Performance Evaluation of a full-scale DEWATS plant in South Africa

Diplomarbeit

im Studiengang “Umwelt-, Hygiene- und Sicherheitstechnik”

am Fachbereich

Krankenhaus- und Medizintechnik, Umwelt- und Biotechnologie

der TH Mittelhessen

vorgelegt von

Lars Schöbitz

Kanzleistr. 84

8004 Zürich

Schweiz

Referent:	Prof. Dr. Harald Platen
Korreferentin:	Dipl.-Ing. Gisa v. Marcard
externe Betreuer:	Prof. C.A. Buckley
	Dr. Sudhir Pillay

Zürich, den

Erklärungen des Kandidaten zur Diplomarbeit

(1) Prüfungsrechtliche Erklärung zur Anfertigung der Arbeit:

Hiermit erkläre ich, Lars Schöbitz, dass ich die vorliegende Arbeit mit dem Titel:

Performance Evaluation of a full-scale DEWATS plant in South Africa

selbstständig verfasst und keine anderen als die zulässigen und angegebenen Quellen und Hilfsmittel verwendet und dieses auch vollständig angegeben habe. Das Dokument zum Thema „Plagiate in Lehre, Studium und Forschung“ habe ich erhalten, zur Kenntnis genommen und mich verpflichtet, identische oder annährend identisch wörtlich übernommene Passagen aus anderen Quellen entsprechend unter Angabe der Quelle ausnahmslos kenntlich zu machen.

Ort, Datum, Unterschrift Kandidat

(2) Erklärung zur Einsichtnahme und Ausleihe in die Arbeit

Ich erkläre mich damit einverstanden, dass die Arbeit zu wissenschaftlichen Zwecken eingesehen bzw. ausgeliehen werden darf. Ich erkläre damit mein Einverständnis, das die Arbeit weiteren als nur den im Prüfungsverfahren involvierten Personen zugänglich gemacht werden kann. Diese Erklärung kann von mir jederzeit widerrufen werden.

Ort, Datum, Unterschrift Kandidat

(3) Erklärung zum Urheberrecht

Ich erkläre mich damit einverstanden, dass einzelne Inhalte oder Ergebnisse dieser Arbeit zu wissenschaftlichen Zwecken von der Technischen Hoschschule Mittelhessen verwendet werden können.

Hinsichtlicht einer anderen Verwendung der Inhalte und Ergebnisse (z.B. freizügige Verteilung, wirtschaftliche Verwertung u.a.) ist mir bekannt, dass hierzu vorher das ausdrückliche Einverständnis der Referenten und eventuell beteiligter externer Institutionen von mir eingeholt werden muss.

Ort, Datum, Unterschrift Kandidat

Erklärung des Referenten bzw. der Referentin zu Einsichtnahme bzw. Ausleihe

Ich erkläre mich damit einverstanden*) / nicht einverstanden*), dass die vorliegende Arbeit zu wissenschaftlichen Zwecken eingesehen*) bzw. ausgeliehen*) werden darf, sofern der Kandidat bzw. die Kandidatin diesem unter Punkt A(2) nicht widersprochen hat.

Es besteht ein striktes Ausleihverbot.*) / Es besteht kein Ausleihverbot.*)

Ort, Datum, Unterschrift Referent/in / ggf. Stempel

Acknowledgements

First of all I want to thank Durban and everyone I have met during my time staying in this precious city. The happiness I have achieved during my time in this often unvalued city of South Africa changed me into a different person and gave me lots of advices for the rest of my life

Lots of gratitude for Prof. Chris Buckley, who always motivated me and is the best person to meet when arriving on the Durban shores. A big thank you to everyone at the Pollution Research Group, sitting in an office with you was a great pleasure. Thanks to my project team members Sudhir Pillay, Nicolas Reynaud and Phatang Sananikone. Doing field work with you was always great fun and I appreciate all your help and support during the write up of my thesis a lot.

Thanks to Stefan Reuter and the BORDA team, you made it possible for me to work, stay and live in South Africa. I want to thank everyone from the eThekwini Municipality and co-contractors for the support and help when it was needed.

Vielen Dank an meine Familie, eure Unterstützung und das Vertrauen das ihr mir geschenkt habt.

Abstract

As a part of a project funded by the Water Research Commission (WRC Research Project K5/2002) was the aim of this thesis the interpretation of the performance of a full-scale DEWATS evaluation plant in Durban, Newlands-Mashu, South Africa.

Laboratory analyses were performed to determine the treatment efficiency of the plant and compared with the initial design performance and local discharge standards. Operational and maintenance issues that influenced the research activities were elaborated.

The treatment plant was operated under operation conditions three times above the design maximum in order to study if the system collapses under increased hydraulic loadings.

It was shown that the system was affected by rainfall, which led to an overstatement of the performance through a dilution effect. The general performance of the system has not shown satisfying results compared to the estimated performance of the design spreadsheets. The effluent complied with local discharge limits concerning the reuse of wastewater for irrigation. Increased hydraulic loadings have not affected the treatment plant in a way that a collapse could be observed. It was moreover observed that no difference in the reduction of organic matter under higher hydraulic loadings appeared.

Key words: DEWATS, Anaerobic Baffled Reactor (ABR), anaerobic treatment, domestic wastewater, developing countries, low-cost, irrigation,

Table of contents

List of tables

List of figures

Abbreviations

Abbreviation	Explanation
ABR	Anaerobic Baffled Reactor
AD	Anaerobic Digestion
AF	Anaerobic Filter
AIDS	Acquired Immune Deficiency Syndrome
AM	Ante Meridiem
AMU	Agricultural Management Unit
BOD	Biochemical Oxygen Demand
BOD5	Biochemical Oxygen Demand after 5 days
BORDA	Bremen Overseas Research and Development Association
C	Compartment
CAD	Computer-Aided Design
cap	capita
Cmpts.	Compartments
COD	Chemical Oxygen Demand
COD in	Chemical Oxygen Demand inflow
COD out	Chemical Oxygen Demand outflow
DEWATS	Decentralised Wastewater Treatment Solutions
e.g.	example given
EWS	eThekwini Water & Sanitation
HFW	Horizontal Flow constructed Wetland
HIV	human immunodeficiency virus
HRT	Hydraulic Retention Time
ID	inner diameter
IHI	Internationales Hochschul Institut
IP	Ingress Protection
KHP	Potassium hydrogen phthalate
max.	maximum
MDGs	Millenium Development Goals
min.	minimum
O&M	Operation and Maintenance
OL	Organic Loading
OLR	Organic Loading Rate
PM	Post Meridiem
PVC	Poly Vinyl Chloride
Set. 2a	Settler 2a
Set. 2b	Settler 2b
SS	Settleable Solids
Std. Dev.	Standard Deviation
T1E	Train 1 Effluent
T2E	Train 2 Effluent
T3E	Train 3 Effluent
THM	Technische Hochschule Mittelhessen
Treatm. effic.	Treatment efficiency
TS	Total Solids

Abbreviation	Explanation
UASB	Upflow Anaerobic Sludge Blanket
UKZN	University of KwaZulu-Natal
US$	United States Dollar
VFA	Volatile Fatty Acids
VFW	Vertical Flow constructed Wetland
VS	Volatile Solids
WRC	Water Research Commission
ww	Wastewater
RP	Research Phase

1. Introduction

The first section will give an overview about the purpose of this study and the used technologies. Control parameters that were used for the investigations of this work will be described and the importance in terms of the performance of the system will be discussed with existing literature about this topic.

1.1 Problem statement

In 2000 all members of the United Nations states have signed the declaration for the Millennium Development goals (MDGs). One of the sub-goals of goal 7 is to halve the proportion of the population without sanitation by 2015 [1]. Achieving this goal would have an influence on the other goals since there is evidence about the fact that sanitation provision is fundamental to personal dignity, security, social and psychological issues, public health, poverty reduction and gender equality [2]. Basic sanitation prevents diseases like diarrhoea and cholera, which puts stress on the weakened immune system of people with HIV / AIDS.

In 2004, Hutton et al. [3] have studied the costs and benefits of water and sanitation facilities. The results show that an investment of US$1 would show around US$5 to US$11 of economic benefit. It is stated that these benefits must be seen as a long term effect, due to more time spend in school and at work resulting of avoided illness.

However, the Millennium Development Goals Report of 2012 shows that many countries are behind the target for basic sanitation and if the trend continues the number of people worldwide who lack access to basic sanitation will grow by 100 million from 2008 to 2015 [4].

In South Africa, the right for basic sanitation services has been stated in the Constitution of South Africa in 1996. The South African government aims to provide universal sanitation access by 2014. By March 2009, the proportion of people with basic sanitation services increased from 50% to 77%. In terms of the MDGs, the goal of halving the population without access has been fulfilled but local governmental goals are more ambitious than the MDGs [5]. A problem of South Africa is that in urban areas there is a strong resistance to any sanitation provision other than flush toilets. Generally there are a lot of sanitation options that would be better for the realities of water scarcity that South Africa is facing. But due to the country's history of apartheid, underdevelopment and discrimination, flush toilets are a strong symbol of aspiration to a better life and any other sanitation facility is seen as discrimination since flush toilets are associated with a white privilege [6].

Besides the water scarcity that the country is facing, there is also a lack of financial and human resources to develop an infrastructure that could provide the services of waterborne sanitation. Additionally it was shown that centralised existing treatment systems are often not reaching the discharge limits set by the South African Department of Water Affairs. The country therefore needs to find affordable, low maintenance interim sanitary solutions that are accepted by the local communities [6].

DEWATS, namely Decentralised Wastewater Treatment Systems, is being considered as an alternate waterborne solution for areas in South Africa where septic tanks, centralised sewered systems and dry toilets are not appropriate. The system is based on the principle of low maintenance and is seen as a technical approach rather than a technology package [7]. In the past 15 years BORDA Bremen has installed more than 700 DEWATS systems, mainly in India and Indonesia, treating wastewater of dense urban settlements, hospitals, schools and agricultural industries. The system has shown a reliable and long lasting alternative which is fulfilling discharge standards, tolerant against inflow and load fluctuations and requires minimal maintenance and low operation costs [7].

The eThekwini Metro Municipality, which is serving greater Durban, has a great track record in water and sanitation provision using innovative solutions, which has been awarded several times [10]. The municipality is considering the use of DEWATS in a partnership between the eThekwini Water and Sanitation Unit (EWS) and BORDA Bremen. In 2006, the University of KwaZulu-Natal and EWS signed an agreement to jointly investigate research water and sanitation services of interest to the municipality.

Over the last ten years, the Pollution Research Group, based at the University of KwaZulu-Natal in Durban, has conducted various laboratory and pilot studies with anaerobic baffled reactors (ABRs), which form the treatment core of the DEWATS system. In 2007, BORDA Bremen tasked the Pollution Research Group to direct scientific activities on BORDA DEWATS worldwide. A technical evaluation plant was built in Newlands-Mashu according to BORDA design guidelines [8]. The plant is situated in a lower middle income area and is connected to an existing local trunk sewer. The multipurpose property site is being managed the eThekwini Agricultural Management Unit (AMU) to serve as an agriculture learning centre, used as a community garden in social upliftment schemes, and as a multidisciplinary research centre looking at DEWATS applications, struvite recovery and the reuse of the treated wastewater for horticulture and education purposes.

The land and construction costs of the plant were financed by the eThekwini Municipality. The plant was built with the purpose of investigating the limitations of a typical BORDA DEWATS plant and to serve as a demonstration plant to interested stakeholders and local water authorities. Research activities are funded by the South African Water Research Commission (WRC Research Project K5/2002) and involve a multidisciplinary team looking at various aspects of DEWATS performance and implementation.

1.2 The concept of the DEWATS system

The term DEWATS is a common name that was developed by an international network of organisations [7].

The principle behind DEWATS is to have a system that includes local communities and authorities in the process of the decision making and further maintenance. Systems are generally installed on a community level in comparatively poor countries and therefore maintenance requirements need to be as low as possible. DEWATS can be seen as an approach that takes specific local economic and social issues into consideration. The system provides treatment for wastewater flows from 1 m^3 to 1 000 m^3 per day with the ability to provide a renewable energy source in form of collected biogas, which can be reused on-site for cooking, lighting or power generation. Additionally, the system generally functions without energy input and therefore in itself has a positive energy balance. DEWATS can be constructed from locally available materials that suit the quality standards of the plant. Pollution load can be reduced to local requirements but generated solid waste, in form of sludge, needs be treated and disposed in a way so as to meet local hygiene and environmental standards [7]. Common high flow fluctuations and organic shock loads for domestic wastewater sources are not a problem for DEWATS [8]. DEWATS is a modular system and the flow to the plant can be realised by simplified sewer systems operated under a hydraulic gradient. The modules often get installed in form of ship containers, which makes the implementation suitable in case of space requirements and transportation.

1.3 Treatment systems of DEWATS

The DEWATS system is based on anaerobic digestion and includes four different treatment steps shown in Figure 1.

- Primary treatment step and sedimentation in sedimentation ponds, septic tanks, Imhoff tanks or bio-digester.
- Secondary anaerobic treatment step in form of a modified septic tank, known as ABR.
- Secondary anaerobic treatment step in fixed bed filters, known as anaerobic filters (AF)
- Secondary and tertiary aerobic/anaerobic treatment step in form of vertical flow or horizontal flow constructed wetlands

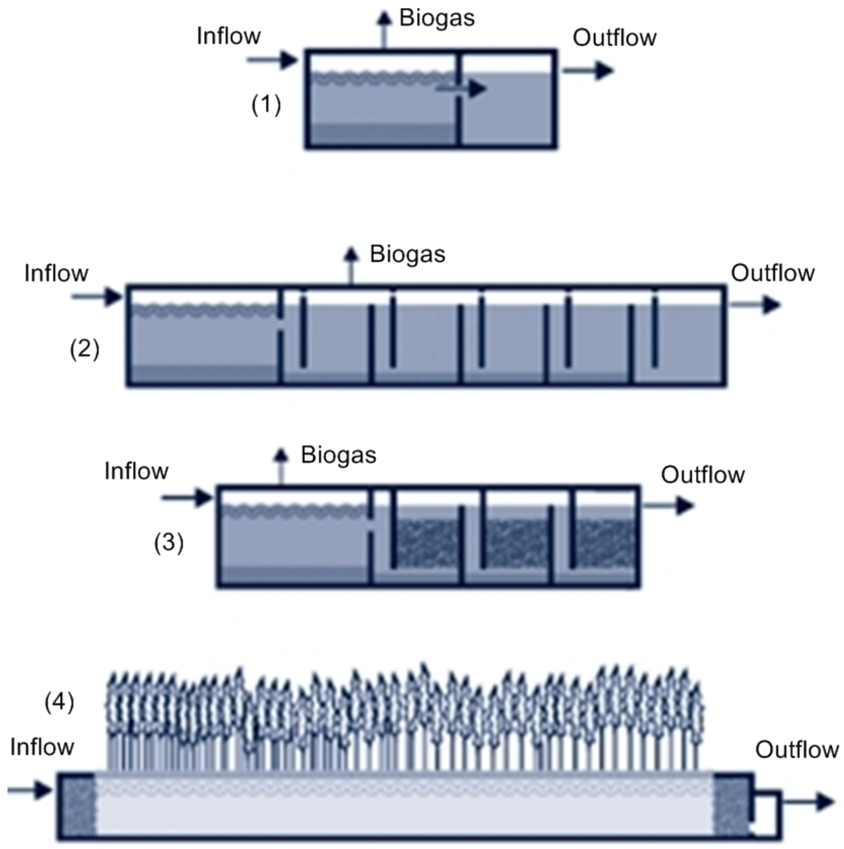

Figure 1: Main DEWATS-modules for physical and biological wastewater treatment. (1) Settler, (2) ABR, (3) AF, (4) Planted Gravel Filter. Adapted from [7].

Depending on the required local discharge standards and on the purpose of treatment, these modules can be combined in different ways. If the effluent wastewater should be used for irrigation, special post treatment requirements need to be considered in order to minimise the health risk. The wastewater characteristics in DEWATS plants differ from centralised systems. Higher organic loadings are expected due to a lower water usage in poorer households. Furthermore DEWATS is designed to only treat domestic wastewater without an influence of stormwater, which in the case of South Africa is ensured by separate storm- and wastewater sewers [8].

1.3.1 Wastewater characteristics concerning design parameters

A typical DEWATS system is suitable for industrial wastewater as well as domestic communal wastewater. Due to the scope of this thesis further sections will concentrate on domestic wastewater. Domestic wastewater in this case is the mix of black and greywater leading through a sewer to the DEWATS without any influence of stormwater.

The wastewater components can be divided into different main groups such as microorganisms, biodegradable and non-biodegradable organic materials, metals and other inorganic materials and odour, such as hydrogen sulphide. Variations in the composition of domestic wastewater are significant in terms of place and time. Daily, weekly and monthly fluctuations are important for the design and operation [11].

The actual design of DEWATS is calculated after a spreadsheet developed by Ludwig Sasse [8]. In this procedure several parameters need be investigated in order to calculate the dimensions of the different treatment modules. Data that are important for the correct DEWATS design after [8] are:

- Daily wastewater flow (calculations are often made with the per capita wastewater consumption if appropriate flow data are unavailable. Special attention needs to paid in case of system where it is possible that not all used water ends up as wastewater in the sewer system [7])
- Hours of major wastewater flow
- Average organic loadings in form of COD and the ratio to BOD_5
- Suspended solids content and fraction of settleable solids
- Ambient temperature and temperature of wastewater at source

It is not always possible to get all this information before designing the DEWATS plant. While it is far more important to have knowledge about the hydraulic loading, average local estimations for COD can be used. Hydraulic loading rates have direct effect on the hydraulic retention time (HRT) of the system and maximum peak flows determine the maximum upflow-velocity. Since there is some well researched knowledge about these parameters, the treatment plants are mainly designed on the hydraulic load characteristics rather than on the organic loading. The organic loading rate only becomes the limiting factor if the influent wastewater is characterised as a strong wastewater, which is not the case for the treatment of domestic communal wastewater. Nevertheless, problems can occur if a system is under- or overloaded. At too high loading rates, the microorganisms might not be able to consume the end products from the prior fermentation step. Too low loading rates, on the other hand, result in low production of sludge and the bacteria will degrade. Consequently, incoming wastewater does not get in adequate contact with the microorganisms for decomposition [8].

1.3.2 Treatment modules used by the investigated DEWATS plant

Besides the gravel filter, all other DEWATS modules are based on anaerobic digestion. The treatment aspects in the different modules will be explained in the following sections. Furthermore, only the modules, which are used by the investigated plant, will be included in this section.

1.3.2.1 Septic Tank

The septic tank, also known as a settler, is the first treatment step if an anaerobic digester is not included in the treatment system. Figure 2 shows a typical design for a septic tank. Basically it serves as a sedimentation tank. When wastewater flows into the tank, the heavy particles sink to the bottom and over time the organic particles degrade to more stable compounds. At the top, a scum layer builds up which is caused by the lower density of fat, oil and grease and the process of flotation due to gas evolution from the anaerobic process. Dissolved and suspended matter leaves the tank mainly untreated [8, 12].

The septic tank typically consists of two chambers. The first chamber should be at least 2/3 of the total length of the second chamber. The separation between the two chambers serves as a grease trap to prevent scum and solids escaping with the effluent [12].

With turbulent flow, the contact between fresh and already active substrate is more intense and degradation starts more quickly. Fermented solids that are not completely digested leave the tank and the effluent is malodourous. The septic tank needs to be emptied every two to five years, depending on the designed volume. If accumulated sludge fills up to 2/3 of the total volume, the removal of settleable solids decreases drastically [8, 12].

The septic tank is often used as the primary treatment step in a full DEWATS system, but can also stand by itself. After [8] the HRT should be around 48 hours to achieve a proper treatment. This would require very large reactor volumes in case of a high wastewater inflow to the plant. Therefore an HRT of 1.5 to 2 hours is often chosen for systems where the effluent of the septic tank gets treated in secondary and tertiary treatment steps. Other literature has shown that after a retention time of three to four hours the BOD removal efficiency does reach 40% and does not increase significantly anymore [25].

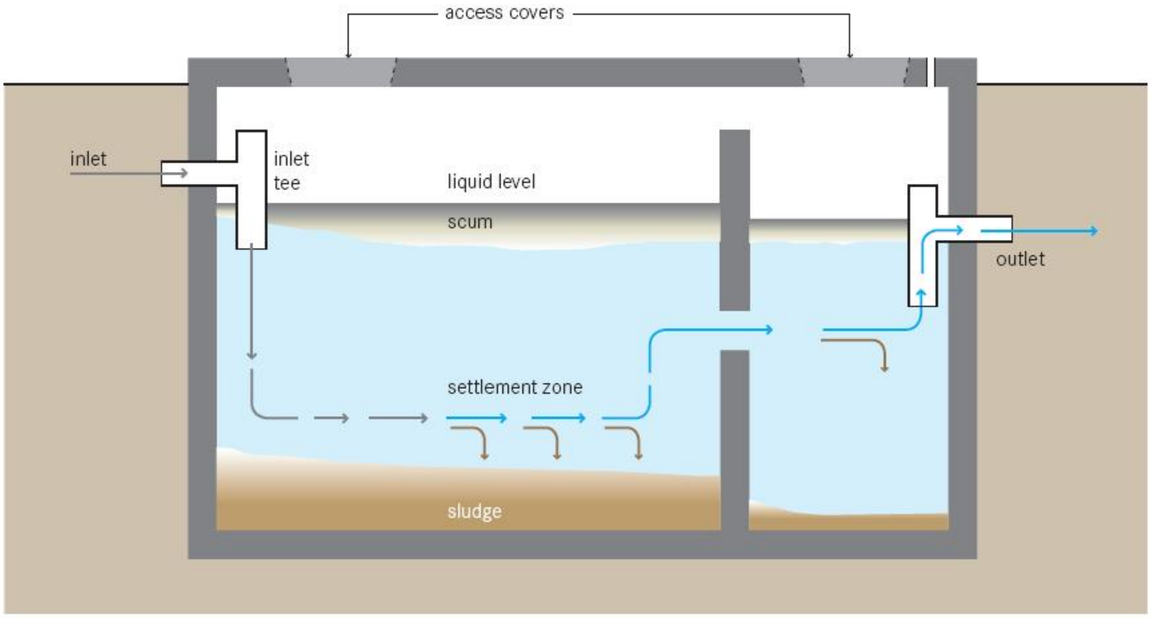

Figure 2: Schematic design of a septic tank. Adapted from [12].

1.3.2.2 The Anaerobic Baffled Reactor

The ABR is the main core treatment step of DEWATS and usually follows as a secondary treatment step after the septic tank. The ABR is basically a modified septic tank where the wastewater is forced to flow through a vertical baffle, which ensures an increased contact time with active biomass [8]. The ABR can be described as a series of upflow anaerobic sludge blanket (UASB) reactors. The main difference to the UASB is that the sludge blanket in an ABR does not need to float and that granulation of the biomass is not required. Particles can settle and accumulate at the bottom of each chamber, which is a main advantage over the UASB reactor in terms of control problems and costs [8]

Figure 3 shows a typical ABR reactor with three chambers. In many cases the first settling step occurs in the septic tank and there is no need for another sedimentation step if the systems are combined. The ABR system is known to have a good resilience to hydraulic and organic shock loadings [16]. Further advantages against systems like the AF or the UASB are longer biomass retention times, lower sludge yields and the ability to partially separate the different phases of anaerobic digestion [16].

The first studies on ABRs have been investigated in the 1980s [14].It is assumed by [15] that the ABR has the ability to separate the anaerobic processes of acidogenesis and methanogenesis. This separation makes the system more resistant to changes in pH and temperature. More recent studies conducted by [17] have shown similar results, where the first compartment had the greatest variety of hydrolysing and acid producing microorganisms. The microbial populations decreased over an eight chamber ABR. It suggested that biomass activity and therefore the reduction of organic matter is highest in the first compartments [17]. Limited or no phase separation occurs in the treatment of high particulate wastewater, such as domestic wastewater. This is because hydrolysis is the rate-limiting step [30].

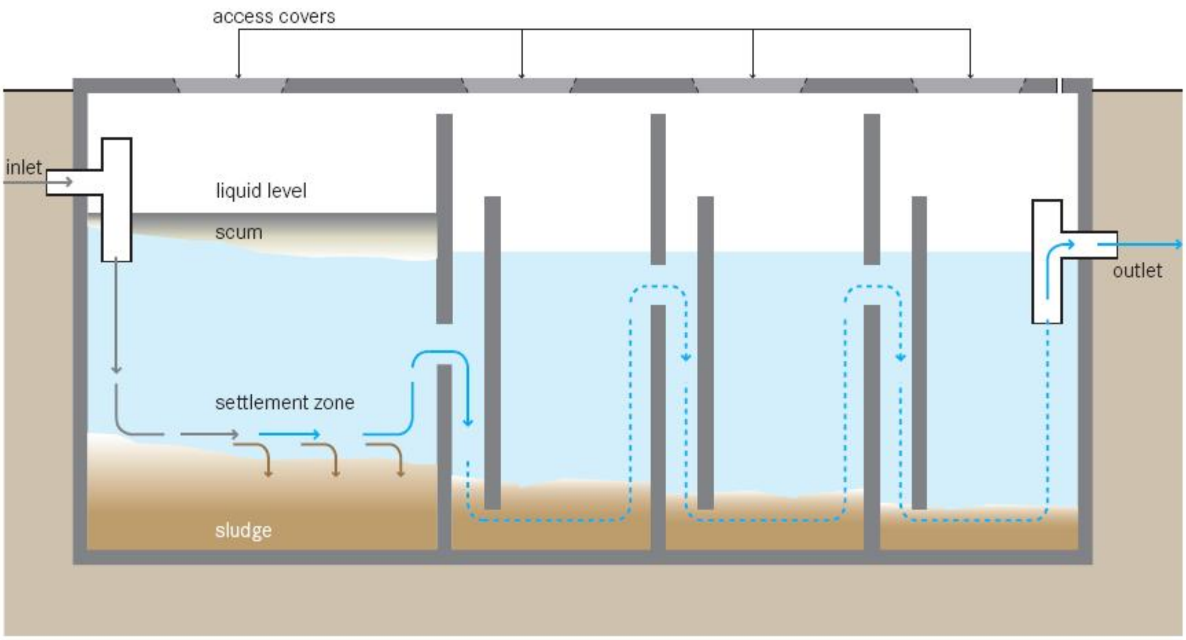

Figure 3: Typical three chamber ABR with vertical baffles and a settling step as the first chamber. Adapted from [12].

It is well known that an ABR reactor should consist of at least three chambers but little research has been done on the comparison of reactors with different compartments treating the same wastewater source with similar loading rates [16].

An important parameter in the decision making for the right number and the size of the compartments is the HRT of the system. It describes the volume of wastewater per day applied per volume of reactor. Sasse [8] states in the DEWATS design manual that HRT should not be below 8 hours. Values for the optimum HRT for anaerobic digestion in ABRs vary in literature from 6 to 36 hours. Some studies have shown COD removal efficiencies up to 90% at HRTs of 10 hours while other studies could not reach these removal efficiencies at retention times of more than 20 h [17, 20]. Since the studies have been investigated under different circumstances such as reactor volume, source of wastewater, temperature, these values should not be seen as absolute.

The biochemical and microbial processes inside an ABR are well documented and the system generally shows satisfying removal rates. A comparative study of [19] has shown that COD removal rates are more likely to change with changing hydraulic retention times than changing feed concentrations. An increase in the organic loading rate (OLR) from 9.6 kg COD/(m^3·d) to 19.2 kg COD/(m^3·d) has shown a drop in the COD removal rate from 96% to 90% for a constant HRT, while for a constant feed concentration and a changing HRT the efficiency dropped from 90% to 54%.

Another important design consideration of an ABR is the liquid upflow-velocity. This factor is influenced by the hydraulic loading rates and calculated by dividing the wastewater flow by the available area of one ABR chamber. The main design criteria is that liquid upflow-velocities need to be lower than the settling rate of the sludge particles in order to prevent biomass wash-out. Since wastewater flow does not stay constant during the day, the maximum upflow velocity is calculated from the maximum peak during one day. This varies with countries, location and the specific use of the ABR system. DEWATS plants designed by BORDA for domestic wastewater generally have an upflow-velocity that is not higher than 1 m/h [8]. An experimental pilot scale setup by [30] has shown how vital upflow velocities are to the performance of an ABR. Velocities of 0.55 m/h were shown to be too high for stable operation and to prevent microorganism washout. The results in [30] suggested that the upflow velocity is the limiting factor that determines the organic and hydraulic loading rates that may be applied in an ABR design. The main advantages of the ABR system against other wastewater treatment systems are shown in Table 1 [16].

Table 1: Main advantages of the ABR reactor technologies against other systems like the UASB. Adapted from [16].

Construction	Biomass	Operation
• simple design	• Low sludge generation	• Low HRTs possible
• no moving parts	• high solid retention times	• intermittent operation possible
• no mechanical mixing	• retention of solids without fixed media	• extremely stable to hydraulic shock loads
• cheap to construct	• no special gas or sludge separation required	• protection from toxic material in influent
• high void volume		• long operation times without sludge wasting
• reduced clogging		• high stability to organic shocks
• reduced sludge bed expansion		
• low capital and operating costs		

1.3.2.3 The Anaerobic Filter

The AF is the next secondary treatment step that can be involved in a combination of a septic tank and an ABR. It is often the first polishing step. The term AF is often misleading, since it is actually a fixed-bed reactor. AFs are used for wastewater with low fraction of suspended solids and therefore need a primary treatment step. Commonly used filter materials are gravel, crushed rocks, cinder or specially formed plastic packing material. The material size ranges from 12 to 55 mm and ideally provides 90 to 300 m^2 of surface area per 1 m^3 [8]. Filters usually have two or three layers of material with decreasing size and the water level above the filter media should be at least 0.3 m to guarantee an even flow regime through the filter [21]. Figure 4 shows a typical design for a one chambered AF, including a sedimentation step. The wastewater usually flows from the bottom to the top, comes in contact with the biomass on the filter and is subjected to anaerobic degradation [8, 21].

The number of AF chambers that are included in a treatment system can be decided for the specific requirements of effluent quality and the nature of the post treatment. However, usually no more than 2 to 3 compartments are installed into a DEWATS system, due to the higher investment costs of filter material [8]. Additionally, AFs can clog when subjected to high pollution loads, especially with a high content of suspended solids. Back washing is a possibility to clean the filter, but this requires an additional outlet pipe and makes the design of the AF more complicated. Usually the filter material gets removed, washed and replaced after cleaning. In general this needs to be done every 5 to 10 years [8, 21].

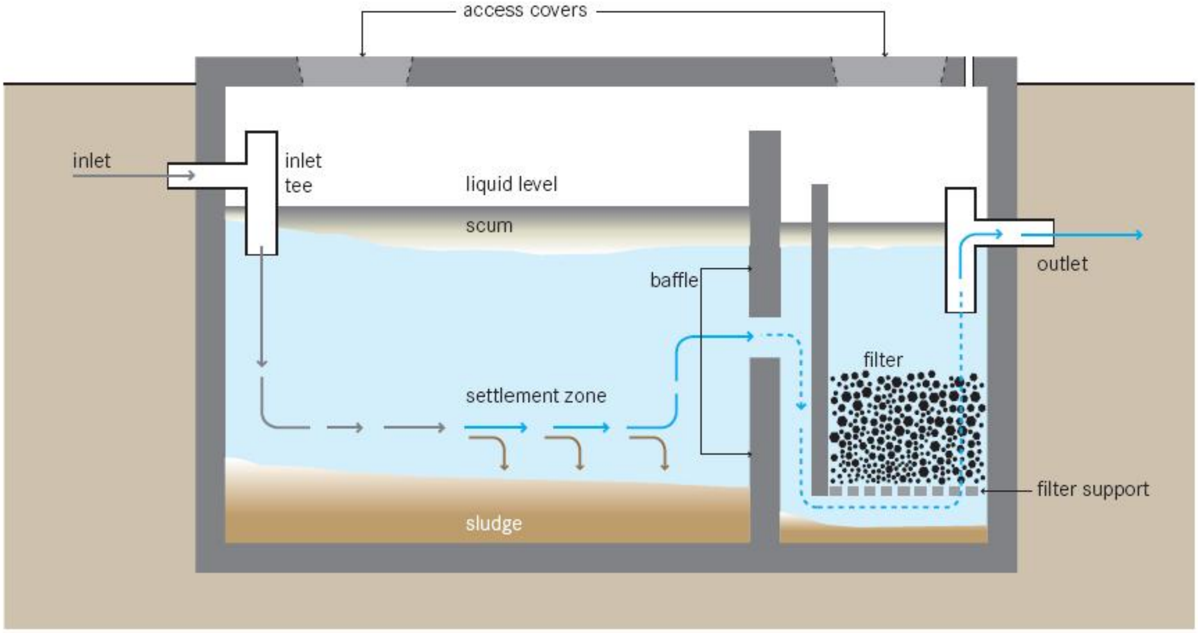

Figure 4: The AF in combination with a sedimentation step. Adapted from [12].

Studies on AFs report different suggestions for HRTs and surface loading rates. Harindra Corea (cited in [21]) suggested a maximum surface loading rate of 2.8 $m^3/(m^2{\cdot}d)$ and a minimum retention time of 0.7 to 1.5 days, while [8] reports a maximum HRT of 1.5 to 2 days. COD removal rates can be as high as 85 to 90% and generally range within 50 to 80% [21].

A disadvantage of the AF is the long start up time of six to nine months due to the time required for the anaerobic biomass to stabilise. Active sludge should be sprayed on the filter material before starting continuous operation. The flow rate should be slowly increased, starting with a quarter of the total flow [8].

1.3.2.4 Vertical flow constructed wetland

The vertical flow constructed wetland (VFW) serves as a tertiary treatment step in the DEWATS system and is usually the final polishing step. Water must be applied intermittently and evenly on the filter surface. Feeding in doses ensures even water distribution over the system and resting times are necessary to enable oxygen to enter the filter after wastewater has percolated. The doses must be large enough to completely flood the filter for a short while. Vertical filters are often planted with macrophytes (aquatic plants) in order to improve the treatment performance of the filter [8, 21]. The most important difference between a VFW and a horizontal flow wetland is not only the direction of the flow, but also the aerobic conditions inside the VFW. Due to the intermittently fed flow the filter goes through stages of aerobic and anaerobic conditions [12]. Aside from the disadvantage of eventually “bad odour” from the anaerobic pre-treatment, the VFW is the more efficient and reliable treatment system [8].

Figure 5 presents a typical VFW. The filter material usually consists of a rough bottom layer (e.g. stones), a medium middle layer (e.g. gravel) and a fine top layer such as sand, topsoil or mulch [8, 21]. The area below the filter media is a free flow area, which is connected to the drainage pipe. The drainage pipes have openings above ground and as a reason of that the free flow area is also connected to the open. The water gets distributed by a mechanical dosing system. A siphon is the most common installation and ensures intermittently fed volume with appropriate volumes and time intervals. Equal water distribution is assured by the fine top layer, while the middle layer is the actual treatment zone. The bottom layer provides open pores in order to reduce the capillary forces which would decrease the effective hydraulic gradient [8].

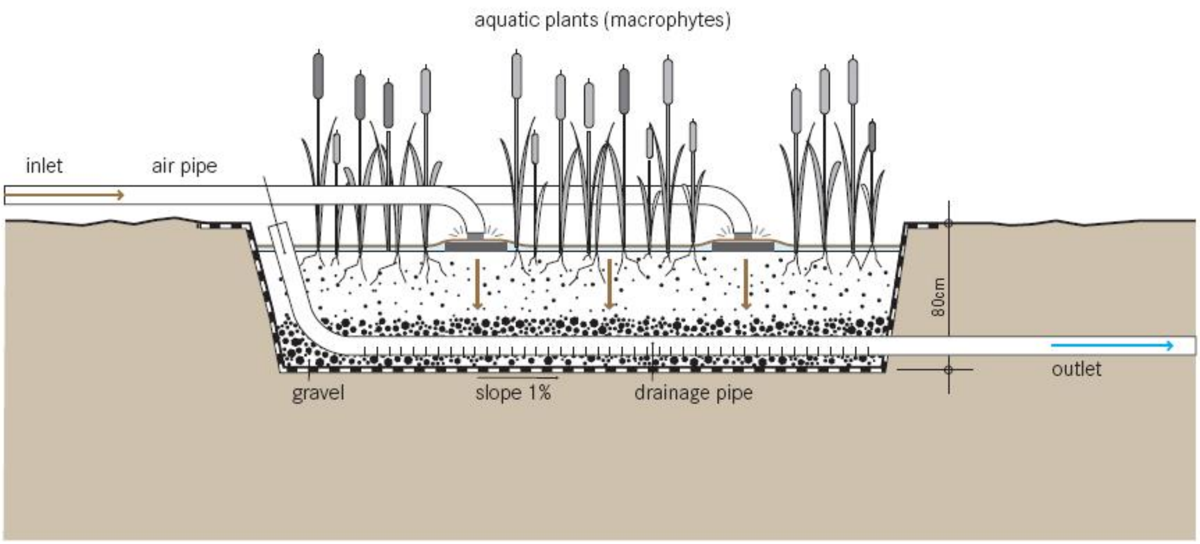

Figure 5: Typical vertical flow constructed wetland, planted with macrophytes and fed by a mechanical dosing system. Adapted from [12].

The wastewater undergoes physical, chemical and biological treatment in the VFW. Mechanical filtering and sedimentation remove suspended solids and degradation occurs by microorganisms which are colonising the filter bed. Microorganisms can grow and reproduce while the organic matter is mineralized and nutrients get partly removed [21].

One of the main risks of the VFW is clogging. Pre-treatment is necessary to avoid filter operational problems. This is ensured if the VFW is installed after an ABR system.

Usual depths of vertical flow filters are 1 to 1.2 m. But if there is enough ventilation and a good natural slope, the filters can also be built up to a depth of 3 metres. Hydraulic loading rates should not exceed 50 L/m^2·d with organic loads of around 20 g BOD/m^2·d. If the wastewater is pre-treated, the hydraulic load is the deciding design factor [8].

Vertical flow wetlands, if working properly, can produce high quality effluents with less than 10 mg BOD/L and a removal efficiency of 90 to 95%, while nitrogen removal is limited to 30 to 40% [21].

A study investigated by [22] has shown that the system of a VFW is capable of treating organic loads up to 200 g COD/m^2·d, with removal rates of 140 to 195 g COD/m^2·d. It was also shown that an extended sand layer, above the gravel layers, of 30 cm thickness significantly improves the performance of the system. A total bed thickness of 80 cm proved to be adequate.

1.4 The treatment process of anaerobic digestion

Anaerobic digestion is carried out by a large variety of microorganisms that convert organic compounds to methane and carbon dioxide [11]. It has long been used for the stabilisation of wastewater sludge and is now successfully integrated into the treatment of domestic and industrial wastewater. The process has several advantages over aerobic processes. This includes lower treatment costs since no artificial oxygen supply is required, lower sludge production, production of biogas, suitable for high strength wastewater and preservation of anaerobic microorganisms even after an extended time without any feed. The disadvantages of anaerobic digestion are that it requires a long start-up period, the generally slower treatment process and a high sensitivity against factors like temperature, pH and upsets due to toxicants [13].

The microbiology of the anaerobic digestion is very complicated. The consortia of microorganisms that are involved in the process are mainly bacteria and methanogens [13]. Anaerobic digestion can roughly be divided into four categories of microorganisms that transform complex materials into simple molecules like methane and carbon dioxide. The overall simplified reaction is shown in Eq. 1

$$\text{Organic matter} \rightarrow CH_4 + CO_2 + H_2 + NH_2 + H_2S \qquad \text{(Equation 1 [13])}$$

The first stage of the process is called hydrolysis. Complex organic molecules like proteins, cellulose, lignin and lipids get broken down into soluble monomer molecules like amino acids, glucose, fatty acids and glycerol. These molecules are directly available for the next group of bacteria. The process is enzymatic and can be catalysed by extracellular enzymes [11, 13].

The second stage is named acidogenesis. It is a biological process where acid-forming bacteria convert carbohydrates, amino acids and fatty acids into organic acids such as butyric, propionic acids and others. Acetate and other end products (CO_2 and H_2) are formed as the main products of the carbohydrate fermentation. The formed products vary with the conditions concerning pH, temperature and redox potential [11, 13].

The bacteria of the third step (acetogenesis) and microorganisms in the fourth step (methanogenesis) have a symbiotic relationship. Acetogenic bacteria convert the organic acids and alcohols into acetate, hydrogen and carbon dioxide. These end products are then used by the methanogens. Acetogenic bacteria require low hydrogen partial pressure for the conversion process which methanogens help to achieve by consuming hydrogen. The reduction of CO_2 by hydrogen forms the first group of methane producing microorganisms. The second group uses the produced acetate to form methane and CO_2 as a second end product [13].

The whole process and the interactions of the different stages of anaerobic digestion are very complex. The methane-forming microorganisms in particular are highly sensitive in changes of their environment. They are strict anaerobes and extremely sensitive to changes in alkalinity, pH and temperature. Several other conditions that should be monitored to maintain optimum conditions are gas composition, HRT, and volatile and acid concentration [24]. Further parameters influencing the anaerobic microorganisms are the chemical composition of the wastewater, competition of methanogens and sulphate-reducing bacteria and the presence of toxicants [13].

1.5 Control parameters for DEWATS systems

Several control parameters are suggested for the investigation of DEWATS plants. Depending on the defined objectives the parameters can differ. This section presents the control parameters that were used in the scope of this thesis.

1.5.1 Chemical Oxygen Demand

The Chemical Oxygen Demand (COD) is the most common parameter used to measure organic pollution in wastewater. It reflects how much oxygen is needed to oxidise all organic and inorganic matter that can be found in water [8].

Organic substances that can be found in wastewater are mainly proteins, carbohydrates, fats and oils as well as some synthetic organic molecules that are not easily biodegradable. Concentrations are highly depended on the amount of water and products that are used in the households. Anaerobic digestion processes only degrade the biodegradable fraction of the COD. The ratio of COD and BOD is a good indicator to check the biodegradability of the wastewater. A ratio COD/BOD below 2 to 2.5 indicates easily degradable wastewater and is common for most domestic wastewater sources. If the wastewater of the households contains a high amount of oil, fat and synthetic surfactants which are used in detergents and may not be replaced by biodegradable detergents in middle and low-income countries, the amount of the non-biodegradable COD rises and results in lower treatment efficiency [8, 21].

Wastewater can be defined as slight/weak, medium or high depending on the COD concentration. Sasse [8] defined domestic wastewater with less than 500 mg COD/L as weak, whilst [23] defined domestic wastewater as highly, medium and slightly contaminated at COD concentrations of 800, 600 and 400 mg COD/L, respectively. DEWATS systems are also used for high strength industrial wastewater sources with over 80 000 mg BOD/L [8]. Therefore the definitions of high and low strength wastewater in terms of DEWATS are not as clear.

More of an interest is the organic loading rate of the system, which represents the amount of COD per m^3 digester volume per day. Easily degradable substrate can be fed at higher loading rates, since the bacteria involved multiplies fast and consumes organic matter quickly. At too high loading rates the end products of one anaerobic digestion step cannot be consumed by the microorganisms of the next step and the process could collapse. The system could turn sour (pH below 6) as methanogens cannot consume the acid end products of the acidogenesis phase [8].

The soluble and particulate parts of the COD give further information about the nature of the wastewater. The more soluble COD, the easier the substrate can be digested by the anaerobic food chain.

1.5.2 pH

The pH value is described as the negative logarithm to base ten of the hydrogen ion activity in mol/l. The value of 7.0 is defined as neutral in pure water. With the presence of acids and alkalis the pH changes [23]. A neutral pH in the effluent of the treatment plant is an indicator for of optimum treatment performance [8].

The value of pH has a major influence on anaerobic processes. The optimal pH range for methanogens is between 6.7 and 7.4, and the process may fail if the pH gets close to 6.0. Acceptable enzymatic activity for acid-forming bacteria occurs above pH 5.0 [13, 24]. Thus acidogenic bacteria are not inhibited by lower pH values and degradation and thus acid production does not stop. Since the methanogens would not consume the produced acid, the system pH lowers, turns sour and collapses. The increase of volatile acid levels serves as an early indicator of system upset [13]. Additionally, values above 8.0 are toxic and restrictive to methane-forming bacteria and would also lead to collapse of the system [24].

1.5.3 Alkalinity

The concentration of alkalinity in domestic wastewater is directly influenced by the composition of the wastewater feed. Alkalinity results from the presence of hydroxides, carbonates and bicarbonates. Dissolved in water they derive mainly from calcium and magnesium, while bicarbonates are the most common at pH values between 6 and 10, due to the carbonate equilibrium. Alkalinity is commonly expressed in terms of calcium carbonate (mg $CaCO_3$/L) [25].

Domestic wastewater, with high quantities of proteinaceous wastes results in high concentrations of alkalinity. During the degradation of protein, amino groups get released and ammonia is produced. Ammonia dissolves in water and forms ammonium bicarbonate along with carbon dioxide [24].

The stability of the DEWATS system is enhanced by a high alkalinity concentration. The alkalinity can decrease by an accumulation of organic acids due to the failure of methanogens. Moreover, a discharge of organic acids can cause a decrease. A drop in alkalinity usually causes a rapid change in pH values [24].

The optimum alkalinity values for methanogens in anaerobic digestion are 1500 to 3000 mg $CaCO_3$/L and the marginal conditions are 1000 to 1500 mg $CaCO_3$/L [24]. However [26] have studied the relationship between volatile fatty acids (VFA), alkalinity and pH and suggest alkalinity to be not lower than 800 mg $CaCO_3$/L in order to have a stable pH above 6.0.

1.5.4 Solids

Solids play an important role in anaerobic digestion. They can generally be separated into total solids (TS) and volatile solids (VS). Volatile solids represent the organic fraction of TS [24]. To determine the amount of TS, a sample is dried. The inorganic fraction is found as ash after ignition. TS minus the amount of ash will result in the amount of VS in a sample [8].

Another parameter to describe solids in wastewater is suspended solids (SS), and represents the amount of organic and inorganic matter that is not dissolved in water. SS include settleable solids and non-settleable solids. Settleable solids get removed by sedimentation and sink to the bottom of the treatment plant. Non-settleable suspended solids play an important role in DEWATS. If the upflow velocity of the treatment system is higher than the settling velocity, which is usually the case for these particles due to their small diameter, the solids get washed out and cause turbidity in the water which may cause clogging of the pipes and AFs [8]. VS, whether dissolved or suspended are reduced by anaerobic digestion. The parameters soluble COD and particulate COD provide a measure of these fractions.

1.5.5 Temperature

Temperature in DEWATS system is important for bacterial growth and especially for methane-forming bacteria, which are active in two temperature ranges. These ranges are the mesophilic range from 30 to 35°C and the thermophilic range between 50 and 60°C. Since DEWATS is a common treatment system in tropical areas, wastewater temperatures usually do not fall below 25°C. However, the optimum temperature for methane-forming bacteria is 35°C [24]. In principle a digester temperature above 18°C is acceptable [8]. Studies have shown have shown little difference in the methane production at different temperatures of 10, 15 and 25 °C [27]. Treating low strength domestic wastewater with an AF at 13°C has furthermore shown COD removal rates of 81% at an HRT retention time of 4h and methane concentrations of the Biogas as high as 82% [28].

1.5.6 Nitrogen

Nitrogen, and its different forms in wastewater, is a good indicator of what happens during wastewater treatment. As the major component of proteins, nitrogen can mainly be found in the form of free ammonia and ammonium. The ratio depends on the pH of the water and is illustrated in Figure 6. Since the wastewater in DEWATS plants usually has a pH of 7, ammonia-nitrogen is the most present form within the treatment plant. Ammonium evaporates into the atmosphere and would, in case of irrigation purposes of the effluent, lead to unwanted nitrogen losses [8, 24]. Although ammonia-nitrogen is needed for bacterial growth of microorganisms, there is no net removal inside an anaerobic treatment plant [24].

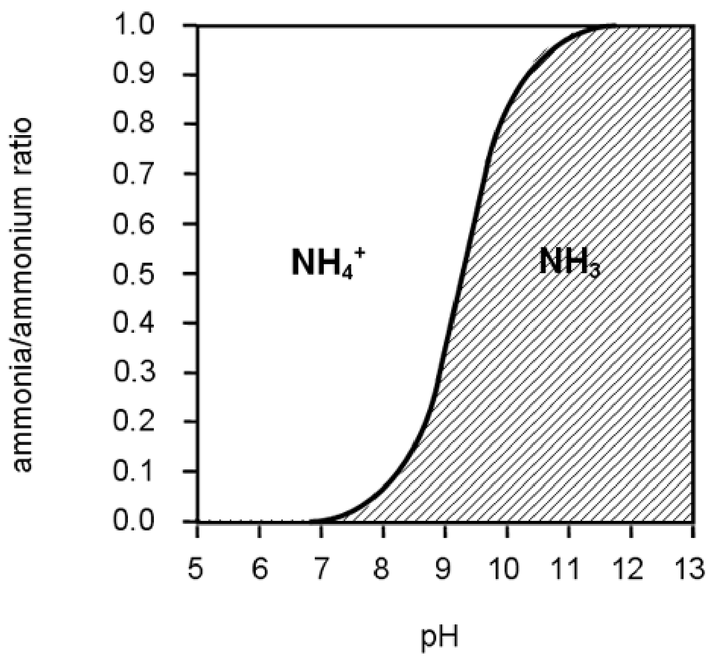

Figure 6: Ratio of ammonia and ammonium at rising pH values. Graphic adapted from [29].

High effluent values of nitrogen can cause eutrophication in open waters if directly discharged, but there is an opportunity to reuse these nutrients for agriculture purposes. In case of reuse purposes the nitrogen would not need to get removed through nitrification and the subsequent denitrification. The DEWATS plant is therefore independent from electricity supply [30].

1.5.7 Phosphate

Another macronutrient that is from interest inside DEWATS is phosphorous, mainly available as orthophosphate-phosphorous in wastewater. It is important for bacterial growth with an approximate relation for BOD/P of 100, or an N/P ratio of 5. The activity of microorganisms will be less if not enough phosphorous is available and result in a lower removal of BOD [8]. However, anaerobic treatment plants are not designed for phosphate removal and effluent concentrations are from minor interest if the water is not discharged into open water bodies. Furthermore phosphate rich effluents can be used for agriculture.

1.6 Scope of work and hypothesis

This thesis forms part of the larger research project (WRC Research Project K5/2002) evaluating the DEWATS process. The long-term objective of the research project is to evaluate the suitability of DEWATS processes for on-site sanitation in South Africa. This will be achieved by evaluating the operational boundaries of a BORDA DEWATS system and its treatment performance according discharge standards. The research was split into different research phases. This thesis includes the data collection and interpretation from research phases 2 and 3 and shows the performance of the DEWATS plant from October 2011 to May 2012. The outcomes from this thesis serve as a source of information for possible plant design improvements, plant operation and maintenance and future research activities.

A main part of this thesis deals with the problems that occurred during the research activities. The focus of research phase 2 was to evaluate the effect of rainfall events on plant performance and the overall performance of the plant in relation to design assumptions and South African discharge guidelines.

The aim of research phase 3 was to test the operation limit of the system. The objective was to assess the performance by evaluating different physico-chemical parameters through the DEWATS plant.

Hypothesis deriving from the aims and objectives of both research phases are:

1. Rainwater influences the performance of the plant.
2. The treatment plant shows satisfying results compared to South African discharge standards for the reuse of wastewater for irrigation.
3. The actual treatment performance meets the theoretical design performance.
4. The treatment plant cannot handle increased hydraulic loadings and collapses.
5. High upflow-velocities lead to migration and washout of sludge.

2. Material and methods

2.1 DEWATS treatment plant in Newlands-Mashu

The Newlands-Mashu DEWATS plant was built as an evaluation plant in October 2010. It is situated on 71 John Dory Road in Newlands, Durban. The site is used by the eThekwini Municipality as an educational facility for emerging farmers and small-scale growers. Practical assistance is provided and people from the community are supported by food programs that are running in the area.

The plant was designed for 84 households plus an additional safety factor of 10%. From Sasses' [8] design spreadsheet, the plant was designed for a total number of 462 persons. The houses are linked to an existing main trunk sewer to which the DEWATS plant was connected. The effluent of the treatment plant flows back into the sewer, ensuring that no wastewater is released into the surrounding environment. A barrier can be used at the influent of the plant to bypass the incoming wastewater and let it directly flow back into the main trunk sewer.

Since the plant was built for experimental purposes, the design of the Newlands-Mashu plant differs from other implemented DEWATS plants. Figure 7 presents a photograph of the installed DEWATS plant. The plant does not consist of a typical anaerobic digester. Instead a settling step which consists of two chambers was installed as the primary treatment step and also serves as a biogas collection point. The water is distributed into three parallel ABR treatment trains, of which two are identical in size and consist of seven compartments (train 1 and 2). Train 3 consists of four compartments. The first three chambers of treatment train 3 have the double length of train 1 and 2, while the fourth compartment is identical in size. Two chambers of AFs follow each ABR train. The effluent of each train flows through a magnetic induction Safmag flow meter [31]. Before reaching the final polishing steps of the VFW and HFW filter, the effluent of one train flows into a siphon chamber, which serves as a mechanical dosage system.

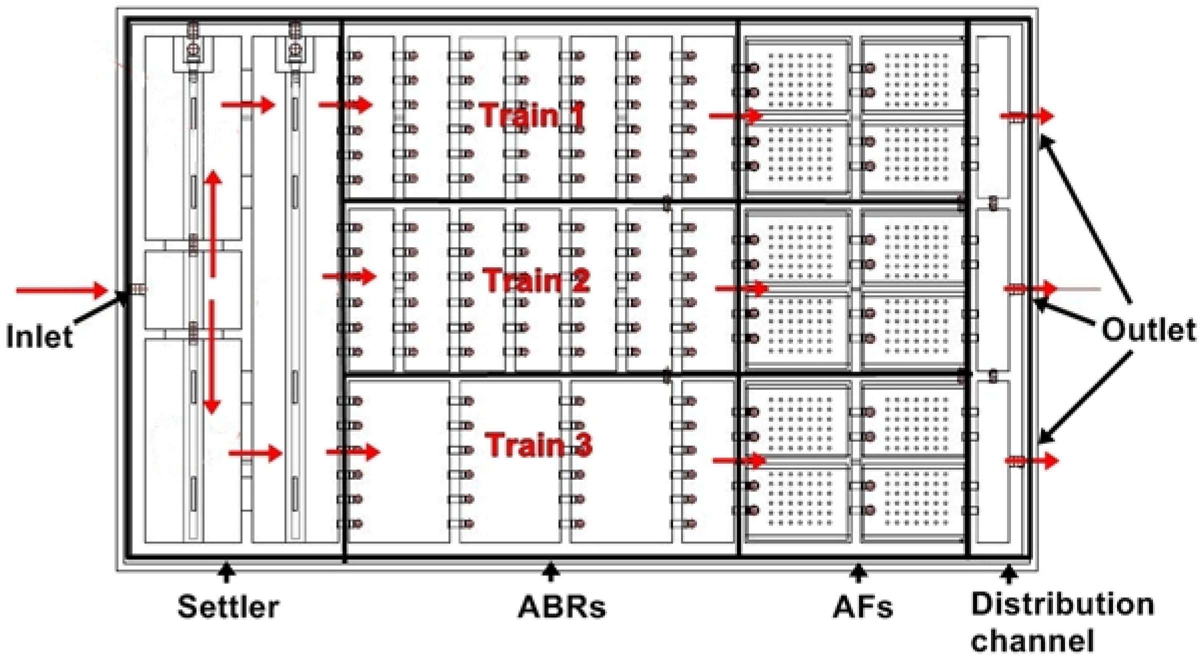

Figure 7: Technical design drawing of the Newlands-Mashu DEWATS plant, showing the settler, ABR and AF trains. Red arrows indicate the flow directions. (BORDA, 2012)

Pipes from the settler are connected with the agricultural hub on the site and the produced biogas can be reused for cooking purposes. A BORDA emergency container in form of an ABR is also connected with the settler and further research investigations can be done with sludge directly fed into the container. The container system was not within the scope of this thesis.

The effluent of the HFW can be pumped into a JoJo Tank, which serves as a water source for irrigation and can be used once the wetlands show acceptable results for the reuse in agriculture.

Figure 8 shows the side view of a treatment train with seven ABR compartments. The first compartment of the settler serves to hold back big solid material, while the second chamber is built gastight. This is ensured by sleeves from the top of the manhole cover.

Wastewater flows through 110 mm downflow pipes within the whole plant. The water level is kept constant on 180 cm. ABR compartment 6 and AF2 have PVC end caps with an eccentric hole of 50 mm on the outflow pipes. The end caps ensure that the water level can be controlled within each chamber individually and therefore the distribution of flow can be adjusted.

A brick wall with holes is installed at the bottom of the AFs and ensures an existing water level for sedimentation below the filtermaterial. The filtermaterial consists of two layers of crushed stones with a diameter of 60 to 80 mm for the first layer and 30 to 50 mm for the second layer. No specific information about the used material is available.

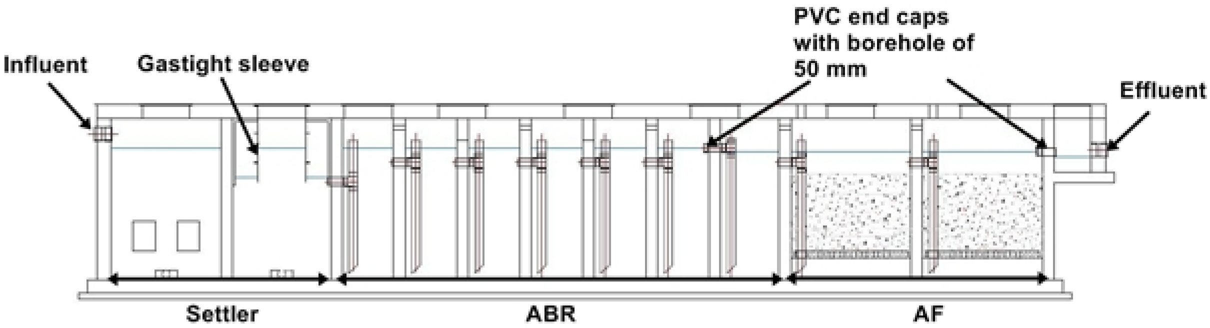

Figure 8: Technical drawing of the settler, an ABR train with seven compartments and the AF as a side view. (BORDA, 2012).

Figure 9 illustrates the siphon chamber and the VFW. The VFW was designed to treat one third of the total design wastewater flow volume. This is realised by splitting the effluent streams of the three trains. The siphon chamber serves as a storage tank. A mechanical floating device is installed inside chamber, which sinks down at a certain water height and feeds the VFW in doses. The water flows through four pipes that are above the ground and have little holes which distribute the wastewater evenly over the whole area of the wetland. The material of the VFW consists of three layers. At the bottom a drainage layer of washed gravel with a grain size of 19 to 25 mm and a thickness of 150 mm is used. The top layer consists of fine washed gravel with a grain size of 7 to 13 mm and a thickness of 50 mm. A double laid shade net separates the top and bottom layer from the middle layer which consists of sieved river gravel with a grain size of 2 to 4 mm and a thickness of 550 mm. A drainage pipe with drilled holes for ventilation above the ground is installed at the very bottom of the VFW and allows the treated wastewater to flow off to the HFW or can be distributed back into the main sewer. The VFW was planted with the macrophytes *Typha Capensis* by co-workers of the AMU in the end of May 2011. The plants were growing in a water dam before they got planted into the VFW and have been watered with tab water until the effluent of one train was connected to the siphon.

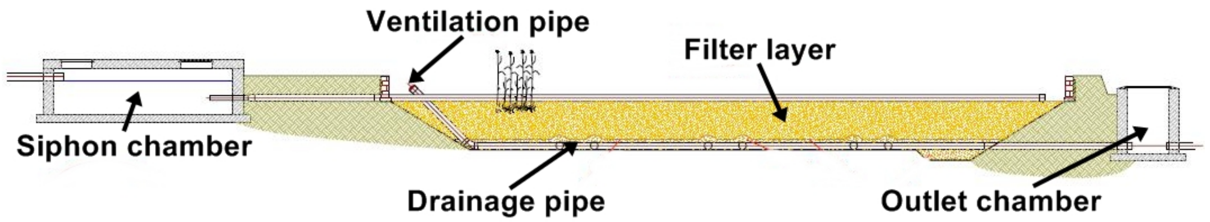

Figure 9: Technical drawing of the siphon chamber and the VFW as a side view. The scale of the siphon chamber does not reflect the real size. (BORDA 2012)

2.2 Hydraulic and organic loading design parameters

The Newlands-Mashu DEWATS plant is designed after a spreadsheet developed by Ludwig Sasse [8]. The parameters, the dimensions of the system are based on, are presented in the following tables. Important hydraulic parameters are defined in Table 2. The equations that were used for the calculations of the design parameters in Table 3 and Table 4 are also included in this section.

Table 2: Definition of important hydraulic parameters for the dimensioning of a DEWATS plant.

Parameter	Definition
Average daily flow [m³/d]	Average wastewater volume flowing into a treatment plant in 24 hours.
Average hourly flow [m³/h]	Average daily flow divided by 24 hours.
Maximum peak flow [m³/h]	Highest hourly flow to a treatment plant during 24 hours. In purpose of the dimensioning of DEWATS it is the daily wastewater flow, divided by the hours of maximum flow (8 hours).

Hydraulic Retention Time

$$\text{HRT [h]} = \frac{\text{Volume of the reactor } [\text{m}^3]}{\text{Average daily flow } \left[\frac{\text{m}^3}{\text{d}}\right]} \cdot \frac{24\text{ [h]}}{1\text{ [d]}} \quad \text{(Eq. 2)}$$

Upflow-velocity

$$\text{v} \left[\frac{\text{m}}{\text{h}}\right] = \frac{\text{Flow } \left[\frac{\text{m}^3}{\text{h}}\right]}{\text{Reactor area } [\text{m}^2]} \quad \text{(Eq. 3)}$$

Organic load

$$\text{OL} \left[\frac{\text{kg}}{\text{d}}\right] = \text{COD inflow } \left[\frac{\text{mg}}{\text{L}}\right] \cdot \text{Average daily flow } \left[\frac{\text{m}^3}{\text{d}}\right] \cdot \frac{1000\text{L}}{1\text{ m}^3} \cdot \frac{1\text{ [kg]}}{10^6\text{ [mg]}} \quad \text{(Eq. 4)}$$

Organic loading rate

$$\text{OLR} \left[\frac{\text{kg}}{\text{m}^3 \cdot \text{d}}\right] = \frac{\text{OL } \left[\frac{\text{kg}}{\text{d}}\right]}{\text{Volume of the reactor } [\text{m}^3]} \quad \text{(Eq. 5)}$$

The hydraulic design parameters in Table 4 show the values for the entire treatment plant. Each ABR treatment train has a volume of 22.05 m³. Train 1 and 2 consist of seven equally built compartments. The first three compartments in train 3 have double the volume as compartment 4, which is built the same as the compartments of train 1 and 2. The upflow velocity in the first three compartments of train 3 is therefore 0.5 m/h instead of 1.0 m/h. The volume of the AF is based on the water volume without the voids in filter mass. Furthermore is the maximum upflow-velocity the maximum velocity inside the filter voids.

Table 3: Planning base for the design of the Newlands-Mashu DEWATS plant.

Parameter	Unit	Value
Connected people	/cap	462
Wastewater production	$/L \cdot cap^{-1} \cdot d^{-1}$	90
Average daily flow	$/m^3 \cdot d^{-1}$	41.6
Organic matter	$/g \cdot cap^{-1} \cdot d^{-1}$	110
COD inflow	$/mg \cdot L^{-1}$	1222

Table 4: Hydraulic design parameters of the Newlands-Mashu DEWATS plant.

Parameter	Unit	Settler	ABR	AF
Volume	$/m^3$	31.5	66.15	26.66
Max. peak flow	$/m^3 \cdot h^{-1}$	5.2	5.2	5.2
Max. upflow-velocity	$/m \cdot h$	-	1	1.1
HRT	/h	2	36	15.4

2.3 Operational periods

Presented in Figure 10 are the research activities on the Newlands-Mashu DEWATS plant. The results of research phase 1 did not from part of this thesis, but operational knowledge was gained from this phase and is included in section 2.10. Research phase 1 involved the seeding and start-up of the Newlands-Mashu DEWATS plant. The plant was continuously fed from the 2nd November 2010 to the 18th July 2011. Other project team members were responsible for operation and maintenance, sampling and analysis of data. The plant underwent major revision construction at the end of research phase 1 to resolve some of the problems experienced during operation. The Newlands-Mashu DEWATS plant was restarted on the 27th October 2011 as part of research phase 2.

Analytical measurements of research phase 2 and 3 are included in this thesis. Research phase 2 was from the 27th October 2011 to 8th February 2012. During this period wastewater was running through all three treatment trains. The period was influenced by several technical and operational difficulties. Operational issues and the resolving of them are shown in section 2.10.

The plant was operating with all flow going through treatment train 1 during research phase 3. Data included in this thesis includes the period from the 22nd February 2012 to 1st May 2012.

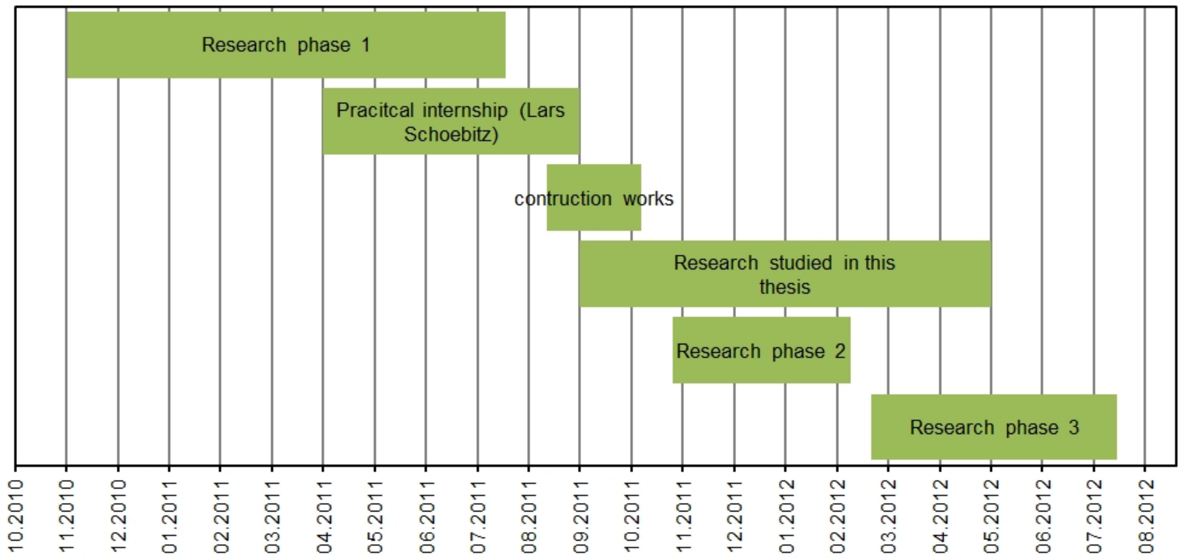

Figure 10: Timeline of research on Newlands-Mashu DEWATS plant. The plot shows the different research phases conducted on the research plant. This dissertation presents the data from research phases 2 and 3.

Several people were involved in the research of the Newlands-Mashu DEWATS plant. Table 5 shows the position of the different team members and Table 6 shows the responsibilities for the conducted research, studied in this thesis.

Table 5: Research team member of the Newlands-Mashu DEWATS project between 27th October 2011 and 1st May 2012.

Team Member	Position
S. Pillay (SP)	Project Manager (PRG, UKZN)
N. Reynaud (NR)	Ph.D. Student (BORDA, Universität Dresden)
L. Schöbitz (LS)	Diploma Student (BORDA, THM)
P. Sananikone (PS)	Design Engineer (BORDA)
B. Pietruschka (BP)	M.Sc. Student (BORDA, IHI Zittau)

Table 6: Responsibilities of the different team members during the operation periods of the research, studied in this thesis.(1) operation and maintenance, (2) sampling, (3) analyses, (4) data entry interpretation, (5) design drawings. Team members are abbreviated according to Table 5.

	Start	End	Team members	Responsibilities
Research Phase 2	27.10.2011	29.11.2011	SP, LS	(1), (2), (3), (4)
			PS	(1), (5)
	29.11.2011	08.02.2012	LS	(1), (2), (3), (4)
	08.02.2012	22.02.2012	NR, LS	(1), (2), (3), (4)
Research Phase 3	22.02.2012	02.04.2012	SP, LS	(1), (2), (3), (4)
	02.04.2012	01.05.2012	SP, NR	(1), (2), (3), (4)
			BP	(1), (2)
			PS	(1), (5)

2.4 Sampling campaigns at the Newlands-Mashu DEWATS plant

2.4.1 Equipment

- Beaker 1L
- Camera
- Cooler Box filled with Ice cubes
- Ethanol 70%
- Grab sampler, Volume 200 mL
- Latex gloves
- Merck alkalinity test kit (range 0.1 – 10 mmol/L; 1.11109.0001)
- Notebook and pencil
- pH-Meter (WTW, 340i)
- Plastic bottles 1000 mL
- Plastic buckets, Volume 10 L

2.4.2 Sampling points and procedure

The sampling of the DEWATS was planned to happen on a weekly basis, while trying to ensure that each sampling campaign was at the same day and time of the week. This would have given the most representative samples according to the wastewater flow entering the plant. Due to staff and maintenance issues was this often not realisable. Nevertheless were samples always taken during the morning between 9:00 and 11:00 AM, which also represents the daily peak flow of the treatment plant.

Influent wastewater samples were taken from the head of the DEWATS plant (compare Figure 11). Every ten minutes, six samples were taken with a 1 L beaker over an hour and placed in a 10 L plastic bucket. The contents of the bucket were mixed, stirred, placed in a 1 L plastic bottled and stored in the cooler box.

All plastic bottles at each sampling point were rinsed with the respective wastewater before the sample was placed into the bottle. Furthermore the bottles were filled up to the top to eliminate air bubbles. The bottles were labelled with the respective sampling chamber and reused at every campaign after cleaning in the laboratory.

No samples were taken out of the first settling chamber. During research phase 2, samples were taken at 2 different points in chamber 2 (Figure 11). Samples from settler 2a were taken inside the sleeve which is going down into the chamber using the 200 mL grab sampler. The accumulated top scum layer was removed and samples taken from the supernatant. During research phase 3, the method was changed on the 24th April 2012 by using the sludge sampling device in order to take the samples below the sleeve. The method was change because the scum layer on top of settler 2a could not easily be removed anymore and scum was caught into the sampling bottles. According to the design drawings no manhole should have existed at the sampling point settler 2b (further elaborated in section 2.13), but it was decided to take samples for comparison reasons.

Samples from the ABR and AF compartments were taken from the supernatant with the 200 mL grab sampler. Accumulated scum on top of the chamber was removed before sampling. This became a problem during research phase 3, since scum was pushing from the settler into the first ABR chamber. The samples were furthermore taken as close as possible to the outflow of the chamber in order to have a representative sample of the effluent conditions of each chamber. The characterisation of the effluent after each train was done by sampling the supernatant in AF2.

Effluent samples of the VFW were taken inside the manhole which is situated directly at the outflow of the wetland. The sample was taken with a 1 L beaker, by placing it directly under the outflow pipe.

Sludge heights were investigated by using a specially designed sludge sampler. It was made of a clear PVC tube with an inner diameter of 50 mm, including a metal rod with a rubber at the end of the rod. The metal rod was slowly lowered into the chamber until the rubber plug reached the bottom. It was then waited a couple of minutes in order to let the dispersed sludge settle again and the PVC tube was slowly lowered onto the metal rod until it reached the plug, which formed a seal at the bottom. The column was then removed from the chamber and the sludge was allowed to settle for another five minutes until visible solids had settled inside the PVC tube. The sludge bed height was recorded inside the data book.

Another parameter that was measured onsite was the pH-value using the WTW 340i pH-meter. The pH electrode was directly immersed into the ABR chamber. After each measurement destilled water was used to clean the electrode before measuring the next compartment.

Alkalinity investigations were also performed onsite, using the Merck alkalinity test kit. The procedure was followed after the enclosed Merck procedure. Wastewater was taken directly out of the respective chamber and the test tubes were rinsed with destilled water after each measurement.

After each sampling campaign the samples were taken back to the laboratories at UKZN and stored inside a cold room at a constant temperature of 4°C.

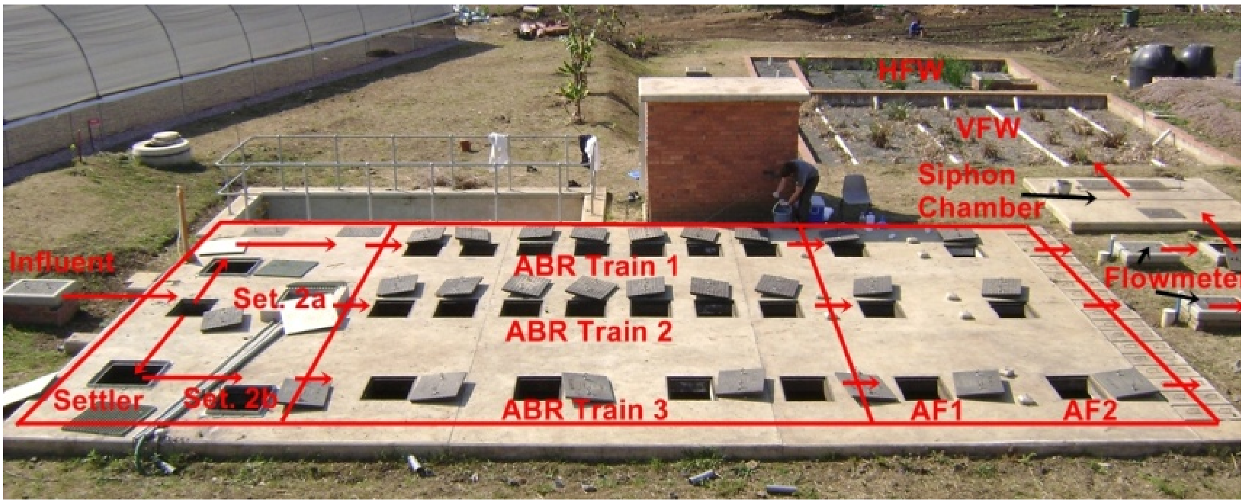

Figure 11: Photograph of the Newlands-Mashu DEWATS plant. The red arrows show the flow pattern through the plant. Incoming wastewater flows through a settling step, three ABR trains and flow velocities are detected by flow meters after each treatment step. The photograph was taken on the 23rd July 2012 by N. Reynaud.

2.5 Water Consumption

Data about the water consumption of the connected households were provided by EWS. Monthly water meter readings were available and calculated for the period between June 2011 and May 2012

2.6 Ambient temperatures

Temperatures have not been measured on site. The ambient temperatures of a climate station approximately 7 km away from the DEWATS plant were used. The data is available on http://www.wunderground.com/history/airport/FALE/2011/11/1/DailyHistory.html?req_city=NA&req_state=NA&req_statename=NA

2.7 Rainwater Investigations

Data about precipitation was available through the eThekwini Municipality who monitor rainfall events around Durban. The data is available on http://www.avitrack.co.za/dur/showmodels.php. The data used in this thesis if from Newlands3 reservoir at Seabass Road, Newlands East, approximately 600 m away from the plant. Data can be detected for every 5 minutes and gives results for the minimum amount of 0.2 mm, which equals 0.2 L/m².

2.8 Flow measurements

Overall seven Safmag electromagnetic flow meters were installed at the DEWATS plant. The effluent flows were detected after each ABR-AF train. The installed flow meters were sized to match the actual pipe diameter of 50 mm. Once the electrical supply was ensured, the flow was be detected by BETA converters and was available online through the beyond wireless system see http://portal.beyondwireless.co.za/. From the stage of research phase 2 it was learned that flows through these lines were far too low for measurement with the current setup. Trial tests have been done with the high end signal DCMPU unit which is able to convert the very small signal generated by the flow condition. Flow measurements became one of the biggest maintenance issues during the project and the handling of this problem is further elaborated in section 2.10.4

During research phase 1, flow measurements with buckets have been investigated but could not show satisfying results, due to flow fluctuations that could be observed within two measurements of several minutes.

2.9 Laboratory analysis of physico-chemical parameters

2.9.1 Equipment

- COD glass test tubes
- Centrifuge Z323, Hermle, Germany
- Distiller (4 L/h), Boeco, Germany
- Eppendorf pipette (range 0.2 to 1.0 ml)
- Eppendorf pipette (range 1.0 to 5.0 ml)
- General glass test tubes
- Glas beaker, volume 1 L
- Latex gloves
- Magnetic stirrer
- Magnetic stirrer bar
- Mechanical blender
- Merck NH4-N kit (range 2.0 to 75.0 mg/L NH4-N, 1.00683.0001)
- Merck PO4-P kit(range 1.0 to 100. mg/L, 1.00798.0001)
- Merck 1. COD solutions (range 100 to 1500 mg/L, 14538.0065)
- Merck 1. COD test tube method (range 100 to 1500 mg/L, 14541.0001)
- Merck Spectroquant® Nova 60
- Merck Spectroquant® Thermoreaktor TR 320
- Paper towels
- Plastic flasks for centrifuge, 40 mL
- Rectangular cells (10 mm), Merck, Germany
- Test tube rack

2.9.2 Chemicals

- Distilled water (produced in the laboratory)
- COD solution A (114538), Merck, Germany
- COD solution B (114539), Merck, Germany
- Ammonium test kit (100683), Merck, Germany
- Phosphate test kit (100798), Merck, Germany

2.9.3 Sample preparation and analysis

The analytical measurements were performed on the same day as the sampling campaign. Samples with an obvious amount of solids (feed and settler) were transferred into a mechanical blender and homogenised. The samples were then transferred into a 1 L beaker including a magnetic stirrer bar and placed on a magnetic stirrer. For the measurement of soluble COD, NH_4-N and PO_4-P, 40 mL of each sample was placed into a plastic flask and ultra-centrifuged for 15 minutes at 10 000 g to remove suspended and particulate matter.

The steps for the actual measurements have been followed after the respective Merck information sheets (see section 2.9.1). The samples of the feed and settler were diluted 1:1 with distilled water.

All parameters were analysed with the Merck Nova 60 Spectroquant®. In order to financial restrictions in chemical supply and due to a performed standard KHP test for COD measurements by Pillay and co-workers during research phase 1, no replicates have been measured. Nevertheless, the test tubes for all parameters were measured three times in order to check the accuracy of the photometer.

2.9.4 Preparation of the COD testkits

Total and soluble COD measurements were performed using the Merck Spectroquant® method. The test cells had to be prepared in the laboratory by using the Merck COD solutions A and B for the Merck COD cell test method 14541 and a measuring range of 100 to 1500 mg COD/L. To prepare the test cell 0.30 mL of solution A and 2.30 mL of solution B were pipetted into cleaned test cells and swirled until any bottom sediment was suspended. The cells could then be used for COD analysis.

2.10 Operational, maintenance and design issues influencing the outcome of this thesis

While monitoring the Newlands-Mashu DEWATS plant, several technical difficulties were experienced. These difficulties had a major influence on the research progress and the sampling campaigns. Some problems were related to design and construction flaws whilst others concerned with operational and maintenance problems. Since the plant was built as a research plant, some of the technical difficulties would not be present in a field based system. This section shows how the problems were handled and solved and focuses on the influence of research results. This information could also be used to assist designers and operators of anaerobic and large-scale experimental systems.

2.10.1 Flow measurements and uneven flow distribution

The Newlands-Mashu DEWATS plant was designed to have three parallel trains. An advantage of the design is that studies can be done on parallel ABR-AF trains, which are loaded with the same wastewater source and operating under similar OLRs and HRTs. However, this became a major problem while operating the plant. This chapter shows all issues that are related to the flow measurements and how it influenced the research activities concerning the obtained results.

2.10.2 Uneven flow distribution

During research phase 1 uneven flow distribution occurred through the plant with around 80% flowing through treatment train 1, and 15 and 5% through train 2 and 3, respectively (measured by bucket and stopwatch). Several modifications and surveying of the plant took place. The overflow part of the respective ABR outlets was checked to ensure that each train had similar elevation. However, the investigations could not show any problems that would result in an uneven flow distribution.

During later observations the project team opened an end cap inside a distribution channel that is linking the effluents of the AFs. It was observed that water was flowing from the compartment of train 2 into the compartment of train 1. This suggested different elevations of the trains resulting in uneven hydraulic gradients. Moreover, when the heights inside the distribution channel got checked with a dumpy level it was observed that there was a 50 mm difference between the exit pipes of the flow meters between train 1 and train 3 (compare Figure 12) . The height difference between train 1 and 2 was 40 mm and another 10 mm difference could be observed between train 2 and train 3.

In order to solve this problem 110 mm end caps, with a cut weir that eliminates the height difference, were placed at the end of Train 1 and Train 2. Another check was performed with the dumpy level to ensure that water levels were similar.

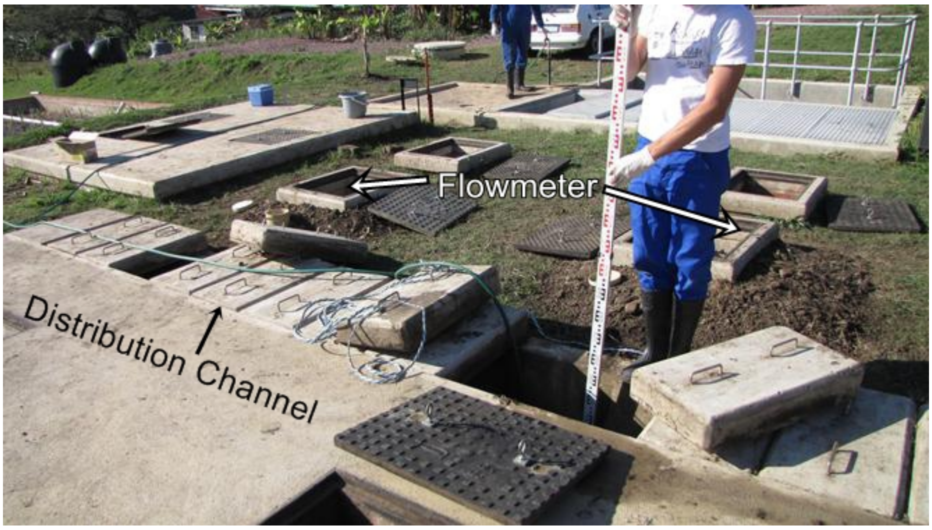

Figure 12: Photograph of measuring the water level with a dumpy level inside the effluent distribution channel after AF2 of ABR train 2. Photograph taken by P. Sananikone on the 28th June 2011.

At the start of research phase 2 it was assumed that readjusting the water gradient through the plant would result in even flow distribution. However, this did not occur. Bucket measurements have shown periods of even flow alternating with uneven flow distribution. It was hypothesized that following situations could have led to uneven flow distribution through the plant:

- The settler compartments were filled up with too much sludge and needed to be emptied. During the peak flow times the sludge could have blocked the outflow pipes, which leading into the different trains. Since train 1 was operating at highest flow rates, channelling could have occurred through the plant and caused the higher flow rates in train 1.
- Flow could have been restricted due to scum inside the downflow pipes of the ABR chambers, which resulted in restricted flow in the respective train.

By cleaning the downflow pipes of train 1 during research phase 3, a significant amount of flow entering the train could be observed. Regular cleaning of the pipes should be added to the maintenance programme of DEWATS plants that are designed with downflow pipes instead of vertical baffles.

2.10.3 Clogging of flow meter pipes

Another problem that concerned the uneven flow distribution was the clogging of the flowmeter pipes. This would especially have influenced treatment train 2 and 3, which experienced lower flow rates and therefore washed out sludge could have accumulated faster inside the pipes.

This was encouraged by the fact that the diameter after the AFs changed from 110 mm to 50 mm in order to provide the right size for the flowmeter. Cleaning of the pipes was difficult, due to 90° bends inside the pipework. Therefore the piping system was rebuilt before the start-up of research phase 2. By installing a T-piece it was ensured that the pipes could be cleaned with a hose pipe from above the ground (compare Figure 13). After this modification no further blockages were observed and the pipes were cleaned on a regular basis.

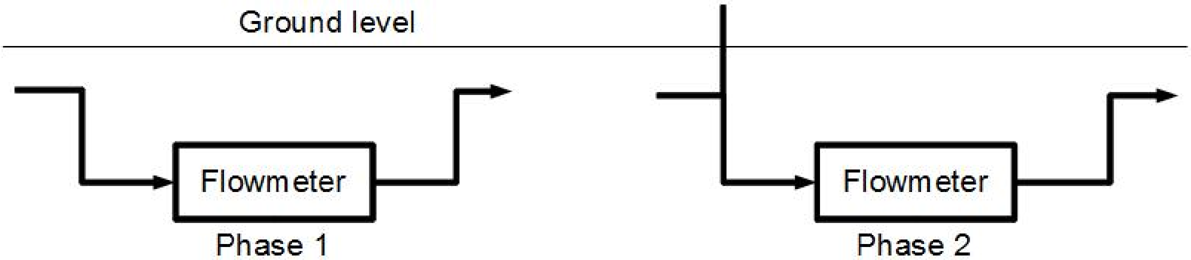

Figure 13: Schematic comparison of the piping leading to the magnetic induction flow meter in research phase 1 and 2. Arrows indicate the direction of the flow.

2.10.4 Installation of inappropriate flowmeter

The flowmeters that were chosen during the construction of the plant were not suited for the actual flow conditions during operation. On the one hand, large diameter piping was required to prevent clogging by solids and the ability to measure high flow velocities during later research phases. A diameter of 50 mm was chosen, which would be able to measure operating flow rates between 0.97 and 15.56 L/s (compare Table 7). However, this problem was only noticed after the start of research phase 2. Although the flowmeters were able to detect flows as low as 0.3 L/s, no statement could be made about the accuracy of these results. Furthermore, flow volumes were only slightly higher than 0.3 L/s during the morning peak flow hours and rain events.

Although flowmeters with a diameter of 15 mm would have been more suitable for the low flow volumes, this would have been even more susceptible to clogging, since clogging already appeared with 50 mm pipes, as shown in section 2.10.3. Additionally the maximum volumes during later research phases were suggested to be over the maximum detectable flow rate of 1.78 L/s.

Table 7: Minimum and maximum flow rates that can be detected by the installed flowmeter for the respective inner diameter (ID) of the pipe.

Meter size	Operating flow rate			
ID /mm	**min /$m^3 \cdot h^{-1}$**	**min /$L \cdot s^{-1}$**	**max /$m^3 \cdot h^{-1}$**	**max /$L \cdot s^{-1}$**
10	0.14	0.04	2.8	0.78
15	0.32	0.09	6.4	1.78
25	0.88	0.24	14	3.89
40	2.25	0.63	36	10.00
50	3.5	0.97	56	15.56
80	9	2.50	144	40.00
100	14.5	4.03	232	64.44
150	32.5	9.03	520	144.44

In order to find a solution, compromises had to be made. Trial measurements with a DCMPU converter have shown success concerning the detection of flow rates during the whole day (compare 2.8). The DCMPU is usually used for an inner diameter of 10 and 15 mm and the results would therefore lead to a loss of accuracy, but this was accepted and it was decided to change every BETA converter into DCMPU units. The responsibilities for the covering of the investment costs had to be discussed and no progress was made in order to change all converter units.

Furthermore, the information that was gained from the new converter in treatment train 1 and the peak flow information from train 2 and 3 have shown that even flow distribution still was not ensured during research phase 2. The objectives of research phase 2 therefore could not be fulfilled and it was decided to use the investigated physico-chemical parameters to show the observed influence of rainwater events to the system.

Moreover it was decided to move on to research phase 3, which initially would have been the closing off treatment train 2 in order to slowly increase the hydraulic and organic loading rates in train 1 and 3. But as a result of appropriate existing flow data only in train 1 and the unknown problem of uneven flow distribution, it was decided to let all flow run through treatment train 1, with the purpose of testing how the system would behave with flows 3 times above the design capacity.

2.10.5 Runoff and groundwater leakage into the manholes

Another problem that was encountered during research phase 1 was that groundwater was running off into the manholes which housed the flowmeters. The sump was constructed out of layered concrete blocks that would allow easier access to the flowmeter if it had to get changed. Water leaked through these gaps due to the high water table and low water drainage. Due to the leakage these areas were sealed with a combination of plastex, bitumex and finally a foil. Additionally, a drainage line was placed under the flooring to divert ground water away from the sump. However, during research phase 2 the same problem appeared again. Additionally was observed that water was flowing into the electrical part of the flowmeters and damaged them. As a result no flow could be detected after treatment train 2 and the VFW.

The damaged flowmeters only got replaced during research phase 3 and the terminal housing was potted with FR707 potting gel and therefore to have it compliant to the protection category IP68 which makes the flowmeters water tight and submersible up to 1 meter.

2.10.6 Stormwater entering the DEWATS plant

In South Africa separate stormwater and wastewater sewers exist. Hence, stormwater should not have entered the DEWATS plant. During research phase 1 sludge movement inside the ABR chambers was observed and it was hypothesised that illegal stormwater connections might have been the reason. EWS conducted a drainage inspection in the catchment area. Cherne smoke machines were used over five different wastewater manholes at different times to cover the entire catchment area. Out of 86 households seven properties had illegal stormwater connections to the sewer system could be found. The households were informed about the sewage bylaws and been advised to remove the connections.

During the rest of research phase 1, it looked like the problem would have been solved since no sludge movement was observed anymore. However, when the flowmeters were able to detect flow rates an obvious amount of additional flow could be observed related to rainwater events. This influenced the research results of research phase 2 and 3.

2.11 Siphon chamber feeding the vertical flow constructed wetland

The mechanical siphon was not working properly during the operation of research phase 2. The swing arm of the siphon was supposed to rise to a certain height until wastewater flows into the siphon box. Consequently the swing arm would have been pushed down and the wetland would have been fed in doses (compare Figure 14). Instead, the siphon box lifted higher than the inlet pipe to the siphon chamber and the wastewater was flowing continuously into the air vents, which linked the distribution pipes to the VFW. The wastewater was additionally flowing back into the distribution channel, which is leading into the siphon chamber.

Several options were considered to solve this problem. As the cheapest and easiest option, longer air vents were installed in order rise the water level inside the siphon chamber. This option has shown little success, since the inlet pipe was now completely under water and the backflooding was at a degree were the distribution channel started to overflow. The only way to handle this problem was to install a device that would prevent the swing arm to rise above the air vents. A stainless steel construction was built at the workshop at UKZN and installed during research phase 3.

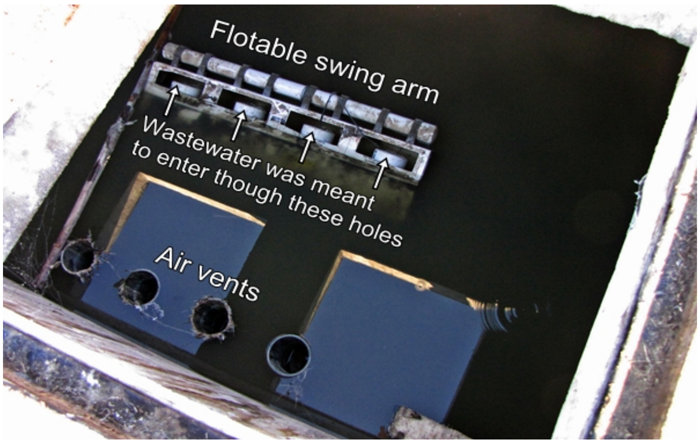

Figure 14:

Photograph of the inside of the siphon chamber. Wastewater is entering the air vents instead of the swing arm. Photograph taken by P. Sananikone on the 7th July 2011

Resulting from this problem is that investigations on the wetland during research phase 2 are not representing the conditions the VFW was designed for. Instead of feeding the VFW in doses and provide the required conditions, it was mainly fed continuously. Since the flow going into the wetland could also not be detected, statements about the loading rates need to be made by assumptions.

2.12 Influent sewer line

Fat and grease deposit often built up in the sewer line leading to the influent of the plant. This accumulated and caused blockages of the treatment plant (compare Figure 15). A continuous monitoring of the flow conditions was necessary to notice this problem and solve it.

Figure 15:

Photograph of the trunk sewer leading to the DEWATS plant. The wastewater was running off through the overflow pipe if the system was closed or blocked. Photograph taken by P. Sananikone on the 15th July 2011.

2.13 Gas chamber

The DEWATS plant was producing biogas, which could be smelt and seen when opening the manhole cover of settler 2b. But the biogas could not be harvested and it was suggested that a leakage caused the runoff. To examine this problem, a water meter was attached to the gas inlet of the stove, which is approximately 100 m away from the plant. The system was pressurised using motor vehicle fumes and the valve opened. This was also repeated at the gas outlet of the DEWATS plant, but both tests did not show any pressure built up. The observations suggested that there was no leak in the gas lines to the stove and the problem was rather no pressure built up inside the settling chamber.

Further investigations on the as-built drawings revealed that the technical drawings were not matching the actual plant construction. The gas was leaking through the manhole cover at the sampling point settler 2b. Looking at the AutoCAD drawings this manhole should not have existed but was built for a sewage pump to transfer domestic wastewater to the emergency container ABR system. The problem occurred because this manhole did not have a shroud going to the wastewater, which would have kept the system gastight. In order to solve this problem the municipality agreed to install neoprene gasket seals and a steal construction that allows putting pressure on the gasket seal and closing the manhole properly. Additionally the gas lines from the settler to the kitchen were re-laid since the previous line did not have any condensation traps.

3. Results and Discussion

This chapter is divided in three sections. First, the overall water consumption for the households connected to the Newlands-Mashu DEWATS plant is presented in section 3.1. The precipitation data near the site is presented in section 3.2. Operational details for research phase 2 and 3 are presented in sections 3.4 and 3.5, respectively. Analytical measurements of research phase 1 are not part of this thesis, but the gained knowledge will serve as a reference. An overview of the research phases is described in section 2.3.

3.1 Water consumption

The average water consumption has an impact on the inflow of the treatment plant. Although not all consumed water is ending up in wastewater and entering the DEWATS plant the data can give an idea about monthly fluctuations.

Monthly water meter readings were investigated by the EWS. Table 8 shows the average water consumption between June 2011 and May 2012. It can be observed that the average water consumption increases until March 2012 and decreases again afterwards. February and March 2012 show distinctively higher water consumption than all the other months.

Date	Water consumption $/m^3 \cdot d^{-1}$
06.2011	37.9
07.2011	37.7
08.2011	40.3
09.2011	41.9
10.2011	43.0
11.2011	38.2
12.2011	49.9
01.2012	46.9
02.2012	62.4
03.2012	62.3
04.2012	48.4
05.2012	39.5

Table 8:

Monthly water consumption of the connected households between June 2011 and May 2012.

It was observed that the high consumption in February and March is basically caused by three households and shown in Figure 16. The values seem very unrealistic, especially when looking at the consumption in previous and later months. The bars show that a single household has consumed 600 m^3 water in March 2012, which complies with an average of 20 m^3 per day. After a personal communication with the responsible persons at EWS, it was concluded that the readings must have been done wrong.

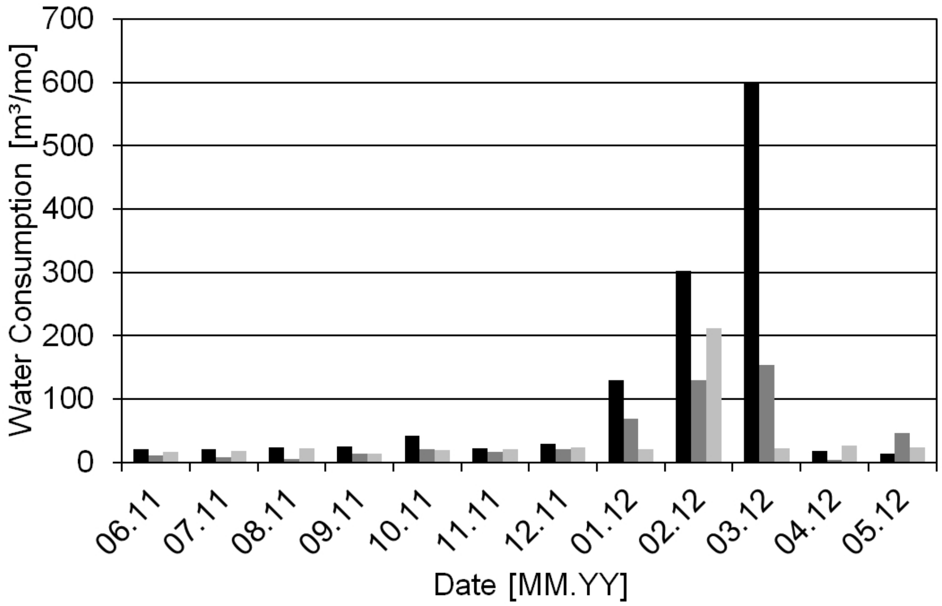

Figure 16: Water meter readings of three households connected to the Newlands-Mashu DEWATS plant. The different bar colours indicate the different households.

3.2 Precipitation

The Newlands-Mashu DEWATS plant is designed according to Sasse [8] guidelines to treat only domestic wastewater. Stormwater flow is not included into the design spreadsheet. Rain water events could therefore influence the hydraulic conditions in the plant if there are illegal stormwater connections.

The sampling campaign occurred during Durban's rain period between the summer months of December and February. Table 9 shows the monthly rain data for the months of research activity. For comparison, the data of previous years was also included. The rain in December 2011 and January 2012 was significantly less than in previous years, while the months of November 2011 and March 2012 experienced higher rainfall. Rainfall in February was unexpectedly low over the last 4 years and February 2012 does not show an exception.

Table 9: Monthly precipitation for the months of sampling between 2009 and 2012. Bold numbers highlight the months that were investigated during research phase 2 and 3.

Month	2009 /mm·mo^{-1}	2010 /mm·mo^{-1}	2011 /mm·mo^{-1}	2012 /mm·mo^{-1}
January	85	59	121	**38**
February	59	47	6	**30**
March	21	10	53	**175**
April	17	22	70	**18**
November	60	78	**212**	-
December	118	163	**59**	-

3.3 Ambient Temperatures

Durban has a subtropical climate and the minimum temperatures during the winter months seldom drop below 10°C. Table 10 presents the average ambient temperatures during the months of research activity.

Table 10: Ambient temperatures at Virginia Airport in Durban North between the months of November 2011 to April 2012.

	November 2011 /°C	December 2011 /°C	January 2012 /°C	February 2012 /°C	March 2012 /°C	April 2012 /°C
Average Maximum	24	26	27	27	27	24
Average Medium	20	22	24	24	23	19
Average Low	17	19	21	21	19	14

3.4 Research Phase 2

Research phase 2 started on the 27th October 2011 and was completed on the 8th February 2012. This chapter presents the results from measured parameters and will evaluate the influence of rainwater on the performance of the treatment plant.

3.4.1 Rainfall overview

Figure 17 shows the overall precipitation for the sampling period of research phase 2. As discussed earlier in section 3.2, high rainfalls were experienced in November 2011 compared to earlier years. There was rainfall on 21 days of the month and 77 mm of rain, which is one third of the entire rainfall for that month, fell between the 17th and 19th of November 2011 with a peak of 45 mm on the 19th November 2011. The second high peak rainfall occurred on the 27th November 2011 with a total amount of 56 mm. Within the whole sampling period several minor rainwater events around 10 mm/d happened.

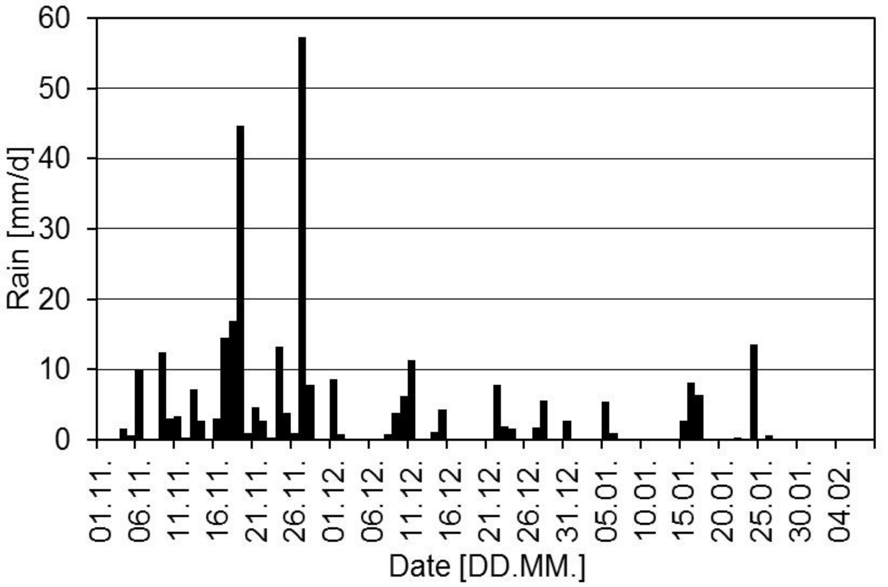

Figure 17: Rainfall overview for the sampling period of the 1st November 2011 until the 4th February 2012. The data was collected from a meteorological station close to the DEWATS plant.

Looking at the intensity and duration of the two major rainfall periods that occurred during research phase 2 shows a difference in the nature of the rainfall. The first period between the 17th and the 19th November 2011 was characterised by continuous rainfall over 3 days with two distinct intense peak rainfall periods. During the night from the 18th November 2011 to the 19th November 30 mm of rainfall was recorded in 8 hours. While on the 19th November 2011 itself 44 mm of rain was recorded.

During the second major rainfall period, an intense period of rainfall occurred on the 27th November 2011 with 57 mm of rainfall recorded over a period of 4 hours of which almost half fell within 20 minutes.

The significance of these rainfall events becomes more obvious when the hydraulic conditions of the plant are presented in the next section. .

3.4.2 Recorded flow data and influence of rainwater on the hydraulics of the DEWATS plant

This section presents the hydraulic loadings of the settler-ABR-AF during research phase 2. Flow data after each treatment train was recorded by magnetic induction flow meters, transmitted using a wireless telemetry. As mentioned earlier section 2.10.4, the flowmeters that were chosen were inappropriate to register the flow that was going through the plant. Although the minimum flow rate that could possibly be detected by the installed converting units was 0.97 L/s, the existing data has shown values down to 0.2 to 0.3 L/s. The accuracy of these data is unknown and flows above these values could only be detected and transmitted during the morning peak flow time and rainfall events. Nevertheless, hydraulic loadings are the most important parameters to make a statement of the treatment conditions inside the DEWATS plant and it will be discussed if the existing flow data allows any assumptions in order to test the defined hypothesis.

Figure **18** presents the different periods of available flow data and the configurations that were made at the treatment plant. Due to start-up problems with the system flow data was not available until the 10th November 2011. The first period of existing data was from the 10th November to the

28th November 2011, where peak flow data was recorded from all three trains until the flowmeter in train 2 got water damaged on the 28th November 2011 (this elaborated further upon in Section 2.10.5) and was not replaced until the end of research phase 2. On the 24 January 2012 the converter of train 1 got replaced and was able to detect the flow over 24 hours (further explanations in Section 2.10.4). Besides the data of train 1 after the 24 January 2012, no flow data was recorded during non-peak times during research phase 2. These technical issues made it difficult to compare the hydraulic loadings of the different trains and to describe the actual flow that occurred through the plant.

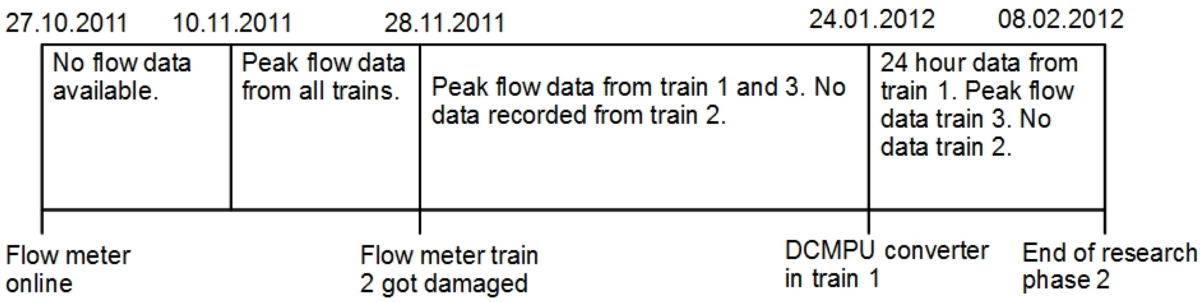

Figure 18: Overview of changes applied to the flowmeter and available flow data during research phase 2.

An overview of the flow characteristics of the entire plant during research phase 2 is presented in Figure 19. As it no flow was recorded during non-peak periods and it was difficult to examine the flow distribution through each treatment train, the flows are examined as the total flow over all three trains. This furthermore is underestimated after the 28th November 2011, due to the damage of flow meter 2.

For several days no flow data is available at all. On the 7th December 2011 and between the 18th and 24th January 2012 blockages of the influent sewer were observed during field trips. On other days without any recorded flow, the blockages may have been unblocked with the inflow itself or technical problems with the flowmeters may have occurred.

The flow meter of train 1 was able to register flows over 24 hours after the converting unit got changed on the 24th January 2012. The registered data shows an average flow of 15.10 ± 1.56 m^3/d until the 8th February. This is a bit more than a third of the design flow (41.3 m^3), which suggests that the hydraulic loading rates of treatment train 1 have been suitable for the research objectives during this period. Nevertheless, this data is not enough to make assumptions about the overall flow distribution in the treatment plant.

Figure 19 also includes the rainfall data calculated in section 3.4.1. A correlation between the main rainfall events in November 2011 and the detected flow can be seen. On the 19th and 28th November 2011 the plant experienced almost 4 times more flow than it is actually designed for. Besides these events no distinct correlation between rainfall events and flow conditions can be observed from the graphs. But from investigating the data more detailed in the calculation spreadsheet it can be observed that almost every rain event has an influence on the flow that is going through the plant. This is not a surprise, since it is assumed that the additional stormwater flows are due to unauthorised stormwater connections and no drainage inspections have been done during research phase 2 in order to solve this problem. These effects suggest that for the determination of the performance of the DEWATS plant, days with rainfall should not be included. Thus, out of 100 days of research phase 2, 42 days would need to be erased to calculate the actual wastewater loadings through the plant.

Comparing the evaluated flow with the actual design flow shows how little is known about the real flow conditions in the treatment plant. Only the two main rainfall events will be examined in the next chapter, since the existing data allows investigations over a 24 hour period.

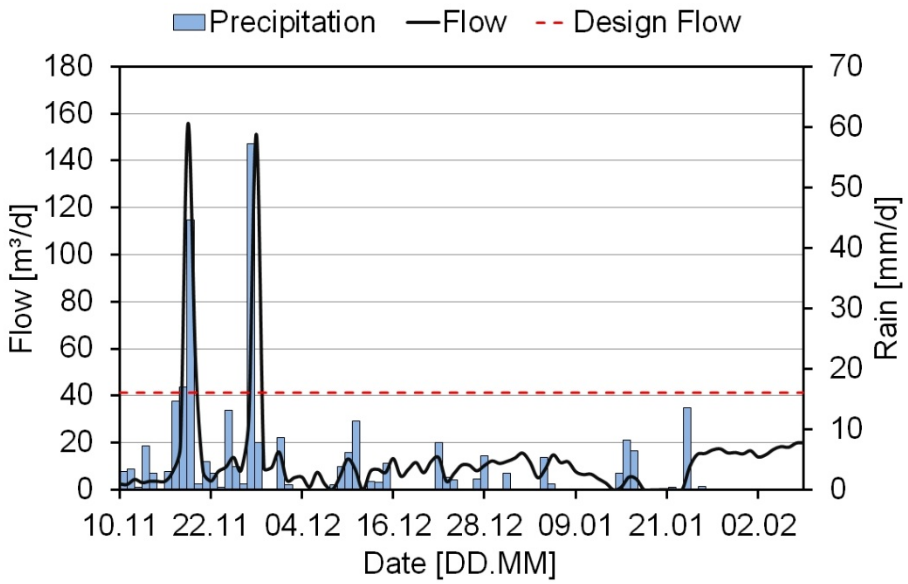

Figure 19: Detected flow over all three trains through the DEWATS plant and precipitation during research phase two. The red line shows the design flow.

3.4.3 Influence of rainwater during heavy rainfall events

Although it was shown that every rainwater event shows an effect on the hydraulic loadings of the DEWATS plant only the heavy rainfall events of the 18th to 20th November and the 28th to 29th November will be investigated further since flow was registered for 48 and 33 hours, respectively.

Figure 20 and Figure 21 show the influence of intense rainfall events on the flow of the settler-ABR-AF. The flow graph in both figures is calculated as the total flow over all three trains. Since the flowmeter in train 2 broke at 12:00 AM on the 28th November, Figure 21 is also including a graph that shows the estimated flow through the plant. The flow through treatment train 2 has been calculated by assuming 0.7 as the distribution factor between treatment train 1 and 2. This distribution was determined by the observed flows between 10:00 PM and 11:50 PM on the 27th November 2011, where the values for treatment train 2 were 70% of the values of train 1. The flows could be observed because the treatment plant was experiencing some high flows, which could be recorded by the flowmeters. The flow over all three trains is summed up after the correction of the data and shown as the total flow through all three trains. The data has been calculated and added because the hydraulic conditions would otherwise have been underestimated.

The plots show that there is a correlation between rainfall and intensity of the flow pattern. Between the 18th and 19th November 2011, 61.4 mm of rain resulted in a total amount of 224 m^3 of flow in 48 hours, while 64.2 mm on the 27th November 2011 caused 237 m^3 (estimated flow) in 33 hours. As discussed before the characteristics of the rainwater events are different. Both figures show that the flow is instantly influenced by rainfall, but Figure 21 also shows for how long a heavy and short rainfall event can influence the treatment plant. Five hours of rain caused an increased flow for 15 hours with maximum peak flows up to three times above the designed maximum of 5.2 m^3/h.

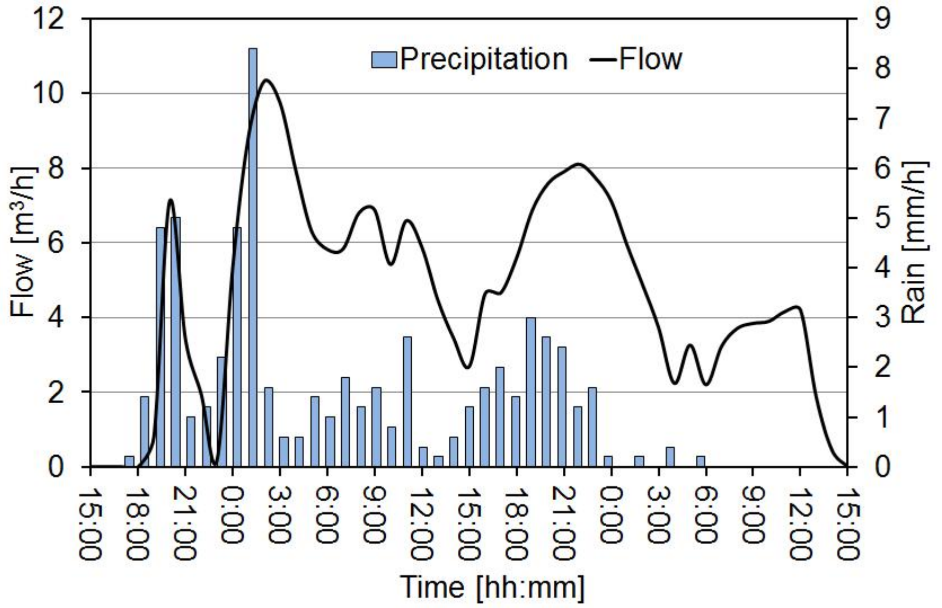

Figure 20: Hourly rainfall and flow between the 18th and 20th of November. Flows of the different trains are summed up.

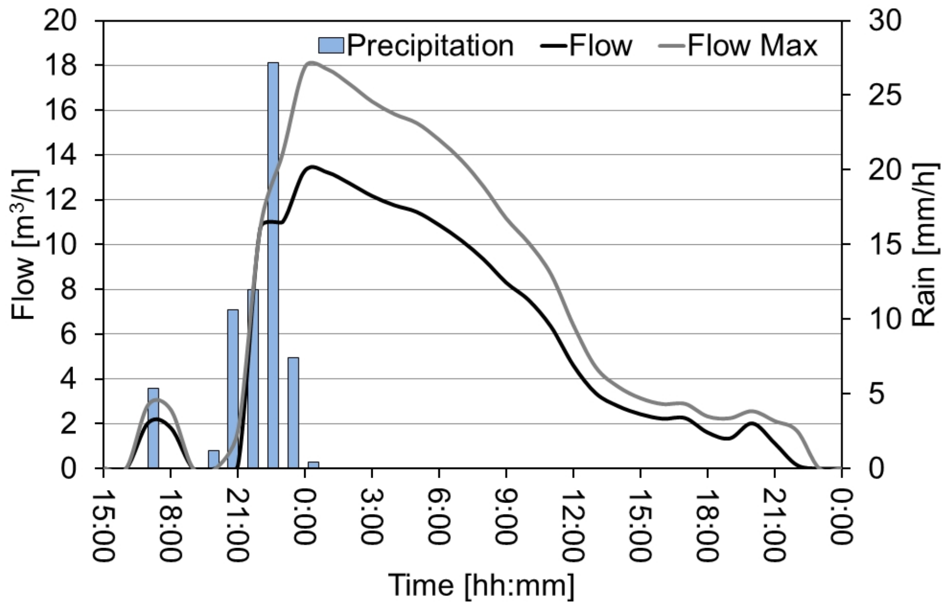

Figure 21: Hourly rainfall and flow between the 28th and 29th of November, including the estimated maximum flow. Flows of the different trains are summed up.

The hydraulic loading rates are presented in Table 11. The calculations have been done with the same dataset that was used for the figures in this section. The average wastewater flow per day is determined by dividing the total flow of each period by the ratio of investigated hours (48 and 33 hours) and the hours of one day (24 hours). Since the area in the first three compartments of treatment train 3 is double as much as in train 1 and 2, the upflow velocities had to be analysed individually. The last compartment of train 3 would show the same upflow velocities as train 1 and 2.

The calculated values show the high difference of the hydraulic conditions during the rain events compared to the design values. Upflow velocities have been 2 to 3.5 times higher for the respective rain period. Although only maximum peak flows are investigated, the figures in this chapter show that the flow conditions have been above the designed maximum for a long time.

Table 11: Hydraulic loading rates of the ABRs during the two main rainwater events in November. Calculations of HRT and upflow-velocities were done after the equations in section 2.2.

Parameter	Unit	18. – 20.11.2011	28. – 29.11.2011	Design
Wastewater flow	$/m^3 \cdot d^{-1}$	111.5	172.4	41.6
HRT	/h	14	9	36
Max peak flow	$/m^3 \cdot h^{-1}$	10.34	17.93	5.2
Upflow-vel. train 1 and 2	/m·h	1.97	3.42	0.99
Upflow-vel. train 3	/m·h	0.98	1.71	0.50

Taking into account that the treatment plant is designed for 41.6 m^3 of wastewater per day, the calculations lead to about 347 m^3 of additional stormwater during these two rain events.

The total volume of the ABR chambers itself is 66.2 m^3, which means that the DEWATS plant has been filled 5 times with rainwater within a period of 14 days. Having an HRT of 36 hours in the ABR chambers on days with a normal flow of 41.6 m^3, the dilution process caused by rainwater influences the treatment process for a long time, since not all rainwater gets directly washed out again and rather mixes with the incoming wastewater. As a result it can be assumed that the hydraulic conditions have influenced the performance of the Newlands-Mashu DEWATS plant, which is assessed in the next section, using physico-chemical data collected over research phase 2. The overall performance of the plant is examined as well as the effect of rainfall events according to measured parameters.

3.4.4 Performance-Assessment of the DEWATS plant

This section presents the treatment performance of the Newlands-Mashu DEWATS plant. The performance of the plant was assessed by measuring different physico-chemical parameters and comparing the data to South African discharge guidelines for agricultural irrigation [35]. Pathogen indicator analyses were not performed as the Biochemical Engineering laboratory at Chemical Engineering (UKZN) was not equipped for microbiological analyses. These microbiological tests, however, will be performed by microbiologists examining the health risk assessment of crops irrigated with treated wastewater from a DEWATS plant.

3.4.4.1 Assessment of COD results of the ABR-AF trains

In the previous sections it has been shown that rainfall near the plant site influences the hydraulic conditions in the DEWATS plant. For this reason, COD results have been split into three time periods (stages 1 to 3) to show the influence of rainfall events on the COD concentration through the plant. The COD data was collected over different periods during research phase 2 with a total of eight sampling datasets.

The results for feed concentrations only show the concentration for the moment but cannot directly be related to the removal efficiency of the following treatment steps. The values of the second settler are much more representative in order to get an idea of the organic loading during the specific sampling day and have been used to compare influent and effluent concentrations of the ABR-AF chambers.

Due to the small sample number, general trends have been reported with no statistical validation of the COD data set performed. Table 12 shows the COD results for stage 1, the first three weeks of sampling.

The concentration of the wastewater entering the three ABR trains from the settler on the 15th November 2011 was 1768 ± 112 mg COD/L and the effluents from the three trains were around 400 mg COD/l.

Heavy rainfalls occurred on the 18th and 27th November 2011 (see section 1.4.3) with samples taken on the 21st and 29th November 2011. The COD values for those dates were comparatively lower than those reported before the rainfall event. The results indicate that the illegally connected stormwater flow causes a dilution of the feed wastewater and thus lower effluent average COD concentrations. Moreover the lower COD concentrations in settler 2a and settler 2b on the 21st and 29th November 2011 show that the treatment plant was still filled with rainwater. It is therefore possible that the treatment performance of the plant in terms of COD removal could have been overestimated by these rainfall events.

Table 12: Stage 1 of the measured concentrations of COD values at the specific sampling points. The effluent samples were taken from the outlet pipe of AF 2. Rainfall data is included as the sum of the 5 days before the respective sampling campaigns.

Date	**Rainfall /mm**	**Feed /mg·L^{-1}**	**Set. 2a /mg·L^{-1}**	**Set. 2b /mg·L^{-1}**	**T1E /mg·L^{-1}**	**T2E /mg·L^{-1}**	**T3E /mg·L^{-1}**
15.11.2011	17	1899	1847	1688	407	321	399
21.11.2011	80	1983	483	632	167	198	181
29.11.2011	83	926	382	345	104	98	97

Table 13 presents the COD data from sampling stage 2. The results show that the average COD concentrations in the settlers doubled 2 weeks after the previous sampling period, where heavy rainfall occurred. Although 21 mm rainfall was recorded during the 5 days before the 14th December 2011, the influence on the treatment performance cannot be observed which might be due to the fact that the 11th December was the last day of that rain period. The low value for settler 2b on the 21st December cannot be explained with rainfall since no rain fell during the week before. The average COD concentrations through the plant are generally higher than during stage 1, which presumes that the influence of stormwater on the results is less during stage 2.

Table 13: Stage 2 of the measured concentrations of COD values at the specific sampling points. The effluent samples were been taken from the outlet pipe of AF 2.Rainfall data is included as the sum of the 5 days before the respective sampling campaigns.

Date	**Rainfall /mm**	**Feed /mg·L^{-1}**	**Set. 2a /mg·L^{-1}**	**Set. 2b /mg·L^{-1}**	**T1E /mg·L^{-1}**	**T2E /mg·L^{-1}**	**T3E /mg·L^{-1}**
14.12.2011	21	1077	929	1027	265	203	224
21.12.2011	0	927	938	568	261	308	218

Table 14 shows the last sampling stage of the research phase. On the 18th January 2012, the sewer line to the Newlands-Mashu DEWATS plant became blocked by fat and grease deposits which had

accumulated near the stop gate to the plant. This blockage closed off the feed for a period of 7 days and therefore the rain that fell (14 mm) during the night before the 24th January 2012 could not have caused any dilution within the reactor, since incoming water was redirected through an overflow into the main sewer. Furthermore no other rain periods have occurred between the 17th January and the 8th February 2012.

Since no stormwater events could have influenced the treatment performance during stage 3, the effluent COD values give a reliable idea about the performance of the plant. Mentioned earlier in section 3.4.2 flow conditions for treatment train 1 could be registered during this stage and have shown that a bit more than a third of the total design flow was going through this train (15.10 ± 1.56 m^3/d). Nevertheless was the distribution between the other trains still unknown and as section 2.10.2 has explained the reasons for the generally uneven flow distribution, it cannot be presumed that the flow was as stable in train 2 and 3.

The relatively low COD values of the effluents on the 24th January 2012 could be explained by the blockage which could have resulted in longer wastewater residence time and therefore longer contact time with the biomass within the ABR chambers. Effluent COD values of all trains rose after the 24th January 2012. This suggests that microorganisms are not able to degrade the incoming wastewater to an extent that was achieved during the blockage of the plant.

The COD decrease of T2E after the 31st January 2012 could be explained by changing flow conditions in treatment train 2 and 3. The flow in train 2 could have been throttled due to blockages in the piping system and caused higher flows in train 3.

Table 14: Stage 3 of the measured concentrations of COD values at the specific sampling points. The effluent samples have been taken at the outlet pipe of AF2.

Date	Feed /mg·L^{-1}	Set. 2a /mg·L^{-1}	Set. 2b /mg·L^{-1}	T1E /mg·L^{-1}	T2E /mg·L^{-1}	T3E /mg·L^{-1}
24.01.2012				229	195	191
31.01.2012	1147	749	763	379	358	250
08.02.2012	725	876	1019	398	256	385

The overall COD removal rates for each ABR-AF train are shown in Table 15. The 24th January 2012 is not included, since no data of the settler was available. Without any stormwater interferences the removal rates are generally lower than during rainfall events, which supports the hypothesis that removal rates during rain events are overestimated and do not show the actual performance of the treatment plant. A trend of decreasing removal efficiencies towards the end of research phase 2 can be observed. The removal efficiencies during rainwater events are basically equal in all three trains. Without stormwater influence the trend shows that train 1 is the least efficient train, without knowing the actual flow distribution in train 2 and 3.

Table 15: COD removal efficiency of all three ABR-AF trains calculated as the percentage of removed COD from the average of settler 2a and 2b and the effluent of the AF, according to Table 12, 13 and 14. Rainfall data is included as the sum of the 5 days before the respective sampling campaigns.

Date	Rainfall /mm	Removal efficiency Train 1 /%	Train 2 /%	Train 3 /%
15.11.2011	17	77	82	77
21.11.2011	85	70	64	68
29.11.2011	83	71	73	73
14.12.2011	23	73	79	77
21.12.2011	0	65	59	71
31.01.2012	0	50	53	67
08.02.2012	0	58	73	59

3.4.4.2 Assessment of COD results of the VFW

Although problems existed with the siphon chamber feeding the VFW, COD loading rates and the treatment efficiency was calculated. Mentioned in section 2.11, the siphon was not working properly and was feeding the wetland continuously. The VFW was designed for one third of the flow going through the plant and was started on the 15th November 2011 and loaded with the flow of treatment train 2. Although the flow going through this train is unknown, calculations were made with one third of the total design flow, resulting in 13.87 m³ ABR effluent entering the VFW per day. This is presumably overestimated, but will give an idea about the performance of the removal of organic matter through the wetland.

Rainfall influences the VFW not only through the assumed diluted effluent of treatment train 2. It is furthermore influenced by the rain that is entering through the surface area. Although this process cannot be prevented and is a natural influence, samples taken directly after rainwater events need to be examined with caution. The investigated data is presented in Table 16. The effluent of treatment train 2 equals the influent to the VFW. After rainwater events the removal efficiency is around 40%, while 47 to 68% of total COD get removed when no rainfall was recorded.

On days without rain influence between the 21st December 2011 and the 8th February the average influent concentration of the wetland was 279 ± 117 mg COD/L and resulted in an average effluent concentration of 116 ± 21 mg COD/L. The South African discharge standards for irrigation require a COD below 400 mg/L, which is fulfilled after the treatment step of the VFW [35].

Table 16: Treatment efficiency of the VFW, calculated on the basis of the effluent of ABR-AF train 2. Rainfall data is included as the sum of the 5 days before the respective sampling campaigns.

Date	Rainfall /mm	T2E /mg·L^{-1}	VFW /mg·L^{-1}	Treatm. effic. /%
29.11.2011	83	98	57	42
14.12.2011	23	203	119	41
21.12.2011	0	308	99	68
24.01.2012	0	195	97	50
31.01.2012	0	358	135	62
08.02.2012	0	256	135	47

3.4.4.3 Assessment of the nutrient data

The nutrient quality of the DEWATS final effluent (after wetland polishing) is of importance as the treated wastewater is envisaged for re-use in community gardens. Although the wetland was mainly installed to reduce the pathogen load in the anaerobically treated wastewater, this section presents the nutrient data through the VFW. The performance of the VFW is therefore assessed according to nutrient removal and the influence of rainfall events on wetland performance.

Table 17 presents the investigated data for NH_4-N. All sampling days besides the 21st and 29th November 2011 show the same trend. NH_4-N concentrations increase within the ABR-AF train and decrease after the VFW. The included raindata shows that concentrations have been diluted with stormwater during the 21st and 29th November 2011. This dilution cannot be observed from other days, where rainfall occurred during the days before the sampling campaigns. The variation in feed concentrations shows again that the sampling method needs to be revised. But, since no data is available from the settler this information was used to characterise the influent of ammonium to the treatment plant.

Table 17: NH4-N concentrations during research phase 2. Rainfall data is included as the sum of the 5 days before the respective sampling campaigns.

Date	Rain /mm	Feed /mg·L^{-1}	T1E /mg·L^{-1}	T2E /mg·L^{-1}	T3E /mg·L^{-1}	VFW /mg·L^{-1}
15.11.2011	17	32.5	62.9	62.4	61.4	
21.11.2011	80	32.0	23.8	19.0	18.0	
29.11.2011	83	20.1	16.9	14.9	15.2	10.5
07.12.2011	1	34.3	76.4	72.2	74.1	42.3
14.12.2011	21	9.2	51.7	51.0	49.8	21.1
21.12.2011	0	44.5	54.3	52.1	53.7	24.0
06.01.2012	6	24.3	51.4	53.3	51.3	22.5
31.01.2012	1	17.4	58.7	56.7	58.2	27.6
08.02.2012	0	49.5	57.4	54.8	58.3	16.1

Not including the two stormwater influenced days, the average ammonium feed concentration entering the treatment plant was 30.2 ± 14.4 mg/L. All three trains show basically the same concentrations in the effluent. This suggests that the treatment conditions inside the ABR-AF trains are the same. The increase through the trains shows that particulate solids were being digested and released as free and saline ammonia. The observed average effluent concentrations of Train 1, 2 and 3 were 59.0 ± 8.7 mg/L, 57.5 ± 7.5 mg/L and 58.1 ± 8.2 mg/L, respectively. From the effluent of the VFW an average concentration of 25.6 ± 9.0 mg/L was observed. Calculating the removal efficiency of the VFW with the effluent of Train 2 shows a reduction of 55% of a NH_4-N.

PO_4-P concentrations have been measured on the same samples as the NH_4-N concentrations. As a result the same days have been influenced by stormwater. Table 18 presents the investigated data. Besides the sampling days of the 21st and 29th November 2011 the trend of increased concentrations can be observed in all three treatment trains, which can be explained by the digestion processes. The average effluent concentration of Train 2, leading into the wetland, is 7.4 ± 0.2 mg/L. The average effluent concentration of the VFW is 5.3 ± 1.0 mg/L, which results in a PO_4-P removal rate of 28 %, excluding the days that have been influenced by stormwater.

Table 18: PO4-P concentrations during research phase 2. Rainfall data is included as the sum of the 5 days before the respective sampling campaigns.

Date	Rain /mm	Feed /mg·L^{-1}	T1E /mg·L^{-1}	T2E /mg·L^{-1}	T3E /mg·L^{-1}	VFW /mg·L^{-1}
21.11.2011	80	11.8	2.9	1.7	1.8	
29.11.2011	83	5.1	2.1	1.7	1.2	1.4
07.12.2011	1	4.1	6.8	7.1	7.4	3.0
14.12.2011	21	2.3	6.9	7.3	8.5	5.4
21.12.2011	0	5.8	6.8	7.5	7.0	4.3
06.01.2012	6	3.1	7.4	7.6	7.0	5.4
31.01.2012	1	3.4	7.6	7.6	8.0	6.2
08.02.2012	0	5.2	7.3	7.1	7.2	5.2

3.4.4.4 Sludge heights

The sludge bed height inside the ABR chambers is another parameter that is used to monitor the performance of ABRs. Useful information about the sludge accumulation and the influence of hydraulic loadings can be gained. Furthermore can be observed when desludging of the treatment plant is required.

Figure 22 presents the sludge heights during research phase 2. The measured heights of 2 cm in train 1 (compartment 1) and train 2 (compartment 4) are implausible and assumed to not reflect the actual sludge height in the compartment.

It can be observed that there is no distinct sludge bed migration towards the end of the plant in all trains during the beginning of research phase 2. This, despite the intense hydraulic loadings the plant was experiencing in November 2011. It also contradicts with the experience gained from research phase 1, where the stormwater influence on the system has shown sludge migration in all trains.

After the 14th December 2011 only train 1 shows a migration towards the end of the treatment train. The sludge volume inside the first three chambers decreased and accumulation could be observed in the 6th chamber. This cannot directly be related to a specific rainwater event, but suggests that more flow was going through train 1.

Furthermore, no obvious sludge accumulation can be observed during research phase 2. Sludge heights stay mainly the same in all three trains besides in train 1 at the end of the research phase.

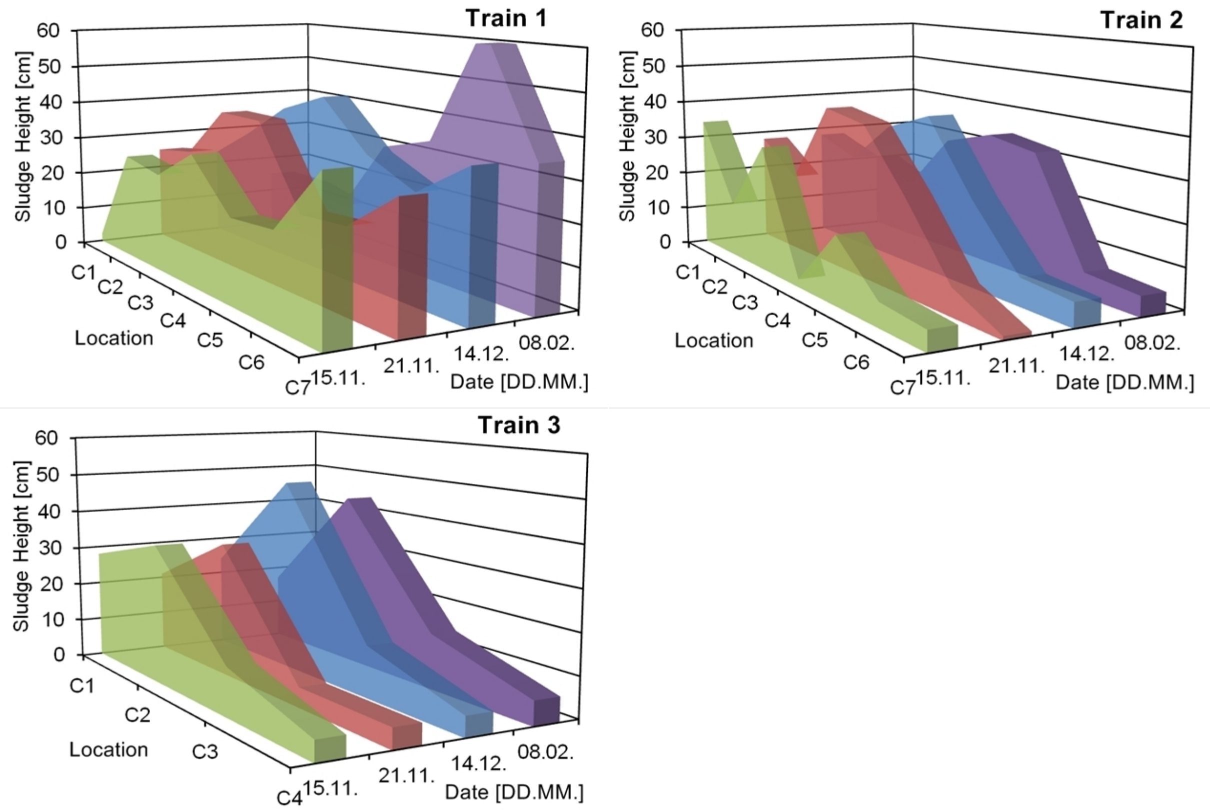

Figure 22: Sludge heights inside each compartment of train 1, 2 and 3 of the Newlands-Mashu DEWATS plant during research phase 2.

3.5 Research Phase 3

3.5.1 Rainfall Overview

During research phase 3, March was experiencing unusual high rainfall (see section 3.2). From Figure **23**, three main rainwater events can be seen. The meteorological station recorded 79.2., 31.4 and 21.2 mm of rain on the 4th, 11th and 31st March 2012, respectively. A total of 26 days rain days occurred, of which 11 have been with values below 1.0 mm/day. In April 2012, there were 10 days of rain with no rainfall above 4.8 mm/day. It could be therefore be envisaged that April 2012 would be less affected by stormwater events.

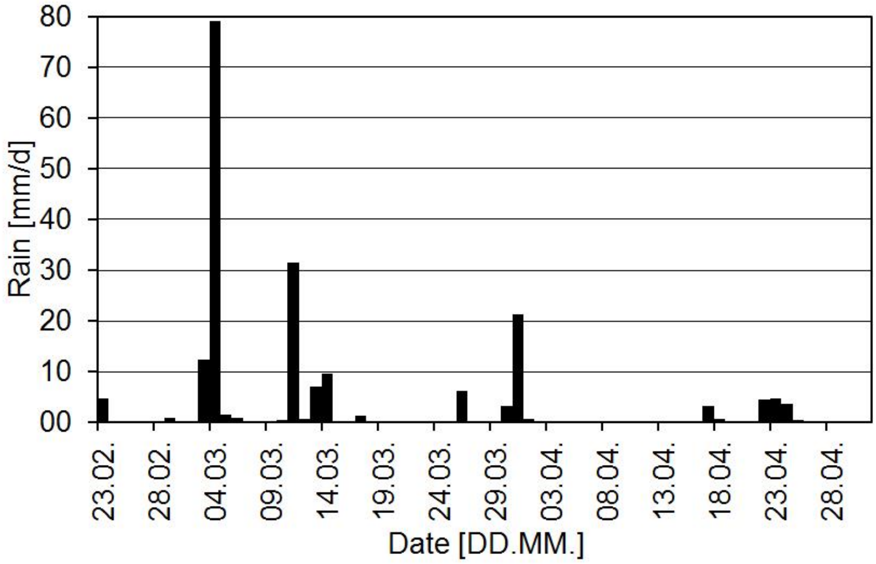

Figure 23: Overview of rainwater events during research phase 3.

3.5.2 Flow characteristics

This chapter analysis the flow conditions in research phase 3. Due to errors with the flow detection units, several hours on the 7th and 8th March 2012, the 13th, 26th and 30th April 2012 and the 1st May 2012 did not pick up any flow rates and have therefore been excluded from Figure 24 and Table 19.

From the height of the flow bars in Figure 24 it can be presumed that flow in March was generally higher than in April even without considering the rain impacted days. Days of rain in March show a strong relation to the magnitude of flow, while no direct influence can be seen in April. The plot shows that 3 sampling days could have been influenced by surge stormwater flows and associated dilution effects (marked in red).

Since research phase 3 started on the 23rd February 2012 only a flow dataset of 5 days is available for this month. The 25th February shows abnormal characteristics to every other day. Flows in the morning are comparatively low and flows during the afternoon up to 3 times higher than normally. No stormwater influence could have affected that day. The flow of the other 4 days shows the same characteristics as flow in March. For further determinations the total average flows for February and March will therefore be calculated as one flow period.

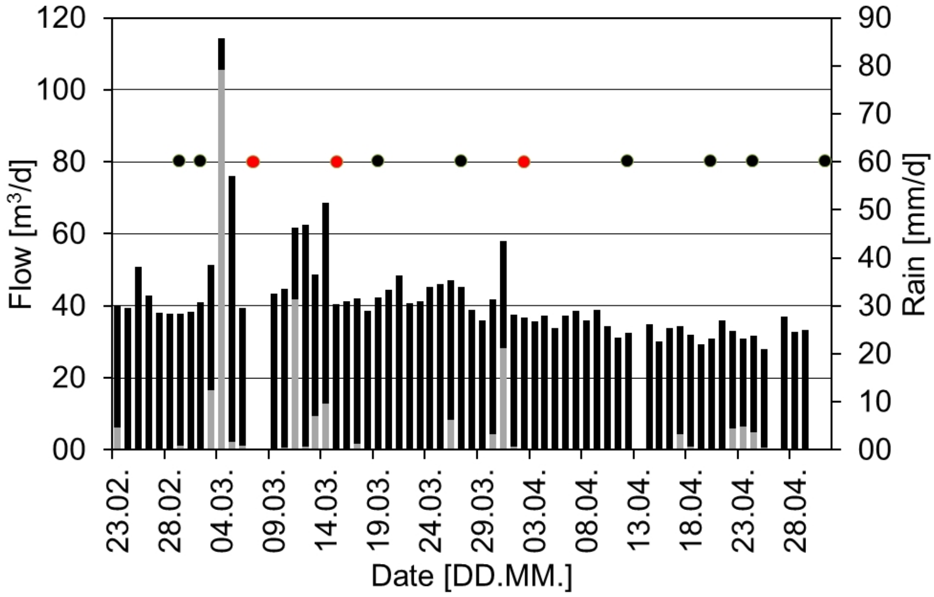

Figure 24: Flow overview and rainfall in research phase 3. The black bars show the flow data and the grey bars represent the rainfall on the specific day. The dots show the days of the sampling campaigns, marked in red if a performance influence could be suggested.

The flow characteristics with and without rainfall days are summarised in Table 19. Average daily flows without rainfall in February/March 2012 are 7.4 m³ higher than in April 2012. Furthermore it can be observed that February and March are highly influenced by rainfall. This influence cannot be observed for April. Average daily flows during the 8 rain influenced days are even lower than during the 20 days without rainfall. This could be explained with the fact that the trend of generally lower flows in April can be seen especially towards the end of the month. Furthermore no rainfall was recorded until the 17th April and overall only 18 mm of rain was recorded for the whole month, as shown in section 3.2. Although this might suggest that April has not been influenced by rainfall, further determinations need to be made with caution if rain was recorded during the days before the sampling campaigns.

Table 19: Flow characteristics of train 1 during research phase 3 with and without days of rainfall. The number in parenthesis shows the quantity of days that are included. The data was collected in 2012.

Parameter	Days without rainfall February/March (20) /m^3·d^{-1}	Days without rainfall April (20) /m^3·d^{-1}	Days with rainfall February/March (16) /m^3·d^{-1}	Days with rainfall April (8) /m^3·d^{-1}
Flow	41.8	34.4	55	32.6
Std. dev.	3.8	2.8	19.5	3.2
Max. Flow	50.9	39	114.2	37.5
Min. Flow	35.8	29.4	37.7	28.1

Figure 25 shows the average hourly flow for research phase 3. Two outliners could be found in the dataset and have been excluded from the raw data, due to an obvious problem with the datalogger. On the 9th March 2012 at 10:50 AM and on the 5th April 2012 at 7:30 AM, the datalogger has registered flows of 12.0 and 16.5 l/s, respectively. Since there was no rain influence and flow data 10 minutes before and after shows flows below 1.0 l/s, it can only be suggested that the flowmeter were detecting wrong flows.

The hourly standard deviations in Figure 25 show basically the same trend, therefore the error bars have only been included for April in order to have a clear overview of the graphs. A difference in flow characteristics for March and April can be seen. The total sum of flow in April 2012 was generally lower. Morning peak flows in March were between 7:00 and 10:00 AM while in April morning peaks occurred between 8:00 and 11:00 AM. The average morning peak flow for February/March and April was 2.72 ± 0.39 m^3/h and 2.33 ± 0.39 m^3/h, respectively.

There is no specific knowledge about flow variability within the year. A possible explanation could be that people generally use less water in April than in March due to the change in season from summer to autumn.

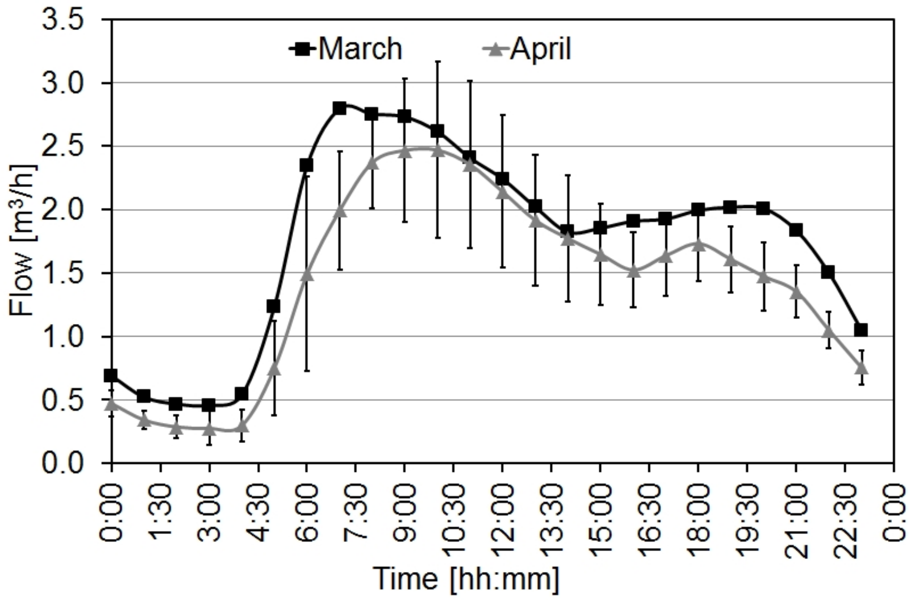

Figure 25: Average hourly flow of March and April 2012. Error bars are included for April.

3.5.3 Comparison of hydraulic parameters

Sasses' spreadsheet for the design of DEWATS plants was originally used to calculate the dimensions for the design of the Newlands-Mashu DEWATS plant [8]. The same spreadsheet was therefore used to calculate the investigated hydraulic loads within the treatment plant. This section presents the effect on the hydraulics of the DEWATS plant due to closing off treatment train 2 and 3.

The hydraulics of the two settlers should not be influenced by closing off treatment train 1 and 2. The same volume is still available which leads to the same flow rates within the settlers and no change in the theoretical treatment performance.

The ABR-AF chambers were originally designed for a width of 7.5 m divided into three different. Since the entire flow was going through train 1 the actual width of the ABR-AF chambers was changed to 2.5 m. Therefore the actual hydraulic parameters differ from the original design and need to be recalculated in order to have comparable parameters.

Table 20 gives an overview of the most important parameters which are used for the hydraulic load calculations. By distributing all inflow through train 1, the available treatment volume inside the ABRs divides by three to 22.05 m^3. Consequently the HRT changes to 12 h and the maximum upflow-velocity during the time of most wastewater flow rises to 2.97 m^3/h.

The table also contains the investigated flow rates for the months of research. In February and March 2012 the average flow is almost the same as expected in the original design spreadsheet. This leads to the same HRT, but since the measured maximum peak flow per hour is less than expected, the maximum upflow-velocity drops to 2.38 m^3/h. The lower average wastewater flow in April results in an HRT of 15 hours. Accordingly, the maximum peak flow is lower and results in a maximum upflow-velocity of 2.04 m^3/h.

Table 20: Hydraulic parameters of the initial design for all three ABR trains, the actual dimensions of train 1 during research phase 3 and the measured wastewater flows of train 1 during February/March and April 2012. Calculations are based on the equations in section 2.2.

Parameter	Unit	Design	Actual dimensions	February/March	April
Width	/m	7.5	2.5	2.5	2.5
Length	/m	0.7	0.7	0.7	0.7
Height	/m	1.8	1.8	1.8	1.8
Volume of 7 ABR chambers	$/m^3$	66.15	22.05	22.05	22.05
Average daily wastewater flow	$/m^3 \cdot d^{-1}$	41.6	41.6	41.8 ± 3.8	34.4 ± 2.8
Time of most wastewater flow	/h	8	8	8	8
HRT	/h	36	12	12	15
Max. peak flow	$/m^3 \cdot h^{-1}$	5.2	5.2	4.16	3.57
Max. upflow-velocity	$/m \cdot h^{-1}$	0.99	2.97	2.38	2.04

Maximum peak flows above 4.0 and 3.5 m^3/h were the exception. The average flow during morning peak flow hours was 2.72 ± 0.39 and 2.33 ± 0.39 m^3/h and was leading to upflow-velocities of 1.55 ± 0.22 and 1.33 ± 0.22 m^3/h for February/March and April, respectively. Figure 26 shows the average hourly upflow-velocities during research phase 3 compared with the design maximum after [8]. The defined limit for upflow velocities after [8] is 1.0 m^3/h. The average daily flows for February/March and April 2012 are above the design value during 16 and 8 hours, respectively.

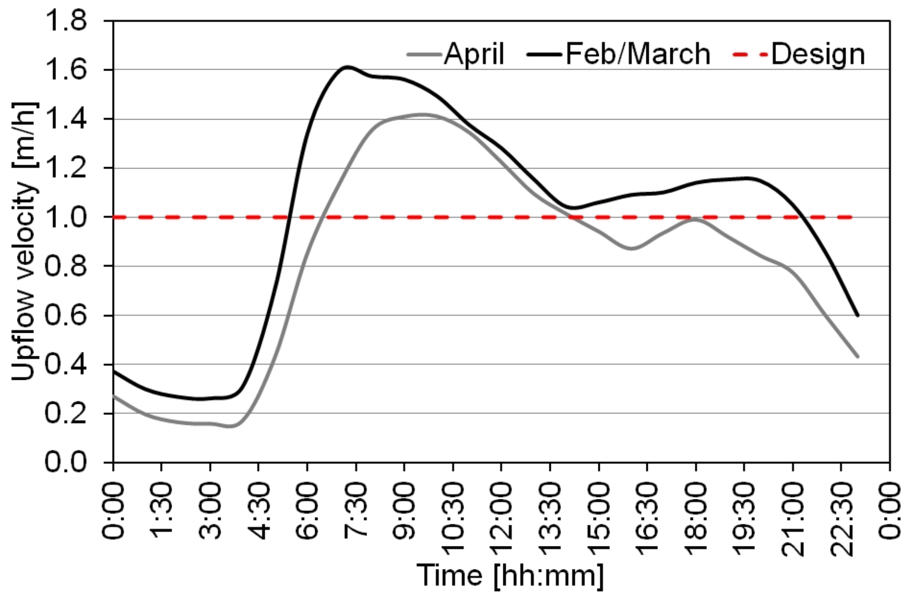

Figure 26: Average hourly upflow-velocities during in February/March and April 2012. The maximum design upflow-velocity after [8] is included. The Upflow-velocity calculates from the data out of Figure 25.

3.5.4 Hydraulic loading rates during major rainwater events

Section 3.5.2 has shown the influence of stormwater on the flow during research phase 3. This chapter presents the hydraulic loading rates during the main rainwater events in March 2012.

Table 21 shows the observed flow conditions during the three main rainwater events in March and the main hydraulic loading parameters. The average hourly flows have been calculated for the whole day and therefore underestimate the actual flow during the hours of rain. The maximum peak flows ranged from 3.76 m^3/h up to 6.68 m^3/h with only 3 days below 4.0 m^3/h. This resulted in upflow-velocities between 2.15 and 3.82 m/h, which was way above the maximum design upflow-velocity of 1.0 m/h.

Table 21: Hydraulic loading parameters and precipitation during the three main rainwater events in research phase 3.

Date	Precipitation /mm	Max. peak flow $/m^3 \cdot h^{-1}$	Max .upflow-velocity $/m \cdot h^{-1}$	Average hourly flow $/m^3 \cdot h^{-1}$
03.03.2012	12.4	3.93	2.25	2.14 ± 1.16
04.03.2012	79.2	6.68	3.82	4.76 ± 2.01
05.03.2012	1.6	5.66	3.24	3.17 ± 1.51
11.03.2012	31.4	5.11	2.92	2.57 ± 1.76
12.03.2012	0.6	4.72	2.69	2.61 ± 1.12
13.03.2012	7	3.76	2.15	2.03 ± 1.01
14.03.2012	9.6	4.01	2.29	2.86 ± 0.91
31.03.2012	21.2	4.43	2.53	2.42 ± 1.42

The hydraulics during stormwater events suggest that the performance of the plant has been influenced during most of the time in March. Washout of sludge and microorganisms can be presumed.

3.5.5 Performance-Assessment of the DEWATS plant

Due to the objectives of research phase 3 different physico-chemical parameters have been investigated then in research phase 2. Besides total COD and sludge height measurements the other investigated parameters were soluble COD, Alkalinity and pH. No nutrient measurements have been investigated because the VFW was not fed with wastewater. This is because the flow of train 1 could not get separated and would have overloaded the wetland.

In section 3.5.5.1 the whole COD dataset will be compared with the rainfall data and analysed on the feasibility to use the datasets for the calculations of organic loading rates and the treatment efficiency of the ABRs and AFs. The cleared dataset will be used to compare the actual treatment performance with the theoretical design performance after Sasses' design spreadsheet [8].

The investigated data of soluble COD will be used to examine the sampling method and show trends concerning the removal of dissolved and particulate organic matter.

The parameters pH and Alkalinity will be used to examine the stability of anaerobic digestion processes and will give an idea about the condition of the DEWATS plant.

3.5.5.1 Assessment of the COD

Overall ten COD values were measured during research phase 3. Only every second compartment was measured due to financial limitations in chemical supply and the expectation that not much difference was expected in between every compartment. The results are presented in Table 22.

Ten individual samples only exist for settler 2a, ABR compartment 1 and 7, and both AFs. Out of the ten sampling sets of settler 2a, three outliners can be observed with COD values between 3 000 to 5 000 mg/l. It was assumed that scum was caught into the sampling bottles due to the thick scum layer that was building up inside the settler. This led to unrepresentative high values concerning the calculations for the organic loading rates of the ABR compartments. As a result, the sampling method of the settler has been changed for the last two sampling campaigns, where samples were not taken with the grab sample stick anymore and instead the PVC sludge height measuring device was used to grab samples below the scum layer. Therefore, the results of settler 2a on the 24^{th} and 1^{st} May 2012 cannot be compared with the rest of the investigated data.

Two additional outliners can be observed on the 2^{nd} March 2012. Compartment 1 and AF1 show COD concentrations that have no relation to the foregoing or following concentrations and will be excluded for further calculations.

Comparing the COD values with the respective rainfall shows that the 7^{th} and 15^{th} March 2012 have lower concentrations through the ABR and AF compared to all other days. The 2^{nd} April was expected to show similar dilution effects on the system, since high surge flows were observed on the 31^{st} March, but the effect is not as distinctive as during the other major rainfall events and the dataset does not need to be excluded to calculate the treatment efficiency.

Two more days could have been affected by rainfall in April. Values for compartment 1 on the 20^{th} and 24^{th} April are lower than during foregoing and following sampling campaigns but the overall concentrations in the ABR and AF compartments do not show the same trend. A general high variation of concentrations inside ABR compartment 1 could be the explanation for the measured results. It was additionally shown in section 3.5.2 that flows in April generally have not been influenced by rainfall, which makes the data reliable to calculate the organic loading rates.

Furthermore have March and April shown a significant difference of the daily average wastewater flow (see section 3.5.2). Concerning the organic loading rates the months would need to be analysed individually. If also all outliners were revised, very little information about the treatment performance in March could be gained. As a reason of that organic loading rates and the treatment efficiency will only be calculated for April 2012

Table 22: Investigated COD data during research phase 3. Rainfall data is included as the sum of the 5 days before the respective sampling campaigns.

Date	Rainfall /mm	Feed /mg·L^{-1}	Set. 2a /mg·L^{-1}	C1 /mg·L^{-1}	C3 /mg·L^{-1}	C5 /mg·L^{-1}	C7 /mg·L^{-1}	AF1 /mg·L^{-1}	AF2 /mg·L^{-1}
02.03.12	0.8		3097	3391	574	556	632	903	450
07.03.12	94	636	785	599			431	337	290
15.03.12	49	1734	1049	442			442	346	297
19.03.12	10.8	632	836	811	539	526	710	517	371
27.03.12	6.4	836	1275	574			482	458	424
02.04.12	25	1091	757	616	714	507	506	507	401
12.04.12	0		5385	1329	641	595	648	504	442
20.04.12	3.8	878	3395	751	608	613	560	537	431
24.04.12	9.2		843	692	562	503	558	558	406
01.05.12	0.4		1483	1416	658	644	749	555	466

Due to the clearance of the dataset a few assumptions needed to be made to calculate the organic loading rates and treatment efficiency of the plant.

1. The total exact number of households, connected to the DEWATS system, is unknown and calculations will be done with the design parameter of 462 persons.
2. The sampling method for feed concentrations was not changed after research phase 2 and results are not useful to calculate the organic loading of the plant. Therefore the feed will be characterised by using the design value of 110 g COD/(cap·day) which equals a total COD concentration of 1.477 mg/l, taking the average daily flow of 34.4 m^3 into account.
3. The existing data of the settler is not appropriate, due to an assumed accumulation of solids inside the samples. Therefore the design spreadsheet was used to calculate the removal inside the settler. A COD removal rate of 29% is expected with a given HRT of 2 hours. This results in a COD concentration of 1046 mg/L that would enter the first ABR compartment.

Table 23 illustrates the average COD concentrations of the sampling campaigns between the 2nd April and the 1st May 2012. Compartment 1 shows very high variations in COD concentrations and a COD removal rate of 9% compared to the assumed influent concentration of 1046 mg/L. A removal rate of 34 % can be observed in compartment 3. Until the AFs almost no COD removal can be observed. Concentrations even show an increase after compartment 5. Comparatively low standard deviations also support that concentrations stay more or less the same inside the ABR compartments. However, AF 1 and 2 show removal rates of 12 and 19%, which results in an effluent concentration of 429 ± 27 mg COD/L and a total removal efficiency of 55% inside the ABR and AF compartments based on the assumed value of 1046 mg/L for the settler.

A total removal of 36.1 kg COD/d was calculated with the difference of the average daily inflow and outflow COD concentrations and the total average daily wastewater flow of 34.4 m³. This complies with a removal rate of 70% through the treatment plant after AF2 and a per capita removal of 78 g COD/d on the basis of 462 persons.

Table 23: Average COD concentrations between the 2nd April and the 1st May 2012. The treatment efficiency of compartment 1 has been calculated with the theoretical value of 1046 mg/L out of the design spreadsheet. A total quantity of 5 samples was available.

Location	Average /mg·L^{-1}	Std. dev. /mg·L^{-1}	Max. /mg·L^{-1}	Min. /mg·L^{-1}	Treatm. effic. /%
C1	961	380	1416	692	9
C3	637	57	714	562	34
C5	573	64	644	503	10
C7	604	96	749	506	0
AF1	532	26	558	504	12
AF2	429	27	466	401	19

The treatment performance of the plant was further compared with the theoretical performance after Sasses' design spreadsheet (Figure 27) [8]. The hydraulic parameters of April 2012 have been used for the calculations (Table 20).

The graphs show the same trend until compartment 3, while the first two compartments actually remove more COD than theoretically expected. In between compartment 3 and 7 a total removal of 315 mg COD/L was expected while the observed values show a removal of 33 mg COD/L. Sasses' calculations show a COD removal of 206 mg/L for the first AF and 32 mg/L for the second AF. The investigated data shows a COD removal of 72 and 97 mg/L, respectively.

The little removal within the ABR compartments indicates that the system was overloaded and proper anaerobic digestion was not taking place anymore. The contact time between incoming wastewater and active anaerobic microorganisms could have been too low to result in a removal of organic matter.

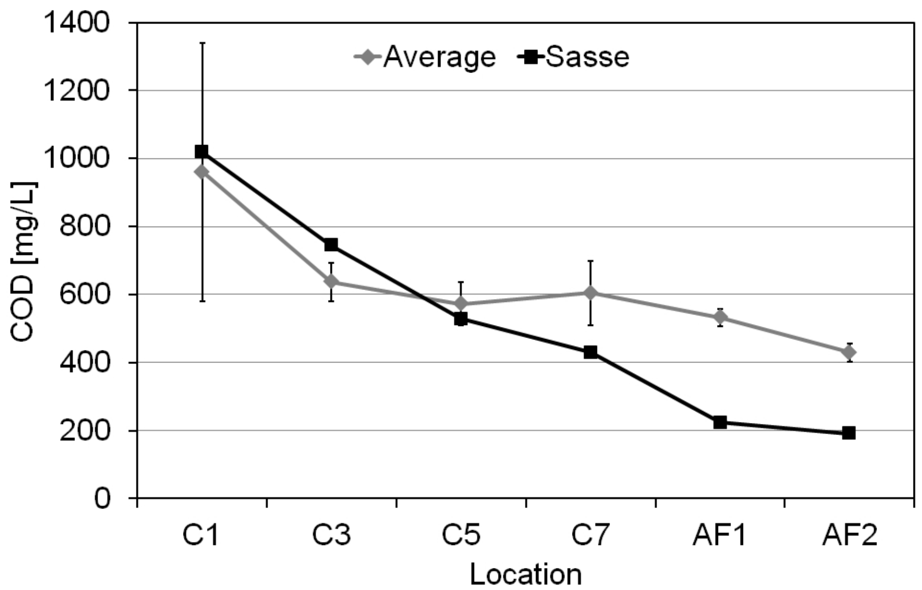

Figure 27: Average measured COD concentrations with included standard deviations and theoretical COD values calculated after the Sasse design spreadsheet in the different locations of the treatment plant.

3.5.5.2 Comparison of the COD fractions

This section will analyse the correlation of soluble and particulate COD as a part of the total COD. Due to the observed outliners in the last section, statements about the actual treatment efficiency of the settler and compartment 1 need to be made with caution. The data will be used to show trends of which parts of the total COD was removed through the plant. The data can give an idea about the proportion of solids inside the respective samples and therefore give explanations for the high total COD values that were measured during research phase 3.

High total COD values with the respective soluble and particulate COD values are presented in Table **24**. It can be observed that for every sample, where a high total COD has been investigated, the values for particulate COD are comparatively high. This proves the assumption that a high proportion of solids got caught into the sample. This happened because of the built up of a thick scum layer inside the settler. When the layer got larger the scum was pushed through into the first ABR compartment and has shown the same scum built up effect.

Table 24: Total, particulate and soluble COD concentrations of samples where the percentage of particulate COD was higher than 60%.

	total COD			particulate COD			soluble COD		
Date	**Set. 2a /mg·L^{-1}**	**C1 /mg·L^{-1}**	**C7 /mg·L^{-1}**	**Set. 2a /mg·L^{-1}**	**C1 /mg·L^{-1}**	**C7 /mg·L^{-1}**	**Set. 2a /mg·L^{-1}**	**C1 /mg·L^{-1}**	**C7 /mg·L^{-1}**
02.03.12	3097	3391		2659	3003		439	388	
19.03.12		811	710		520	476		291	233
12.04.12	5385	1329	648	4882	911	414	503	417	234
20.04.12	3395			2969			426		
01.05.12	1483	1416		794	979		688	437	

Table 25 presents the results for the average particulate COD during the entire research phase. High particulate COD concentrations can be found at the front of the system. On average, a high proportion of particulate COD gets removed until compartment 3, which can be expected. Nevertheless are the standard deviations for settler 2a and compartment 1 too high to calculate the actual removal of particulate COD. This moreover supports the assumption that the results are totally depended on the sampling method and the influence of scum built up inside the chamber. The high standard deviations of compartment 7 cannot directly be related to scum built up, since this was mainly observed in the first compartments of the ABR. High turbidity of the wastewater, due to high upflow-velocities could be the reason for the measured results.

On average no removal was observed after compartment 3 until the wastewater reached the AFs. It can be assumed that the AF was acting on COD through solids retention. Due to a longer contact time between the substrates and the biomass inside the filter material a higher reduction of particulate COD could be assumed.

The minimum particulate COD concentration of 73 mg/L in compartment 1 has been measured on the 15th March 2012. As shown in the last section, rainfall on the 14th March caused a dilution of the system resulting in comparatively low total COD values. It was suggested that the effect of stormwater would cause a dilution of soluble COD. The results contradict with that and it could be assumed that the solids already settled again or got washed out and the dilution effect can be observed on the particulate fraction of the COD.

Table 25: Average particulate COD concentrations during research phase 3. The data is calculated as the difference of total and soluble COD for the whole dataset of March, April and the 1st May 2012. The number in parenthesis represents the quantity of available samples.

Location	Average /mg·L^{-1}	Std. dev. /mg·L^{-1}	Maximum /mg·L^{-1}	Minimum /mg·L^{-1}
Set. 2a (4)	2826	1673	4882	794
C1 (9)	731	907	3003	73
C3 (6)	266	37	329	219
C5 (6)	238	37	284	192
C7 (9)	272	125	476	135
AF1 (9)	200	58	287	94
AF2 (10)	148	26	171	95

Average soluble COD concentrations are shown in Table 26. The standard deviations show that the measured soluble COD concentrations are more reliable than the particulate COD concentrations. Compartment 1 shows an average removal of 149 mg/L. In between compartment 3 and 7, values decrease by an average of 55 mg/L. The AFs remove another 51 mg COD/L. Generally can be seen that the reduction of soluble COD after compartment 3 is very low. A reason for the poor treatment could be that the contact time between microorganisms and the substrates is not long enough for a complete digestion.

However, a reduction of particulate COD should also result in an increase of soluble COD. It could be assumed that the COD contains a high proportion of non-biodegradable COD, but this parameter has not been measured during the sampling campaigns.

Table 26: Average measured soluble COD concentrations during research phase 3. The data is calculated out of the whole dataset of March, April and the 1st May 2012. The number in parenthesis represents the quantity of available samples.

Location	**Average /mg·L^{-1}**	**Std. dev. /mg·L^{-1}**	**Maximum /mg·L^{-1}**	**Minimum /mg·L^{-1}**
Set. 2a (4)	514	121	688	426
C1 (9)	365	59	437	272
C3 (6)	356	50	403	269
C5 (6)	336	57	407	242
C7 (9)	301	52	364	233
AF1 (9)	280	45	345	201
AF2 (9)	250	49	326	173

3.5.5.3 pH values

Figure 28 presents the measured pH values. Over the sampling period the values have always been between 6.5 and 7.0. High values have been measured at the beginning of the plant while the pH is decreasing within the ABR compartments and increasing again towards the end. The treatment plant generally shows stable and good pH conditions for the anaerobic digestion processes.

The decrease of pH within the compartments is due to the production of acidity in the first steps of the anaerobic digestion processes. The increase towards the end of the plant can be explained by the process of methanogenesis, which is using the acids produced in the stages before. This can mainly be observed in ABR compartment 7 and the AFs. It could be possible that organic matter was being withheld at the AF step and the longer retention could have caused enough time to breakdown the particles. Generally increasing pH values over time could be explained by less acidifying processes and therefore less reduction of organic matter.

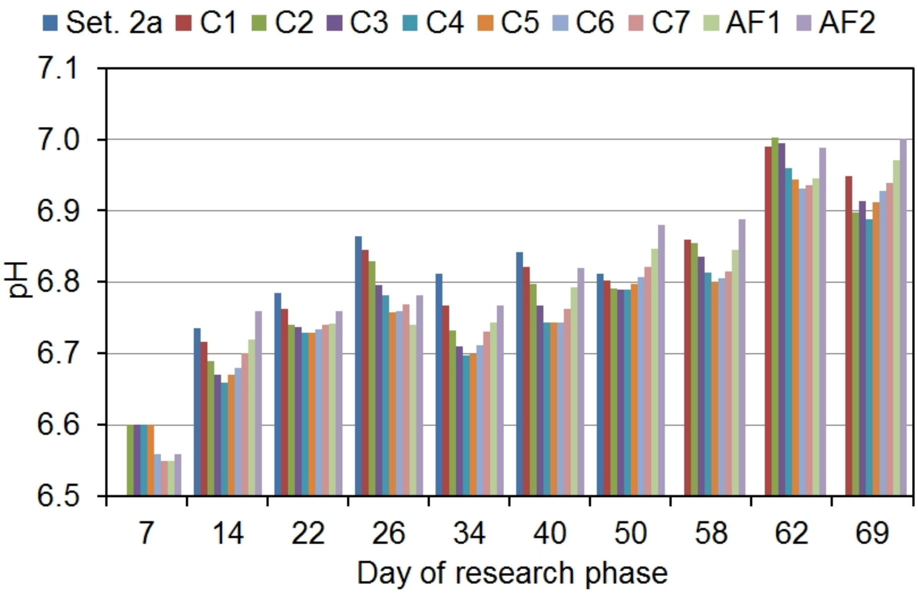

Figure 28: pH values over time of research phase 3. Single bars show measured values from settler 2a to AF 2, with data points missing for the settler on day 7, 58, 62, and 69 and on day 7 for ABR compartment 1.

3.5.5.4 Alkalinity

The alkalinity tests have only been investigated from compartment 1 to AF2 and are measured in "mg $CaCO_3$/L". No information about the feed or settler concentrations was available. Figure 29 presents the investigated data. During the first two sampling campaigns, concentrations show a rising trend from around 300 mg $CaCO_3$/L to 350 mg/l. The next three sets show almost equal values of around 400 mg $CaCO_3$/L. Results of the last sampling campaign are slightly above 400 mg $CaCO_3$/L.

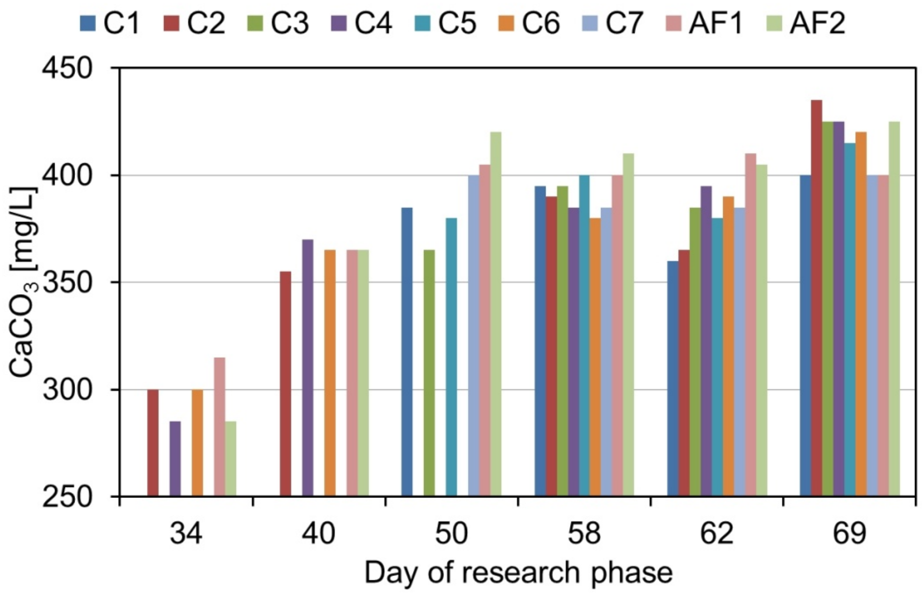

Figure 29: Alkalinity concentrations over time from ABR compartment 1 to AF2. Day 34, 40 and 50 have missing data on every second compartment.

3.5.5.5 Sludge heights

Sludge heights inside the ABR compartments were investigated ten times during the research phase and are presented in Figure 30. The sludge heights in the first three compartments were all around 20 cm and stayed almost the same until the end of the research phase. Sludge built up when reaching the 4th compartment with values between 30 and 40 cm, while compartment 5 showed a decrease from 40 down to 29 cm half way through the research phase. Compartment 6 with values between 50 and 60 cm had the highest sludge volume of all compartments, while volumes decrease to around 40 cm in compartment 7.

Although expected, sludge accumulation inside the compartments could not be observed. This might be an indicator for little bacterial growth and low treatment performance.

Furthermore was no sludge migration noted, which was expected due generally higher upflow velocities and the influence of stormwater entering the system. Previous events may have caused a selection pressure for those organisms, which could not be washed out.

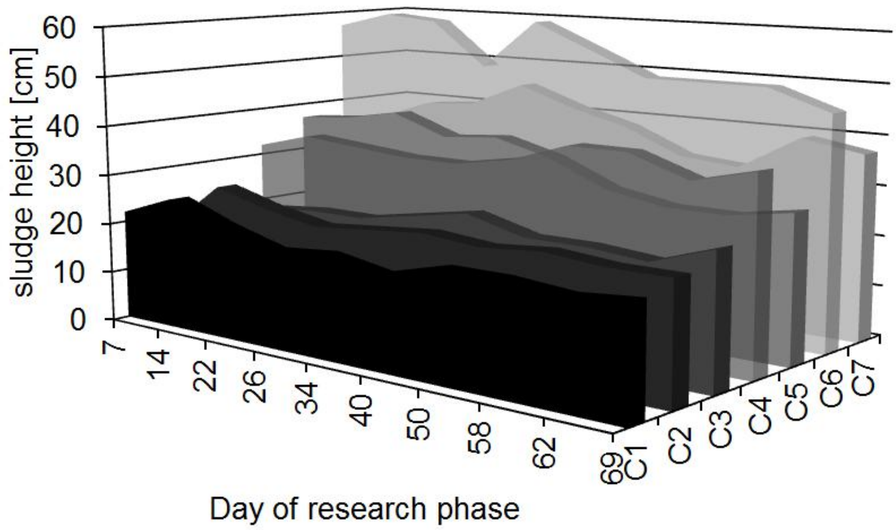

Figure 30: Sludge heights of every compartment in train 1 during research phase 3.

3.5.5.6 Settling velocities and the formation of granular sludge

The forming of granular sludge plays an important role on the settling characteristics of particles. It is the determining factor if solids get washed out during high upflow velocities or are remaining inside the ABR compartments.

Solid settling rates are influenced by different factors.

- Granulation: The size of the granules and the proportion of microorganisms is a factor in determining the average settling velocity of the sludge [31].
- Fixation: Microorganisms, attached to walls, flocs and films will retain within the compartment under much higher upflow-velocities than suspended microorganisms [31].

Neither settling velocities nor studies on granular sludge have been investigated in the scope of this thesis

At the end of research phase 3, Pillay and co-workers [33] observed sludge from the Newlands-Mashu plant using scanning electron microscopy. No granular sludge was observed in all compartments. However, *Methanosaeta*-like microorganisms were tentatively identified in all ABR compartments and decreasing from compartment 2 onwards. A review by [16] has shown similar results for most pilot-scale ABR studies. The microorganisms were found in aggregates but not complete granules. It is plausible that granular sludge formation was hindered by the high upflow velocities as shown in other related work [34].

4. Conclusions

This section summarises the gained knowledge for each hypothesis, presented in section 4.1 and compares the treatment efficiency of the Newlands-Mashu DEWATS plant with other systems in section 4.2.

4.1 Assessment of the produced hypothesis

1. Rainwater influences the performance of the plant

It was shown that the recorded rainfall had an influence on additional stormwater entering the system during both research phases. Observed heavy rainfall events had a strong influence on the hydraulic conditions of the treatment plant. Although expected that every rainfall would cause higher flows inside the treatment plant, a statistical correlation could not be evaluated. It was observed that the intensity and duration of the rainfall events played a major role on the influence of the hydraulic conditions. Intense rain over a short period influenced the treatment plant more than less intense rainfall over a long period. This is not a surprise, since the sewer line would fill up much faster during heavy rainfalls and the influence on flow could directly be noticed. Nevertheless, it was assumed that every rainfall would cause a dilution of the wastewater inside the system. It must be decided for each event individually if the additional volume that would enter the plant has a distinctive influence concerning the analysis of collected datasets. In general should the design of a DEWATS plant exclude any stormwater entering the system, since it not only dilutes the incoming wastewater and therefore the organic load that is needed for anaerobic digestion, but also carries substantial amounts of silk and rubbish into the treatment system [8].

2. The treatment plant shows satisfying results compared to South African discharge standards

The treated wastewater of the DEWATS plant is envisaged for the re-use in community gardens. Therefore it is important that the effluent meets local discharge standards. Table 27 summarises the investigated data from both research phases. The only discharge limit that exists for irrigation after [35] is the COD concentration. For the wastewater use of up to 500 m³/d the discharge limit is 400 mg COD/L. The combined modules of the DEWATS process meet this discharge limits. Concerning the concentrations of NH_4-N, no discharge limits but suggestions are given for different water quality ranges. The VFW effluent concentration of 25 ± 9 mg NH_4-N/L is suspected to have a negative effect on most crops and an increasingly serious likelihood of ground water contamination [36]. Suggestions or discharge limits for irrigation concerning PO_4-P could not be found in the governmental documents for the reuse of wastewater.

Although the treated wastewater of the Newlands-Mashu DEWATS plant is not meant to enter any open water sources, comparisons with the general effluent discharge standards give an idea about the effluent quality of the treatment plant. Regarding the COD effluent concentrations a limit of 75 mg/L exists, which was not achieved at any stage of the operation. NH_4-N and PO_4-P limits are at 3 and 10 mg/L, respectively [35]. Measured NH_4-N concentrations are up to ten times higher than the limit. The average PO_4-P effluent concentration of wetland was 4.9 ± 1.1 mg/L. Concerning this parameter the effluent would be allowed to get discharged into an open water source. Therefore it can be assumed that the PO_4-P effluent concentration is useful for irrigation.

It cannot be concluded if the effluent shows satisfying results compared to the discharge standards. Concerning the use for irrigation, the NH_4-N concentrations are too high to reuse the treated wastewater. The discharge standards for open water bodies furthermore show that the treatment plant has a poor removal of organic matter, especially after the ABR-AF treatment step.

Table 27: Average effluent COD, NH4-N and PO4-P concentrations and the respective std. dev. during research phase 2 and 3. The number in parenthesis shows the quantity of samples.

Research Phase	Location	COD $/mg \cdot L^{-1}$	PO4-P $/mg \cdot L^{-1}$	NH4-N $/mg \cdot L^{-1}$
2	VFW	117 ± 21 (4)	4.9 ± 1.1 (5)	25 ± 9.0 (6)
	T1E	317 ± 84 (4)	7.1 ± 0.3 (5)	59.0 ± 8.7 (6)
	T2E	280 ± 70 (4)	7.4 ± 0.2 (5)	57.5 ± 7.5 (6)
	T3E	261 ± 86 (4)	7.5 ± 0.6 (5)	58.11 ± 8.2 (6)
3	T1E	429 ±27 (5)	-	

3. The actual treatment performance meets the theoretical design performance

Sampling of the feed and settler has shown difficulties in order to gain knowledge about the COD influent concentrations of the treatment plant and therefore the calculations of the treatment performance.

Overall was no feed data used to characterise the COD inflow concentration of the treatment plant. The results have shown that this method needs to be revised and it was realised that feed measurements over an hour only show the concentrations for the moment of the measurement and were not useful to correlate with the investigated data from the following treatment steps.

Settler concentrations were considered to be more useful for the characterisation of the influent to the ABR compartments. The measured settler values have been used for research phase 2.

The used sampling method for the settler has not shown satisfying results during research phase 3 anymore. The comparison of total and soluble COD data has shown that samples of the settler and the first compartment were containing a very high amount of particulate COD. It was assumed that scum was caught inside the sampling bottles during some research campaigns. The scum layer inside the sampling point settler 2a built up to an extent, where the sampling method was not appropriate anymore. The method was changed for the last two sampling campaigns in April 2012, but the results of total COD for settler 2a were considered to be not useful to calculate the loading of the ABR.

Therefore, the theoretical design inflow concentration was calculated after [8] and used to determine the removal of COD during research phase 3. Although this was not reflecting the real conditions inside the settler either, the data seemed to be more useful than the actual collected data and was also reflecting the values that were determined during research phase 2.

Table 28 summarises the hydraulic and organic loading conditions of train 1 during both research phases, compared to the theoretical design performance. Since the AFs are included in the total volume, the HRTs are higher than in the ABRs itself. The COD concentrations for research phase 2 are based on the measured effluents of settler 2a and AF2 of the last three sampling campaigns between the 24th January and 8th February 2012. This data could be used for the calculations of organic loading rates, because flow data was available for treatment train 1 from the 24th January onwards. The ABR COD influent concentrations for research phase 3 are based on the design spreadsheet, as calculated in section 3.5.5.1. COD outflow concentrations reflect the investigated results of AF2 between the 2nd April and 1st May 2012.

Although the results for train 1 in research phase 2 only show the trend of COD concentrations, it can be observed, that the general treatment performance of the ABR-AF train 1 has not changed during research phase 3. This even though HRTs were 2.3 times lower and upflow velocities well above the design maximum of 1.0 m/h. Comparing both research phases with the theoretical performance shows that the treatment plant is generally not functioning well. A theoretical expected removal of 89% compares with 59% during both research phases. Less removal efficiency during research phase 3 is plausible because of higher organic loadings, but the outcomes assume that the treatment plant was already not performing well during research phase 2.

Table 28: Hydraulic and organic loading parameters for research phase 2, 3 and the initial design. Calculations are based on the equations in section 2.2. The removal efficiency is calculated for the entire ABR-AF train 1. The number in parenthesis presents the quantity of samples.

Parameter	Unit	RP2 Train 1	RP3 Train 1	Design Train 1
Average Daily Flow	$/m^3 \cdot d^{-1}$	15.1 ± 1.56	34.4 ± 2.8	13.9
HRT	/h	49	22	53
Max. peak flow	$/m^3 \cdot h^{-1}$	1.26	3.57	1.73
Upflow-velocity	$/m \cdot h^{-1}$	0.72	2.04	1.0
COD in	$/mg \cdot L^{-1}$	819 ± 111 (3)	1046	866
COD out	$/mg \cdot L^{-1}$	335 ± 93 (4)	429 ± 27 (5)	92
OLR	$/kg \cdot m^3 \cdot d^{-1}$	0.40	1.16	0.39
Removal efficiency	/%	59	59	89

4. <u>The treatment plant cannot handle increased hydraulic loadings and collapses</u>

In terms of COD removal inside the ABR compartments the results have shown that the treatment plant was not functioning well when exposed to flow three times above the design maximum. It was suggested that this was not only due to the hydraulic overloading but also because the start-up conditions of train 1 were already influenced by poor treatment performance during research phase 2. Earlier experiences in research phase 1 have shown that treatment train 1 was generally experiencing higher hydraulic loadings than the other two treatment trains. It could be concluded that treatment train 1 was generally overloaded during the whole operation of the DEWATS plant.

During research phase 3 it was shown that the removal of total COD mainly takes place in the first two ABR compartments and the AFs. Very little removal was observed between compartment 3 and 7. The comparison of particulate and soluble COD fractions has shown that the settler and the first two ABR compartments remove a high amount of particulate COD. Furthermore was shown that the first compartment was also efficient in removing soluble COD, while poor removal was observed in the following ABR compartments.

Analyses on the sludge at the end of research phase 3 have identified *Methanosaeta*-like microorganisms in the first four compartments, while decreasing from the second compartment onwards [33]. This could also be indicated by the measured pH values which decrease within the first four compartments. Although the difference in pH units was only around 0.1 the trend of decreasing and increasing pH values could be observed during every sampling campaign. Higher pH values inside the AFs also support the measured COD removal rates, since it is suggested that increasing pH values show the presence of methanogens which remove the acids produced during the acidification [30]. It could therefore be concluded that the high load of biodegradable organic material to the first compartments of the ABR, led to a large number of anaerobic microorganisms which were able to degrade the particulate organic matter [30]. It could be assumed that less observed methanogens in the following compartments could not remove the acids produced during acidification and due to an accumulation of VFA the pH decreased.

The overall results show that the system is not functioning well when exposed to hydraulic and organic loadings 3 times above design maximum, but none of the measured parameters indicate that the system is about to collapse. The measured alkalinity values of around 400 mg $CaCO^3$/L are suggested to be very low for anaerobic digestion [24, 26]. Nevertheless was no negative effect on the pH values observed. Studies with $NaHCO_3$/COD ratios between 0.5 and 0.05 have shown no significant variations in the overall COD removal, while $NaHCO_3$ concentrations were as low as 100 mg/L for the influent [37].

5. High upflow-velocities lead to migration and washout of sludge.

It was suggested that heavy rainfall events and the increased hydraulic loadings, causing high upflow-velocities inside the ABR compartments, would cause carry over and washout of sludge and microorganisms. But neither research phase 2, nor research phase 3 have shown a migration of sludge. Studies on sludge of the ABR compartments of train 1 at the end of research phase 3 have shown that microorganisms formed bioaggregates [33]. It was assumed that during research phase 1 a selection pressure occurred, which caused the washout of slow-growing microorganisms, such as methanogens, and inhibited the formation of granules inside the compartments. This effect could also be shown by related work [34]. Nevertheless are these aggregates less sensitive to high surge flows and remain at the bottom of the compartments, which could explain that no sludge movement was observed.

Related studies have shown that the combination of high OLR and low inflow alkalinity concentrations affect the critical upflow velocity that determines if slow-growing microorganisms can establish in the system or get washed out [30].

4.2 Comparison of the treatment performance with other studies

Overall very little research has been done on full-scale DEWATS systems so far. Most studies deal with pilot-scale systems and concentrate on a specific treatment step of the overall system.

Table 29 gives an overview of different studies that assessed the treatment performance of ABRs concerning the removal of COD. The data of research phase 3 is included and shows the removal efficiencies for the entire ABR-AF train 1 and for the ABRs on their own. Pilot-scale studies generally show better removal efficiencies than full-scale system. Treating wastewater under defined simplified laboratory conditions is usually not influenced by operational problems that appear on full-scale plants. The inflow-characteristics are controlled and processes inside the ABRs better understood. It is therefore difficult to compare the performance of the Newlands-Mashu DEWATS with pilot-scale systems.

A full-scale system was studied by [9] and as cited by [16] has shown a 70% to 80% COD reduction. The system was constructed of eight ABR compartments with an upflow-velocity of 3.0 m/h, an HRT of 10.3 hours and an OLR of 0.85 $kg/m^3{\cdot}d$. Furthermore has a variation of the OLR between 0.4 and 2 $kg/m^3{\cdot}d$ not shown an effect on the removal efficiency. The COD reduction shown in this study, and treating the wastewater with seven ABR compartments and two AFs, could not achieve a reduction above 59% at similar organic loadings but higher HRTs and lower upflow-velocities. Another full-scale system, studied by [38] has shown similar treatment efficiencies as the DEWATS plant studied in this thesis.

Table 29: Overview of full-scale and pilot-scale studies on ABRs treating synthetic and domestic wastewater. The investigated data of research phase 3 of this study is included.

Reference	ww source	Volume /L	Cmpts. N	HRT /h	COD in /$mg \cdot L^{-1}$	OLR /$kg \cdot m^3 \cdot d^{-1}$	Removal /%
[19]	synthetic	10	8	10	4000	9.6	52
[19]	synthetic	10	8	5	4000	19.2	90
[17]	synthetic	10	8	8	500	1.5	90
[20]	domestic	32	9	6	400	1.6	84
[31]	domestic	15	5	24	682	0.67	82
[31]	domestic	15	5	8	682	2.1	68
[38]	domestic	42000	not listed	36	2914	0.46	47
[16]	domestic	394000	8	10	315	0.85	70
RP3 ABR+AF	domestic	31000	9	22	1046	1.16	59
RP3 ABR	domestic	22000	7	15	1046	1.63	43

5. Recommendations

The results have shown that the DEWATS plant was not performing well in terms of COD removal inside the ABR compartments. Further investigations of different parameters need to be done to examine the conditions inside the ABR. Possible investigations are:

- A COD mass balance in order to investigate the amount of produced methane.
- Measurements of volatile fatty acids (VFA) inside each compartment in order to gain knowledge about the acidifying processes inside the ABR compartments.
- Settling tests of the sludge to determine the settling rates and make comparisons with the measured upflow-velocities.

Concerning the sampling method of the feed and settler, following investigations and tests could be done:

- Characterise feed concentrations by a 24-hour sampling campaign. Collect samples every 10 minutes over one hour. Homogenise the contents and measure the concentrations for each hour of one day. This campaign should be done for every following research phase.
- Consider the desludging of the settler and measure the sludge heights of the settler continuously.

Operation and maintenance were influenced by several issues concerning the design of the plant. Most of these problems would have not occurred in a full-scale system, which is not built for research purposes since the design would be simplified. Nevertheless can following recommendations be made:

- If downflow pipes instead of standing baffles are installed, a regular cleaning of the pipes is necessary. Sludge and scum pushes through the pipes and leads to blockages that could cause uneven flow distribution inside the compartment and the throttling of flow.
- Where rainwater is likely to enter the treatment plant, a stormwater overflow is essential. Additionally a safety factor for rain influence could be involved into the design for new plants

6. References

[1] United Nations Department of Economic and Social Affairs (2010). Goal 7: Ensure Environmental Sustainability. http://www.un.org/milleniumgoals/environ.shtml, opened 10.07.2012

[2] Tissington, K. (2011). Basic Sanitation in South Africa: A Guide to Legislation, Policy and Practice. Socio-Economic Rights Institute of South Africa (SERI), Johannesburg, South Africa

[3] Hutton, G., Haller, L. (2004). Evaluation of the Costs and Benefits of Water and Sanitation Improvements at the Global Level. Water, Sanitation and Health Protection of the Human Environment. World Health Organization, Genf, Switzerland

[4] World Health Organization (2011). 10 facts on sanitation. http://www.who.int/features/factfiles/sanitation/en/index.html, opened 10.07.2012

[5] Statistics South Africa (2010). Millenium Development Goals, Country Report 2012. South Africa

[6] Eales, K. (2008). Rethinking sanitation improvement for poor households in urban South Africa. IRC Symposium, 19 – 21 November 2008, Delft, The Netherlands.

[7] Ulrich, A., Reuter, S., Gutterer, B. (2009). Decentralised Wastewater Treatment Systems (DEWATS) and Sanitation in Developing Countries. A Practical Guide. Water, Engineering and Development Centre (WEDC), Loughborough University, Leicestershire, United Kingdom

[8] Sasse, L. (1998). DEWATS Decentralised Wastewater Treatment in Developing Countries. Druckerei Verwohlt, Bremen, Germany

[9] Orozco, A. (1997). Pilot and Full-scale Anaerobic Treatment of Low-Strength Wastewaters at Sub-Optimal Temperature (15 °C) with a Hybrid Plug Flow Reactor. Proceedings of the 8th International Conference on Anaerobic Digestion, Vol.2, Sendai, Japan, pp.183-191

[10] United Nations Department of Economic and Social Affairs (2011). “Water for Life” UN-Water Best Practices Award. http://www.un.org/waterforlifedecade/winners2011.shtml opened 05.07.2012

[11] Henze, M., Harremoës, P., Jansen, J.l.C., Arvin E. (1996). Wastewater Treatment. 2. Edition. Springer Verlag, Berlin, Germany pp. 11-33

[12] Tilley E., Lüthi C., Morel A., Zurbrügg, C., Schertenleib R. (2008). Compendium of Sanitation Systems and Technologies. Mattenbach AG, Winterthur, Switzerland

[13] Bitton, G. (2005). Anaerobic Digestion of Wastewater and Biosolids. In: Wastewater Microbiology. 3. Edition. John Wiley & Sons, Inc., Hoboken, New Jersey, USA: 345 – 371.

[14] Bachmann A., Beard, V. L., McCarty, P. L. (1985). Performance characteristics of the anaerobic baffled reactor. Water Research **19(1):** 99 - 106.

[15] Weiland, P., Rozzi, A. (1991). The start-up, operation and monitoring of high-rate anaerobic treatment systems: discussers report. Wat. Sci. Technol. **24(8)**: 257 - 277.

[16] Barber, W.P., Stuckey D. C. (1999). The use of the anaerobic baffled reactor (ABR) for wastewater treatment: a review. Water Research **33(7)**: 1559-1578.

[17] Krishna, G.V.T.G., Kumar, P., Kumar, P. (2009). Treatment of low-strength soluble wastewater using an anaerobic baffled reactor (ABR). Journal of Env. Man. **90**: 166 – 176.

[18] Stuckey, D.C. (2012). Recent developments in anaerobic membrane reactors. Bioresource Technology. Available online 5 June 2012

[19] Stuckey, D.C., Nachaiyasit, S. (1997). The effect of shock loads on the performance of an anaerobic baffled reactor (ABR). 2. Step and transient hydraulic shocks at constant feed strength. Water Research **31(11)**: 2747 – 2754.

[20] Bodkhe, S.Y. (2009). A modified anaerobic baffled reactor for municipal wastewater treatment. Journal of Env. Man. **90**: 2488 – 2493.

[21] Morel, A., Diener, S. (2006). Greywater Management in Low and Middle-Income Countries, Review of different treatment systems for households or neighbourhoods. Swiss Federal Institute of Aquatic Science and Technology (Eawag). Dübendorf, Switzerland.

[22] Stefanakis, A.I., Tsihrintzis V.A. (2012). Effects of loading, resting period, temperature, porous media, vegetation and aeration on performance of pilot-scale vertical flow constructed wetlands. Chem. Eng. Journal **181 – 182**: 416 – 430.

[23] Rump, H.H. (1999). Laboratory Manual for the Examination of Water, Waste Water and Soil. Laboratory manual book. 3. Edition. WILEY-VCH Verlag GmbH, Weinheim, Germany

[24] Gerardi, M.H. (2003). The Microbiology of Anaerobic Digesters. 1. Edition. John Wiley & Sons, Inc., Hoboken, New Jersey, USA

[25] Metcalf & Eddy, Inc., Tchobanoglous, G., Burton, F.L., Stensel, H.D. (2003). Wastewater Engineering: Treatment and Reuse. 4. Edition. McGraw Hill Companies, Inc., Boston, USA

[26] Su, D.L., Wang, J.L., Huang, Y.H., Zhou ,D. (2006). Variation of alkalinity and its regulation of ABR. http://www.ncbi.nlm.nih.gov/pubmed/17256603#, opened 12.07.2012

[27] Kayranli, B., Aysenur, U. (2011). Effects of temperature and biomass concentration on the performance of anaerobic sequencing batch reactor treating low strength wastewater. Desalination **278**: 77 – 83.

[28] Elmitwalli, T. A., Oahn, K.L.T., Zeeman, G., Lettinga, G. (2002). Treatment of domestic sewage in a two-step anaerobic filter/anaerobic hybrid system at low temperature. Water Research **36**: 2225 – 2232.

[29] Virginia State University (editor). Ammonia Emissions from Animal Agriculture. http://www.ammonia.ext.vt.edu/sources.php, opened 13.07.2012

[30] Foxon, K.M. (2009). Analysis of a pilot-scale anaerobic baffled reactor treating domestic wastewater. Ph.D. Thesis. School of Chemical Engineering, University of KwaZulu-Natal, Durban, South Africa

[31] Nasr, F.A., Doma, S.D., Nassar, F.H. (2009). Treatment of domestic wastewater using an anaerobic baffled reactor followed by a duckweed pond for agricultural purposes. Environmentalist **29**, 270-279.

[32] Foxon, K M., Buckley, C.A., Brouckaert, C.J., Dama, P., Mtembu, Z., Rodda, N., Smith, M.T., Pillay, S., Arjun, N., Lalbahadur, T. and Bux, F. (2005): Evaluation of the Anaerobic Baffled Reactor for Dense Peri-Urban Settlements. WRC Report No. 1248/1/06. pp. 1-224, University of KwaZulu-Natal, Pollution Research Group, Durban, South Africa

[33] Pillay, S., Reynaud, N., Schöbitz, L., Pietruschka, B., Foxon, K.M., Buckley, C.A. (2012). Evaluation of the DEWATS process for decentralised wastewater treatment. WRC Report K5/2002 (in progress), University of KwaZulu-Natal, Pollution Research Group, Durban, South Africa

[34] Pillay, S. (2006). Microbial studies of an Anaerobic Baffled Reactor (ABR) testing domestic wastewater. M.Sc. thesis. University of KwaZulu-Natal, School of Biological and Conservation Sciences, Durban, South Africa

[35] Muller, A.M., (1999). General authorisations in terms of section 39 if the national water act, 1998 (Act no. 36 of 1998). Government Gazette no. 20526, Department of Water Affairs and Forestry, South Africa

[36] Department of Water Affairs and Forestry (1996). South African Water Quality Guidelines, Volume 4: Agricultural Use: Irrigation. 2. Edition. Pretoria, South Africa

[37] She, Z., Zheng, X., Yang, B., Jin, C., Gao M. (2006). Granule development and performance in sucrose fed anaerobic baffled reactors. Journal of Biotechnologie, **122**(2): 198-208.

[38] Singh, S., Haberl, R., Moog, O., Shrestha, R.R., Shresta, P., Shresta, R. (2008). Performance of an anaerobic baffled reactor and hybrid constructed wetland treating high-strength wastewater in Nepal – A model for DEWATS. Ecological Engineering **35**, 654-660.