficult to estimate the *E. coli* removal considering that the AF effluent indicated an absence of *E. coli*. However, *E. coli* were found in the VFCW effluent of some systems. This was possibly due to wash out during heavy rainfall events during the monitoring period from previous loadings of the first eight weeks of operation before monitoring.

Table 5-3: Final effluent quality during the start-up performance in comparison to the anaerobically treated domestic wastewater (AF effluent) from Trains 2 and 3 (Weeks 9-15)

HLR = 0.19 m d^{-1}			**Single-stage VFCWs**								**Two-stage VFCW**			
	Media		Filter depth: 0.5 m (2-3 mm)				Filter depth: 1 m (2-3 mm)				1st stage Filter depth: 0.5 m (2-3 mm) 2nd stage Filter depth: 0.6 m (0.5-2 mm)			
	AF effluent		CW 1		CW 3		CW 2		CW 4		CW 5b (2nd stage)		CW 6b (2nd stage)	
	n	Mean (± Std Dev) / Min-Max	n	Mean (± Std Dev) / Min-Max	n	Mean (± Std Dev) / Min-Max	n	Mean (± Std Dev) / Min-Max	n	Mean (± Std Dev) / Min-Max	n	Mean (± Std Dev) / Min-Max	n	Mean (± Std Dev) / Min-Max
COD [mg L^{-1}]	7	238.6 (± 38.5)	7	195.5 (± 31.2)	7	200.1 (± 25.9)	7	182.8 (± 29.2)	7	212.8 (± 33.3)	6	97.7 (± 25.9)	4	111.5 (± 33.8)
Removal efficiency %				18.1%		16.1%		23.4%		10.8%		59.1%		53.4%
bCOD [mg L^{-1}]	3	211.9	3	129.4	3	98	DNM				3	39	3	60
Removal efficiency %				25.5%		53.8%						81.6%		71.7%
TSS [mg L^{-1}]	6	26.7 (± 14.2)	6	14.2 (± 3.3)	6	13.2 (± 2.8)	6	4.7 (± 2.7)	6	13.3 (± 2.3)	5	0 (± 0)	3	0 (± 0)
Removal efficiency %				46.8%		50.6%		82.4%		50.2%		100%		100%
NH_4-N [mg L^{-1}]	7	64.7 (± 39)	7	41.8 (± 2.8)	7	43.1 (2.4)	7	62.2 (± 6)	7	55.4 (± 4.4)	6	4.9 (± 2.6)	4	10.8 (± 2.1)
Removal efficiency %				35.4%		33.4%		3.9%		14.4%		92.4%		83.3%
NO_3-N [mg L^{-1}]	7	Not detected	7	20.1 (± 1.9)	7	19.1 (± 3.2)	4	0.6 (± 0.6)	6	4 (± 2.4)	6	40.6 (± 7.4)	4	39.1 (± 6.3)
TN [mg L^{-1}]	7	74.9 (± 3.1)	7	77.9 (± 1.5)	7	78.6 (± 2.1)	7	72.4 (± 3.6)	7	73.8 (± 4.7)	6	57.7 (± 5.7)	4	61.3 (± 5.4)
Removal efficiency %				-4%		-5%		3.3%		1.5%		23%		18.2%
PO_4-P [mg L^{-1}]	6	8.3 (± 0.8)	6	8.7 (± 0.4)	6	10 (± 2.1)	6	9.4 (± 2.1)	6	10 (± 0.8)	5	5.5 (± 0.3)	3	5.3 (± 0.2)
Removal efficiency %				-4.8%		-20.5%		13.3%		-20.5%		33.7%		36.1%
E. coli (CFU 100 mL^{-1})	6	0 (± 0)	6	24 (± 42)	6	1.2 (± 2.4)	6	7.7 (± 16)	6	0 (± 0)	5	1 (± 1)	3	0 (± 0)
pH	7	7.5-7.7	7	7.5-7.8	7	7.5-7.7	7	7.2-8.5	7	7.1-7.4	6	6.8-7.3	4	7-7.1
EC (mS m^{-1})	7	113.4 (± 4.4)	7	106 (± 3.4)	7	105.2 (± 4.2)	7	114.2 (± 3.5)	7	112.3 (± 3.7)	6	74.2 (± 3.2)	4	71 (± 3.8)
ALKY [mg $CaCO_3$ L^{-1}]	7	6.1 (± 0.7)	7	4.5 (± 0.2)	7	4.4 (± 0.4)	7	7.1 (± 0.4)	7	6.6 (± 0.5)	6	0.9 (± 0.4)	4	1.3 (± 0.2)

DNM = Did Not Measure; Negative removal efficiencies indicate an increase in concentration due to possible wash out, and are thus, regarded as 0% removal.

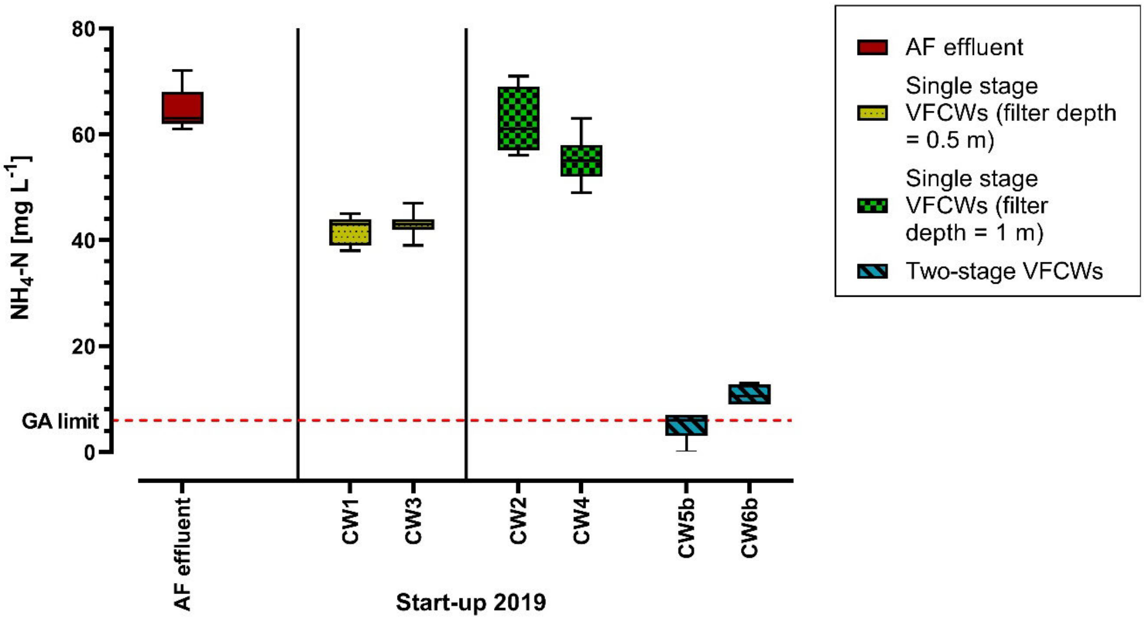


Figure 5-5: Comparison of ammonium-N concentration in the effluent of each pilot-scale VFCW in relation to the AF effluent (Trains 2 and 3) during the start-up performance. The whiskers represent the minimum and maximum concentrations while the boxes represent the 25-75% range of data

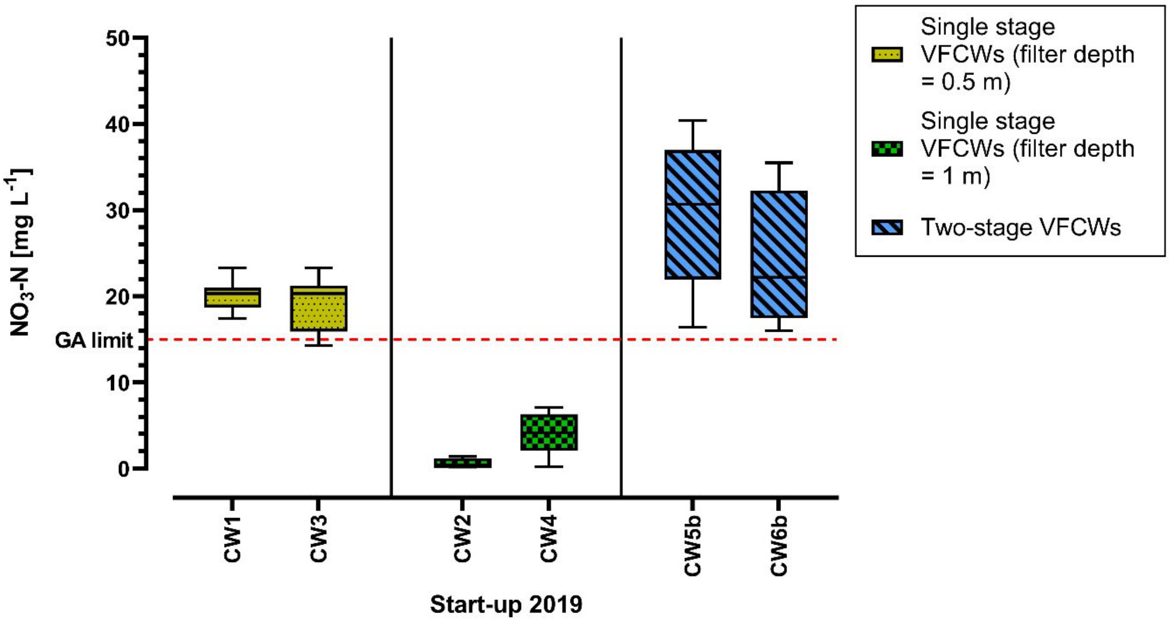


Figure 5-6: Comparison of nitrate-N concentration in the effluent of each pilot vertical down-flow constructed wetland during the start-up performance. The whiskers represent the minimum and maximum concentrations while the boxes represent the 25-75% range of data

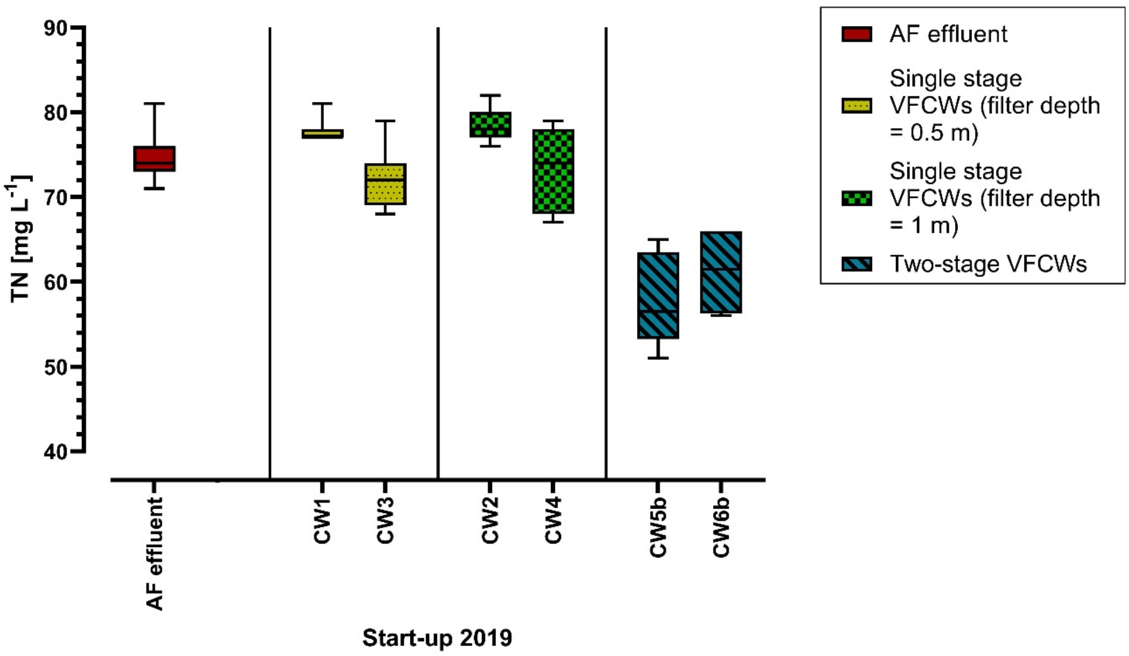


Figure 5-7: Comparison of total nitrogen concentration in the effluent of each pilot-scale VFCW in relation to the AF effluent (Trains 2 and 3) during the start-up performance (2019). The whiskers represent the minimum and maximum concentrations while the boxes represent the 25-75% range of data

5.3.2 Impact of higher loading on performance

5.3.2.1 Daily flow

The total flow into the DEWATS ranged from 23.2-67.7 m^3 d^{-1} with an average flow of 38.7 (± 5.7) m^3 d^{-1} over the monitoring period. The cumulative flow through Trains 2 and 3 feeding the pilot-scale VFCWs ranged from 15.3-38.6 m^3 d^{-1} with an average flow of 23.8 (± 3.7) m^3 d^{-1} equating to 61.6% of the total flow into the DEWATS, similar to the start-up performance evaluation period (Figure 5-8).

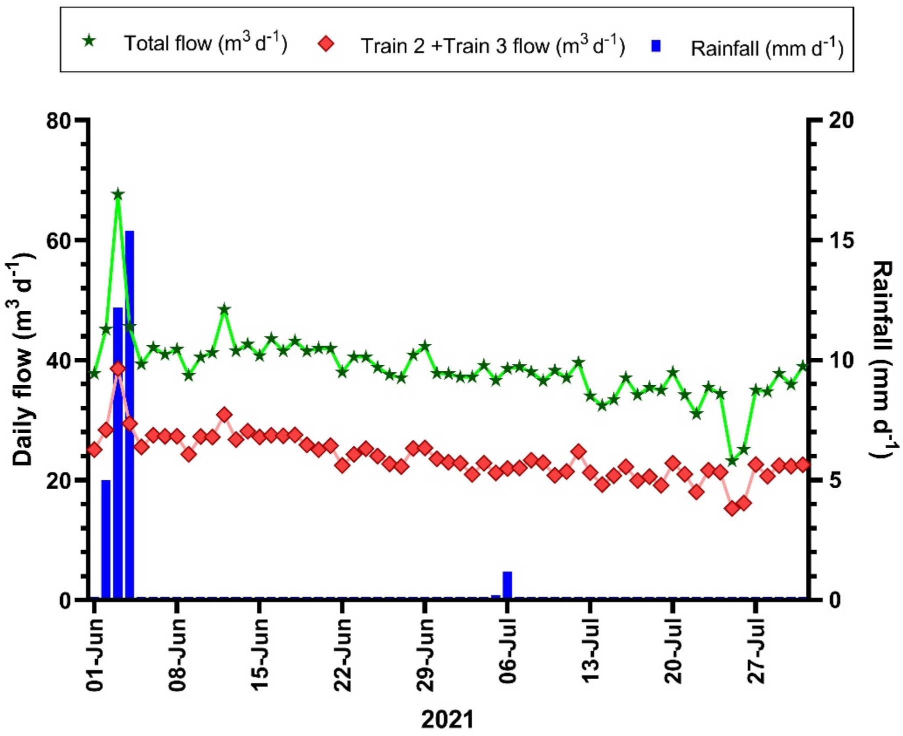


Figure 5-8: Total daily flow (m^3 d^{-1}) to the DEWATS and through Trains 2 + 3 (combined flow) feeding the pilot-scale VFCWs in relation to rainfall (mm d^{-1}) (June-July 2021)

5.3.2.2 AF effluent quality (Trains 2 and 3)

The AF effluent quality is presented in Table 5-4. Compared to the start-up performance in 2019, the COD, TSS, PO_4-P and *E. coli* loading was substantially higher during this sampling period. The mean COD concentration was 590 (± 122) mg L^{-1} which equated to a mass loading of 112.1 g COD m^{-2} d^{-1} in the VFCWs operating at the design HLR and 336.3 g COD m^{-2} d^{-1} at 3*q*. The bCOD fraction was unknown***. The mean TSS concentration was 61 (± 20.2) mg L^{-1}.

The NH_4-N was the highest of the TN fraction equating to 85.4% at a mean concentration of 69 (± 5) mg L^{-1} while NO_3-N was not detected in the AF effluent. The mean PO_4-P concentration was 13.8 (± 0.8) mg L^{-1}. Unlike during the start-up performance, the average *E. coli* colony forming units were 1.1 x 10^6 (± 6.4 x 10^5) CFU 100 mL^{-1}.

*** The instrument was out of service at the time of sampling.

5.3.2.3 CW effluent quality

A comparison of VFCW effluent quality from the different designs is shown in Table 5-4 (more detailed data is presented in Appendix G and Appendix H). The highest removal efficiencies of each parameter measured were noted in the two stage VFCWs, while the lowest removal efficiencies were observed in the single-stage design without an extended filter depth, inclusive of the nitrogen species (Figure 5-9 and Figure 5-11).

5.3.2.3.1 Single-stage (0.5 m filter depth) CW1 vs. CW3

Overall, removal of COD, NH_4-N, TN, PO_4-P and *E. coli* was higher at the design HLR (CW1). Removal of TSS was higher when operating at 3*q*.

5.3.2.3.2 Single-stage (1 m filter depth) CW2 vs. CW4

At design *q,* CW2 generally performed better than CW4 which operated at 3*q*.

5.3.2.3.3 Two-stage CW5 vs. CW6

Similarly, the two-stage VFCW at design HLR (*q*) at 0.19 m d^{-1} performed marginally better than the two-stage VFCW operating at 3*q* at 0.57 m d^{-1}. Notable treatment differences were observed for NO_3-N removal. It appears that at design *q*, partial NO_3-N removal is possible as seen with the reduction in TN concentration (refer to Appendix G and Appendix H).

Table 5-4: Impact of higher hydraulic loading (*q* verses 3*q*) on performance of the pilot-scale vertical down-flow constructed wetlands treating anaerobically treated wastewater

			Single-stage VFCWs								**Two-stage VFCW**			
	Media		Filter depth: 0.5 m (2-3 mm)				Filter depth: 1 m (2-3 mm)				1^{st} stage Filter depth: 0.5 m (2-3 mm) 2^{nd} stage Filter depth: 0.6 m (0.5-2 mm)			
	AF effluent		CW 1 (*q* or 0.19 m d^{-1})		CW 3 (3*q* or 0.57 m d^{-1})		CW 2 (*q* or 0.19 m d^{-1})		CW 4 (3*q* or 0.57 m d^{-1})		CW 5b (*q* or 0.19 m d^{-1})		CW 6b (3*q* or 0.57 m d^{-1})	
	n	Mean (± Std Dev) / Min-Max	n	Mean (± Std Dev) / Min-Max	n	Mean (± Std Dev) / Min-Max	n	Mean (± Std Dev) / Min-Max	n	Mean (± Std Dev) / Min-Max	n	Mean (± Std Dev) / Min-Max	n	Mean (± Std Dev) / Min-Max
COD [mg L^{-1}]	6	590 (± 122)	6	407.6 (± 92.9)	6	432.7 (± 141.3)	6	353.2 (± 112)	6	385 (± 140.6)	6	273 (± 136.9)	6	311 (± 125)
TSS [mg L^{-1}]	4	61 (± 20.2)	4	33.3 (± 13.9)	4	25 (± 10)	4	27.5 (± 12.6)	4	26.3 (± 6.3)	4	13.8 (± 11.8)	4	13.8 (± 4.8)
NH_4-N [mg L^{-1}]	6	69 (± 5)	6	50 (± 2.7)	6	61.6 (± 2.8)	6	38.7 (± 3.5)	6	48.7 (± 1.8)	6	5.6 (± 1.5)	6	7.2 (± 1.9)
NO_3-N [mg L^{-1}]	6	Not detected	6	13.3 (± 3.5)	6	3.4 (± 0.6)	6	17.7 (± 2)	6	11.4 (± 2.8)	6	42.9 (± 5.7)	6	53.1 (± 5.6)
TN [mg L^{-1}]	4	80.8 (± 6)	4	71.8 (± 2.2)	4	72.8 (± 1.5)	4	71.3 (± 2.5)	4	72.5 (± 1.3)	4	54.5 (± 4.1)	4	71.5 (± 1.7)
PO_4-P [mg L^{-1}]	4	13.8 (± 0.8)	4	9.4 (± 1.5)	4	12.9 (± 0.7)	4	8.7 (± 0.5)	4	10.4 (± 0.5)	4	6 (± 0.3)	4	8.4 (± 2.4)
E. coli (CFU 100 mL^{-1})	6	1.1×10^6 (± 6.4×10^5)	6	7.4×10^5 (± 6.1×10^5)	6	9×10^5 (± 5.6×10^5)	6	5.6×10^5 (± 4.7×10^5)	6	8.5×10^5 (± 5.3×10^5)	6	1.2×10^5 (± 1.2×10^5)	6	2.5×10^5 (± 1.3×10^5)
pH	4	7.76-7.85	4	7.97-8.15	4	7.74-7.92	4	7.24-7.42	4	7.52-7.62	4	7.12-7.37	4	6.6-6.67
EC (mS m^{-1})	4	91.6 (± 3.5)	4	75.1 (± 14.3)	4	73.3 (± 19.2)	4	58.4 (± 9.6)	4	78.8 (± 14.7)	4	60.9 (± 6.5)	4	57.6 (± 12.5)
ALKY [mg $CaCO_3$ L^{-1}]	4	383.8 (± 31)	4	299.2 (± 38)	4	367 (± 6.8)	4	208.4 (± 20.9)	4	277.1 (± 23.9)	4	49.6 (± 5.6)	4	30.5 (± 11.4)

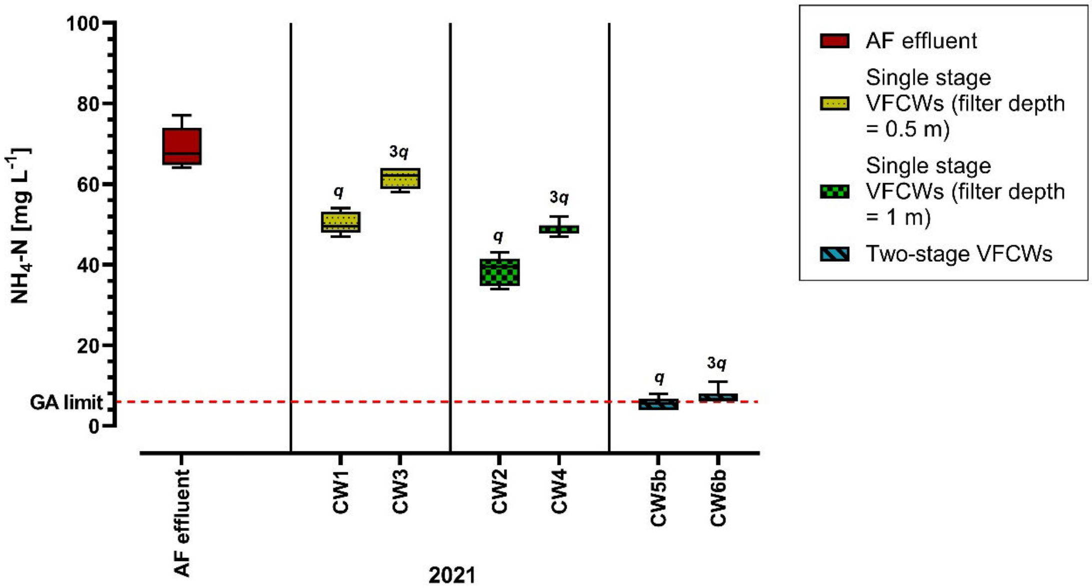


Figure 5-9: Comparison of ammonium-N concentration in the effluent of each pilot-scale vertical down-flow constructed wetland in relation to the AF effluent (Trains 2 and 3) and hydraulic loading rate (*q* and 3*q*). The whiskers represent the minimum and maximum concentrations

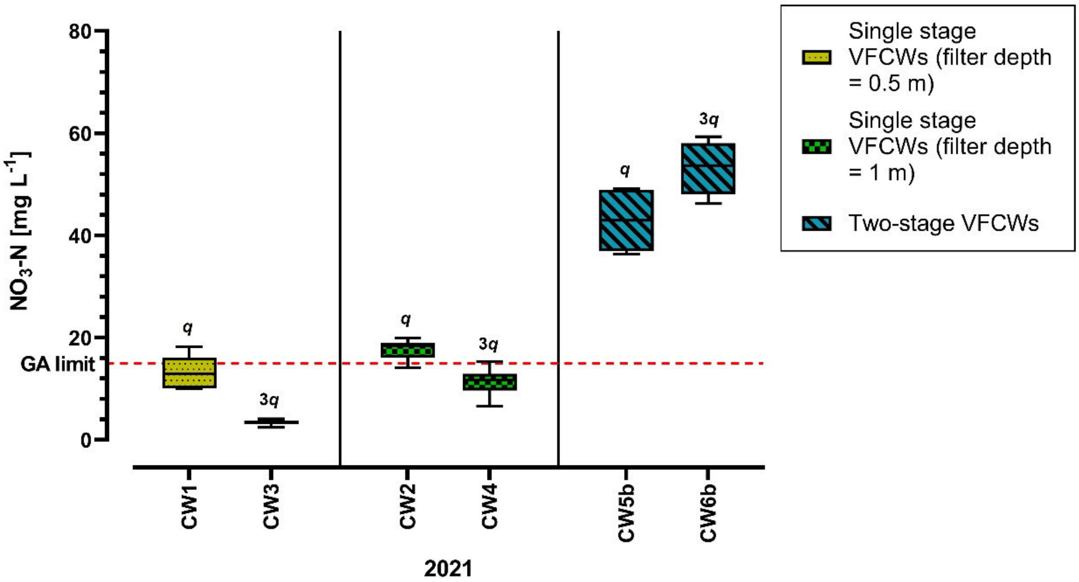


Figure 5-10: Comparison of nitrate-N concentration in the effluent of each pilot-scale vertical down-flow constructed wetland in relation to hydraulic loading rate (*q* and 3*q*). The whiskers represent the minimum and maximum concentrations

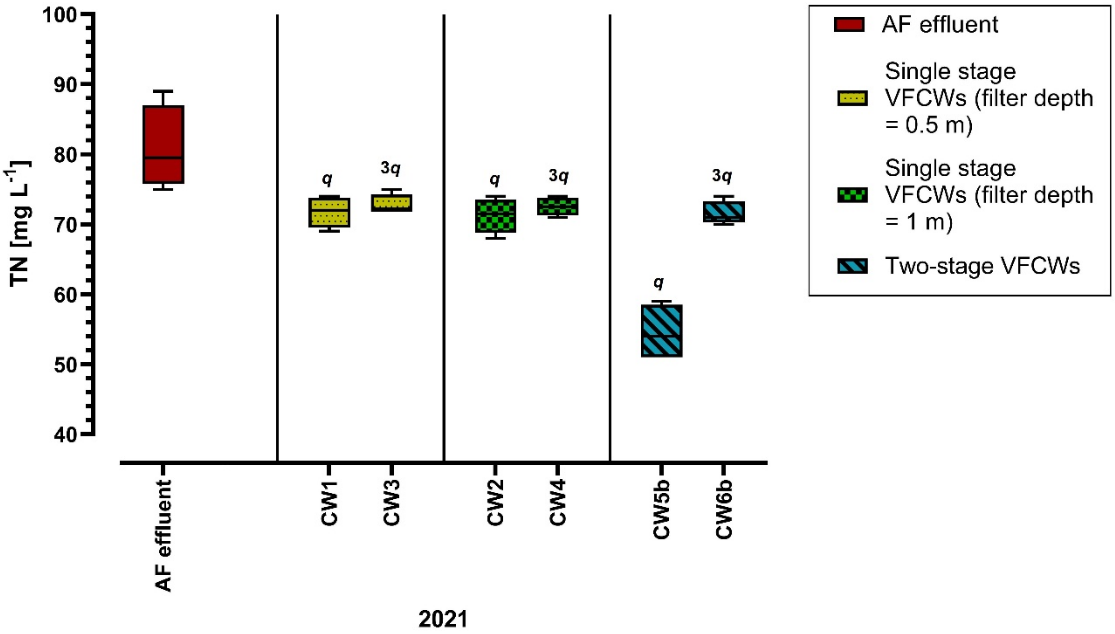


Figure 5-11: Comparison of total nitrogen concentration in the effluent of each pilot-scale VFCW in relation to the AF effluent (Trains 2 and 3) and hydraulic loading rate (*q* or 3*q*). The whiskers represent the minimum and maximum concentrations

5.4 DISCUSSION

During the start-up performance evaluation, the VFCWs operated at an organic loading rate of 45.3 g COD m^{-2} d^{-1}. The presence of bCOD in the effluent of the single-stage VFCW confirmed that incomplete organic degradation was possible using 2-3 mm coarse sand media as suggested by Langergraber et al. (2014). Unfortunately, bCOD concentration could not be measured in the effluent of the single-stage VFCWs with extended filter depths due to instrument downtime. Therefore, it is unclear if the lack of denitrification in these VFCWs was due to the absence of available carbon (Table 5-3). Overall, nitrification was also limited in the single-stage VFCWs with extended filter depths compared to the single-stage VFCWs with 0.5 m filter depths. It is possible that these VFCWs required a longer start-up time. In comparison, almost complete nitrification (> 90% in CW5b) was observed in the two-stage VFCWs after the 15 weeks of continuous operation (start-up performance), increasing by almost 60 percentage points compared to CW 1 and 3 (which represented the first stage of the two-stage VFCW) (Table 5-3). Based on this observation, it can be assumed that the longer retention time in the second stage of the two-stage VFCW as a result of the lower hydraulic conductivity of the sand media compared to the first-stage VFCW, resulted in better nitrification. Overall, the single-stage VFCWs with extended filter depths performed poorly compared to the two-stage VFCW and thus, hypothesis 1 is not supported by the findings of this study. Moreover, it is assumed that the start-time of the VFCWs with extended filter depths are greater than the other designs due to the poor NH_4-N removal (Figure 5-5).

Denitrification was however, limited in the two-stage VFCW, which was overall the best performing design compared to the single-stage VFCWs. Since the VFCWs were not seeded with any inoculum, it is believed that the VFCWs still require a longer time to reach optimum performance. Moreover, a microbial community analyses would have been beneficial to understand the microbial populations established within the system during start-up and explain some of the negative removal efficiencies (Table 5-3). Wang et al. (2022) recently published the major microbial species responsible for the main nitrogen removal processes in CWs. Microbial degradation of nutrients, especially nitrogen, can account for as much as 90% removal (Tan et al., 2021). Thus, a clear understanding of the communities' present, will aid in determining the treatment limitations of each VFCW designed in this study.

In terms of total nitrogen removal, approximately 20% was removed in the two-stage VFCW compared to only approximately 3% removal in the single-stage VFCW with extended filter depths (Table 5-3; Figure 5-7). It is clear, that extending the filter depth had no impact on the TN removal efficiency of the single-stage VFCW. None of the VFCWs were able to produce compliant effluent with the Revised General Authorisations for safe discharge to a water resource, which was expected since this was only the start-up evaluation.

During the assessment of high HLRs on the performance of each VFCW design, the organic loading rate was 112.1 g COD m^{-2} d^{-1} (Table 5-4), almost triple that during the start-up phase at the design HLR (q). The high OM in the AF effluent was potentially due to reduced anaerobic degradation in the primary and secondary treatment modules, upstream of the VFCWs. This implies that those VFCWs operating at $3q$ operated at an organic loading in excess of 300 g COD m^{-2} d^{-1}. The removal efficiency of COD was the lowest in the VFCWs operating at $3q$ (Table 5-4; Appendix H). In Europe, VFCWs are designed for maximum organic loading rates of 20 g COD m^{-2} d^{-1} in Austria with 0.06-4 mm sand (Langergraber et al., 2007), including Germany with 0-2 mm sand (Nivala et al., 2018), and 27 g COD m^{-2} d^{-1} in Denmark with washed well graded 0.25-4 mm sand (Brix and Arias, 2005). Langergraber et al. (2008) recommended a maximum areal organic loading of 80 g COD m^{-2} d^{-1} for VFCWs with 2-3 mm sand.

Nevertheless, removal of NH_4-N was above 90% in the two-stage VFCW operating at design *q* (Figure 5-9) with TN removal the highest, compared to the single-stage VFCWs at 32.5% (Figure 5-12; Appendix G). The increase in alkalinity in the effluent of the second stage of CW5 (CW5b) demonstrated that partial NO_3-N removal did occur as also seen in the change in TN concentration (Table 5-4). This implies that the higher organic loading did increase the carbon availability for denitrification and thus, hypothesis 2 is supported. However, the NO_3-N concentration was still above the GA limit for safe discharge (Figure 5-11). Overall, none of the VFCWs during this phase of the study was able to produce safe discharge quality effluent (Appendix G and H). The design of the two-stage VFCW operating a design HLR needs to be optimised for higher NO_3-N removal.

5.5 CONCLUSION

This study evaluated the nitrogen removal capacity of two designs of intermittently fed down-flow vertical flow constructed wetlands receiving anaerobically treated domestic wastewater from a demonstration-scale decentralised wastewater treatment system in eThekwini, South Africa. These designs were single stage VFCWs with an extended filter depth (1 m) and two-stage VFCWs, adapted from the Austrian design. Depth had no measurable impact on the nitrogen removal capacity of the VFCWs. The two-stage VFCW demonstrated the highest NH_4-N (> 90%) and TN (32.5%) removal efficiencies. However, the final effluent quality of the two-stage VFCW was not compliant with the South African regulatory requirements for safe discharge. This study also demonstrated that organic loading rates of 112.1 g COD m^{-2} d^{-1} do improve the denitrification potential of the two-stage VFCW. However, long term performance monitoring is necessary to determine the sustainability and robustness of this design of VFCW in the application of DEWATS in South Africa for community sanitation.

CHAPTER 6. OVERALL DISCUSSION

In South Africa, more than half of the centralised wastewater treatment works (WWTWs) are failing (DWS, 2018). Moreover, in 2022, only 2.3% of the 995 WWTWs evaluated by the regulator achieved Green Drop Status, where 90% of the effluent samples analysed were compliant with the General Authorisations (GA) (DWS, 2022). In addition, there is no published data on excreta flows in cities and the actual percentage of domestic wastewater that reaches safe disposal or reuse after treatment (WHO and UNICEF, 2021). Without this data, South Africa is unable to track their progress towards the United Nation's (UN) Sustainable Development Goal (SDG) 6.2.1.a which highlights the *proportion of population using safely managed sanitation services* (WHO and UNICEF, 2021). More recently, many Municipalities in the KwaZulu-Natal (KZN) province experienced severe damage to its bulk water and sanitation infrastructure during the April and May 2022 flooding. Many citizens were without basic services such as water and electricity for extended periods of time (more than 90 days), while raw domestic wastewater infiltrated natural water bodies. In addition to already existing delays in service delivery, as a result of population growth and rapid urbanisation, an urgent action sanitation services plan is required, especially for informal settlements, who are the most affected.

The South African Government plans to upgrade informal settlements *in situ* to formal housing with waterborne sanitation. This study contributed to developing an appropriate design for a decentralised wastewater treatment system (DEWATS) that produces fully compliant effluent for safe discharge to a water resource. DEWATS based on the BORDA design principles have no electrical or chemical demands and thus, offers wider applicability for where waterborne sanitation is provided. Although waterborne sanitation is not sustainable for a water-stressed country like South Africa, the legacies of Apartheid still spur on the desire of a "flushing toilet" amongst the indigent, who suffered the most in terms of service delivery.

The research was conducted on a Municipal owned research site that demonstrates the application of a BORDA-designed DEWATS for the South African context which was commissioned in 2010 and serves a community of 84 households. The design of the demonstration-scale DEWATS includes a settler (primary treatment), anaerobic baffled reactor and anaerobic baffled filter (secondary treatment), and vertical down-flow constructed wetland (VFCW) and horizontal flow constructed wetland (HFCW) (tertiary treatment) (Pillay et al., 2013). Arumugam and Buckley (2020) undertook minor design upgrades, particularly to increase the hydraulic retention time (HRT) in the demonstration-scale CWs, in an attempt to effectively improve the treatment efficiency. This study focussed on the feasibility of VFCWs in four design configurations to determine which produced fully compliant effluent for safe discharge. These designs were:

1. A single-stage demonstration-scale VFCW (design 1) compared to its hybrid configuration with a horizontal flow CW (HFCW) (design 2).
2. VFCWs with extended filter depths (1 m) consisting of 2-3 mm sand media (at pilot-scale) (design 3).
3. Two-stage VFCWs (at pilot-scale, operating under field conditions) (design 4):
    c. First stage: 0.5 m filter depth consisting of 2-3 mm coarse sand media.
    d. Second stage: 0.6 m filter depth with 0.5-2 mm fine to coarse sand media.

This section discusses the main scientific findings of the study and its implications on the DEWATS design for *in situ* informal settlement upgrades to formal housing in eThekwini, and South Africa in general.

6.1 KEY SCIENTIFIC FINDINGS

The demonstration-scale VFCW under continuous operation (i.e., no interruptions in flow during the 2019 sampling campaign) (design 1) indicated high removal efficiencies of total chemical oxygen demand (COD) and ammonium-N (NH_4-N) concentration, at 85% and 91.7%, respectively (Table 3-2). Evidently, physically stamping the surface of the bed by foot was sufficient to compact the sand media and cause temporary surface flooding after each dose of the mechanical float siphon. Thus, the oxygen demand for almost full nitrification was met through diffusion of atmospheric oxygen as the wastewater percolated through the sand media. Pillay et al. (2013) operated the DEWATS with tertiary treatment in the VFCW alone between October 2011 and February 2012 and reported COD and NH_4-N removal efficiencies of 55.7% and 54.4%, respectively. Moreover, no clogging was observed during peak flows in this study, suggesting that the hydraulic and organic loading were adequate for continued operation. However, the higher residence time resulted in complete degradation of bCOD from the AF effluent. The high nitrate-N (NO_3-N) concentration in the VFCW effluent at 29.6 (± 13.8) mg L^{-1}, implied that even if there were anaerobic conditions toward the bottom of the filter depth, lack of endogenous carbon limited denitrification. Moreover, the poor

Escherichia coli reduction in the VFCW concurred with other studies (Decamp and Warren, 2000; Headley et al., 2013; Vera-Puerto et al., 2021) as a result of the sand media. The VFCW effluent, under continuous operation, only achieved compliance with the GA limits for orthophosphate (PO_4-P), pH and electrical conductivity (EC) (Table 3-2). In its present design, the demonstration-scale VFCW (design 1) cannot achieve fully compliant effluent.

When comparing design 1 (single-stage demonstration-scale VFCW) with design 2 (the hybrid CW system: the demonstration-scale VFCW operating in series with a single HFCW), there were minimal increases in removal efficiencies of COD and NH_4-N, but substantially higher removal efficiencies of total suspended solids (TSS) and *E. coli* (Table 3-3). The TSS removal efficiency of the HFCW was only 58.4%. It is suspected that the loosely packed, irregular-shaped, mixed gravel of 8-20 mm and 25-80 mm aggregates/broken stones in the HFCW contributed to low residence times even after permanently impounding the outlet. In fact, maintaining the saturated level of the wastewater at 0.15 m below the surface of the gravel media resulted in surface water accumulation during peak flows. It is also suspected that the growth of the roots resulted in poor hydraulic conductivity of the media. In the BORDA design guideline, Sasse (1998) recommend round gravel of uniform size, preferably between 6-12 mm or 8-16 mm medium gravel.

Manios et al. (2003) found more than 90% TSS removal in HFCWs with a combination of sand and gravel media (ranging from river sand and 6 mm, 12 mm, and 30 mm gravel) compared to HFCWs without gravel (i.e., with sand or compost media). They attributed the high removal efficiencies to higher residence times in the combined sand/gravel matrix. However, the combination of sand and gravel in the HFCW may increase the possibility of logging.

In terms of *E. coli* removal, the hybrid CW system demonstrated 2.8 $\log_{10}$ reduction compared with only 0.8 $\log_{10}$ reduction in the VFCW alone (design 1). Decamp and Warren (2000) found higher *E. coli* log removal in planted gravel HFCWs compared to HFCWs with sand media. Headley et al. (2013) compared non-planted and planted HFCWs with or without aeration and 8-16 mm medium gravel media. They found the highest *E. coli* log removal in non-planted aerated HFCWs. The demonstration-scale HFCW in this study was fully vegetated with *Cyperus sexangularis* and *Typha capensis*. The dissolved oxygen (DO) concentration in the HFCW effluent was 2.1 ($\pm$ 0.2) mg L^{-1} and thus denitrification was not expected. It is assumed that the surface water accumulation during peak flows, due to root growth, and larger void spaces in certain aeras of the filter layer led to passive aeration by diffusion of atmospheric oxygen. Kadlec and Wallace (2009) argue that although atmospheric transfer of oxygen into the gravel media of HFCWs is possible, it is inadequate to have an influence on the anoxic conditions within the HFCW. If the DO concentration in the HFCW was attributed to passive aeration, then this may have contributed to the higher $\log_{10}$ reduction of *E. coli*, despite the mixed gravel media in the HFCW compared to that used by Decamp and Warren (2000) and Headley et al. (2013).

However, as in the VFCW, NO_3-N removal was poor in the HFCW, with only 9% removal efficiency compared with the VFCW effluent concentration (Table 3-3). The rate-limiting factors affecting denitrification were the DO concentration and lack of endogenous carbon. Bertino (2010) document that DO concentrations above 0.3-0.5 mg L^{-1} cause denitrifying bacteria to utilise DO instead of NO_3-N as the terminal electron acceptor in their respiration. Similarly, the absence of any endogenous carbon in the HFCW and no bCOD in the VFCW effluent resulted in very little to no carbon as the electron donor for denitrification (Table 3-3). Overall, the HFCW effluent was 100% compliant with the GA limits for NH_4-N, PO_4-P, pH, and EC while the percentage of samples achieving compliance for COD and *E. coli* was 87% and 83.3%, respectively. Design 2 (hybrid CW system) therefore, produced better quality effluent than design 1 (single-stage VFCW).

An estimated recovery time of the hybrid CW system (design 2) was 16 weeks, which is important data for design engineers and operators of DEWATS if applied in scenarios where flows are disrupted. In informal settlements, circular migratory patterns are still common (Posel and Marx, 2013) and thus, can affect the hydraulic loading to the DEWATS and the CWs applied for tertiary treatment. The 16-week estimation was based on the time taken for the concentration of NH_4-N in the HFCW effluent to decrease below the GA limit of 6 mg L^{-1}. Due to this evaluation being a recovery assessment, the % of samples compliant with the GA limits for NH_4-N, NO_3-N and *E. coli* were negligible. The DO concentration was still above 0.5 mg L^{-1} at 1.8 (± 0.6) mg L^{-1} (Appendix B). At week 16, the NO_3-N centration was still above the GA limit of 15 mg NO_3-N L^{-1}. Therefore, based on the performance data under continuous operation and during recovery after long-term shut down (162 days), DEWATS design 2 was not able to produce fully compliant effluent for safe discharge to a water resource.

Converting the drainage regime of the HFCW from being permanently impounded to maintaining the saturation level at 0.3 m below the surface, did appear to decrease the DO concentration in the HFCW effluent to 1.2 (± 1.4) mg L^{-1} but again, still above 0.5 mg L^{-1} (Table 4-3). However, it was unclear if this was the result of the lower saturation level inside the HFCW or addition of the dried plant material of *Arundo donax* L. the latter of which provided a carbon source for facultative denitrifying bacteria. One week after adding the dried plant material, COD concentration in the HFCW effluent increased by 172 mg L^{-1} (Figure 4-5a). In week 4, the COD concentration was 860% higher than the base line concentration on Day 1 (Table 4-3). Considering that there was minimal reduction on NO_3-N concentration in the HFCW effluent (Figure 4-5a) while the DO and COD concentrations decreased over time, it is assumed that the present denitrifying bacteria used oxygen as the terminal electron acceptor in their respiration. This is supported by analysing the change in NO_3-N concentration as a function of the COD concentration over the monitoring period, as illustrated in Figure 4-5b. The Spearman Correlation Coefficient (r) was -0.1816; $p = 0.5498$. Although this only indicates a minor inverse correlation between both parameters, this does show that COD:NO_3-N are to some degree, inversely proportional.

As expected, the consequence of lower saturation levels in the HFCW was poor removal of *E. coli*. During the 13-week monitoring campaign, only 16.7% of the HFCW effluent samples achieved compliance with the GA limit for *E. coli*. As indicated by literature, lower HRTs will negatively effect pathogen removal (Tanner et al., 1995; Kansiime and Bruggen, 2001; Vymazal, 2005b; Díaz et al., 2010). Overall, the HFCW effluent was compliant with the GA limits for TSS, PO_4-P, pH, and EC (Table 4-3).

Despite having a minimal effect on NO_3-N removal, the natural anaerobic fermentation of the air-dried plant material of *A. donax* L., added at the inlet of the HFCW in waterlogged conditions, was successful in rapidly increasing COD concentrations in the field. In fact, where the hybrid CW design is implemented in conjunction with DEWATS, and plant material of *A. donax* L. is readily available, then the invasive species can be used as a practical and efficient source of carbon to augment COD:NO_3-N for denitrification in HFCWs provided the DO concentration is below 0.3-0.5 mg L^{-1}.

Based on the data from this study, the main design limitation of the HFCW was, and is still currently, the mixed, irregular-shaped gravel media in the filter layer, potentially contributing to the low HRT and thus, limiting the treatment efficiency. If the hybrid CW system (design 2) is to be included in the DEWATS design, then the aggregate size would need to be uniform, consist of round, regular shaped gravel. Moreover, smaller (below 25 mm) gravel size will reduce the void spaces in the media, improve the residence time and reduce the potential of passive diffusion of the atmospheric oxygen into the HFCW. In a consolidated effort to provide a practical and technical guideline of the implementation of the main CW types, based on years of research, Langergraber et al. (2019) recommend 8-16 mm medium gravel in filter layer with 50-200 mm large aggregate media in the inlet and outlet regions of HFCWs.

The pilot-scale VFCWs with extended filter depths (1 m) using 2-3 mm coarse sand (design 3) demonstrated no effect on total nitrogen (TN) removal. The highest TN removal efficiencies were observed in the final effluent quality of the two-stage VFCWs (design 4), which also contained the lowest and highest concentrations of NH_4-N and NO_3-N, respectively. This implied almost full nitrification compared to the single-stage VFCWs (with or without extended filter depths). However, removal of NO_3-N was poor, despite the presence of bCOD in the effluent of each CW, implying that the coarse sand (2-3 mm) did limit full organic degradation of the influent OM. At higher hydraulic loadings, the pilot-scale VFCWs operating at the design HLR (*q*) was 112.1 g COD m^{-2} d^{-1}, compared to 45.3 g COD m^{-2} d^{-1} during the start-up, while the VFCWs operating at 3*q* received an organic loading of 336.3 g COD m^{-2} d^{-1}. Langergraber et al. (2008) recommend a maximum areal organic loading rate of 80 g COD m^{-2} d^{-1} for a filter layer consisting of 2-3 mm coarse sand while Langergraber et al. (2007) suggest a maximum of 20 g COD m^{-2} d^{-1} for 0.06-4 mm sand. The sand media used in the second stage of the two-stage VFCW was 0.5-2 mm since it was readily available.

Due to the usually high organic mass loadings from the AF effluent of Trains 2 and 3 in 2021, the VFCWs operating at design *q* performed better than the VFCWs at 3*q*. The two-stage VFCWs again demonstrated the highest NH_4-N and TN removal efficiencies compared to the single-stage VFCWs. Overall, the pilot-scale VFCWs operating at the design (*q*) HLR performed marginally better than at higher loadings (3*q*). Unfortunately, none of the pilot-scale VFCWs were able to achieve compliance with the GA limits for safe discharge into the receiving environment and long-term performance monitoring is required. Therefore, at its current design, both design 3 (single VFCWs with extended 1 m filter depths using 2-3 mm coarse sand) and design 4 (two-stage VFCW adapted from the Austrian design) cannot be integrated into DEWATS with the intention to produce effluent quality that is safe discharge in relation to the Revised General Authorisation limits (DWS, 2013).

6.2 IMPLICATIONS ON THE DESIGN OF DEWATS

It is important that the simple, practical design and low operating and maintenance principles (no electrical or chemical requirements) of DEWATS be maintained for it to be easily adopted as an affordable and sustainable solution for Municipalities. Considering that the hybrid CW design (VFCW connected in series to a HFCW) produced the highest quality effluent compared to the other VFCW configurations, the following design amendments to the current demonstration-scale CWs are suggested.

It is suggested that the VFCW be 0.8 m in depth. Based on the incomplete organic degradation in the first-stage of the two-stage VFCW, 2-3 mm is recommended for the top filter layer. Moreover, a middle organic layer will provide addition biodegradable chemical oxygen demand as a carbon source for denitrification. It was difficult to conclude if the dried plant material of *Arundo donax* (Chapter 4) was a sustainable source of bCOD, considering the DO concentration in the HFCW was very high. If applied toward the bottom of the filter depth in the VFCW, the DO levels may be low enough to promote nitrate-N removal via denitrifying bacteria. Biochar has been used to augment CW treatment efficiency (Gupta et al., 2015; Vijay et al., 2017).

In summary, the VFCW should be designed as:

VFCW: 0.8 m total depth (excluding freeboard at the top)

- Top: 0.5 m filter layer of 2-3 mm coarse sand.
- Middle: 0.1 m organic layer (like dried plant material of *Arundo donax* L. or biochar).
- Bottom: 0.2 m drainage layer with 16-30 mm gravel.
- Outlet permanently impounded to maintain the saturation level at 0.5 m below the surface to allow for natural anaerobic fermentation of the organic layer to increase the COD:NO_3-N for denitrification downstream.

The design of the HFCW is based on recommendations from literature (Gutterer et al., 2009; Kadlec and Wallace, 2009; Langergraber et al., 2019; Vymazal, 2022) and should be designed as follows:

HFCW: 0.7 m total depth (excluding freeboard at the top)

- Top: 0.5 m filter layer of 8-16 mm medium gravel.
- Bottom: 0.2 m drainage layer of 16-30 mm gravel.

- The inlet and outlet regions of the HFCW must have 50-80 mm gravel.
- Outlet permanently impounded to maintain the saturation level at 0.2 m below the surface for reduction of NO_3-N and *E. coli*.

The sizing of the CWs will be dependent on the community size (person equivalent). However, influent characterisation is important and must be combined with testing this design at demonstration-scale to assist in developing design guidelines for DEWATS implemented for domestic wastewater treatment.

If DEWATS final effluent quality meets the regulatory requirements for safe discharge (DWS, 2013), then this will unlock the barriers associated with service delivery backlogs, failing WWTWs and damaged and ageing bulk infrastructure. Moreover, through adequate data ecosystems, Municipalities can track their domestic wastewater treatment through the sanitation value chain using shit flow diagrams (SFDs) in smaller decentralised systems and thus, provide an estimate of the proportion of households with access to safely managed sanitation services. South Africa will then be in a position to track its progress against SDG 6.2.1a which refers to the "*Proportion of population using safely managed sanitation services*" (WHO and UNICEF, 2018; 2021). Smaller, decentralised systems are more manageable from an operation and maintenance point of view, allowing for any community vulnerabilities to be identified and thus, effectively addressing sanitation challenges in South Africa.

CHAPTER 7. CONCLUSIONS AND RECOMMENDATIONS

For DEWATS to be adopted in South Africa, and serve as a sustainable sanitation option, the low operation and maintenance principles must be preserved. This means that energy demands for flow or treatment augmentation as well as chemical inputs must be avoided. It is for this reason, that this study looked at the application of vertical down-flow constructed wetlands in different design configurations to determine which design would be appropriate to achieve effluent quality safe for discharge. This is important for dense informal settlements undergoing formal housing upgrades where space is restricted, and DEWATS integration with agriculture is not possible. Based on land area availability, VFCWs are more suitable as they require less surface area compared to other CW designs, and also have a higher oxygenation capacity required for ammonium-N removal.

This research was carried out at a demonstration-scale DEWATS operating within the eThekwini Municipality, treating raw domestic wastewater from 84 households. The DEWATS was designed for research purposes and the final effluent is diverted back into the main sewer line.

7.1 REVISTING THE AIMS AND OBJECTIVES

The aim of this study was to evaluate the performance of vertical down-flow constructed wetlands (VFCWs) as a tertiary treatment module in the DEWATS receiving anaerobically treated domestic wastewater, in different design configurations. These configurations formulated four DEWATS designs consisting of a settler, anaerobic baffled reactor, anaerobic filter and:

1. A single-stage VFCW (design 1).
2. A hybrid configuration comprising a single-stage VFCW with subsequent treatment in a horizontal flow CW (HFCW) (design 2).
3. Single-stage VFCWs with extended filter depths (design 3).
4. Two-stage VFCWs (design 4).

The specific research objectives were to:

1. Monitor the overall performance of the demonstration-scale VFCW against its hybrid configuration with the HFCW under the continuous operation of the

DEWATS and compare the effluent quality of both with the Revised GA limits for safe discharge into a water resource.

2. Determine the feasibility of increasing nitrate-N removal by the addition of dried plant material of *Arundo donax* L. as a plant-based carbon source in the CW.
3. Design and construct pilot-scale VFCWs with different designs (single and two-stage) and operation configurations (specifically, media gradation, depth, and hydraulic loading rate) to determine the total nitrogen removal capacity of each.

7.2 GENERAL CONCLUSIONS

This section revisits the hypotheses tested during this study and summarises the main findings.

1. The effluent quality of the demonstration-scale VFCW (design 1) will achieve fully compliant effluent for safe discharge into a water resource.
2. The addition of dried plant material at the inlet of the demonstration-scale HFCW will meet the COD:N demand for denitrification, resulting in the hybrid CW system (design 2) achieving fully compliant effluent quality that is within the GA limits for safe discharge into a water resource.
3. The single-stage VFCW with an extended filter depth (design 3) will achieve the same, if not better effluent quality than a two-stage VFCW (design 4).
4. Increasing the hydraulic loading rate will increase carbon availability in the incoming wastewater and thus, improve nitrate-N removal across the pilot-scale VFCWs (single-stage and two-stage systems).

Under continuous operation, the VFCW did not achieve compliance with the GA limits for all chemical parameters including *E. coli*. In terms of NO_3-N removal, complete degradation of the available biodegradable chemical oxygen demand (bCOD) in the VFCW limited any potential of denitrification toward the bottom of the filter depth. Thus, the VFCW with its current media gradation cannot be operated alone and thus, hypothesis 1 was not supported.

In terms of full treatment in the hybrid CW system (design 2), the HFCW effluent was of better quality compared to the VFCW effluent, but not fully compliant with the GA limits. Like in the VFCW, NO_3-N removal was limited by the lack of endogenous carbon and high dissolved oxygen (DO) (> 0.5 mg L^{-1}). Permanently impounding the outlet of the HFCW at 0.15 m below the gravel media, together with root growth of the planted vegetation, resulted in surface water accumulation during peak flows. Temporary surface water potentially resulted in passive diffusion of atmospheric oxygen, although minimal, into the CW. An unexpected interruption of flow to the DEWATS indicated a recovery time of at least 16 weeks upon resumption of flow. This estimation was based on the time taken for the NH_4-N concentration in the HFCW effluent to decrease below the GA limit of 6 mg L^{-1}, indicating full nitrification in the VFCW.

After reducing the outlet elevation of the HFCW to maintain the saturation level at 0.3 m below the surface, and adding the pre-dried combined (top leaves, mid leaves and stems and bottom stems) of the invasive *Arundo donax* L. nitrate-N removal did not increase. Regardless of the COD concentration in the HFCW effluent increasing by 860% in Week 4, high DO concentrations were still present which resulted in poor denitrification rates. It was then postulated that the mixed, irregular-shaped gravel and potentially loose packing of the filter media of the HFCW, resulted in passive diffusion of atmospheric oxygen, thus, elevating DO concentrations above the threshold for denitrification. This was surmised as a current design limitation. At the inception of the DEWATS, Pillay et al. (2013) stressed the importance of preventative measures against clogging and surface water accumulation due to operational instability and public health concerns of human contact with the wastewater. Hypothesis 2 was therefore rejected.

Finally, single-stage VFCWs with extended (1 m) filter depths (design 3) had no effect on TN removal and thus, the third hypothesis was not supported in this study. The two-stage VFCWs (design 4) demonstrated the highest removal efficiencies of NH_4-N (> 90%) and TN (32.5%) during a hydraulic loading rate of 0.19 m d^{-1} and an organic loading rate of 112.1 g COD m^{-2} d^{-1}. However, NO_3-N was still above the GA limit of 15 mg L^{-1}. This implied that using the coarse sand (2-3 mm) did limit full degradation of the available bCOD in the feed. Considering that the maximum recommended areal organic loading rate for 2-3 mm coarse sand is 80 g COD m^{-2} d^{-1}, then the added OM did contribute to higher TN removal in the two-stage VFCW, compared to the start-up performance when the organic loading was 45.3 g COD m^{-2} d^{-1} and TN removal efficiency was on average 20%. Therefore, the fourth hypothesis was supported. However long-term performance monitoring of the two-stage VFCW will determine its optimum treatment efficiency and operational stability.

In conclusion, based on the performance evaluation of all four design configurations integrating VFCWs as the tertiary treatment module in the DEWATS, the hybrid CW system (design 2) was the best performing configuration and is therefore, recommended for future

DEWATS in South Africa provided that the denitrification potential of the entire CW system is enhanced.

7.3 CHALLENGES

Due to the intention of simplifying the design for DEWATS purposed for safe discharge of the final effluent, this research only considered simple techniques to enhance the treatment efficiency of the already established demonstration-scale CWs, despite the wealth of innovative methods and designs presently available in the literature. In addition, it is surmised that originally purposing the DEWATS for integration with agriculture resulted in no emphasis on the proper packing of the media in the CWs as well as reducing the risks of clogging, which impacted on the treatment efficiency of the system.

Moreover, since this was the first field application of DEWATS as a sanitation solution at the community-level in South Africa, long-term performance data was desired to test the operational stability, sustainability, and resilience of the system. However, the National Lockdown restrictions due to the COVID-19 pandemic disrupted sampling, prevented access to the site between March-August and November-December 2020, with additional restrictions in effect during November-December 2020 and January-February 2021. Under these conditions, operations of the entire DEWATS was halted. Thus, long-term performance monitoring (after a year of continuous operation) was not possible to evaluate the treatment efficiency and sustainability of the DEWATS as a whole. However, for future application, employing a community member to maintain the DEWATS operation would have alleviated this shut down.

Maintaining the minimalistic approach, recirculation was not an option as it would require a pump, nor the option of alternating the serial operation of the hybrid system (i.e., connecting the HFCW in series with the VFCW) as the gravitational flow to the CWs would not be possible. Moreover, the option of mixing the feed to the HFCW with VFCW effluent and AF effluent to augment the availability of bCOD may also be unsuccessful due to high pathogen loading to the HFCW. Despite the high DO concentrations in the HFCW, which will be beneficial for nitrification, nitrifiers have a slower respiration rate than denitrifiers. The facultative denitrifying bacteria would have then preferentially used the available DO rather than NO_3-N in their respiration, still limiting the overall removal of total nitrogen from the hybrid CW system by inhibiting nitrification (the first transformation step in total nitrogen removal).

In terms of the pilot-scale VFCWs, the major inhibition in the overall research timeline was time (disrupted by the National COVID-19 lockdown restrictions) and the inability to determine the long-term performance of the two-stage VFCW design. Since these CWs received combined AF effluent from Train 2 and Train 3 (with larger chambers) of the ABR, it is evident that the COD loading was much higher than from Train 1 of the ABR. Nonetheless, the total nitrogen removal efficiency of the two-stage VFCW is promising and needs to be evaluated over a longer period to determine if this design is able to achieve compliant effluent.

Furthermore, due to the high scum accumulation, composed of fats, oil, and grease (FOG) as well as sanitary and plastic material, at the inlet of the DEWATS, descumming was scheduled for twice a week in this study, which added to the maintenance demands of the system since no grit chamber or screen was included in the design. Srivastava (2018) found that lower income communities have higher fat diets compared to the affluent groups. This will have a direct impact on the DEWATS as more scum will accumulate within the system. However, including a grit chamber in future DEWATS will reduce the scum accumulation and alleviate its impact on operation.

7.4 RECOMMENDATIONS FOR FUTURE RESEARCH

As an immediate design, DEWATS with the hybrid CW system can still be employed for *in situ* formal upgrading of informal settlements through the installation of urine diversion flushing toilets at the household level. European household wastewater characterisation indicates that approximately 80% of the total N load in wastewater is derived from urine (Henze, 1997). If urine is separated from the blackwater and solid fraction, then this will reduce the nitrogen loading to the DEWATS and essentially improve the total nitrogen removal capacity of the CWs in whatever configuration is chosen (hybrid CW system or two-stage VFCW, although in the latter, the treatment efficiency needs to be optimised). Moreover, the separated urine at a large scale will also unlock the resource recovery potential from the urine similar to the Valorisation of Urine Nutrients in Africa (VUNA) project (Udert et al., 2015). Although, the latter was in combination with dry sanitation (urine diverting dry toilets) within the eThekwini Municipality (Udert et al., 2015). The urine diversion flushing toilet is a low flush technology and will satisfy the desire for waterborne sanitation while the mixture with greywater from the households will facilitate the conveyance of the raw wastewater to the DEWATS and permit gravitated flow.

Moreover, micropollutant (MP) removal must be included in monitoring the treatment efficiency of the hybrid CW system and the two-stage VFCW, and all future DEWATS in general. The awareness around the challenges of MP pollution in receiving water bodies is rising (Schwarzenbach et al., 2006; Jonsson et al., 2014; Späth et al., 2021b). Späth et al. (2021a) recently measured the MP removal efficiency of the demonstration-scale DEWATS but only until the AF effluent. If DEWATS are to be adopted for informal settlements predominantly comprising communities where the prevalence of HIV and other diseases are high, then as Späth et al. (2021a) argues, these DEWATS will become hotspots for MPs (predominantly from the use of antiretroviral drugs and antibiotics). Removal of MPs in two-stage CWs will contribute to wider acceptance of the DEWATS approach and its suitability for *in situ* formal housing upgrades of informal settlements in South Africa.

The use of DEWATS is a step forward towards providing basic sanitation services, without the need to impede on the existing water service, sewer system and failing WWTWs. Its low capital, operating and maintenance requirements make this sanitation option a feasible solution for a country facing increasing challenges in water security, continuous degrading of bulk sanitation infrastructure and short supply of electricity.

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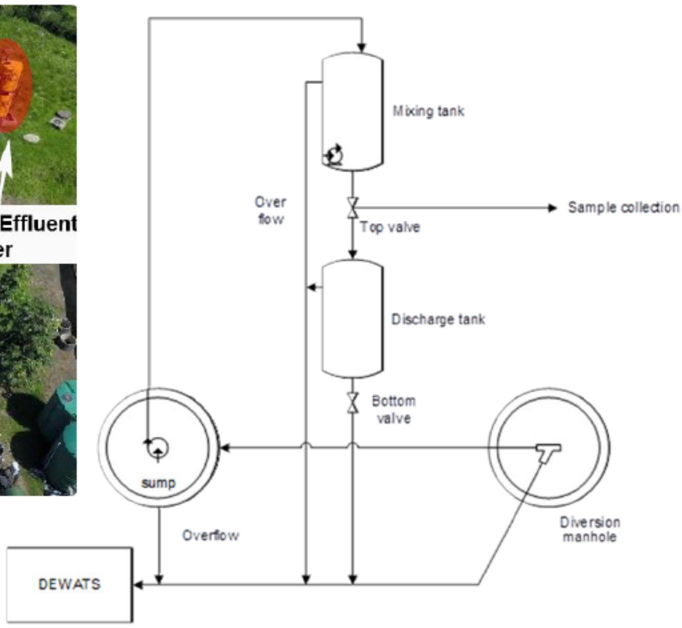
APPENDICES

Appendix A: Whole effluent sampler (WES) at the demonstration-scale DEWATS in eThekwini purposed for composite sampling of the raw wastewater

a.

b.



c.

Figure A-1: a. aerial view of the WES; b. process flow diagram of the WES and connection to the DEWATS and c. side view of the WES

Appendix B

Table A-1: Effluent quality from the horizontal flow constructed wetland (September-December 2020)

	n	Range (Min-Max)	Mean ± Std Dev	GA limit
COD [mg L^{-1}]	6	44-87.3	62.7 (± 14.3)	75
bCOD [mg L^{-1}]	3	Not detected		
TSS [mg L^{-1}]	5	10-25	19 (± 5.5)	25
NH_4-N [mg L^{-1}]	6	2-24	**11.3** (± 7.3)	6
NO_3-N [mg L^{-1}]	6	6.8-45.9	**27** (± 15.7)	15
TN [mg L^{-1}]	5	30-54.7	42.9 (± 10.7)	
PO_4-P [mg L^{-1}]	5	2.8-5.5	4.4 (± 1.1)	10
E. coli (CFU 100 mL^{-1})	6	5 x 10^2-8.8 x 10^4	**1.9 x 10^4** (± 3.9 x 10^4)	1000
pH	5	6.8-7.4		5.5-9.5
EC (mS m^{-1})	5	54.4-85.9	71.7 (± 13.4)	150
AKLY [mg $CaCO_3$ L^{-1}]	5	20-80	29.7 (± 18.3)	
DO [mg L^{-1}]	5	0.9-2.8	1.8 (± 0.6)	

Red text denotes mean concentration above the GA limit for discharge

Appendix C

Figure A-2: Preparation of the oven-dried plant material of *Arundo donax* L. for the batch fermentation experiment in the oven

Appendix C

Figure A-3: Batch experiments in the oven

Appendix D

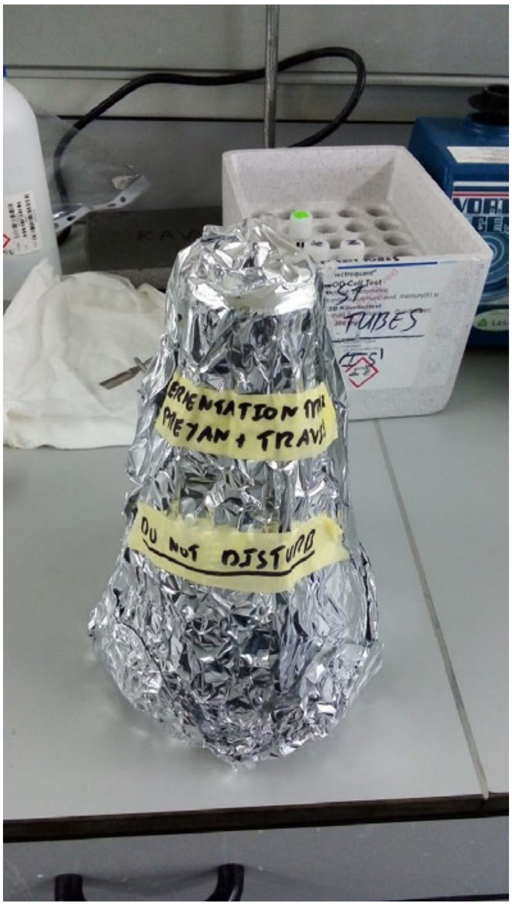


Figure A-4: Batch reactor in the field

Appendix E

Figure A-5: Stainless steel sumps for collection of drainage from the first stage of the pilot-scale two-stage VFCWs. Inset (top view of the sump without the lid demonstrating the inlet, outlet and overflow pipes)

Appendix F

Table A-2: Final effluent quality and performance data from pilot-scale vertical down-flow constructed wetlands (VFCW) receiving anaerobically treated domestic wastewater at design flow (0.19 m d^{-1})

			AF effluent (Trains 2 and 3)	**CW 1** Single-stage Filter depth = 0.5 m			**CW 2** Single-stage Filter depth = 1 m			**CW 5b** Two-stage		
	n	GA limit	Range (Min-Max)	Range (Min-Max)	Removal efficiency (%)	% Samples achieving compliance	Range (Min-Max)	Removal efficiency (%)	% Samples achieving compliance	Range (Min-Max)	Removal efficiency (%)	% Samples achieving compliance
COD [mg L^{-1}]	6	75	464-811	303-572	30.9	0%	242-533	40.1	0%	164-489	53.7	0%
TSS [mg L^{-1}]	4	25	35-84	15-45	45.4	25%	10-40	54.9	25%	5-30	77.4	75%
NH_4-N [mg L^{-1}]	6	6	64-77	47-54	27.5	0%	34-43	43.9	0%	4-8	91.9	66.7%
NO_3-N [mg L^{-1}]	6	15	Not detected	10-18.2		50%	14.1-19.9		16.7%	36.3-49.1		0%
TN [mg L^{-1}]	4		75-89	69-74	11.1		68-74	11.8		51-59	32.5	
PO_4-P [mg L^{-1}]	4	10	13.4-15	7.2-10.6	31.9	50%	8.1-9.2	37.0	100%	5.6-6.3	56.5	100%
E. coli (CFU 100 mL^{-1})	6	1000	2×10^5-2×10^6	1×10^5-1.9×10^6		0%	1×10^5-1.4×10^6		0%	1×10^4-3.6×10^5		0%
Log_{10} reduction						0.2			0.3			1
pH	4	5.5-9.5	7.76-7.85	7.97-8.15		100%	7.24-7.42		100%	7.12-7.37		100%
EC (mS m^{-1})	4	150	86.5-94.6	59.4-94.1		100%	48.2-66.6		100%	52.8-68.8		100%
ALKY [mg $CaCO_3$ L^{-1}]	6		358-463	270.3-370.5			189.8-248.5			41.2-57.5		

Appendix G

Table A-3: Final effluent quality and performance data from pilot-scale vertical down-flow constructed wetlands (VFCW) receiving anaerobically treated domestic wastewater at 3 x design flow (0.57 m d^{-1})

			AF effluent (Trains 2 and 3)	**CW 3** Single-stage Filter depth = 0.5 m			**CW 4** Single-stage Filter depth = 1 m			**CW 6b** Two-stage		
	n	GA limit	Range (Min-Max)	Range (Min-Max)	Removal efficiency (%)	% Samples achieving compliance	Range (Min-Max)	Removal efficiency (%)	% Samples achieving compliance	Range (Min-Max)	Removal efficiency (%)	% Samples achieving compliance
COD [mg L^{-1}]	6	75	464-811	262-620	26.7	0%	235-638	34.7	0%	206-504	47.3	0%
TSS [mg L^{-1}]	4	25	35-84	20-40	59	75%	20-35	56.9	75%	10-20	77.4	100%
NH_4-N [mg L^{-1}]	6	6	64-77	58-64	10.7	0%	47-52	29.4	0%	6-11	89.6	33.3%
NO_3-N [mg L^{-1}]	6	15	Not detected	2.4-4.1		100%	6.6-15.3		83.3%	46.2-59.3		0%
TN [mg L^{-1}]	4		75-89	72-75	9.9		71-74	10.3		70-74	11.5	
PO_4-P [mg L^{-1}]	4	10	13.4-15	12.2-13.9	6.5	0%	10-11.2	24.6	25%	6.5-11.9	39.1	75%
E. coli (CFU 100 mL^{-1})	6	1000	2 x 10^5-2 x 10^6	1 x 10^5-1.7 x 10^6		0%	4 x 10^5-1.8 x 10^6		0%	1 x 10^5-4.7 x 10^5		0%
Log_{10} reduction						0.1			0.1			0.7
pH	4	5.5-9.5	7.76-7.85	7.74-7.92		100%	7.52-7.62		100%	6.6-6.67		100%
EC (mS m^{-1})	4	150	86.5-94.6	48.8-94		100%	59.3-93		100%	43.9-70.5		100%
ALKY [mg $CaCO_3$ L^{-1}]	6		358-463	359.7-376.3			247.5-310.8			15.5-49.7		

Appendix H

MANUSCRIPTS EMANATING FROM THIS RESEARCH

The potential of decentralised wastewater treatment in urban and rural sanitation in South Africa: lessons learnt from a demonstration-scale DEWATS within the eThekwini Municipality

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ABSTRACT

The low maintenance principles of decentralised wastewater treatment systems (DEWATS) (such as no electrical usage or chemical inputs) make them an affordable and practical sanitation option for municipalities to adopt in fast growing cities. Since 2014, a demonstration-scale DEWATS with a modular design consisting of a settler, anaerobic baffled reactor (ABR), anaerobic filter (AF), vertical flow constructed wetland (VFCW) and horizontal flow constructed wetland (HFCW) has been in operation within the eThekwini Municipality in South Africa. The purpose of this study was to evaluate the long-term operation of the DEWATS over an extended period of time in 2019, including a shut down and subsequent resumption of operation in 2020, and to measure compliance of the final effluent quality with the General Authorisations (GA) limits for safe discharge into the receiving environment. Monitoring of the final effluent quality indicated high (≥ 85%) removal efficiencies of total chemical oxygen demand (CODt), ammonium-N (NH_4-N) and orthophosphate-P (PO_4-P), 75% removal of suspended solids (TSS) and 83.3% removal of *E.coli* bacterial cell counts. Lack of available carbon and high dissolved oxygen (DO) concentrations inhibited denitrification in the HFCW resulting in only 12.5% of effluent samples compliant with the GA limits for nitrate-N (NO_3-N). Recovery response time of the DEWATS after system shut down was estimated to be 16 weeks. Based on effluent quality over both monitoring periods, safe continuous discharge from the DEWATS is not possible in its current state. The passive application of alternative carbon sources needs to be investigated for the reduction of NO_3-N in the HFCW. Furthermore, social household surveys are required to determine migratory patterns of the community and other activities that may impact on the operation of the DEWATS. On the other hand, separation of nutrients through the installation of urine diversion flushing toilets (UDFTs) at the household level will reduce the nitrogen load to the DEWATS thereby potentially achieving fully compliant effluent for continuous discharge into the receiving environment. This technological approach is a feasible option to fill the gap in both urban and rural sanitation, such as informal settlement upgrades to formal housing, social housing developments, and school sanitation, where waterborne sanitation is still desired but connections to conventional wastewater treatment works (WWTWs) are not possible.

Keywords: constructed wetlands, DEWATS, decentralised sanitation, rural sanitation, urban sanitation

INTRODUCTION

Service delivery inequality remains evident in South Africa, spawned by the legacies of apartheid (Sutherland et al. 2014). Compounded by rapid, uncontrolled urbanisation in fast-growing metropolitan cities like the eThekwini Municipality (Durban), the increasing prevalence of informal settlements within the urban periphery has left municipalities with limited resources to meet the demand of servicing these communities (eThekwini Municipality 2021). This leads to backlogs in basic service delivery such as water and sanitation.

Sanitation statistics in South Africa indicate that 83.2% of the population have access to improved sanitation defined as either a flush toilet connected to a sewer or a septic tank, or a ventilated improved pit latrine (STATS SA 2020). Reaching the remaining 17% appears to be the most problematic, in large,

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